Department of the Army
U.S. Army Corps of Engineers
Washington, DC

Engineer Manual* 1110-2-4000

CECW-EC

14 February 2025

Engineering and Design

Sedimentation Investigations of Rivers and Reservoirs

FOR THE COMMANDER:

Digitally signed by DELAROSA.DAMON.ANDREW.1134345566
Date: 2025.02.13 15:56:14 -05'00'

DAMON A. DELAROSA
COL, EN
Chief of Staff

Purpose. Sediment-related problems are complex, costly, and can pose significant risks to the sustainability of U.S. Army Corps of Engineers projects and the benefits they provide to the nation. This engineer manual provides guidance to be used in planning, conducting, and documenting river and reservoir sedimentation studies that are performed for U.S. Army Corps of Engineers projects.

Applicability. The guidance presented and procedures described in this manual apply to all Headquarters USACE elements, major subordinate commands, Districts, laboratories, and separate field operating activities having Civil Works responsibilities.

Distribution statement. Approved for public release; distribution is unlimited.

Proponent and exception authority. The proponent of this manual is the Headquarters, United States Army Corps of Engineers, Engineering and Construction Division. The proponent has the authority to approve exceptions or waivers to this manual that are consistent with controlling law and regulations. Only the proponent of a publication or form may modify it by officially revising or rescinding it.

*This manual supersedes EM 1110-2-4000, dated 15 December 1989 and EM 1110-2-4000 Change 1, date 31 October 1995.

SUMMARY of CHANGE

EM 1110-2-4000, Sedimentation Investigations of Rivers and Reservoirs
United States Army Corps of Engineers

Boards, Commissions, and Committees: Headquarters, United States Army Corps of Engineers Engineering and Construction Division

This document revision, dated 14 February 2025

- Replaces the entire document with new content for chapters, appendices, and case studies from the previous version, dated 31 October 1995.
    - Adds content regarding sedimentation key concepts and project formulation for Corps projects.
    - Adds content regarding sedimentation considerations in the climate change assessment.
    - Adds content regarding sediment study investigations and developing the sediment studies work plan.
    - Updates sediment measurement techniques to reflect the current practice including measurement of bedload, suspended sediment, and erodibility.
    - Updates content for sediment yield computation including the effect of wildfire.
    - Adds substantial content regarding river sedimentation processes and evaluation for Corps project design and maintenance.
    - Adds substantial content regarding reservoir sedimentation processes and evaluation including impacts, dam removal, survey, and storage capacity depletion.
    - Adds content for evaluation of sediment management at reservoirs to mitigate capacity loss.
    - Adds substantial numerical modeling guidance.
    - Adds a new chapter with guidance describing sediment considerations for risk and uncertainty analysis.
- Adds 22 case studies demonstrating evaluation of sedimentation processes for Corps project design and maintenance.
- Updates references to match the current document and includes electronic links.
- Revises document format to follow guidance provided within ER 25-30-1, Guidance for Preparation and Processing of Publication and Forms, dated 1 March 2021.

TABLE OF CONTENTS

Chapter 4: Sediment Measurement Techniques

Chapter 5: Sediment Transport Mechanics and Analytic Transport Functions

Chapter 6: Sediment Yield

Table List

Figure List

Chapter 1
Introduction

1-1. Purpose. Sediment-related problems are complex, costly, and can pose significant risks to the sustainability of U.S. Army Corps of Engineers (USACE) projects and the benefits they provide to the nation. This engineer manual provides guidance to be used in planning, conducting, and documenting river and reservoir sedimentation studies that are performed for USACE projects.

1-2. Applicability. The guidance presented and procedures described in this manual apply to all Headquarters USACE (HQUSACE) elements, major subordinate commands (MSC), Districts, laboratories, and separate field operating activities having Civil Works (CW) responsibilities.

1-3. Distribution Statement. Approved for public release; distribution is unlimited.

1-4. References. See Appendix A.

1-5. Records Management (Recordkeeping) Requirements. The records management requirement for all record numbers, associated forms, and reports required by this publication are addressed in the Army Records Retention Schedule – Army (RRS-A). Detailed information for all related record numbers is located in the Army Records Information Management System (ARIMS)/RRS-A at https://www.arims.army.mil. If any record numbers, forms, and reports are not current, addressed, and/or published correctly in ARIMS/RRS-A, see DA Pam 25-403 for guidance.

1-6. General. Preparation of this guidance document benefitted from the direct contributions of over 50 USACE engineers and scientists over the course of several years. Document development relied on the efforts of several hundred more USACE staff that contributed relevant content and valuable review comments. Authored content was received from USACE expert staff with broad experience in the analyses and resolution of sediment problems encountered within USACE projects. Sedimentation analyses are complex; typical USACE project evaluation will require an experienced interdisciplinary team and the consultation of additional references

1-7. Scope. This manual identifies typical sediment problems encountered in the development of flood risk management, reservoir operation, environmental restoration, navigation, and hydropower projects for inland waters. While this manual encompasses common problems and analysis techniques, sedimentation analyses are complex and often require consulting additional references. Thorough analysis of sedimentation problems will often require an interdisciplinary team to include experts knowledgeable in the fields of hydrology, hydraulics, erosion and sedimentation, river mechanics and geomorphology, and soil mechanics.

1-8. Manual Content. This manual covers a broad range of topics, discusses the selection of appropriate data collection and analysis methods, provides guidance on determining the level of details, highlights potential study problems, and suggests acceptable approaches for their analyses. This manual includes a sedimentation glossary. Case studies included provide examples of scoping and conducting actual investigations. An extended reference list provides additional resources to aid in evaluating complex sediment problems. Not all investigations require using every section of the manual. Figure 1-1 illustrates the document structure and provides a guide for navigating through the content.

a. Chapter 1, Introduction. This chapter provides a summary of the requirements for sedimentation studies in the reports and continuing authorities for CW projects in USACE. Written as an overview, the chapter provides the basis for sediment studies and serves as a communication tool with management.

b. Chapter 2, Formulation and Planning of Sediment Studies. This chapter explains the typical sediment study formulation process. It includes guidance for identifying the sediment problem, defining the appropriate level of study, estimating the required time and costs for the work, organizing the tasks, and managing the investigation.

c. Chapter 3, Sediment Properties. Guidance is presented on the properties of sediment particles and mixtures. The chapter focuses on the properties of inorganic noncohesive sediments such as sand and gravel.

d. Chapter 4, Sediment Measurement Techniques. This chapter presents techniques for measuring bed-material properties, suspended and bedload discharges, and particle analysis. Careful planning of sediment data collection efforts is necessary. Data collected by new techniques should be compared with other methods to ensure reliability and comparability.

e. Chapter 5, Sediment Transport Mechanics and Analytic Transport Functions. Sediment transport mechanics includes the processes of erosion, entrainment, transportation, deposition, and compaction of sediment. These are natural processes that have been active throughout geological times and have shaped the present landscape of our world. This chapter provides theoretical background on sediment transport mechanics and frequently applied sediment transport functions.

f. Chapter 6, Sediment Yield. Guidance is presented on the selection and application of procedures for calculating sediment yield, along with the underlying assumptions in these methods. Field methods to validate these calculations are discussed along with an examination of the effects of natural and anthropogenic disturbances on sediment yield.

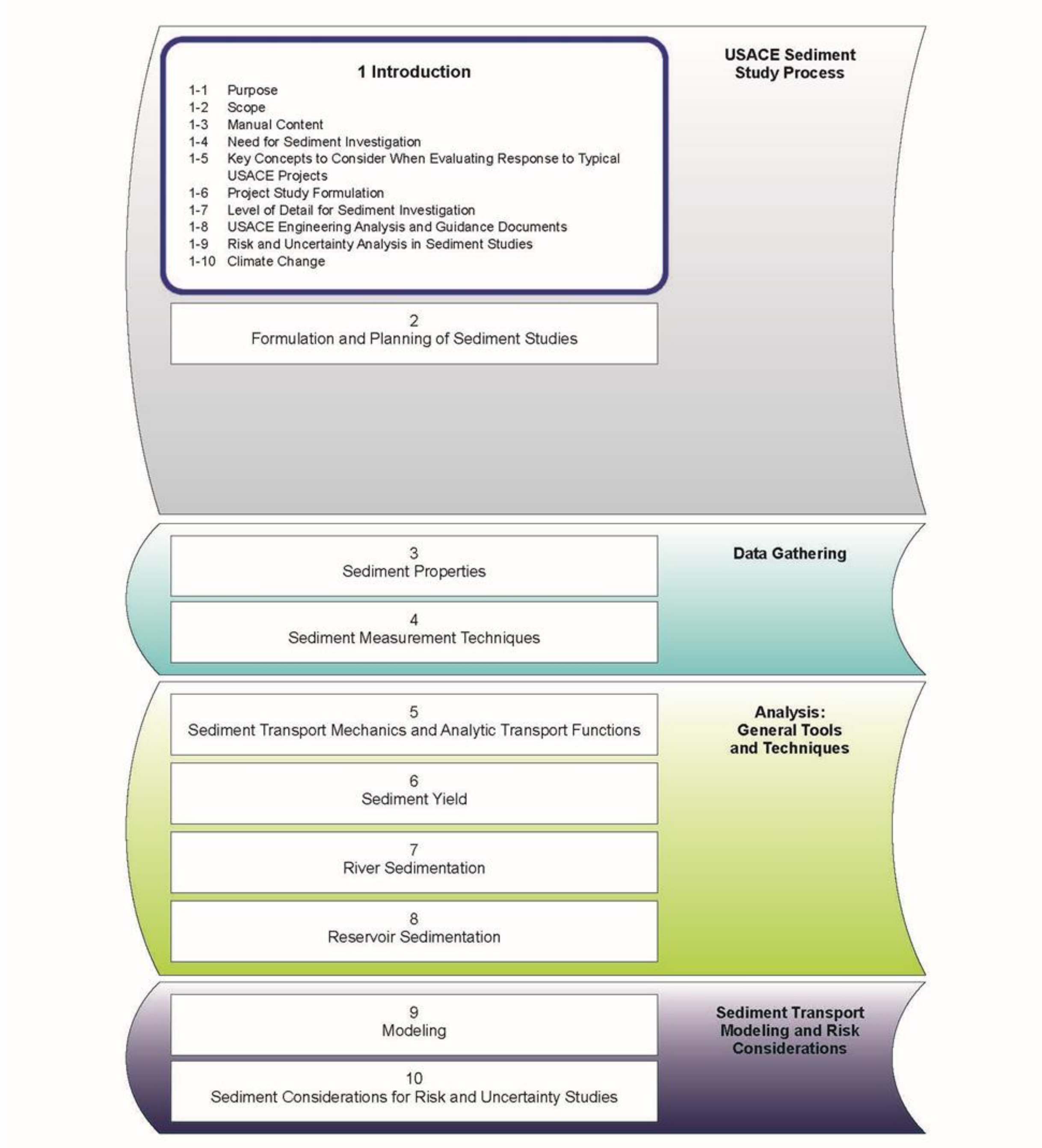


Figure 1-1. Chapter 1 content and general document structure

g. Chapter 7, River Sedimentation. This chapter introduces fundamental concepts of fluvial geomorphology related to sedimentation to provide an understanding of river physical processes. Knowledge of river processes is fundamental to identifying potential river sedimentation problems as they relate to typical USACE projects. Commonly applied procedures such as stability assessments are presented.

h. Chapter 9, Modeling. This chapter describes the proper use and application of sediment models to USACE river and reservoir sedimentation studies. The chapter includes sections on modeling philosophy, the stages of a sediment modeling study, data requirements, and calibration strategies.

i. Chapter 10, Sediment Considerations for Risk and Uncertainty Studies. USACE flood risk management (FRM) projects are planned, designed, constructed, and operated to reduce risk to people and property. USACE policy requires formal risk analysis of parametric uncertainty to identify and formulate optimal alternatives during USACE study phases. This chapter provides supplemental guidance on integrating results from sediment analyses and from uncertainty associated with sediment processes into the typical risk and uncertainty analyses conducted for USACE FRM planning studies.

j. Appendixes. References are contained in Appendix A, additional appendixes are included that contain supportive technical material, and a glossary of sedimentation terms is included at the end of the document.

k. Case Studies. Appendix N includes multiple case studies, which illustrate concepts presented in the technical chapters.

l. Manual Terms. This manual was created over an extended period by numerous primary and secondary authors. While the document was reviewed for consistency, some variation in terms describing sedimentation and geomorphic processes may occur. The reader is encouraged to review entire manual sections rather than individual paragraphs to better understand context. The glossary provides an additional resource for terminology definition. In this manual, sedimentation is used as a broad term that embodies the dynamic processes of erosion, entrainment, transportation, deposition, and compaction of sediment. USACE investigations are performed to evaluate sedimentation in river and reservoir applications.

1-9. Need for Sediment Investigation.

a. USACE projects in the river and reservoir environment are affected by sediment processes that include surface erosion, sediment transport, scour, and deposition. Sediment investigations often require analysis of the existing condition, the altered condition with project features, and identification of maintenance needs to understand if a project is sustainable. Investigations are conducted to evaluate both short- and long-term river and/or reservoir response and project performance.

b. Sedimentation must be considered at the earliest phases of study because minor changes in a stream system may alter stream dynamics and introduce significant risk to project function from sediment processes, including channel migration, erosion, and/or deposition. ER 1110-2-8153 requires that a sediment impact assessment be prepared for all projects. This guideline is implemented to meet USACE planning study objectives that project cost-sharing design costs do not escalate after the cost-sharing agreement has been signed.

c. Significance of Sedimentation Concerns.

(1) USACE water resource projects often impose environmental changes on the stream system that must be considered when developing alternatives during planning and subsequent design phases. Impacts of sedimentation on project performance and stream morphology must be assessed to properly estimate project impacts, risks, and costs. Developing an accurate sediment study cost estimate depends on early identification of sedimentation issues and scoping the appropriate level of effort to address those issues through a Sediment Studies Work Plan (SSWP). Chapter 2 describes this workflow in further detail.

(2) Single-event sediment processes may have long-term socio-economic impacts such as disruption to community access and loss of land use due to deposition. Refer to the extreme event Case Study 10A (Appendix N) for additional presentation of severe impacts that can occur at USACE-constructed projects as a result of sediment processes.

(a) Sediment processes can result in severe consequences to constructed USACE projects in a single event. Sediment deposition can dramatically reduce channel capacity and lateral channel migration can damage adjacent infrastructure (Figure 1-2).

Figure 1-2. Jamestown, Colorado, October 2013, (a) illustrating bridge blockage from sediment post-flood and (b) lateral channel migration and road damage

(b) In other examples of single event devastation, floodplain areas may be threatened by channel-altering primary flow paths, and levee breaches are often associated with scour and landward sediment deposition (Figure 1-3).

Figure 1-3. (a) Lyons, Colorado, channel alteration and (b) Missouri River 2011 sediment damage landward of levee

(3) The following summary examples are from actual USACE projects with a variety of sediment-related problems encountered during project design, construction, and maintenance. These examples are provided to highlight consequences of inadequately considering sediment processes and the impacts of uncertainty. The SSWP and sediment investigations are tools to facilitate developing a better understanding of the range of potential outcomes and river conditions that can develop with and without a project in place. Projects developed under this framework are expected to be more resilient and better able to withstand unexpected events.

(a) Example 1 – Sedimentation Impacts for Alternative Analysis. The study tentatively selected plan (TSP) was determined prior to conducting detailed sediment analysis. Subsequent phase studies found that the TSP increased rates of sediment deposition within various stream reaches. The sediment analysis determined that the increased deposition over time would eventually reduce channel capacity or navigation depths such that mechanical sediment removal would be required to maintain expected benefits. The recurring removal cost was estimated to exceed the project benefits and the TSP was eliminated from further consideration. Delaying the sediment analysis until after plan selection resulted in study delays and increased costs.

(b) Example 2 – Sedimentation Impacts from Implemented Maintenance. Channel maintenance identified a need to restore channel capacity lost to sediment deposition. The initial construction was in a channelized reach but was later extended into an unmodified meandering reach. The meandering reach was improved only by selective removal of trees along the banks using the stream obstruction removal guidelines (C. McConnell et al., 1983) against recommendations from Hydraulic Engineers. Removal of the selected trees destabilized the reach, causing rapid lateral migration of the channel banks and severe erosion. Remediation efforts were required to stabilize the 1.5-mile reach at a cost of approximately $1.5 million (1990 dollars).

(c) Example 3 – Uncertainty Associated with Sedimentation Analysis. Capacity of a flood control channel with levees had declined due to debris and sediment deposition from an upstream channelized reach. Extensive sediment studies and related modeling identified a plan for restoring channel capacity that included two sediment traps with sediment removal projected at a

10-year interval. During project construction, a flood event occurred with an extended duration crest that exceeded the duration modeled during project design. The long-duration event refilled the channel with sands, requiring immediate cleanout before the contractor demobilized excavation equipment. While no further maintenance was needed during the ensuing 10 years, the example highlights sedimentation analysis uncertainty.

(d) Example 4 – Uncertainty Associated with System Response. An urban channel with a highly developed watershed and floodplain was studied for flood reduction measures as well as multiple Section 14 emergency repair actions. Mobile bed numerical and physical models were used to evaluate alternatives for Section 14 repairs. Model predictions and design measures were invalidated between time of contract advertisement and start of construction due to three successive flood events that radically altered channel geometry and alignments. Designs to protect critical infrastructure had to be rapidly adjusted to accommodate continuously changing conditions.

(e) Example 5 – Sedimentation Impacts on Habitat. An upstream series of dams traps nearly 100% of transported sediments. While the project design team predicted downstream degradation, the loss of sand material dramatically reduced sandbar habitat quantity and quality from pre-dam conditions with high impacts to sandbar-dependent species. Reservoir releases are much less turbid compared to pre-dam conditions. A small fish native to the river has become easy prey in the low turbidity environment below the dam and is now a threatened species. The altered river has high bank erosion rates with impacts to critical bank nesting habitat locations used by an endangered bird species. Bank protection projects to address bank erosion have further impacted habitat and sedimentation.

(f) Example 6 – Uncertainty Associated with Project Performance. A reservoir project authorized for flood risk reduction was designed and constructed based on estimated rates of sediment deposition in the flood control pool. Actual rates of sedimentation were higher than predicted, which reduced storage capacity more than expected. A resurvey of sediment ranges confirmed significant loss of storage capacity. The reduction in available flood control storage contributed to more frequent use of the emergency spillway to pass floods during major storm events. Reanalysis of the project under probable maximum flood conditions indicates that the project can no longer provide authorized purposes without significant structural modifications.

d. Water/Sediment Balance.

(1) Nature maintains a delicate balance among a number of variables that affect water and sediment movement in a watershed. These variables include:

(a) Watershed/basin runoff.

(b) Sediment availability and production in a watershed/basin.

(c) Water velocity, depth, and duration.

(d) Concentration, size, and density of sediment particles that move with the water.

(e) Width, depth, hydraulic roughness, planform, and lateral migration of the stream channel.

(2) The natural balance between water and sediment movement through a watershed is dynamic. Lane's balance, or Lane's relationship (Lane 1955b), is a qualitative conceptual model that can be used to assess stream responses to changes in stream flow, slope, sediment discharge, and sediment size. The model is based on the general theory that if force applied by the flowing water on an alluvial channel boundary is balanced with strength of the channel boundary and the delivered sediment load, the channel will be stable and neither aggrade nor degrade. This equilibrium condition in the channel can be expressed as a balance between the four basic factors consisting of sediment load, sediment size, slope, and discharge (Lane 1955b). The balance is dynamic and is often represented with a sliding scale and balance as shown in Figure 1-4 (NRCS 2007).

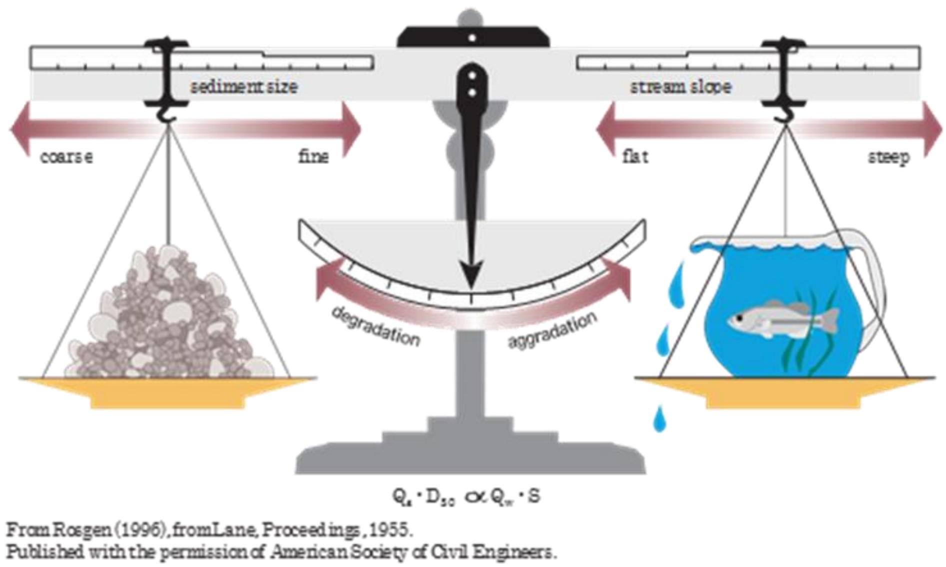


Figure 1-4. Lane's (1955b) sediment balance (NRCS 2007)

Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied. (Also Published in Rosgen (1996), from Lane, Proceedings, 1955 with permission of American Society of Civil Engineers)

(3) The concept of a balance between water and sediment is also expressed by dynamic equilibrium. Wiederhorn et al. (1997) developed a handbook on streambank stabilization that included a definition of dynamic equilibrium. They explained that "… a stable river, from a geomorphic perspective, is one that has adjusted its width, depth, and slope such that there is no significant aggradation or degradation of the stream bed or significant planform changes (meandering to braided, etc.) within the engineering time frame (generally less than about 50 years). By this definition, a stable river is not in a static condition, but rather is in a state of dynamic equilibrium where it is free to adjust laterally through bank erosion and bar building."

(4) Example. Application of these concepts provide qualitative prediction of channel response. For instance, an alluvial channel in a state of dynamic equilibrium may experience urbanization with increased runoff. Lane's balance tilts down to the right due to the increased

flow, causing the pointer to shift left toward degradation. This increased sediment supply may allow downstream reach segments to continue near equilibrium for a time. However, because the banks in urbanized systems typically become armored and impervious area may reduce sediment yield, the long-term sediment supply is likely to be reduced. A reduction in sediment supply further tilts Lane's balance to the right. As a result, many urban areas experience significant downcutting to flatten slope and bed coarsening in order to restore Lane's balance.

1-10. Key Concepts to Consider When Evaluating Response to Typical USACE Projects. The following key concepts must be considered when evaluating the potential for system response to a typical USACE project.

a. Water Resource Project Design and Response.

(1) All water resource projects impose some changes to stream/watershed variables (such as flow, shear, velocity). A successful design evaluates the degree of impact and revises project features to maintain a sustainable project.

(2) Constructed modifications impact stream/watershed parameters that influence the behavior of the stream system. The impact may extend beyond the immediate project footprint. Sedimentation investigations must consider both sediment impacts on project performance and project impacts on system morphology.

(3) Sediment issues can be episodic or gradual. However, over time, even gradual sediment issues often accumulate, impacting project performance.

(4) Sediment impacts start long before the end of a project's design life and continue long after.

(5) Sedimentation processes can have significant influence on project performance. Maintenance during the project operation phase can be one of the most important sediment processes to analyze and can drive the total project cost.

(6) Sedimentation should evaluate system response for a variety of conditions and time scales. In some cases, short observed data record length may require developing synthetic flow and sediment data. Extending historic records or estimating future conditions can be challenging if non-stationarity is occurring. The most common scenarios analyzed are:

(a) Existing conditions.

(b) Future without/with project.

(c) Additional future conditions or time periods to reflect variable sedimentation processes.

(d) Decommissioning or end of life, which can result in large sediment releases over short time periods.

(e) Sediment sustainability/retirement plan.

b. Degree of Study.

(1) No simple formula exists to predict the likelihood or severity of sediment problems. Appropriate analysis tools are available that can be applied at every study stage to predict impact.

(2) Determining the appropriate degree of study and technical analysis tools are critical components of a successful sediment investigation.

(3) The level of investigation depends on historic system stability, degree of anticipated project change, and phase of study, and should be risk-informed.

(4) Sediment studies require spatial and temporal data sets that describe geometry, hydrology, hydraulics, sediment, and land use variables. Sediment studies are often based on sparse data sets. Data sources, reliability, and trends should be carefully considered when selecting methodology.

(5) Sedimentation investigations, with perceived higher cost and uncertainty, are often required to consider performing a lower level detail analysis combined with a qualitative assessment of project performance risk. While appropriate in some instances, caution must be emphasized when extrapolating qualitative assessments to a future project condition. In general, it is not acceptable to eliminate the necessary analysis needed to fully assess the system's likely sediment response to change.

(6) Past system behavior provides a preview of likelihood of sediment problems.

(a) Historically stable channel—magnitude of sediment problems is generally proportional to degree of change from existing state.

(b) Historically unstable channel—magnitude of sediment problems expected to be severe requiring more extensive investigations.

c. Resilience. Resilience is the ability to anticipate, prepare for, and adapt to changing conditions and withstand, respond to, and recover rapidly from disruptions. Recent USACE guidance such as the Resilience Initiative Roadmap Engineer Pamphlet and the Implementation of Resilience Principles in the Engineering and Construction Community of Practice Engineering and Construction Bulletin present the four USACE principles of resilience: Prepare, Absorb, Recover, and Adapt (PARA) that apply to both Civil Works, Military Programs, and International and Interagency Programs.

(1) The Prepare principle considers measures to meet the needs of a project component or system, including reducing risks or costs under loading conditions beyond those required by technical standards or norms.

(2) The Absorb principle considers measures to limit damage to, or loss of function of, a project component or system due to both acute and chronic loading conditions, including conditions beyond those used for the design. This principle can also be used as an opportunity to consider adding system component robustness, redundancy, and increased reliability.

(3) The Recover principle stresses wise and rapid repair or functional restoration of a project component or system.

(4) The Adapt principle considers modifications to a project component or system that maintains or improves future performance based on lessons learned from a specific loading condition or loadings associated with changed conditions.

(5) To properly build resilience into USACE projects, consideration for loading conditions outside of those used for design is necessary. Sediment studies that consider the uncertainties posed by climate change will likely be necessary to understand the resilience of many USACE projects.

d. Project Risk.

(1) Include risk and uncertainty when formulating water resource project alternatives. Regarding sedimentation process impacts on USACE projects, two major sources of uncertainty in water resource projects are future sedimentation processes that may vary drastically from the historic, and the morphologic response of the natural system to changes imposed by USACE projects. Chapter 10 examines these topics in detail.

(2) Uncertainty related to sedimentation and morphologic response of the natural system may result in excessive operations and maintenance (O&M) costs, or in extreme cases, result in project failure due to long-term cumulative changes or single-event project response.

(3) Consider that sediment processes can be significantly different for extreme events compared to normal or more frequent floods. Abrupt shifts in normal processes can have catastrophic project consequences.

(4) Appropriate study analysis risk is generally determined relative to project complexity and scale for each study phase. As the study phase progresses and as project scale/complexity increase, the tolerable risk related to sedimentation processes decreases. As a result, the additional analysis and data collection is increased in order to reduce sedimentation process uncertainty.

(5) Focal Points to Consider. Sediment problems are often unequally distributed within a project. Projects that alter ongoing sedimentation processes often occur with the following project features:

(a) Increased bottom width.

(b) Bridge crossings.

(c) Change in channel longitudinal slope.

(d) Cutoffs/changes in channel alignment.

(e) Any feature in braided reaches.

(f) Upstream and downstream transitions to the project reach.

(g) In-channel structures (weirs, training dikes, etc.).

(h) Tributaries entering project reach.

(i) Water diversions.

- Downstream from dams/grade control structures.

- Upstream from reservoir pools/grade control structures.

(j) Lower reaches of tributaries.

(k) Sediment input/removal points.

(l) Set-back levees for rivers with high sediment loads.

1-11. Project Study Formulation.

a. USACE project formulation, evaluation, and selection for implementation follow procedures that have been established for the various USACE missions and programs. Descriptions of the procedures and different types of planning authorities can be found in ER 1105-2-100. Table 1-1 lists the typical USACE sedimentation study types.

b. Each of the study types listed in Table 1-1 follows a tiered approach to investigating water resources problems. Each tier of investigation is intended to improve understanding of the problems and identifying viable solutions to those problems. Earlier phases of investigation (such as during the reconnaissance phase) are very limited in scope and scale. However, it is imperative that appropriate consideration be given to sediment processes even at this early stage of investigation.

Table 1-1
Typical Sedimentation Study Types

Study or Report Type	Subcategory	Description
Pre-Authorization		• In response to either a study-specific authority or general authority • Funded with General Investigations funds • Conducted in two phases per WRDA 1986
	Reconnaissance Phase	• Determine if problem(s) warrant(s) federal participation • Define federal interest • Complete 905b analysis • Prepare Project Management Plan • Assess level of interest and support from non-federal entities • Negotiate and execute Feasibility Cost-Sharing Agreement
	Feasibility Phase	• Investigate and recommend solutions to water resources problems • Results documented in a feasibility report that includes environmental compliance documentation
Post-Authorization		• Generally funded as part of engineering and design studies under general investigation appropriations • Undertaken per specific construction authorities
	General Re-evaluation	• Re-analysis of previously completed study using current planning criteria and policies required due to changed conditions and/or assumptions • Results are documented in a General Re-evaluation Report (GRR)
	Limited Re-evaluation	• Provides an evaluation of a specific portion of a plan • Results are documented in a Limited Re-evaluation Report (LRR)
	Preconstruction Engineering and Design (PED) phase: Design Documentation Reports (DDRs), Engineering Documentation Reports	• Provides technical basis for the plans and specifications • Serves as a summary of the final design • Designs are documented in DDR • DDR may be used to document other information not included in a decision document if only technical changes are required

Study or Report Type	Subcategory	Description
National Environmental Policy Act Documentation	Various Phases and Studies	• Document scope and nature of changes in environmental effects of the project due to acquisition of new information • May be documented in an Environmental Assessment (EA)
Other Types	Water Resources Needs, Flood Insurance Studies, Planning Assistance to States, Continuing Authorities Program, Review of Completed Projects	• Study water needs of river basins and regions of the United States in consultation with State and local entities • Conducted under various authorities with specific legislative requirements • Results in technical reports

c. The Water Resources Development Act of 1986 (Public Law 99-662) established new requirements of local entities that sponsor USACE water resource projects. Under these requirements, local sponsors are liable for more of the project design and construction costs.

(1) Most federally constructed projects are turned over to the local sponsor for operation and maintenance. Unidentified sedimentation issues can become an insurmountable long-term maintenance cost. Study design should be adequate to identify maintenance issues due to sedimentation. Many local sponsors assume an active role in the design process and should be clearly informed of maintenance costs and risks.

(2) Cost-share requirements necessitate an accurate assessment of study costs before the local cost-sharing agreement (LCA) is signed. Because the LCA must be signed before initiation of project feasibility reports, firm project cost and time estimates must be established during the preparation of the first planning document, typically the reconnaissance report. Therefore, the standard USACE study process requires that the scope, schedule, and budget requirements for sediment studies be established early in the project planning process.

d. The procedures for conducting feasibility studies are heavily influenced by the USACE planning process, currently referred to as the Sustainable, Mobile, Alternative, and Renewable Technologies (SMART) planning guidelines. In the USACE SMART Guide, which resides on the website of the Planning Community Toolbox, SMART planning is:

(1) S: Specific.

(2) M: Measurable.

(3) A: Attainable.

(4) R: Risk-Informed.

(5) T: Timely.

e. Although the USACE planning process acronyms may change, the advent of cost-sharing partnerships with local sponsors and the desire in the USACE plan formulation process (SMART planning or similar USACE process) to reduce the time and cost of delivering a feasibility level study may limit the level of investigation possible.

f. To complete studies within allowable schedules and budgets, some uncertainty is acceptable. The uncertainty is captured in a risk register. The risk register documents the relevant details of risks that could result from actions taken or not taken during each stage of a project's life cycle.

(1) Because sedimentation investigations are perceived to be higher cost, higher uncertainty, and longer duration tasks, study scope development often considers the tradeoff to perform a lower level detail analysis and capture risks in the risk register.

(2) Accurately capturing project performance risk places paramount importance on having skilled river and sedimentation engineering team members with significant experience and understanding of sedimentation processes involved in early studies. However, study scoping must consider that an accurate characterization of risk still requires performing the necessary field investigations, data collection, and analysis efforts to fully assess the system's likely sediment response to change.

(3) An accurate and comprehensive analysis, which is essential to assess project performance risk, cannot be addressed with a simple qualitative evaluation. Early communication of sediment processes that often strongly influence long-term project performance is critical to a successful study.

(4) The risk register is not intended to replace necessary sedimentation analysis with stated risk. Deferring critical sedimentation studies to future design phases, via the risk register, is not acceptable. Chapter 10 provides further details on the risk register.

1-12. Level of Detail for Sediment Investigation.

a. The Project Delivery Team (PDT) must consider the appropriate level of detail that is commensurate with study phase, objectives, and final products. Typically, the PDT considers a series of questions to define decision-making criteria related to project objectives and to set an acceptable level of risk.

(1) Evaluation of existing and project sedimentation processes that affect project performance must be addressed in the initial study phase. Sediment data collection can often be a time-consuming process that relies on the occurrence of meaningful flow events.

(2) Sedimentation investigations are sometimes targeted for deferment until late in the study process because of a perceived high cost for data collection and analysis. This can be a significant misconception. Where the project involves a stream or river, sedimentation processes must be addressed early to ensure that long-term USACE project performance can be reasonably maintained.

b. Sedimentation investigations do not always translate into high-cost tasks. Including measures to assess potential sedimentation issues early can reduce overall study and project costs by helping to identify a project design that achieves an optimum balance between project performance, first cost, and long-term operation and maintenance. In some cases, including sedimentation investigations during the initial study phase helps screen out alternatives that would not be viable before the PDT invests project resources on engineering or economic analysis.

c. Typical questions when considering level of detail are illustrated in Table 1-2.

d. To ensure efficient execution of planning studies without compromising essential sedimentation analyses, it is necessary to carefully outline the SSWP. The SSWP methodically outlines the tasks required to address the study objectives and provides a cost for conducting the necessary analyses. The SSWP addresses the level of detail anticipated at each step of investigation to manage risk during project design phases. Chapter 2 discusses the level of detail more specifically.

Table 1-2
Typical Sediment Investigation Level of Detail Questions

What problems, opportunities and objectives exist for the study or investigation?
Is the level of detail appropriate for the study phase and outcome? a) Detail for project feature formulation. b) Screening of alternatives. c) Project costs and benefits. d) Sustainable project design.
What information is needed? a) Flow Exceedance/Duration. b) Regulated/unregulated discharge. c) Sediment budget/sediment impact assessment/sediment sources and sinks. d) Sediment load (suspended, bedload). e) Morphological trend data (such as repeated cross sections, historic maps, or photographs). f) Sediment characteristics (particle size distribution, median particle size). g) Historic and future basin anthropogenic impacts. h) Identification of sediment sources and/or sinks.
How is this information linked to the study's defined problems, opportunities, objectives, and constraints?
Is the information a high priority need, or a low priority?
Does the information have to be quantitative? If so, why?
Is the data available from other sources or prior studies? a) Yes – is there a good reason available data cannot be used? b) No – what is the least effort needed to get a sufficient amount of data? Is additional data collection viable considering analysis methods?
What is going to change between the existing conditions and future? a) Increased flooding (timing, frequency, duration). b) Increased consequences of flooding. c) Habitats near the river will improve/degrade. d) Trends in geomorphic processes (aggradation, degradation, etc.). e) Are geomorphic processes consistent or will rates change in the future?
What assumptions are being made to answer the above questions for future characteristics and trends?
Are there ways to run the tools with less detailed information where possible and plausible?

1-13. USACE Engineering Analysis and Guidance Documents.

a. The formulation of a Civil Works project must provide a safe, efficient, reliable, and cost-effective design in the most feasible environmentally sustainable manner. The appropriate degree of effort given to each of these design elements varies from one stage of project formulation to the next. USACE engineer regulations and analysis guidance should be reviewed when formulating sedimentation studies. Primary USACE references are:

(1) EM 1110-2-1418, Channel Stability Assessment for Flood Control Projects.

(2) EM 1110-2-1419, Hydrologic Engineering Requirements for Flood Damage Reduction Studies.

(3) EM 1110-2-1420, Engineering and Design Hydrologic Engineering Requirements for Reservoirs.

(4) EM 1110-2-1601, Hydraulic Design of Flood Control Channels.

(5) ER 1110-2-1405, Hydraulic Design for Local Flood Protection Projects.

(6) ER 1110-2-8153, Engineering and Design, Sedimentation Investigations.

(7) Technical Report (TR) 1-28, Hydraulic Design of Stream Restoration Projects.

(8) Technical Note (TN) Ecosystem Management and Restoration Research Program (EMRRP) Stream Restoration (SR) 39, Sediment Sampling and Analysis for Stream Restoration Projects.

b. In addition, a typical sedimentation investigation will often require the use of multiple technical references from various sources dependent on specific project requirements. Development of project models, and the accompanying reference material, should also be consulted.

1-14. Risk and Uncertainty Analysis in Sediment Studies.

a. USACE FRM projects are planned, designed, constructed, and operated to manage risks for people and property. Risk-based analysis is defined as an approach to evaluation and decision-making that explicitly, and to the extent practical, analytically incorporates considerations of risk and uncertainty. Within USACE, risk-based analysis must be used to compare plans in terms of the likelihood and variability of their physical performance, economic success, and residual risks. A risk-based approach to water resources planning captures and quantifies the extent of risk and uncertainty in the various planning and design components of a project.

b. Risk is a computed index representing the product of the probability and consequence of an event such as flooding. USACE policy stated in ER 1105-2-101 requires analysis of parametric uncertainty on alternative selection. The guidance document EM 1110-2-1619 describes USACE risk and uncertainty policy and procedures in detail and should be consulted for the full risk analysis procedures. Chapter 10 of this manual presents supplemental information focused specifically on integrating results of sediment analyses and of uncertainty associated with sediment processes into USACE risk and uncertainty analyses procedures.

c. The combined effects of parameter variability (uncertainty) related to sedimentation for complex projects such as dam removals can be greater than those for the no-action condition because there are more variables affecting the river response. A comprehensive risk analysis considers not only the physical changes predicted by a model but also the effects due to increases in the parameter uncertainty. For example, model predictions of the downstream channel response to a dam removal could vary greatly depending on the inflows during and after removal, the rate and method of dam removal, as well as model parameter uncertainty. A robust sensitivity analysis is often necessary to properly consider uncertainty as part of a sedimentation study.

1-15. Climate Change.

a. Sedimentation investigations involve consideration of the hydrologic cycle that may be influenced by climate change. Variation in climatic conditions can result in two principal changes in basin hydrology and sedimentation. First, storm intensity, depth, and duration may be different from that indicated in past data, which directly impacts sediment yield computations. Second, the seasonality of storms may be different from historic with consequences such as precipitation changing from snow to rain or the occurrence of dry hot periods that affects vegetation. Both types of change affect watershed sediment yield and in-channel sedimentation processes.

b. The significance of climate change on sedimentation investigations is related to various thresholds that pertain to sediment mobility and movement. If climate change produces sufficient change in hydrologic inputs to exceed threshold values, a project may fail to perform as expected. Similarly, climate change effects on basin hydrology may alter land cover by changing vegetation density and species composition, which impacts sediment mobility.

c. USACE guidance continues to evolve in response to conducted studies. Impacts to sedimentation processes may impact USACE projects with many possible outcomes. Water resources management agencies are working together using observed data and model outputs to assess possible future effects from climate change on the runoff and streamflow inputs that could be used in computational assessments of sedimentation processes. When developing study scope, consult current USACE guidance for the latest available methods to evaluate quantitative changes to sediment yield, in-channel sediment routing, or reservoir sedimentation due to changing climate.

d. Future sediment yield is also likely to be heavily influenced by non-climate changes (such as urbanization and changes in land use and land cover), and by indirect climate change effects (such as changes in frequency, duration, intensity, location, seasonality of wild land, and prescribed fire in the basin). There is even less agreement regarding how to incorporate non-climate and indirect climate change effects into analyses of future sedimentation rates. Factors that cannot be accounted for in modeling efforts should be discussed in the risk register.

Chapter 2
Formulation and Planning of Sediment Studies

2-1. Introduction.

a. General. This chapter provides guidelines and concepts to help formulate sediment studies for typical USACE projects. Description of the suitable level of detail for sediment studies is provided that varies with project phase and complexity. Figure 2-1 illustrates the chapter content.

b. Identification of Sediment Problems.

(1) The proper identification of sediment problems is a key step in successful project formulation. There is no simple formula that predicts either the likelihood that sediment problems will occur or the severity of sediment problems when they do occur. However, appropriate analysis tools are available that can be applied at every study stage. A critical component of project formulation is to identify potential sedimentation concerns during the early phases of investigation. Deferring or ignoring sedimentation evaluation during the planning process is not a USACE-accepted practice and creates the risk of omitting the formulation of necessary project features to accommodate sediment processes.

(a) The two aspects of a sedimentation investigation are to understand the potential impacts of: (1) sedimentation on project performance (capacity loss or dredging frequency), and (2) the project on the river system (water quality, habitat, morphology, erosion/deposition, etc.).

(b) Analysis of sedimentation processes and formulation of associated project features early in the planning process is necessary to identify impacts to project cost and benefits, and to avoid reformulating the selected plan during future study phases.

(c) Ignoring evaluation of sediment processes can jeopardize project performance during or after construction.

(d) In extreme cases, sedimentation impacts have resulted in non-performance of constructed USACE projects with catastrophic impacts.

(2) Field reconnaissance and prior experience are an essential part of identifying problems and developing the study scope. Therefore, there is a significant advantage for including highly skilled and experienced River and Sedimentation Engineers when the study initiates.

(3) Table 2-1 may serve as a basic guide in identifying potential sedimentation problems. It provides typical characteristics and stability concerns for different stream channel types and their possible interactions with project features. Information in Table 2-1 can be used as an aid in developing an initial scope for the sediment impact assessment by identifying potential problems and the associated severity of impacts for watersheds with different types of channels.

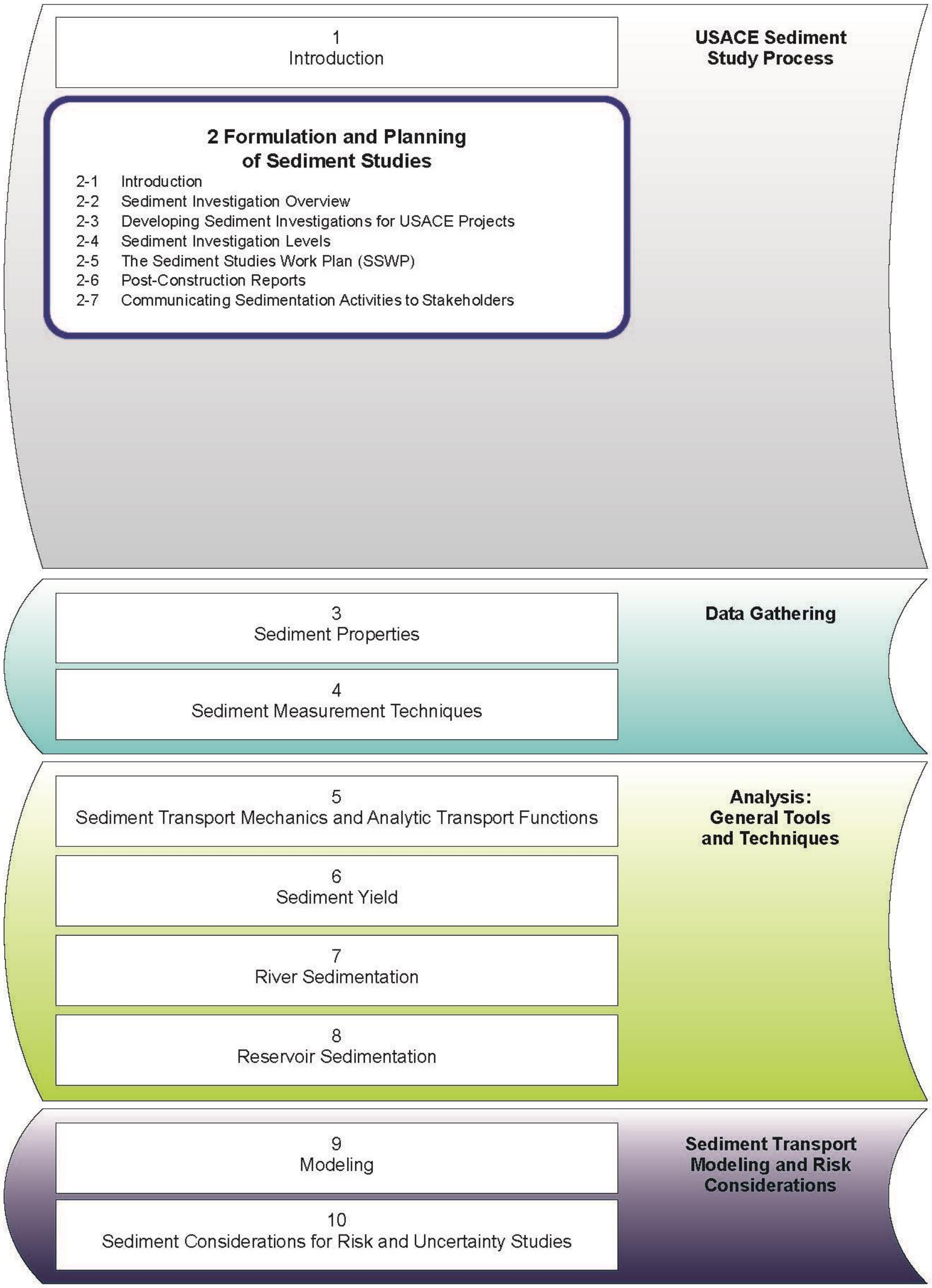


Figure 2-1. Chapter 2 content and general document structure

Table 2-1
Common Stream Channel Types and Characteristic Stability Problems[1]

Channel Type	Typical Features	Stability Problems	Potential Impacts to Water Resource Projects
Mountain torrents	Steep slopes. Boulders. Drops and chutes.	Bed scour and degradation. Potential for debris flow.	Bank instability, shifts in channel alignment.
Alluvial fans	Multiple channels. Coarse deposits.	Sudden channel shifts. Deposition. Degradation.	Rapid change in water profiles, rapid shift in channel location, bank instability, shifts in current direction.
Braided rivers	Interlacing channels. Coarse sediments (usually). High bedload.	Frequent shifts of main channel. Scour and deposition.	Bank instability, rapid shifts in channel location, rapid change in water profiles, shifts in current direction.
Arroyos/ Wash/ Wadi	Infrequent flows. Wide flat channels. Flash floods. High sediment loads.	Potential for rapid changes in planform, profile, cross section.	Bank instability, rapid changes in current pattern and water profiles.
Meandering rivers	Alternating bends. Flat slopes. Wide floodplains.	Bank erosion. Meander migration. Scour and deposition.	Bank instability at bends, gradual changes in channel location and current patterns.
Modified streams	Previously channelized. Altered base levels.	Meander development. Degradation and aggradation. Bank erosion. Headcuts.	Bank instability, changes in water profiles.
Regulated rivers	Upstream reservoirs. Irrigation diversions.	Reduced transport. Degradation below dams. Lowered base level for tributaries. Aggradation at tributary mouths.	Change in water profiles, increased bank height, bank instability.
Deltas	Multiple channels. Fine deposits.	Channel shifts. Deposition and extension.	Changes in water profiles, gradual change in channel location, distributary channel development causing deposition.
Underfit streams	Sinuous channels. Low slope.	Meander migration.	Bank instability.
Cohesive channels	Irregular or unusual planform.	Variable.	Variable, but response typically slower due to cohesive boundary materials.

[1] Revised from USACE, EM 1110-2-1418.

(4) Similarly, the data in Table 2-2 indicates how different types of USACE-constructed project features may affect sediment processes, stream stability, and risk of USACE project performance. Information in Table 2-2 can be used to adapt the scope of the sediment impact assessment to reflect the severity of potential changes in channel stability due to proposed alternative measures.

Table 2-2
Rating of Typical Project Impact on Channel Stability[2]

	Channel Types									
Flood Protection Measures	**Mountain torrents**	**Alluvial fans**	**Braided rivers**	**Arroyos/Wadi**	**Meandering rivers**	**Modified streams**	**Regulated rivers**	**Deltas**	**Underfit streams**	**Cohesive channels**
Non-structural; flood proofing, flood warning, evacuation	0	0	0	0	0	0	0	0	0	0
New levees set beyond stream meander belt[1]	1	2	2	1	1	1	1	2	1	1
Levees set within stream meander belt or along bankline	2	5	5	4	3	3	2	4	2	2
Off-channel flood detention basin	2	3	3	3	2	2	2	2	1	1
Within-channel flood detention basin	4	5	5	5	4	4	3	4	2	2
Major flood storage reservoirs	3	4	4	4	3	3	2	3	1	1
Compound channel – low-flow pilot plus flooding berms	5	8	8	7	7	6	6	7	4	4
Significant channel widening	6	9	9	8	8	6	7	7	5	5
Significant channel widening and deepening	7	9	9	9	9	8	8	8	6	7
Significant channel widening, deepening, and straightening	8	10	10	10	10	8	9	9	7	8
Floodway, diversion, or bypass channel	4	5	5	5	4	4	4	5	3	3

Channel Stability Rating Scale: No Stability Impacts » 0; Major Impacts On Stability » 10

[1] Setback of existing levees can induce local deposition due to local channel widening accompanied by a reduction in flow velocity.

(5) When identifying sediment problems and applying engineering judgment of sedimentation study issues and scope, consider the following concepts:

(a) Stable Channel Historically. When the existing channel is stable, the magnitude of the sediment problem for a project channel is generally proportional to the amount of deviation from the existing channel width, depth, slope alignment, vegetation environment, inflowing water discharge hydrographs, inflowing sediment concentrations, particle sizes in the inflowing sediment load, classification of sediment on the surface of the streambed, downstream stage

[2] Revised from USACE, EM 1110-2-1418.

discharge rating curve, distribution of water between channel and overbanks, and irregularities allowed in the design geometry.

(b) Unstable Channel Historically. When the existing channel is unstable, the study formulation should recognize that the magnitude of the sediment problem for the design channel will be sufficiently severe to require a detailed sediment study.

(c) Future Conditions. Sedimentation studies often rely on sparse historic data sets to determine future conditions. A robust evaluation of potential sediment issues should include consideration of how past trends may not be representative of future conditions.

(d) Sediment Study Limits. Studies should evaluate sedimentation impacts on USACE project performance within the project reach as well as the project influence on stream system geomorphic processes beyond the project construction reach. In addition to factoring into project cost/benefit computations during the planning process, system changes outside the project reach typically migrate into the project reach during future operation.

c. Identification of Potential Problem Areas. Sediment impacts are not equally distributed within a project and usually occur in specific areas with greater consequences. In general, the potential for impacts is greatest in the vicinity of project features that affect the balance of water and sediment movement, as listed in Table 2-2. Typical USACE project features where impacts tend to be focused include:

(1) Existing non-equilibrium locations that are exacerbated by project features.

(2) Increased/decreased channel width.

(3) Channel constrictions (decreased channel width).

(4) Bridge crossings.

(5) Abrupt breaks in channel slope.

(6) Cutoffs and changes in channel alignment.

(7) Braided channel reaches or similar areas showing high levels of instability.

(8) The upstream and downstream transitions to the project reach.

(9) Appurtenant structures in the channel, such as channel training structures.

(10) Within the main channel at tributary confluences and within the lower reach of tributaries.

(11) Water diversion points.

(12) Upstream and downstream from structures such as reservoirs and grade control.

(13) Levees (both near channel and setback) or features that alter floodplain conveyance.

(14) Features that alter natural levee formation near river banks, especially in sediment laden streams.

d. Successful USACE project implementation requires an understanding of the governing physical laws and a knowledge of the far-reaching effects of any attempt to control or modify a river's course.

(1) The complexity of the river processes within the normal hydrologic cycle and sediment movement through the system makes an analytical approach to sediment analysis difficult and time-consuming. Most available river process relationships have been derived empirically. Nevertheless, if a greater understanding of the principles governing the processes of river formation and adjustment is to be gained, the empirically derived relationships must be put in the proper context by employing an analytical approach.

(2) It is not always a requirement to fully understand all processes in great detail; for example, initial screening of alternatives may require only a cursory understanding of river behavior that is sufficient to determine viability of the proposed action (a yes or no answer). The necessary level of "cursory understanding" for initial screening depends on the complexity of the situation. Following the initial screening, more complex analysis is required to assess cause-effect relationships and long-term system behavior. Therefore, a staged investigation approach is warranted. A written plan should be developed to capture the concept of staged investigation and to estimate study costs.

2-2. Sediment Investigation Overview.

a. Purpose.

(1) The purpose of performing sediment investigations within a series of stages is to invest resources in an efficient and cost-effective way to obtain a risk-informed solution. Staged sediment studies identify the appropriate investigation detail to assess project goals, objectives, questions, and concerns with consideration of appropriate risk factors.

(2) ER 1110-2-8153 provides a framework for staged sediment studies in support of the hydrologic analysis and hydraulic design conducted for USACE projects. This guideline is implemented to meet the WRDA 1986 requirement that project cost-sharing design costs do not escalate after the cost-sharing agreement has been signed.

(3) ER 1110-2-8153 includes a description of three stages recommended for engineering evaluation in support of typical planning studies: (1) the sediment impact assessment, (2) detailed sedimentation study, and (3) feature design sedimentation study. In addition to planning projects addressed in ER 1110-2-8153, USACE sediment investigation studies also are performed during subsequent phases of the project life-cycle phases including project implementation, operation and maintenance, and decommissioning/major rehabilitation.

Combining these efforts, the general sediment investigation performed during the USACE project life cycle are:

(a) Sediment Impact Assessment (Planning – Reconnaissance).

(b) Detailed Sedimentation Study (Planning – Feasibility).

(c) Feature Design Sedimentation Study (Planning – PED).

(d) Project Implementation.

(e) Operation and Maintenance.

(f) Decommissioning/Major Rehabilitation.

(4) Sediment investigations can occur in all phases of analysis but are more common early in the project planning phase. Of paramount importance when designing sediment investigations is an understanding of the extent to which the project features deviate from the existing river regime and if this impact can affect project feasibility. Project features with the potential for high impact to sediment processes include reservoir construction, dam removal, river projects that significantly alter the stream cross section and slope, projects in areas with high sediment load, and projects in areas that currently show high sediment instability. Avoid deferring crucial analysis regarding project condition impacts to future study phase during the planning process.

b. Staged Sediment Studies.

(1) Once study objectives have been identified, it is up to the engineer to select an appropriate evaluation procedure. ER 1110-2-8153 requires that a sediment impact assessment be prepared for all reconnaissance-phase USACE projects.

(2) A “staged sediment studies” approach with the appropriate degree of detail should be followed in which contingency factors are assigned and revised as more data and analysis are available to decision-makers. The three sedimentation study stages identified in ER 1110-2-8153 should be regarded as the minimum standard. Complex or high-risk/consequence studies may require greater levels of investigation. The three stages are described as follows:

(a) Stage 1 – Sediment Impact Assessment (Reconnaissance). The sediment impact assessment identifies the magnitude of potential sediment problems that could be induced by the project. Identifying sedimentation processes and potential problems early is key during USACE planning studies to avoid lost effort and future impacts to study costs and schedule. The impact assessment includes an initial evaluation of sediment processes and channel stability. It will require numerical computations (such as rough sediment budget or simple capacity calculations) but generally relies on existing data. When the assessment identifies sediment problems that could affect the project economic viability or cause significant adverse environmental impacts, then this stage should include recommendations for a detailed sediment study scope.

(b) Stage 2 – Detailed Sedimentation Study (Feasibility). A detailed investigation incorporates the principle of sediment continuity through the project reach and is conducted using 1D and sometimes 2D numerical sedimentation models. The detailed sedimentation study further addresses problems reported in the sediment impact assessment, recommends corrective measures, and assesses the effectiveness of these measures.

- A detailed study during Feasibility will be required if the sediment impact assessment predicts a sedimentation problem; if a similar, existing project is experiencing sedimentation problems; or if significant adverse environmental impacts are predicted.

- Typically, the detailed sedimentation study includes analysis using a numerical sedimentation model. Model analysis should be used to fully evaluate project alternatives and to develop the sedimentation related inputs required for the risk and uncertainty analysis (Chapter 10). Studies must be sufficient to identify project features and cost to support Feasibility alternative plan comparison.

- Further information on typical sedimentation study scope can be found in Chapter 14 of the American Society of Civil Engineers (ASCE) Manual No. 110, Chapter 14 (Thomas and Chang 2008), U.S. Army Corps of Engineers-Hydrologic Engineer Center (USACE-HEC) (1992), in Chapter 6 through 10 of this document, and Appendix B.

(c) Stage 3 – Feature Design Sedimentation Study (PED). The feature design study is focused on sedimentation problems associated with project structures and site-specific features. These 3D problems may be evaluated using multidimensional numerical models and/or movable-bed physical models. The purpose of the feature design sedimentation study is to develop and present a detailed plan to identify and avoid sedimentation-related failure modes and to establish special operational procedures, as necessary. This type of study is an extension of the Detailed Sedimentation Study to test the final design of the project and relocation features. These are complex models that will require intensive data sets, both existing and newly collected, to conduct analysis.

(3) These three stages are general guidelines and may not be sufficient for all planning studies. In addition, subdividing these stages may often be appropriate. The engineer is responsible for supplementing these stages as needed to ensure adequate project evaluation.

(4) Details for analysis content and methods for each of the three stages are provided in Appendix B, and specific guidance is provided in Chapters 6 through 9. Risk and uncertainty principles, provided in Chapter 10, should be applied in the staged study approach.

c. Approved Software and Model Selection. Appropriate model selection for use with sediment investigations considers both project objectives and USACE policy. As stated in EC 1105-2-412, the USACE Planning Models Improvement Program (PMIP) was established in 2003 to assess the USACE state of planning models and to make recommendations to assure that high-quality methods and tools are available to enable informed decisions on investments in the nation’s water resources infrastructure and natural environment.

(1) The main objective of the PMIP is to carry out "a process to review, improve and validate analytical tools and models for USACE Civil Works business programs." Enterprise Standard (ES) 08101, Software Validation for the Hydrology, Hydraulics, and Coastal Community of Practice (HH&C CoP) (current version accessible on USACE internal SharePoint site), describes the process the HH&C CoP will follow to use and validate engineering software for use in planning studies and to satisfy the requirements of USACE's Scientific and Engineering Technology (SET) initiative.

(2) Project scope and the Program Management Plan (PMP) should include determination of project analysis software. Software listed on the HH&C CoP's SharePoint site as "CoP Preferred" can be used as long as its capabilities and limitations are consistent with study analysis needs. Software not on the approved list must be justified and vetted following procedures described in the software validation standard. Documents are available on the internal HH&C CoP SharePoint.

(3) Risk and Uncertainty. Risk and uncertainty principles should be applied within the staged study approach. A risk-based approach to water resources planning captures and quantifies the extent of risk and uncertainty in the various planning and design components of an investment project. Within USACE, risk-based analysis must be used to compare plans in terms of the likelihood and variability of their physical performance, economic success, and residual risks. Sediment processes that affect project features should be included in USACE plan formulation and developing the risk register. Chapter 10 of this manual discusses risk and uncertainty parameters analysis methods.

2-3. Developing Sediment Investigations for USACE Projects. The broad range of USACE project applications creates many challenges to develop high-quality and technically sufficient sediment investigations.

a. Available Study Approaches.

(1) Similar to hydraulic studies, each sediment study has specific requirements. However, sediment studies do share many similarities from project to project. Therefore, while individual studies may vary considerably, the basic approaches are similar. The type of approach depends on several variables including:

(a) Purpose of the study; questions that need answers.

(b) Physical setting.

(c) Confidence required in result.

(d) Data available for the study.

(2) The purpose may simply be to determine if a sediment problem does or does not exist within the study area. On the other hand, the project might be quite complex, and the purpose of the sediment study is to calculate as accurately as possible the expected changes in the stream

bed and/or sediment discharge during the life of the project. These two extreme purposes require quite different study approaches.

b. Planning Studies. Sediment investigations are scoped and performed during the USACE planning study process.

(1) The USACE study planning process has evolved considerably with current practice as described in the ER 1105-2-100. Recent focus includes the SMART evaluation process as discussed in Chapter 1 of this manual. Incorporating the sediment study stages with the USACE planning process is a critical element for successful USACE project design, implementation, and operation. Assigning a set of sediment study tasks with an associated planning phase study title is not possible as study needs change for each project. While USACE planning study nomenclature may change, linking sediment study needs with the current planning study definition will facilitate the needed study products.

(2) Regardless of the planning process stage or level of detail, evaluating project impacts on river system morphology is typically determined by comparing the “future with project condition” to a “future without project condition.” The future without project condition is determined by forecasting the stream system without the proposed project, a “no-action alternative.” The future with project condition is made for a period equal to the project life. The future without project condition should be evaluated for the same period of time and should contain all future changes in land use, water yield, sediment yield, stream hydraulics, and basin hydrology, except those associated with the proposed project.

(3) In some cases, evaluation of project impacts should be extended beyond the USACE project planning economic period. For instance, a reservoir project may fill with sediment within the USACE project economic planning analysis period, which is often 50 years. However, the constructed reservoir continues to function and affect sediment processes well past the planning analysis period. Extending the sediment analysis period further into the future during project formulation may identify severe future consequences and lead to the inclusion of mitigating project features for sediment management. Evaluation of sediment processes for the entire project functional life is recommended when project function may appreciably change or when life safety risks could increase relative to the no-action alternative.

(4) Within the USACE study process, it is critical to accurately define costs for data collection, analysis, design, initial project construction, and operation and maintenance. This should include sediment-related study costs, constructed project features to mitigate sediment impacts, and identified operation and maintenance costs. Acquiring adequate data and performing analysis to develop accurate cost estimates can be challenging. The “staged sediment studies” previously presented and analysis components provided in Appendix B should be followed when developing the project staged sediment study scope. Critical sediment processes should be included in the risk register.

c. General Study Framework for USACE Projects.

(1) An explicit framework depicting the relationships between the general study stages and the sediment study process is not feasible. Water resource problems are typically complex and require a multi-disciplinary USACE team. While a general approach is given, this procedure must be tailored to each individual project.

(2) The evaluation and design of water resources projects should advance from a broad evaluation of the river characteristics and design principles to detailed computations and analyses. The evaluation should begin with a qualitative assessment of the river and watershed. As the analysis progresses, it becomes more detailed and, subsequently, more quantitative. At all stages of the investigation and design, qualitative evaluation is important to determine the interrelationships between all aspects of the proposed project and the existing river system. Figure 2-2 depicts the typical USACE study phases within the planning and implementation process. Integration of qualitative/quantitative sediment investigations throughout the life cycle planning process is achieved by following the staged sediment study process.

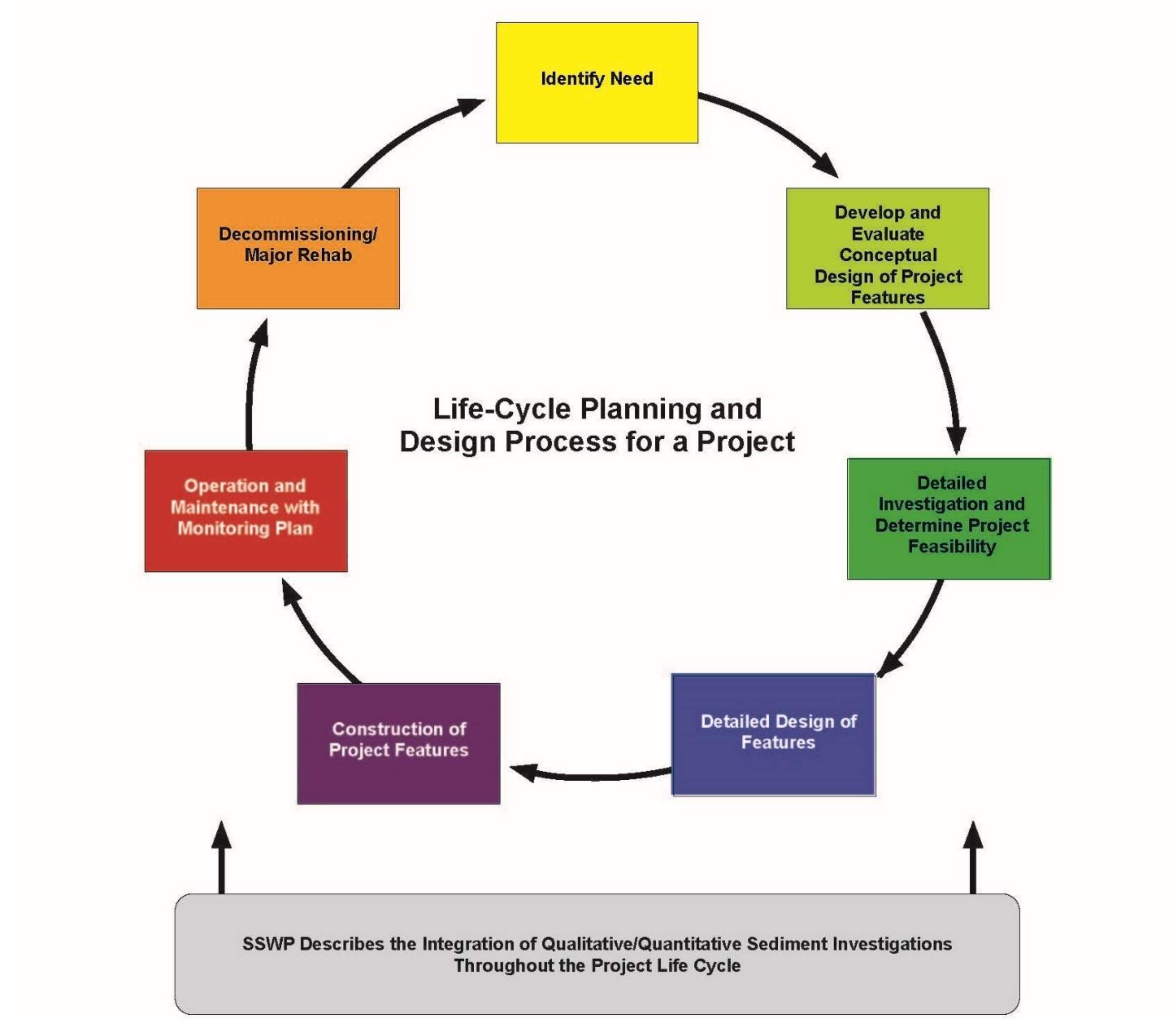


Figure 2-2. Life-cycle planning and design process

2-4. Sediment Investigation Levels. Variable levels of investigation are appropriate in the staged sediment studies that are performed for USACE water resource projects. More than one level of investigation often occurs in a single study phase. Employing investigation levels in the staged sediment study framework recognizes that studies are unique and different approaches are appropriate.

a. The investigation typically proceeds through a series of steps of increasing analysis complexity. The initial analysis should be used to identify the significant principles involved and adjusted/expanded as necessary to evaluate the system given those findings. If system behavior and response is less complex, then additional analysis may be scoped accordingly. More complex systems often require additional data collection in order to conduct more extensive quantitative analysis to reduce risk and uncertainty in the projected study results. Adapt the investigation level to fit study area specifics and include provisions to expand the techniques and methods as necessary to handle increasing complexities.

b. Figure 2-3 schematically shows the levels of investigation and the relationship to typical USACE study phases. Within the framework of the staged sediment study, the variable level of investigation provides for feedback loops to ensure that the interdependence of variables is continually adjusted. A qualitative review of findings at each level is recommended to provide a check to ensure that fundamental principles have been followed. Table 2-3 lists each level of investigation, and further explains the associated typical study processes.

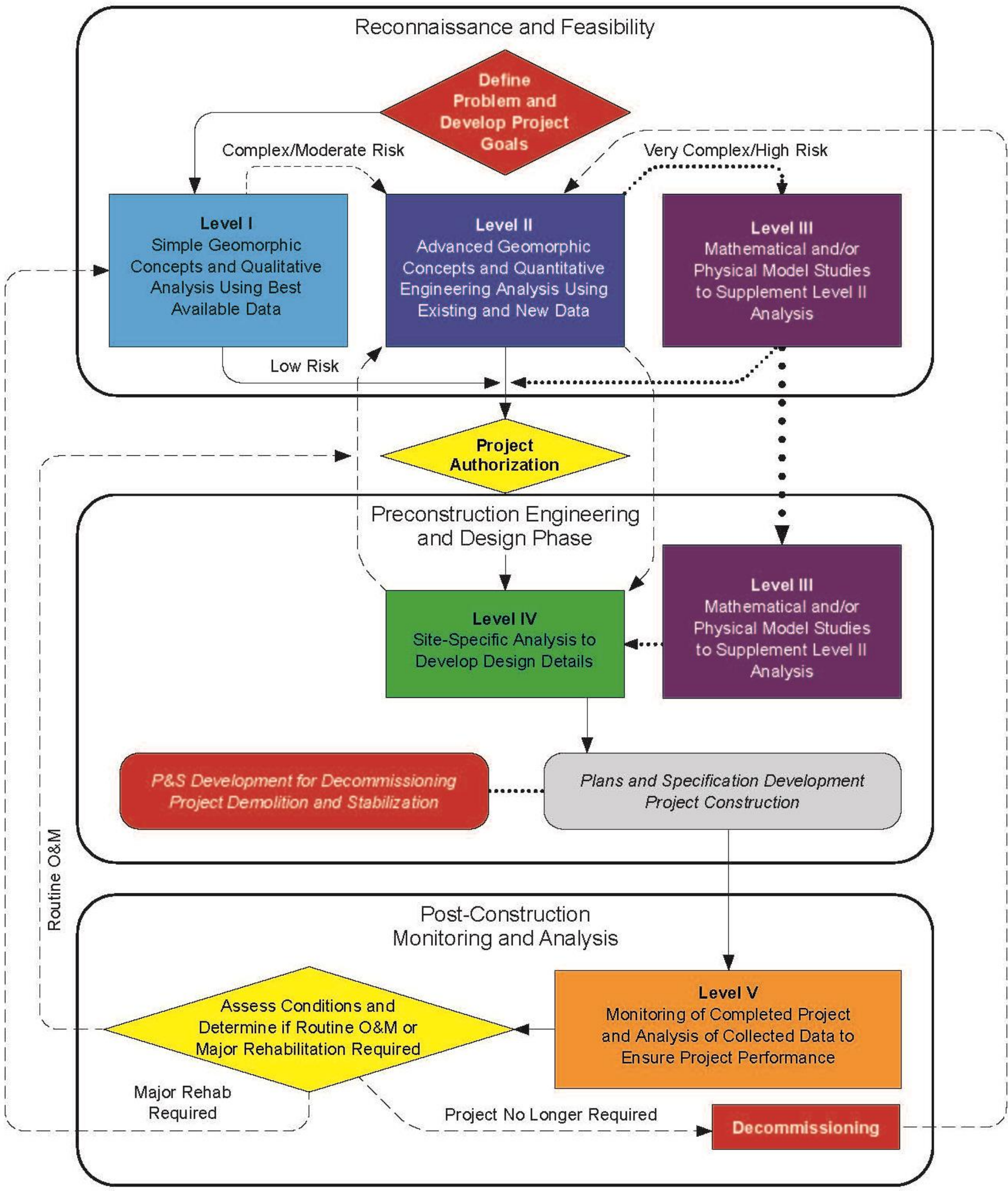


Figure 2-3. Sediment study levels of investigation schematic

Table 2-3
Typical Sediment Study Levels and Components

Study Level/Phase	Typical Sediment Study Components
Level I Recon/Feasibility	Analysis consists of (1) defining the problem; (2) identifying project goals; (3) developing potential solutions to achieve those goals; (4) identifying opportunities and constraints associated with each solution; (5) assembling available data and previous studies; and (6 performing a qualitative assessment. Level I qualitative analysis includes a basic site characterization (assemble previous studies, available data, review aerial photos, perform a site visit, develop an approximate system sediment budget) that results in a conceptual model of the study area with identified responses to various project components. Level I includes basic sediment impact assessments. A potential outcome of Level I is an escalation to Level II for very complex projects. Since sediment data collection is often a long-term process, an outcome of Level I should include identifying data collection efforts and next steps for the more complex Level II and III studies.
Level II Recon/Feasibility	Analysis involves more detailed qualitative analysis (than Level I) combined with a quantitative evaluation. The geomorphic assessment and sediment budget are typically comprehensive as study goals require. Basic data such as grain size distributions or suspended sediment concentration grab samples are sometimes collected as part of Level II investigations. Geographic information system (GIS) analysis of historical trends and computation of water surface profiles and basic sediment transport modeling can be included in this level. Rigid boundary steady and unsteady flow numerical modeling in one and two dimensions are often routinely conducted at this level. The evaluation and analysis can be considered adequate at this level if the goals are met, the interrelationship between different aspects of the project and river system are adequately explained, underlying critical sediment investigation assumptions are confirmed, and all identified analysis problems are sufficiently understood and addressed.
Level III Recon/Feasibility/ PED	Analysis involves more complex numerical models for hydraulics, hydrology, and sediment. Modeling products often address decadal trends or highly complex local phenomena. Level III analysis is necessary when risks are high and/or complexity is such that Level II results are not adequate to define relationships and system response. This level includes advanced multidimensional hydrodynamic (two- and three-dimensional), movable boundary (sediment routing, morphodynamic), and multi-phase (fresh/saltwater) models. In some situations, physical models are required to evaluate highly complex phenomena. This level of analysis often requires historical data with substantial effort to calibrate and verify the models. Scoping and implementation of a formal sediment data collection effort (establishment of new sampling stations, development of rating curves) in support of model calibration would also be considered a level III analysis, however the collection and analysis of the data post-project would be a Level V analysis.
Level IV PED	Conducted in the PED phase, analysis includes a focus on unique aspects that are specific to individual features or measures, to refine more general results obtained from Levels I, II, and III analyses, in support of detailed design efforts. These procedures are not always necessary. However, do not defer significant cost/implementation issues until Level IV. These issues should be resolved during previous study phases.

Study Level/Phase	Typical Sediment Study Components
Level V Decoma/O&M	Analysis is not part of initial project evaluation or design, but in an integral part of the O&M process following construction. This includes ongoing sediment and stream gaging, cross-section surveys, reservoir sediment ranges, and similar field data collection. A planned monitoring program is an integral part of every project; it is not optional. An essential part of this analysis is periodic evaluation and assessment of the data and a comparison with expected project behavior. The objective of this assessment is to make recommendations for subsequent O&M activities or for major project rehab. If the project shows signs of unintended morphological impacts, initiation of Level I–IV analyses may be needed. Data collection should intensify to provide the necessary information (repeated cross sections) to improve the Level I–IV analyses if the process Figure 2-2 cycles to reduce O&M or for a major redesign.

c. The sediment study development may also be illustrated with a flowchart that highlights the typical decision-making required between the different levels of investigation and also how these levels often occur through the study phases (Figure 2-4). Development of study-specific elements and considerations in determining the appropriate study level of detail is required to tailor the flowchart to specific needs.

d. Example Application. USACE guidance documents provide a range of approaches to select the appropriate detail for staged sediment studies. An example application of staged sediment study described for a channel stability evaluation is available in TR-1-28, Hydraulic Design of Stream Restoration Projects (Copeland et al., 2001). Three levels for the channel stability evaluation are given that range from empirical reconnaissance-level methods to more process-based analytical techniques. The appropriate level of detail depends on study status, the perceived magnitude of problems, the project scale, and the available resources. Figure 2-5 shows the study levels presented in TR-1-28 as adapted for staged sediment studies.

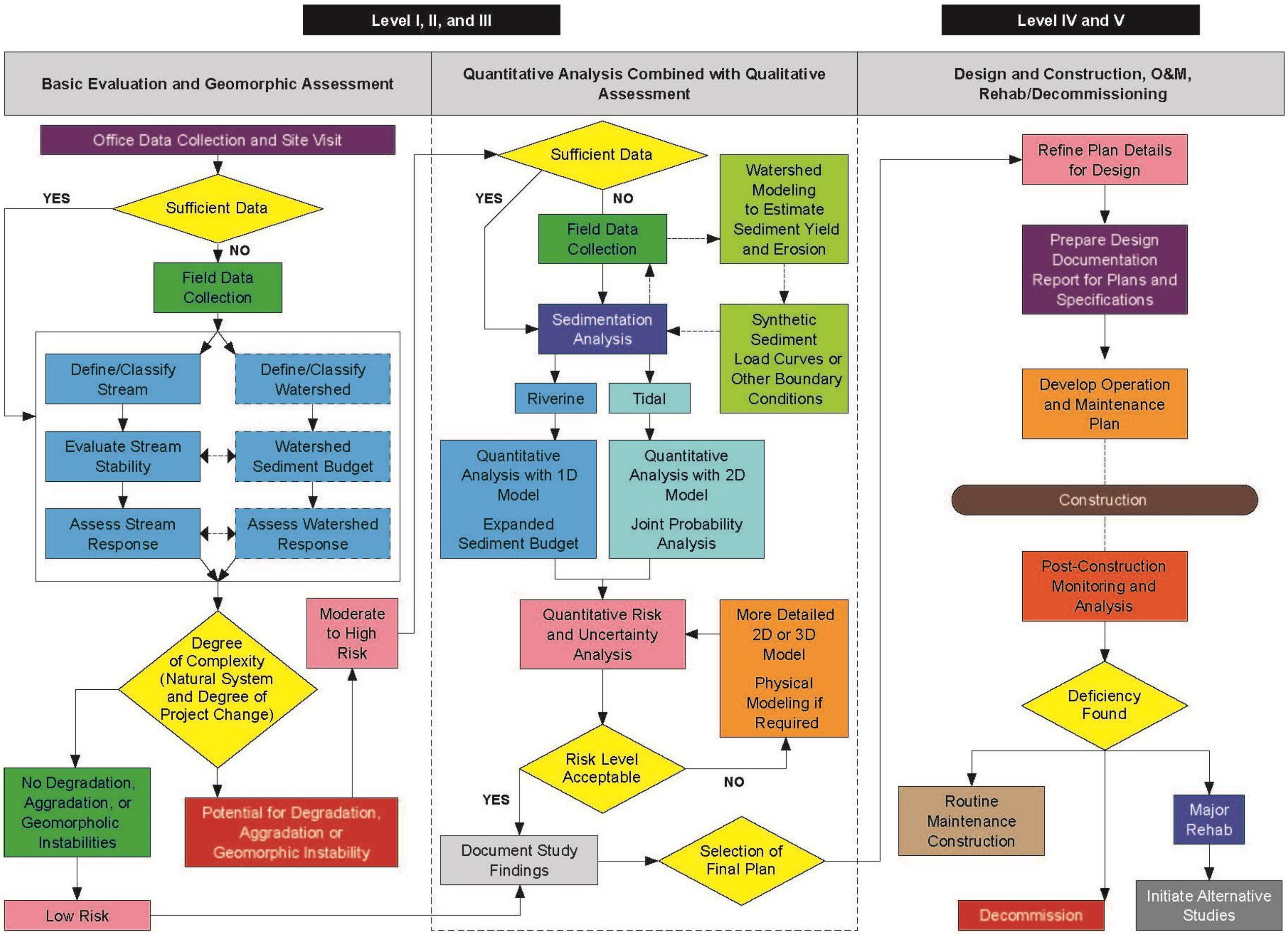


Figure 2-4. Flowchart of decision making with variable sediment studies levels

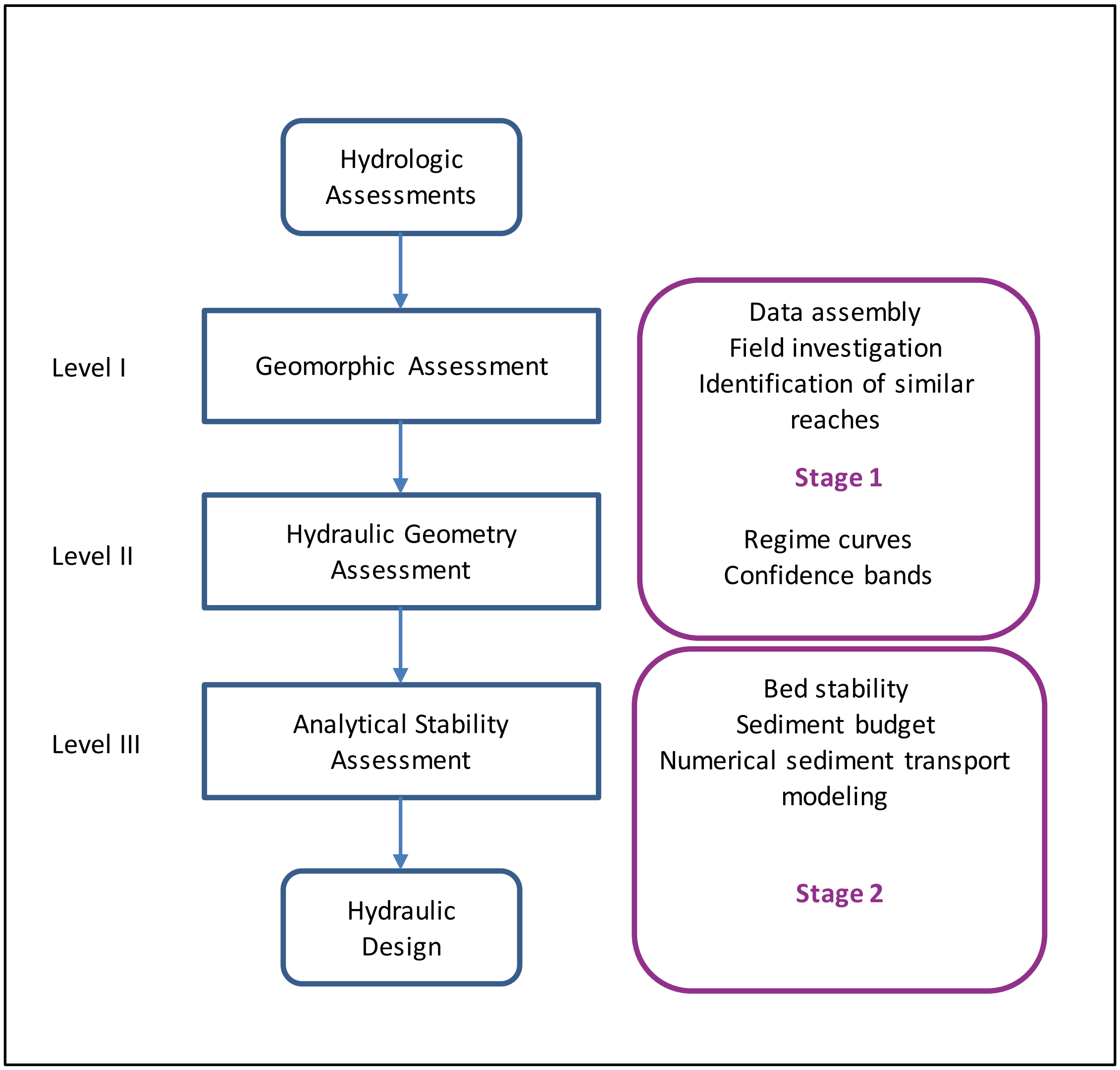


Figure 2-5. Example stream restoration study assessment levels (redrawn from Copeland et al., 2001)

2-5. The Sediment Studies Work Plan. The Sediment Studies Work Plan (SSWP) is the overall linking document that defines the planned sediment investigations and level of detail throughout the entire design and evaluation process for a study. Within the USACE study process, it is critical to accurately define costs for data collection, analysis, design, initial project construction, and operation and maintenance. Sedimentation processes (such as aggradation and degradation) can significantly reduce the project level of protection, jeopardize stability during extreme events, and dramatically increase local sponsor maintenance costs during the project life.

a. Purpose. The purpose of the SSWP is to adequately scope the development of a comprehensive work plan to identify and evaluate sediment analysis needs and problems for USACE projects and to track progress throughout the investigation. The SSWP specifies the study type and required funding. The SSWP should ensure that the study will identify the significant sediment problems and produce a satisfactory analysis of alternatives. The SSWP provides the overall linking document of the various staged sediment studies that will be used throughout the project planning process to:

(1) Document the type and scope of the sedimentation studies identified for evaluation and funding needs.

(2) Produce the overall linking document for the various staged sediment studies within the USACE study planning process.

(3) Provide method(s) to complete study in a timely and efficient manner.

(4) Ensure that study results identify the significant sediment problems, that the level of detail is appropriate, and that the products meet study needs at each study phase.

(5) Ensure that the study will produce a satisfactory analysis of alternatives and sediment processes.

(6) Ensure that technical procedures and products are acceptable to internal and external customers.

b. SSWP Development Overview.

(1) Development of the SSWP will require critical thinking to align the staged sediment study level of detail, including analysis method and data collection, with the project planning study phases. A significant hurdle to developing the SSWP is accurately defining overall study scope, the geographical scale of the affected environment, necessary results accuracy, and the anticipated alternatives. The SSWP should coordinate the sedimentation study needs with those of other disciplines involved in the study process during all phases. An example SSWP is provided in Case Study 2A (Appendix N).

(2) The developed SSWP will establish the sediment study related information and analysis methods, following the staged sediment study approach, that are needed at each stage of the project evaluation within the USACE study planning process. The three main components of the SSWP are to:

(a) Include a plan to collect existing data from both internal and external sources.

(b) Identify the new data requirements required for each level of analysis and the new data acquisition strategy.

(c) Verify that data requirements meet the needs of selected analysis methods for all phases of the staged sediment studies.

(3) Data collection activities and laboratory testing should be scheduled to verify compliance with the overall project schedule. Because data collection and laboratory tests are prerequisites for analysis and modeling, they should be scheduled as early in the project formulation process as possible. Accurately establishing study boundaries is a critical step to determine study methodology and scope. Although initially developed for a specific study need, development of the SSWP should consider how the results of both analyses may impact future analysis needs, data, and study costs.

c. Factors to Consider in SSWP Development.

(1) The potential for project features to impact existing sediment processes should be considered. Evaluate the potential project impacts on the stream balance between water and sediment when developing study methodology using the principles of Lane's balance (paragraph 1-8d(2)). For instance, alternatives that modify the channel cross section, or change stream alignment, or alter the normal or bankfull discharge frequency by water diversions or storage will usually increase both the magnitude and occurrence of sedimentation problems. These possibilities should be reflected in the SSWP.

(2) Factors that will likely result in SSWP modification should also be considered during development. If the SSWP is complex, or if sediment problems are sufficiently large, then the SSWP will likely require modification/updating during the study process. Discovering new issues that arise during study execution or when study objectives change are additional factors that could lead to the need for significant SSWP modification.

(3) Study analysis time and cost estimates in the SSWP can be affected by multiple factors. Use site-specific and general system knowledge to forecast problems and study outcomes during the various study stages. Include an appropriate contingency to reflect study complexity and potential for scope growth. The SSWP cost estimate should be a reasonable balance between attempting to account for every possible problem vs. assuming no unforeseen issues. Include interim study products and future decision points in the SSWP to evaluate if revisions are needed. This process is especially important for complex sedimentation analyses. The SSWP should recognize that early identification of sediment issues is preferred to reduce the risk of identifying issues in future study stages that causes significant lost effort, study cost increase, and schedule delays.

(4) Figure 2-6 shows the primary components and decision processes involved in developing the SSWP.

Development of the Sediment Studies Work Plan (SSWP)

Scope of the SSWP:
Develop a comprehensive scope to identify and evaluate all stages of sediment analysis needs and problems.

1. Document the study type, identify project scope, and determine funding needs.
2. Utilize the staged sediment study approach to develop the scope.
3. Include a study schedule that completes all studies in a timely and efficient manner.
4. Include alternative analysis within the study scope.
5. Specify products for each study phase.
6. Include appropriate quality control reviews throughout the study.

SSWP Technical Considerations Development:
Scope study analysis methods to identify sediment problems at appropriate detail and use acceptable technical procedures.

1. Assemble existing sediment process information, consult with external agencies, and conduct site visit(s).
2. Coordinate sediment study technical analysis with other USACE team members.
3. Align sediment studies with project goals.
4. Define sediment analysis and models at each stage (mathematical and/or physical).
5. For each study stage, assemble existing data and identify any new data collections that will be required including schedule, collection agency/method, and lab testing.
6. Evaluate sediment process impacts and determine O&M costs for sustainability of project features.
7. Identify long-term monitoring needs for future study evaluation and O&M.

Factors to Consider and Reflect in the SSWP:
The SSWP should reflect pre-project stability issues and the likely response to project features (such as extensive modifications to channel geometry, water storage, or similar).

Reasons to Modify SSWP During Study
a. Study plan is complex.
b. Sediment problems are large.
c. Sedimentation studies uncover new issues.
d. Planning study objectives change.

Estimating Study Time and Cost
a. Required in all sediment study stages.
b. Forecast problems/outcomes to define further study needs.
c. Include an appropriate contingency for study change risk.
d. Include interim study products and SSWP review points.
e. Develop study schedule and products to promote the early identification of sediment issues.

SSWP Usage and Application:
1. Followed by the Project Engineer to properly conduct the investigation and develop study products.
2. Used as the basis for contracting scope(s) of work with outside entities to perform study tasks.
3. Used by Project Managers to estimate costs, schedule work, and monitor progress.
4. Developed for use at the District level and signed by the Branch Chief, the SSWP may require Division/HQ review for complex projects.

Figure 2-6. Development of the sediment studies work plan

d. Usage. The project engineer follows the sequence of tasks laid out in the SSWP to properly conduct the investigation and develop task products. The SSWP may also be used as the basis for developing contracting scopes and performing negotiations with outside entities that may be conducting parts of the investigation. The project managers use the SSWP as a basis for estimating costs, scheduling work, and monitoring progress.

e. Process. The SSWP development process is a complex procedure that should be performed in a deliberate manner. Information in the previous sections for SSWP purpose, development overview, factors to consider, usage, and application guidance should all be reviewed during development. A detailed discussion of the process is provided in Appendix B. In many cases, the SSWP may be a collection of scopes for the individual study phases. Example scopes of work for typical projects are included in Appendix C.

f. SSWP and Study Conclusions. The primary purpose of the SSWP is to support study objectives by defining the components and tasks needed to adequately address sedimentation concerns. Sediment studies undertaken according to the SSWP will provide results that can be clearly summarized with definitive study conclusions. Vague study conclusions such as "results are approximate because of funding or data limitations," or "more study is recommended," should not be an outcome with a properly developed SSWP. A well-formulated SSWP will have identified limitations and resolved them during the various study stages such that the study can achieve reliable conclusions.

g. Application. The SSWP will be drafted and used at the District level. However, projects of unusual scope or complexity may require coordination between study stakeholders and representatives of the District, Division, and Chief of Engineers (HQUSACE) to arrive at acceptable criteria and technical procedures. The developed SSWP should be signed at the USACE Branch Chief level.

2-6. Post-Construction Operations and Maintenance and Reporting.

a. Monitoring and reporting requirements for sedimentation should be included in the O&M manuals currently developed for all projects. Monitoring survey extent and type (for example, light detection and ranging (LiDAR), hydrographic, acoustic Doppler current profilers (ADCP)) should be clearly defined based on current methodologies and standard practice. Monitoring methods will require revision as new technologies emerge and best engineering practices change. The location of sedimentation ranges upstream, downstream, and within the project limits should be displayed. Time periods for periodic resurveys should be specified.

b. Acceptable methods and procedures defined in operation and maintenance manuals should include provisions to use future technological advancement in survey data acquisition and processing techniques. As with any use of new technology, appropriate means should be in place to ensure that interpretation of results are comparable with previous monitoring efforts.

c. Any description of monitoring data evaluation methods should include actionable outcome. For instance, if aggradation is expected, then the specific deposition level, or an evaluation process for complex projects at which sediment removal is required, should be stated.

Guidance for dredging intervals for flood control channels should be given. Specific guidance for different project areas and constructed features should be included. Care of vegetation should be described relative to erosion, deposition, and hydraulic roughness.

2-7. Communicating Sedimentation Activities to Stakeholders.

a. The current USACE reporting directive on sedimentation activities described in this section replaces the former practice described in ER 1110-2-4001. Historically, following the format in ER 1110-2-4001, USACE sediment activity reports were combined with data from the other federal agencies and published annually by the Subcommittee on Sedimentation of the Interagency Advisory Committee on Water Data in a publication entitled, “Notes on Sedimentation Activities.” Publication of these combined reports was discontinued in the 1990s.

b. However, USACE has a continuing need for reporting sedimentation activities, including data gathered, studies performed, and relevant research activities. USACE reporting objectives are to provide a clear and concise summary of sedimentation activities to inform other offices of interesting efforts, to stimulate the exchange of information, and to inform management of ongoing activities.

c. The report on USACE activities will be prepared annually by offices and laboratories having Civil Works responsibilities using the following guidelines:

(1) Purpose and Project Location Map. The report should have a purpose statement for each reporting office that outlines responsibilities. Include a project location map.

(2) Sediment Surveys. Information will be prepared regarding sedimentation activity-related surveys of reservoirs, channels, harbors, etc., conducted during the preceding calendar year. In general, the information will describe the following items where applicable: survey purpose (repetitive, single project); type of survey (reconnaissance, range, LiDAR, topographic, etc.); elements measured (depth/elevation, densities, sediment characteristics, etc.); equipment used; and survey scope (complete survey of reservoir or channel, survey of selected ranges, or measurements of deposit depths at selected locations for index information, etc.). Include current status with anticipated date of completion, when applicable.

(3) Sediment Sampling. Information regarding a sediment sampling program in a river basin will be reported. The total number of sediment stations and the number of stations added or dropped from the program during the year should be indicated. The types of information that should be included for the major stations are: location, name of stream, sampling agency, sampling frequency, sampling equipment, type of samples obtained, etc. Other general information about a river basin, such as cooperating agencies, may be included.

(4) Sediment Investigations. Summary descriptions will also be reported regarding sediment investigations that are considered to be of general interest in the sedimentation field. Typical items to be included are a brief resume of current office and laboratory research investigations or any ongoing sediment studies contracted to universities and architect-engineer firms; new equipment; field methods and measuring techniques; important correlations

developed; annotations of important publications and reports not generally publicized; and other pertinent items.

(5) Format. The report should be provided in electronic format. The “Notes on Sedimentation Activities” will be available for public distribution. Therefore, discussions of internal information such as personnel and budgetary matters should be excluded. Case Study 2B (Appendix N) provides an example of a sedimentation activities report.

(6) Reporting Method and Submission Date. Each USACE reporting office should submit an electronic report to the relevant Division. A report of no activities should also be submitted via email when appropriate. At the District level, the Hydrologic Engineering Branch Chief or designated representative is responsible for reporting District sedimentation activities to their Division HH&C CoP lead. Division and research lab offices will provide all reports to the HH&C CoP team leader. In addition, reports will be posted electronically to the USACE central repository. Reports should be provided no later than February 15 for the annual report period ending the previous December 31.

Chapter 3
Sediment Properties

3-1. General.

a. The rate at which sediment is delivered to a reservoir and the resulting loss of storage volume is directly related to the properties of the sediment in transport and storage. The practicing engineer needs to understand how sediment properties affect the performance of the reservoir. This chapter covers the properties of sediment particles and mixtures. Sediment erosion, entrainment, transport, and deposition depend on both the characteristics of the sediment and flow hydraulics.

b. Bulk sediment properties are derived from the properties of individual particles and the relationships among the particles in the mixture. The properties used to characterize individual particles include size, shape, mineralogy, fall velocity, mineral density (or specific gravity), and critical shear stress (in noncohesive sediments). Properties used to characterize sediment mixtures include size distribution (or gradation), bulk density, porosity, critical shear stress (in cohesive sediment mixtures), and erodibility.

c. Many of these properties are highly correlated. Information on the specifics of determining sediment properties is available in most soils textbooks and the discussion below is presented in summary form only. Additional properties define cohesive and organic sediments, including cation exchange capacity and percent of organic content.

d. The information in this chapter is supplemented by information in several other chapters:

(1) Chapter 4 covers sampling techniques and particle size analyses.

(2) Chapter 5 covers additional properties of fine-grained sediments (inorganic and organic).

(3) Chapter 8 covers properties of sediment deposits in reservoirs, such as bulk density and compaction/consolidation.

e. Figure 3-1 illustrates the content of this chapter.

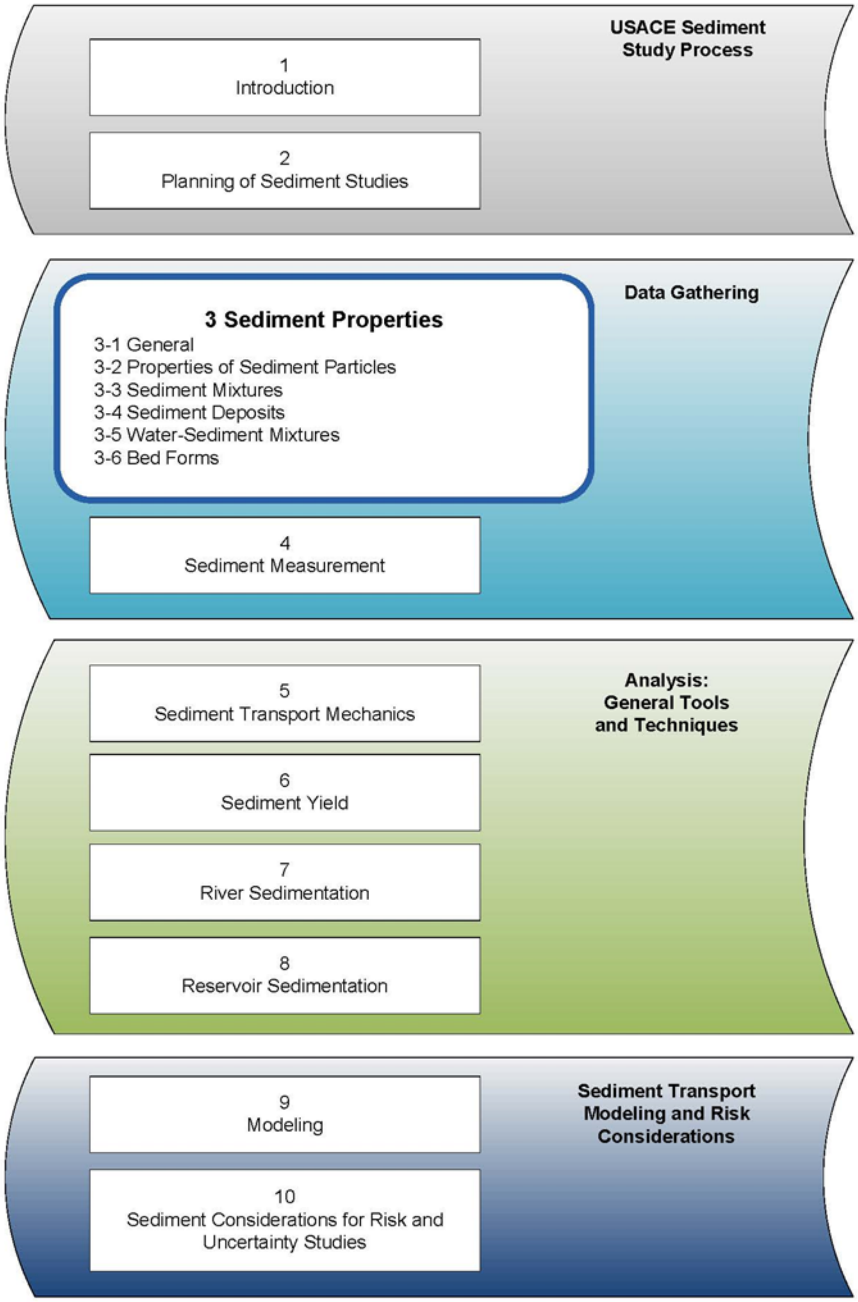


Figure 3-1. Chapter 3 content and general document structure

3-2. Physical Properties of Sediment Particles.

a. Sediment. Sediment refers to the collection of material fragments, both organic and inorganic, that are transported and deposited by the actions of water, wind, or ice. While the term is often used to indicate soil-based mineral matter, decomposing organic substances and inorganic biogenic material are also considered sediments. Most mineral sediment comes from the mechanical weathering of rocks, while organic sediment is typically derived from the chemical secretions of organisms and decomposing material such as algae. The following discusses the most common metrics to characterize sediment particles. Refer to Chapter 4 of this manual for information on sediment measurement techniques including sampling and analysis.

b. Particle Size.

(1) Particle size is one of the most important sediment properties because it varies more than other properties (over seven orders of magnitude from boulders to individual grains of clay), is relatively easy to measure, and exerts control on other parameters such as fall velocity, critical shear stress, porosity, bulk density, and others.

(2) Sediment particle size is typically expressed as a diameter, which is an estimate; because sediment particles are never exactly spherical, their diameters are defined according to the method used to measure it. Diameter can be described as:

(a) Sieve diameter is the length of the side of a square sieve opening (mesh) through which a particle will just pass.

(b) Nominal diameter is the diameter of a sphere having the same volume as the particle.

(c) Sedimentation diameter is the diameter of a sphere, which, in the same fluid, has the same terminal settling velocity as the particle. This is often used to report the diameter of silts and clays since they are not easily sampled otherwise.

(d) Standard fall diameter (or simply fall diameter) is the diameter of a sphere that has a specific gravity of 2.65 and has the same terminal settling velocity as the given particle in quiescent distilled water at a temperature of 24 °C.

(e) Triaxial diameter is the arithmetic average of the three perpendicular axes. The analyses of particle sizes and particle shape parameters are based on the length of three mutually perpendicular particle axes: the longest (a-axis), the intermediate (b-axis), and the shortest (c-axis) axis (Bunte and Abt 2001). The practical application to individual particles is not precise. Differences in visual interpretation and variations in measuring techniques are common (Figure 3-2). Gravel and cobble field measurement relies on the b-axis as discussed in Appendix E.

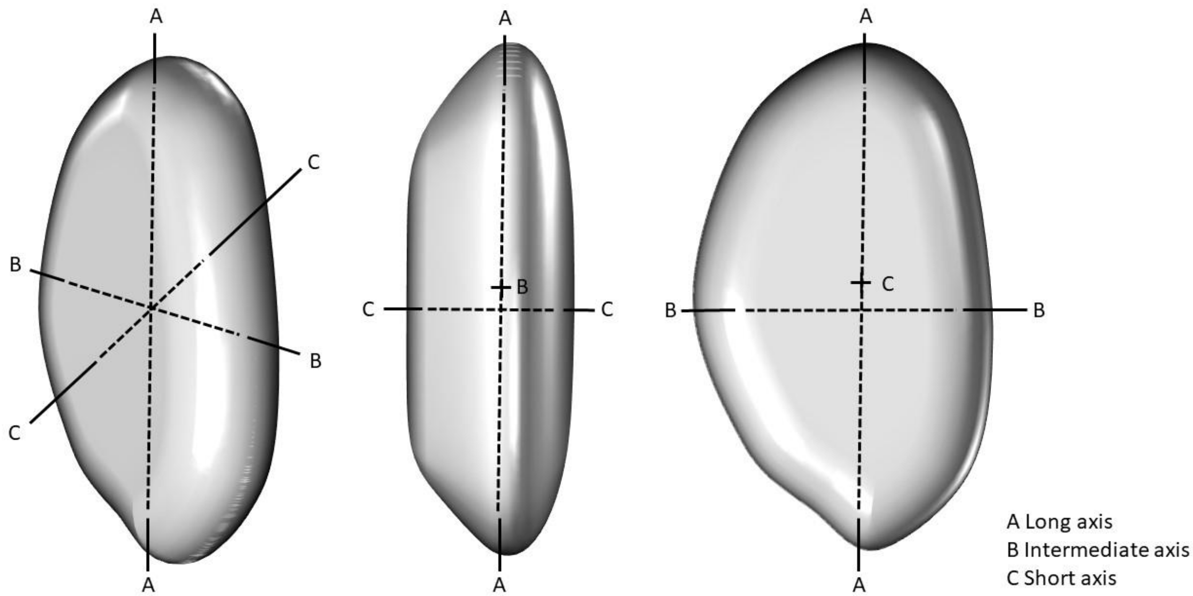


Figure 3-2. Three perpendicular axes

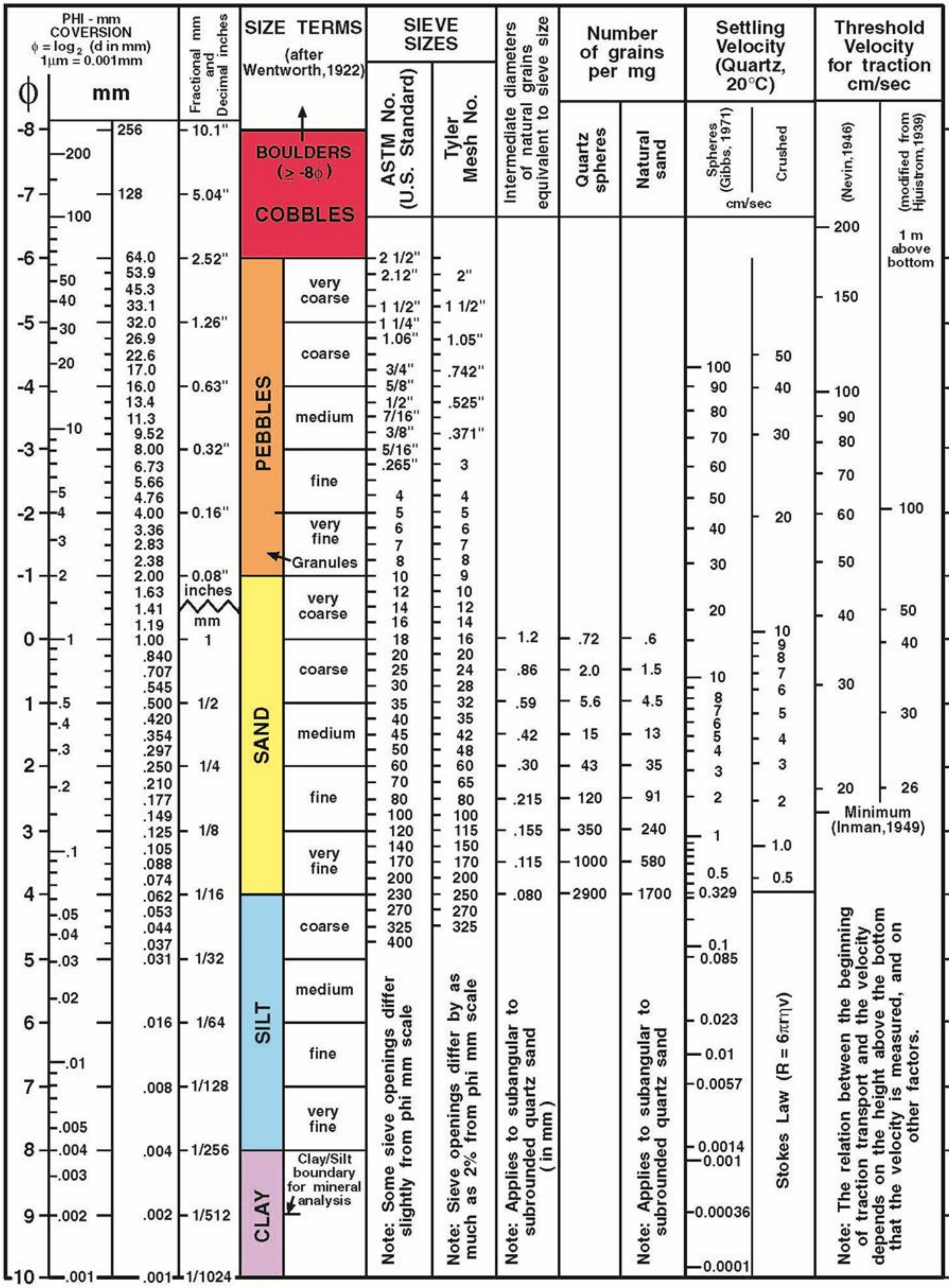


Figure 3-3. Grain size classification chart (from Williams et al., 2006)

c. Sediment Size Classification Systems.

(1) Sediment particles are classified, based on their size, into six general categories: clay, silt, sand, gravel, cobbles, and boulders.

(a) Classifications are essentially arbitrary, and many grading systems are found in the engineering and geologic literature. The most widely used system for sediment transport computations is the American Geophysical Union (AGU) classification system, which arranges sizes in a geometric series with a ratio of 2. This grade scale was originally recommended by the Subcommittee on Sediment Terminology of the AGU (Lane 1947) and has been widely adopted for sediment work.

(b) Table 3-1 presents the AGU size classifications. Note the geometric mean for each size class is based on the actual geometric series, not the Table 3-1 rounded upper and lower bound values. The remainder of this document uses the AGU size classes unless otherwise indicated.

(2) A variety of terms and classification systems have been used to describe the size characteristics of sediments in the past. Overall, the term “coarse sediment” implies sands and gravels, while fine sediment refers to silts and clays. However, the size fractions of gravel, sand, silt, and clay are further broken down into sub-classes of very fine, fine, medium, coarse, and very coarse, and are also identified by texture and plasticity.

(3) It is important to confirm the classification system that has been used for grain size data, since reporting differs slightly between systems and different size boundaries may be used for the break points between classes. Numerous grade scales have been developed to establish the limits of size for each of these classifications. Table 3-2 shows some of the commonly used grade scales for comparison. Note that the range in size for a particular particle class may differ from one classification system to another (NRCS 2012).

(4) Grain size can be as large as several meters for boulders or as small as $1x10^{-6}$ meters for clays. Because of this wide variation, grain sizes in geological studies are often reported using the phi (Φ) classification scale (Krumbein 1934). The phi scale is a geometric grade scale in which phi is expressed in terms of the base 2 logarithm of grain diameter (in mm). A phi of 0 equates to 1 mm, and positive phi values are smaller than 1 mm, whereas negative phi values are larger than 1 mm. A change of 1 phi equates to a doubling (or halving) of particle diameter. Figure 3-3 above shows a grain size classification chart that encompasses many of the typical sediment properties, which the following sections discuss in detail.

(5) While the classifications are based, in part, on particle size distribution, size is often predictive of mineralogy as well. Silt and sand particles are typically composed of quartz. Clay-sized particles have distinct mineralogy and are typically composed of montmorillonite, illite, or kaolinite. Clay particles exhibit varying levels of cohesive behavior that must be analyzed with appropriate methods (see paragraph 3-3h).

Table 3-1
American Geophysical Union (AGU) Sediment Size Classification System (adapted from Lane 1947)

Sediment	Size Range (mm)	Geometric Mean
Very large boulders	4,096–2,048	2,896 mm
Large boulders	1,024–2,048	1,488 mm
Medium boulders	1,024–512	724 mm
Small boulders	512–256	362 mm
Large cobbles	256–128	181 mm
Small cobbles	128–64	90.5 mm
Very coarse gravel	64–32	45.3 mm
Coarse gravel	32–16	22.6 mm
Medium gravel	16–8	11.3 mm
Fine gravel	8–4	5.66 mm
Very fine gravel	4–2	2.83 mm
Very coarse sand	2.0–1.0	1.41 mm
Coarse sand	1.0–0.5	707 µm
Medium sand	0.5–0.25	354 µm
Fine sand	0.25–0.125	177 µm
Very fine sand	0.125–0.0625	88.4 µm
Coarse silt	0.0625–0.031	44.1 µm
Medium silt	0.031–0.016	22.6 µm
Fine silt	0.016–0.008	11.0 µm
Very fine silt	0.008–0.004	5.52 µm
Coarse clay	0.004–0.002	2.76 µm
Medium clay	0.002–0.001	1.38 µm
Fine clay	0.0010–0.0005	0.691 µm
Very fine clay	0.0005–0.00024	0.345 µm

Table 3-2
Particle Gradation Scales for Earth Materials (from NRCS 2012)

inches	U.S. Standard Sieve No.	mm	Unified Soil Classification System[3]	AASHTO[4]	AGU[5]	USDA[6]	Udden-Wentworth[7]
		4,026					
		2,048	boulders		boulders		
		1,024				boulders	boulders
		512		boulders			
12		300					
10		256	cobbles		cobbles		
6		128				cobbles	cobbles
3		75					
		64					
		32	coarse gravel	coarse gravel			
1		25.4			coarse gravel		
0.75		19					
		16					pebble gravel
0.5		12.7	fine gravel	fine gravel	medium gravel	gravel	
0.375		9.5					
		8					
0.25		6.35			fine gravel		
	4	4.76					
		4	coarse sand				granule
	10	2		coarse sand			
		1	medium sand		coarse sand	coarse sand	coarse sand
		0.5					
	40	0.425			medium sand	medium sand	medium sand
		0.25	fine sand	fine sand	fine sand	fine sand	fine sand
		0.125					
	200	0.074			very fine sand	very fine sand	very fine sand
		0.0625					
		0.05					
		0.031		silt			
		0.0156			silt		silt
		0.0078				silt	
		0.005	silt or clay				
		0.0039		clay			
		0.001			clay	clay	clay
				colloids			

[3] Unified Soil Classification System, ASTM D2487
[4] AASHTO, American Association of State Highway and Transportation Officers (AASHTO 1998)
[5] AGU, American Geophysical Union (Lane 1947)
[6] USDA textural classification system (USDA 1951)
[7] Udden-Wentworth classification system (Udden 1914; Wentworth 1922)

(6) Other disciplines (for example, geotechnical engineers and soil scientists) typically use different classification systems, such as the Unified Soils Classification System (USCS), American Association of State Highway and Transportation Officials (AASHTO), and International Standards Organization (ISO) 146880-1 grade scales. For reference purposes, Table 3-3 presents the USCS. Other nations use different grain size classifications. Be aware that numerous classification systems exist both in the United States and globally. Verifying the method used before using the data is required. Additionally, this classification system requires the determination of the Atterberg limits, details of which can be found in geotechnical engineering texts.

(7) Note that GW, GP, SW, and SP soils are typically noncohesive. Refer to paragraph 3-3h for further definition of cohesive and noncohesive mixtures. Refer to paragraph 4-15 for additional information on sediment measurement techniques including sampling and the appropriate analysis methods for different material sizes.

Table 3-3
Unified Soils Classification System (revised from NRCS 2012)

<table>
<tr><th colspan="3">Major Divisions</th><th>Group Symbol</th><th>Group Name</th></tr>
<tr><td rowspan="8">Coarse-grained soils, more than 50% retained on No. 200 (0.075 mm) sieve</td><td rowspan="4">gravel, > 50% of coarse fraction retained on No. 4 (4.75 mm) sieve</td><td rowspan="2">clean gravel, <5% smaller than #200 Sieve</td><td>GW</td><td>Well-graded gravel, fine to coarse gravel</td></tr>
<tr><td>GP</td><td>poorly graded gravel</td></tr>
<tr><td rowspan="2">gravel with >12% fines</td><td>GM</td><td>silty gravel</td></tr>
<tr><td>GC</td><td>clayey gravel</td></tr>
<tr><td rowspan="4">sand, ≥ 50% of coarse fraction passes No. 4 sieve</td><td rowspan="2">clean sand</td><td>SW</td><td>Well-graded sand, fine to coarse sand</td></tr>
<tr><td>SP</td><td>poorly graded sand</td></tr>
<tr><td rowspan="2">sand with >12% fines</td><td>SM</td><td>silty sand</td></tr>
<tr><td>SC</td><td>clayey sand</td></tr>
<tr><td rowspan="6">Fine-grained soils, more than 50% passes No. 200 sieve</td><td rowspan="3">silt and clay, liquid limit < 50</td><td rowspan="2">inorganic</td><td>ML</td><td>silt</td></tr>
<tr><td>CL</td><td>clay</td></tr>
<tr><td>organic</td><td>OL</td><td>organic silt, organic clay</td></tr>
<tr><td rowspan="3">silt and clay, liquid limit ≥ 50</td><td rowspan="2">inorganic</td><td>MH</td><td>silt of high plasticity, elastic silt</td></tr>
<tr><td>CH</td><td>clay of high plasticity, fat clay</td></tr>
<tr><td>organic</td><td>OH</td><td>organic clay, organic silt</td></tr>
<tr><td colspan="3">Highly organic soils</td><td>Pt</td><td>peat</td></tr>
</table>

d. Size Gradation Curves.

(1) The variation in particle sizes in a sediment mixture is described with a gradation curve, which is the cumulative size frequency distribution showing particle size vs. accumulated percent finer by weight. It is common to refer to particle sizes according to their position on the gradation curve. For example: d_{50} is the median particle size; that is, 50% of the sample is finer by weight; d_{84} is one standard deviation larger than the geometric mean size and d_{16} is one standard deviation smaller than the geometric mean size.

(2) In the engineering community, a well-graded sediment refers to a distribution that contains particles across a wide range of size classes (referred to as poorly sorted in the geological community). Conversely, poorly graded (well-sorted) distributions have only a few size classes represented. Gap-graded distributions are often bimodal with a noticeable separation, or gap, between dominant size classes. These determinations are often performed visually. An example illustrating different distributions is shown in Figure 3-4.

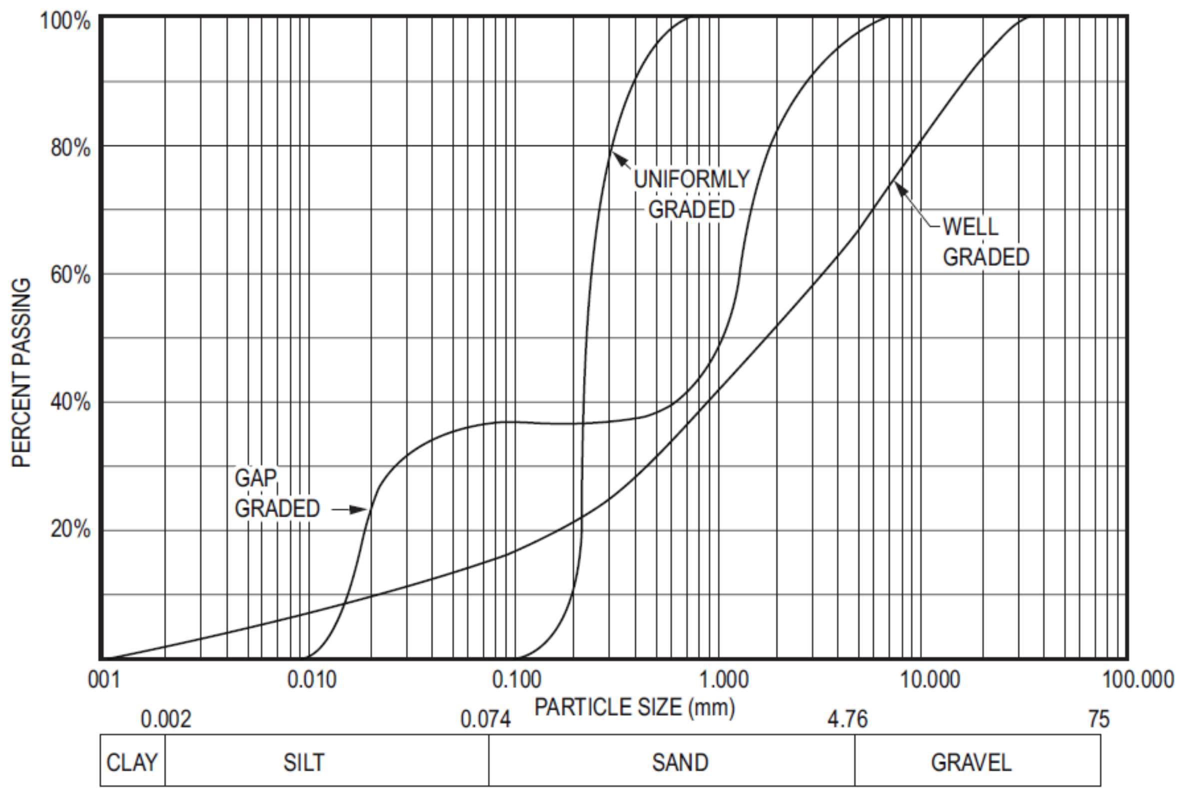


Figure 3-4. Example Gradation Curves

(3) Since natural river sediments often approach log-normal distributions, using a Gaussian (normal) probability function to describe the size distribution is usually not appropriate. Hence, gradation curves are plotted on semi-logarithmic paper, with the customary representation of a plot of percent finer vs. $\log_{10}$ (diameter), with size on the abscissa and percent

finer on the ordinate; the geometric mean and geometric standard deviation are used to describe the distribution. The graphic geometric mean (d_{gm}) of the entire sediment mixture is calculated from the grain sizes one standard deviation below (d_{16}) and above (d_{84}) the mean:

$$d_{gm} = \sqrt{d_{84} \cdot d_{16}} \qquad \text{Equation 3-1}$$

(4) It is common practice to use these definitions for mean sediment size and standard deviation in a mixture, even if the distribution is not log-normal (such as gap-graded). However, a more precise geometric mean can be computed from the entire distribution using the method of moments, paying close attention to the potential errors induced by the tails (Folk 1963):

$$\phi_m = \frac{1}{100}\sum_{i=1}^{k}(\phi_{ci} \cdot m_{\%i}) \qquad \text{Equation 3-2}$$

where:

ϕ_m is the geometric mean in the phi scale, and ϕ_{ci} and $m_{\%i}$ are the phi scale size and relative mass of each grain class *i*

(5) Sediment transport modeling typically computes transport values separately for each size class using the respective midpoint value of the size class boundaries (Table 3-1).

(6) Figure 3-5 shows an example grain size distribution curve, which illustrates the cumulative dry weight of the sample in each size fraction. The particle size is displayed horizontally, and the cumulative weight percent of grains smaller than the stated size is displayed along the vertical axis. For the Naches River, a coarse-bedded river in Washington State, suspended and bedload samples were collected, as were bed material samples (USGS 2008). In Figure 3-5, the two curves shown for suspended load and the two curves for bedload represent the variability between samples taken at different times for similar flows.

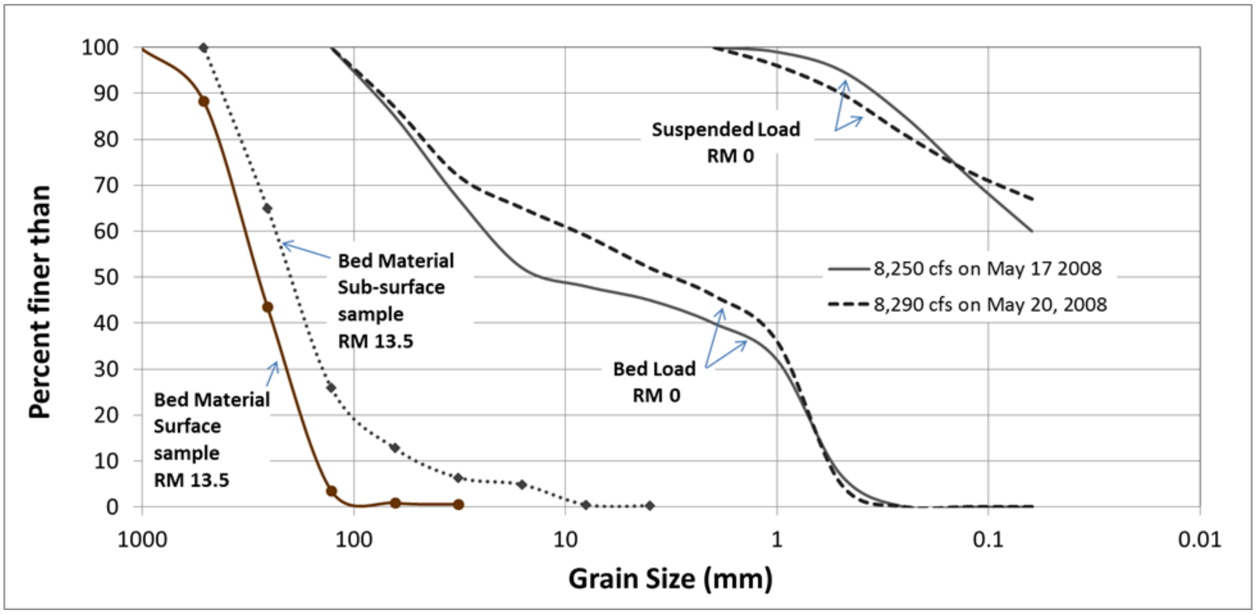


Figure 3-5. Sediment gradations for the Naches River, Washington; RM refers to the river mile above the junction of the Naches River with the Yakima River (redrawn from USACE 2015h)

(7) The finer distribution of the suspended load as compared to bedload is evident in Figure 3-5. Note that the suspended load in a stream tends to coarsen as discharge increases. In many streams, the bed material load constitutes a small fraction of the total load. In addition, gradations show the fining of sediment in downstream locations as compared to sediment samples taken further upstream. This is also shown in Figure 3-5. Finally, the difference between a surface sample and a subsurface sample taken at the same location shows that surface armoring can be a factor in determining the characteristics of sediments available for transport in a channel or reservoir. Refer to paragraph 4-10 for further discussion on bedload measurement.

e. Particle Shape.

(1) Particle shape is another significant sediment property in natural sediments and can be defined by the shape factor, *SF*. A measure of how closely a particle resembles a sphere, it is computed from the longest (a), intermediate (b), and shortest (c) particle axes:

$$SF = \frac{\boldsymbol{c}}{\sqrt{(\boldsymbol{ab})}} \qquad \text{Equation 3-3}$$

where *a*, *b*, and *c* are the lengths of the longest axis, the intermediate axis, and the shortest axis, respectively (Figure 3-2).

(2) These axes are the mutually perpendicular axes of the particle. The shape factor for a sphere is 1.0. Natural sands typically have a shape factor of about 0.7. Particle shape affects the fall velocity and, hence, both the sedimentation diameter and fall diameter of particles. For more information on the effect of particle shape on fall velocity, see Vanoni (1975, 2006).

(3) Roundness, angularity, and sphericity are metrics that roughly describe how much weathering a particle has undergone. Many other shape indices exist that are available in literature.

(a) Roundness can be defined as the ratio of the average radius of the corners and edges of a particle to the radius of a circle inscribed in the maximum projected area of the particle.

(b) Angularity can be characterized by conducting a fracture count on particles that are large enough to examine or by conducting a compaction test on a soil sample. For sedimentation purposes, it is convenient to view angularity as the inverse of roundness. Particles with roundness values less than 0.25 are considered angular, and for values less than 0.17, particles are considered very angular. Coarse sediments in streambeds tend to be more angular in headwaters and less angular farther downstream. Angularity can also be calculated using photogrammetry techniques that automate the determination of the most angular edges.

(c) Sphericity is the ratio of the particle surface area to the area of a sphere with the same volume. Sphericity values range from near zero for flat particles to 1 for a perfect sphere with most sedimentary particles falling in the range of 0.3 to 0.9.

f. Particle Specific Gravity.

(1) Specific gravity, defined as the ratio of sediment particle density to the density of water, varies by mineralogy. Table 3-4 lists some typical specific gravities. As most sands and gravels are composed of quartz, a specific gravity of 2.65 is often assumed for mixtures of coarse-grained sediments. However, site-specific testing is needed when the parent material is not quartz, as is common in systems in mining regions or with sediments derived from volcanic rock.

Table 3-4
Specific Gravity for Different Rock Types and Physical Modeling Media
(values from ASCE Manual No. 110, Chapter 2 (Garcia 2008b))

Rock Type	Specific Gravity
Quartz	2.60–2.70
Limestone	2.60–2.80
Basalt	2.70–2.90
Magnetite	3.20–3.50
Bakelite	1.30–1.50
Coal	1.30–1.50
Ground walnut shells	1.30–1.40
PVC	1.14–1.25

(2) The submerged specific weight of a particle is calculated as the difference between the specific weights of the solid and the surrounding fluid.

g. Particle Fall Velocity.

(1) Fall velocity (also known as settling velocity) is the velocity at which the drag force equals the submerged weight of the particle. The standard fall velocity of a particle is the rate of fall that the particle would finally attain if falling alone in quiescent, distilled water of infinite extent and at a temperature of 24 °C. The settling of sediment grains in a fluid is a drag-dominated process that depends on grain size, shape, and particle density, as well as fluid characteristics. Groups of particles can fall at different speeds than the individual particles in the group. Fall velocity tests can be used to characterize the entire distribution; however, this method is often applied only to the silt and clay fraction.

(2) Small particles such as clays can be flattened or plate-like instead of rounded, and are subject to flocculation, which strongly affects their settling rate. Clay fall velocity is best characterized by sedimentation diameter measured using the same water condition as in the natural condition so that flocculation is correctly represented. To determine the relative amount of clay and silt in a sample, however, testing usually requires the use of deflocculants and de-ionized water, so two different sets of tests may be needed for clay samples, one for settling velocity and one for relative size fraction. The differences in the settling behavior between these two material states is an important consideration for sediment transport calculations (Perkey et al., 2020).

(3) A particle will remain in suspension as long as the vertical components of the bed level turbulence exceed that of the fall velocity. Therefore, the determination of suspended-sediment transport relies heavily on the particle fall velocity. Fall velocity is one of the main parameters that determine the volume and size of the material that becomes trapped in a reservoir, as well as the pattern of sedimentation within the reservoir.

(4) Fall velocity is a fundamental property governing the motion of the sediment particle in a fluid; it is a function of the volume, shape, and density of the particle and the viscosity and density of the fluid. The fall velocity of any naturally worn sediment particle may be estimated with fluid and particle characteristics. Figure 3-6 shows the relationship between sieve diameter and fall velocity of quartz particles in distilled water. This figure shows variation with temperature and shape factor. Values shown are average and fall velocities for individual particles may vary widely.

(5) Similar relationships can be developed for other shape factors and specific gravities using the method outlined by Report 12 of the Interagency Committee on Water Resources (1957). Colder water temperatures lead to higher viscosity and lower settling velocities, as indicated in Figure 3-6.

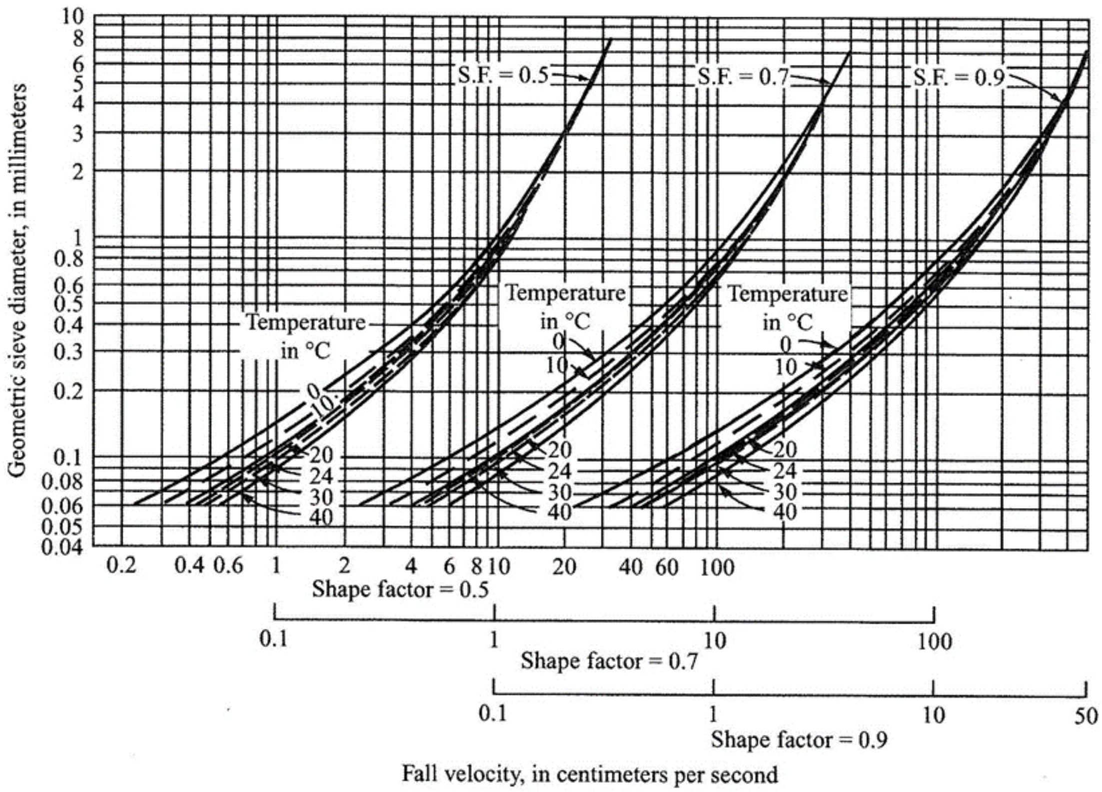


Figure 3-6. Relationship between sieve diameter, fall velocity, and shape factor for naturally worn quartz particles falling in distilled water (Interagency Committee 1957; Vanoni 1975, 2006)

(6) High concentrations of fine sediment in the water column, particularly clay particles, also increase viscosity and decrease fall velocity. Therefore, the settling velocities of clays and silts often deviate from those predicted for single particles. Fine particles such as clays may flocculate and form large, less dense groups of particles. As the larger and faster settling flocs sweep downward, they overtake and entrain the smaller ones. The settling velocities of these will be larger than would be apparent if the grains remained disaggregated in the water column. This is one reason sedimentation diameter tests should be conducted with water samples identical conditions present in the channel or reservoir of interest to obtain realistic floc sizes and settling rates. Refer to Floyd et al. (2016) for additional information on flocculation and settling velocity estimates.

(7) The methods above apply to noncohesive sediments (including silt). Methods for evaluating the settling of fine-grained cohesive sediments must also account for flocculation and are covered in paragraph 3-3.

3-3. Bulk Properties of Sediment Deposits.

a. General. A sediment deposit consists of solid particles and void space occupied by the fluid. Common properties of sediment deposits are defined in the following sections.

b. Porosity. Porosity, p, is the ratio of the volume of voids (filled with air or fluid) to the total volume of a sample. In numerical sedimentation models, porosity (ε) is a common input value used to determine the volume of a sediment deposit from its mass. Porosity or volume concentration (C_V) can be estimated from the specific gravity of sediment particles (S_g) and the specific weight (W) or dry density of sediment deposits:

$$p = \frac{Volume\ of\ voids}{Total\ volume\ of\ sample} \qquad \text{Equation 3-4}$$

c. Void Ratio.

(1) The void ratio, e, is the ratio of the volume of voids (filled with air or fluid) to the volume of solids and is given by:

$$e = \frac{Volume\ of\ voids}{Volume\ of\ solids} = \frac{p}{1-p} \qquad \text{Equation 3-5}$$

(2) The degree of saturation is the ratio of the volume of voids occupied by fluid to the total volume of voids in a sample.

d. Specific Weight.

(1) The specific weight (γ_d) of a deposit is the dry weight per total unit volume. Note that because p is the fraction of voids $(1 - p)$ is the fraction of solids.

$$\gamma_d = (1-p)(1-p)(1-p)SG\gamma \qquad \text{Equation 3-6}$$

or

$$\gamma_d = (1-p)\gamma_s \qquad \text{Equation 3-7}$$

where:

γ_d = specific weight of deposit
SG = specific gravity of sediment particles
γ = specific weight of water (approximately 62.4 lb/ft^3)
γ_s = specific weight of sediment particles

(2) Standard field tests are recommended when major decisions depend on the specific weight of the sediment deposit.

(3) The specific weight of freshly deposited sediment typically increases with particle size. Deposits of uniform coarse sand typically do not exceed 110 lb/ft^3 (Lane and Koelzer

(1943) from American Society of Civil Engineers Manual 54 (Vanoni 1975, 2006)). Thomas (1976) suggests a value of 93 lb/ft^3 for quartz sand deposits in the absence of field data. This value is the default value for noncohesive sediment in the Hydrologic Engineering Center – River Analysis System (HEC-RAS) program (HEC 2016).

(4) Figure 3-7 illustrates general relationships between specific weight (dry density) and porosity (volume concentration) for various sediment types.

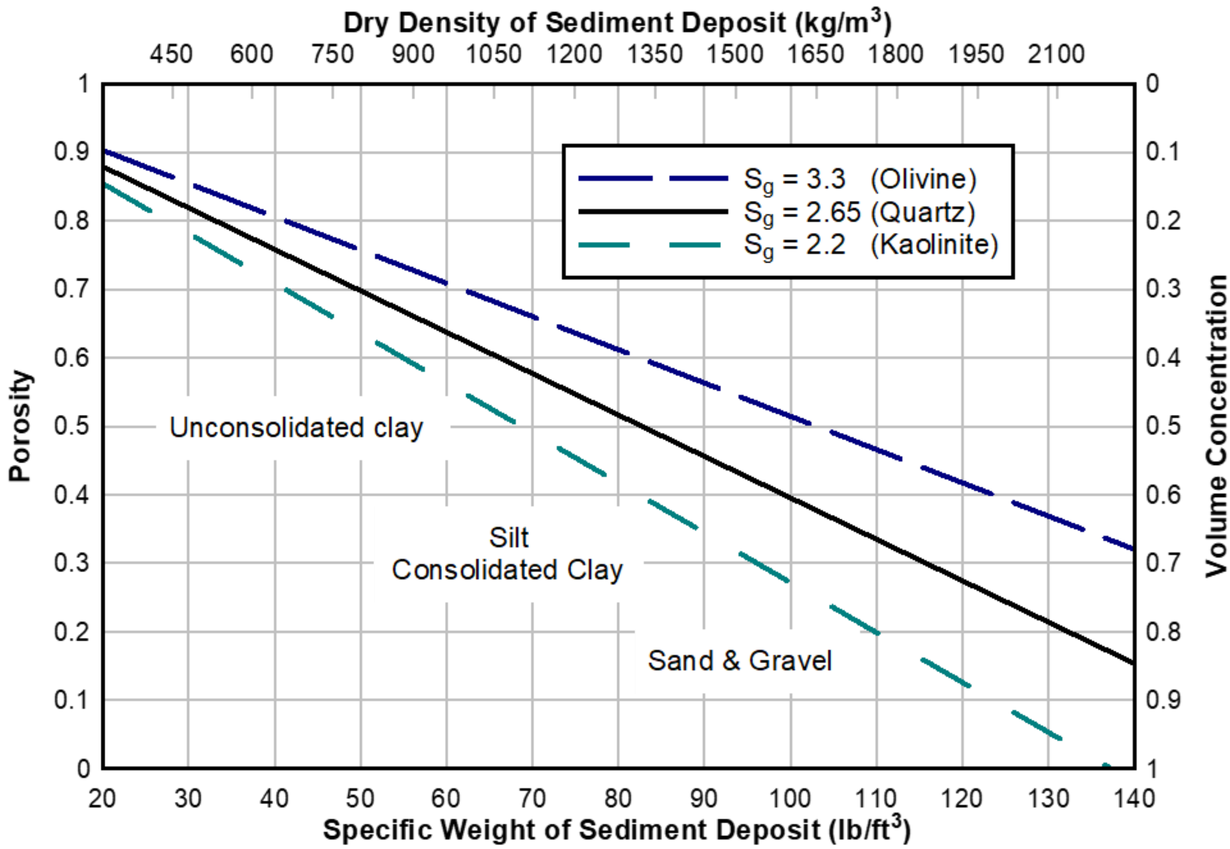


Figure 3-7. General relationships between specific weight and porosity

e. Bulk Density of Sediment Deposits.

(1) Bulk density is a parameter often used to characterize sediments in reservoir studies. Bulk density, along with unit weight and specific weight, are terms used to express the dry weight per unit volume of a bulk sediment sample, including both solid grains and voids, after drying to a constant weight at 105 °C. Bulk density depends on sediment size and shape as well as density, and the packing arrangement or consolidation of the sediment sample.

(2) Sandy sediments attain their ultimate bulk density virtually as soon as they are deposited, but fine sediments may continue to consolidate for many years through dewatering and particle realignment. Accurate bulk density values depend on obtaining undisturbed, in situ

samples of the sediment and drying for sufficient time to evaporate interstitial water. Samples are usually obtained by coring.

(3) There is a wide difference in bulk densities in reservoir sediments, both between sites and within the same reservoir. Highest densities typically occur in coarse delta deposits and lowest densities occur in fine sediment near the dam (Morris and Fan 1998).

(4) The density of solid sediment particles is approximately 2.65 metric tons/m^3. The dry bulk densities for 1,129 samples of reservoir sediments reported by Lara and Pemberton (1963) ranged from 1.8 to 3.0 metric tons/m^3. Table 3-5 lists typical values of bulk density for reservoir sediments.

Table 3-5
Typical Densities for Reservoir Deposits (Geiger 1963)

	Density (10^3 Kg/m^3 or g/cm^3)	
Grain Size	**Always submerged**	**Aerated**
Clay	0.64–0.96	0.96–1.28
Silt	0.88–1.20	1.20–1.36
Clay-silt mixture	0.64–1.04	1.04–1.36
Sand-silt mixture	1.20–1.52	1.52–1.76
Sand	1.36–1.60	1.36–1.60
Gravel	1.36–2.00	1.36–2.00
Poorly sorted sand and gravel	1.52–2.08	1.52–2.08

(5) Bulk densities are affected by the amount of time the sediment spends exposed to air and the duration of submergence. In general, the less time the sediment spends submerged, the higher its specific density will be.

f. Settling and Consolidation.

(1) Fine-grained materials, particularly clays, exhibit three stages in the process of settling. The first stage is flocculation, in which the particles aggregate to form flocs. The second is settling, in which the flocs gently settle, forming a “fluffy,” unconsolidated layer of sediment on the bed. The final stage is consolidation, in which the deposit compacts as grains reorient and water pressure is squeezed from the pores. Mehta and McAnally (2008) provides more information on these three stages. Accurate modeling of fine sediment deposition requires adequate modeling of these processes.

(2) In reservoirs, sand and gravel deposits attain their ultimate bulk density almost as soon as they are deposited. Consolidation of fine-grained deposits in reservoirs can continue for years or decades before the final, fully consolidated density is achieved. The process of consolidation

of reservoir deposits can be described by Equation 3-8 (Lane and Koelzer 1943). The average of consolidation of all deposits over T years can be expressed by Equation 3-9 (Miller 1953).

$$\gamma_{dc} = \gamma_{di} + B \log_{10} T \qquad \text{Equation 3-8}$$

$$\gamma_{dc} = \gamma_{di} + 0.434B\left[\left({}^{T}/_{T\,-\,1}\right) lnT - 1\right] \qquad \text{Equation 3-9}$$

where:

γ_{dc} = consolidated weight of the deposit
γ_{di} = specific weight of the initial deposit
B = coefficient of consolidation, which varies with size classification (see Table 3-6 for average values)
T = age of the deposit, years

(3) Table 3-6 lists consolidation coefficients based on sediment size and reservoir operating condition.

Table 3-6
Consolidation Coefficient for Reservoir Deposits Used in Equations 3-8 and 3-9

	Consolidation Coefficient, B in lb/ft³, (kg/m³)		
Operational Condition	**Sand**	**Silt**	**Clay**
Continuously Submerged	0	5.7 (91)	16 (256)
Periodic Drawdown	0	1.8 (29)	8.4 (135)
Normally Empty	0	0	0

(4) When dealing with mixtures of particle sizes, do not use the percent-weighted specific weight in the γ_d terms of Equation 3-8. It does not conserve the mass of the mixture. Rather, calculate compaction for clay, silt, and sand fractions separately, then calculate the composite-specific weight of the mixture using the following equation:

$$\gamma_d = \frac{1.0}{\left(\left(\frac{F}{\gamma_d}\right)_{clay} + \left(\frac{F}{\gamma_d}\right)_{silt} + \left(\frac{F}{\gamma_d}\right)_{sand}\right)} \qquad \text{Equation 3-10}$$

where F is the fraction by weight.

g. Critical Shear Stress and Erodibility.

(1) The critical shear stress is the stress at which a particle of sediment first detaches. The erodibility of a sediment layer depends first on whether or not it is cohesive. Gravels, sands, and most silts are not cohesive, their resistance to erosion depends largely on grain size and shape and on their degree of compaction. Clays, on the other hand, are cohesive soils that resist erosion much more than would be indicated by their sedimentation diameter. Deposited clays can gain

strength over periods of time through consolidation as the individual particles become rearranged into more stable positions.

(2) Cohesive sediment, with a concentration of fines and colloids sufficient to impart plastic properties, has the ability to resist shear stress. The plastic and cohesive properties of clay are due to colloids. Colloids are extremely small, and do not have a crystalline structure. The plastic and cohesive properties of clay are due to bonding of the clay platelets caused by surface charges. The degree of cohesion depends on numerous factors; these are outlined in the ASCE Manual No. 110, Chapter 2 (Garcia 2008b). Colloids have surface area-to-weight ratios that are so large that behavior is controlled by surface electrochemical forces rather than gravitational forces. In addition, the electrochemical surficial forces in clays may be several orders of magnitude stronger than gravitational forces.

(3) There are numerous formulations to characterize erosion. A recent theoretical investigation of critical shear stress for erosion of sand and mud mixtures with a computational formula is available (Wu et al., 2018). Often, erosion in cohesive sediment deposits is expressed as a linear function of excess shear stress (applied shear minus critical shear), as given in Equation 3-11. Erosion rate (ε) may be expressed as a length/time, volume/time, or mass/time, and may be expressed in metric or U.S. customary units.

$$\varepsilon = k_d(\tau_o - \tau_c) \qquad \text{Equation 3-11}$$

where:

ε = erosion rate
k_d = erodibility constant (how easily sediment is eroded)
τ_o = applied shear stress
τ_c = critical shear stress

(4) The critical shear stress and erodibility are site-specific parameters (Briaud et al., 2001; Huang et al., 2006). Several measurement apparatuses available to measure erodibility are described in paragraph 4-14. The erodibility of a deposit decreases over time, as the sediment consolidates. Erodibility also varies with temperature, with erodibility increasing as temperature increases (Grissinger 1966). Paragraph 5-2 provides more information on critical shear stress.

(5) The erosion rate (Equation 3-11) results in a linear empirical relationship between k_d and τ_c. However, whether or not the assumption of linearity holds over the entire range of shear stress remains to be determined. The erosion rate can also be stated in a dimensionless form (Walder 2016). Khanal et al. (2016) presented an evaluation of nonlinear models for cohesive sediment detachment.

h. Cohesive vs. Noncohesive Sediments, Mixtures, and Special Cases.

(1) Size is highly correlated with cohesiveness. While the threshold between cohesive and noncohesive sediment is usually site-specific, coarse-grained sediments (sands, gravels, and

boulders) are generally noncohesive, and fine-grained sediments (silts, clays, and organic material; also called "fines") have varying degrees of cohesion.

(2) Clays are highly cohesive while most coarse silts are practically cohesionless. Mechanical forces (friction, drag, lift) dominate the sedimentation processes of noncohesive sediments. In contrast, electrochemical forces dominate the behavior of cohesive sediments. Cohesive sediment behavior depends heavily on the properties of the entire sediment mixture (as opposed to the properties of single grains), sediment mineralogy, water chemistry, organic content, particle coatings, turbulence, and other factors. As particle size decreases, the surface area per unit weight increases, which increases cohesion. The plate-like shape of clay particles gives them additional surface area and cohesion.

(3) In situ coarse-grained sediment often includes some fines (silts, clays, or organics). Small percentages of fines will not significantly impact sedimentation processes, and the mixture can be characterized and analyzed as noncohesive. Higher percentages of fines can influence or even dominate sedimentation (particularly the erosion) processes. A gross rule for the threshold percentage of fines to begin cohesive behavior is 8% to 15%. In addition to the fine sediment percentage, other factors such as the nature of the sediment (silt, clay, or organics) and the grain sizes of the coarse fraction will affect the impact the fines have on the overall sedimentation behavior. In reservoirs, high organic content is often associated with a high fraction of fine-grained sediment and the sediments will likely display cohesive behavior.

(4) While sands are generally noncohesive, sand grains can be covered with coatings that produce cohesive forces that influence sedimentation behavior. These coatings may be natural or due to pollution (oil). For hazardous waste sites where oil is involved, there are two important points to consider: (a) the sand particles may behave as cohesive particles, and (b) the contaminant may be transported on sand-size particles.

i. Angle of Repose.

(1) The angle of repose (or friction angle) is the slope angle formed with the horizontal by granular material at the critical condition of incipient sliding. Angle of repose is used in the calculation of sediment stability and motion in channels and reservoir shorelines. The angle of repose for both dry and submerged noncohesive material, as a function of grain size, may be determined from Figure 3-8.

(2) When cohesive sediments are present or when there is a stabilizing matrix such as plant roots or when the soil is vitrified, slopes can be steeper than the angle of repose for cohesionless material. Under other conditions such as earthquake shaking or nearby soil disturbance form construction (pile driving or vibration), sediments can liquefy, producing slopes much lower than the angle of repose.

(3) Morris and Fan (2010) provide additional discussion on angle of repose with respect to reservoir shoreline stability.

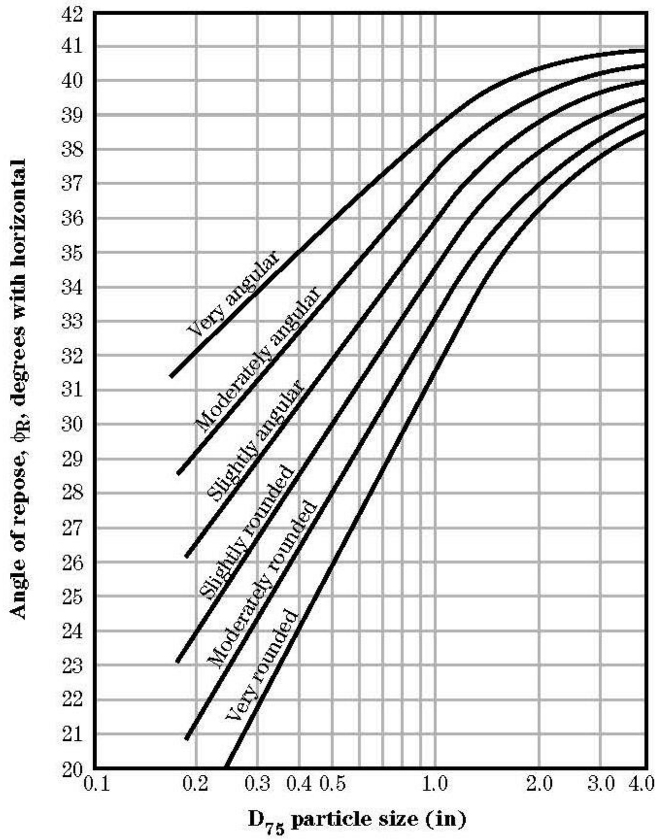


Figure 3-8. Angle of repose for noncohesive sediment (NRCS 2007)
Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

3-4. Water-Sediment Mixtures.

a. Sediment Concentration.

(1) Sediment concentration can be expressed as a mass concentration or volume concentration with respect to either total mass or total volume. The units used in the measurement of sediment concentration vary with the range of concentration and measurement technique. Most commonly, sediment concentration is the ratio of the dry sediment mass in a water-sediment mixture to the volume of the mixture, and is expressed in milligrams/liter (mg/l) (Porterfield 1972). Sediment concentration can be expressed in parts per million (ppm), C_{ppm}, the volumetric sediment concentration C_v, and the concentration by weight, C_w (Julien 2010).

$$C_v = \frac{sediment\ volume}{total\ volume} \quad \text{Equation 3-12}$$

$$C_w = \frac{sediment\ weight}{total\ weight} = \frac{C_v\, {}^{SG_S}/_{SG_W}}{1+\left({}^{SG_S}/_{SG_W}-1\right)C_v} \quad \text{Equation 3-13}$$

(2) If the concentration is less than 16,000 mg/l, then concentration in parts per million is essentially the same as milligrams/liter (Porterfield 1972) and related as:

$$C_{ppm} = 10^6 C_w \qquad \text{Equation 3-14}$$

(3) For concentrations greater than 16,000 mg/l, milligrams/liter and parts per million are related by the following equations:

$$C_{ppm} = \frac{10^6}{SG_w\left(\frac{10^6}{C_{mgl}} + \frac{1}{SG_w} - \frac{1}{SG_s}\right)} \qquad \text{Equation 3-15}$$

$$C_{mgl} = \frac{10^6}{\left(\frac{1.0}{SG_w}\frac{10^6}{C_{ppm}} - \frac{1.0}{SG_w} + \frac{1.0}{SG_s}\right)} \qquad \text{Equation 3-16}$$

where:

C_{ppm} = concentration, ppm
C_{mgl} = concentration, mg/l
SG_s = specific gravity of sediment particles
SG_w = specific gravity of water

(4) Julien also presented equivalent concentrations (Garcia et al., 2008) as listed in Table 3-7.

Table 3-7
Equivalent Concentrations C_v, C_w, C_{ppm}, and $C_{mg/l}$ (Garcia et al. (2008); adapted from Julien (1995)

	C_v	C_w	C_{ppm}	$C_{mg/l}$
Suspension	0.001	0.00264	2,645	2,650
	0.0025	0.00659	6,598	6,625
	0.005	0.01314	13,141	13,250
	0.0075	0.01963	19,632	19,875
	0.01	0.02607	26,069	26,500
	0.025	0.06363	63,625	66,250
Hyper Concentration	0.05	0.12240	122,402	132,500
	0.075	0.17686	176,863	198,750
	0.1	0.22747	227,468	265,000
	0.25	0.46903	469,027	662,500
	0.5	0.72603	726,027	1,325,000
	0.75	0.88827	888,268	1,987,500

Note: Calculations are based on mean density of water, SG_w = 1g/ml, and specific gravity of sediment, SG_s = 2.65

b. Sediment Load.

(1) Sediment load denotes the material that is being transported, whereas sediment discharge denotes the rate of transport. Sediment load is described with a variety of terminology. Sediment load is generally defined based on mode of transport, by its availability in the streambed, or by the method of measurement (Table 3-8).

Table 3-8
Classification of Total Sediment Load by Mode of Transport, Availability in Streambed, and Method of Measurement

<table>
<tr><th></th><th>Mode of Transport
(1)</th><th>Availability in Streambed
(2)</th><th>Method of Measurement
(3)</th></tr>
<tr><td rowspan="4">Total Load</td><td rowspan="3">Suspended</td><td>Wash</td><td rowspan="2">Measured</td></tr>
<tr><td rowspan="3">Bed Material</td></tr>
<tr><td rowspan="2">Unmeasured</td></tr>
<tr><td>Bed</td></tr>
</table>

(2) Based on the mode of transport, sediment load can be divided into bedload and suspended load. Bedload is the sediment load transported close to the bed where particles move intermittently by rolling, sliding, or jumping (saltation). Turbulence supports suspended load throughout the water column, and sediment is swept along at about the local flow velocity.

(3) Based on its availability in the streambed, sediment load can be divided into bed material load and wash load. Wash load consists of the finest particles in the suspended load that are continuously maintained in suspension by the flow turbulence, and thus significant quantities are not found in the bed. Particles that move as suspended load or bedload and periodically exchange with the bed are part of the bed material load. Refer to paragraphs 5-4 and 5-5 for further discussion of bedload and suspended load transport, respectively.

(4) Based on measurement technique, sediment load is described as either measured or unmeasured. Typically, when depth-integrated suspended-sediment samples are used, the lower 0.5 foot of the water column is unmeasured. The unmeasured load includes some of the suspended and usually all of the bedload. The use of the term unmeasured does not mean unmeasurable. Technologies and methods exist for measuring the bedload, which constitutes a substantial portion of the unmeasured load, as explained in Chapter 4.

(5) Note that the relative distribution of the total load listed in Table 3-8 is an example only. The relative distribution of sediment load among the categories varies by site, by flow, and often over time.

c. Spatial Distribution of Sediment Load. Sediment load exhibits significant lateral and vertical variation in concentration, gradation, and transport rate.

(1) Figure 3-9 shows the lateral and vertical distribution of sediment concentration in a large sand-bed river, the lower Mississippi River near the Old River Control Structure. In Figure 3-9, the colors indicate suspended-sediment concentration computed by correlating the acoustic backscatter signals with measured point samples, and the numbers indicate mean grain size in microns obtained from point samples.

(2) As Figure 3-9 shows, sediment concentrations along the point bar on the inside of the bend (left side of the plot) are at least four times greater than concentrations on the outside of the bend. The mean grain size varies by more than an order of magnitude, both vertically and laterally.

(3) Figure 3-9 is provided to illustrate lateral and vertical variability and is not necessarily representative. The relative location of maximum concentration is flow-dependent and may shift over time.

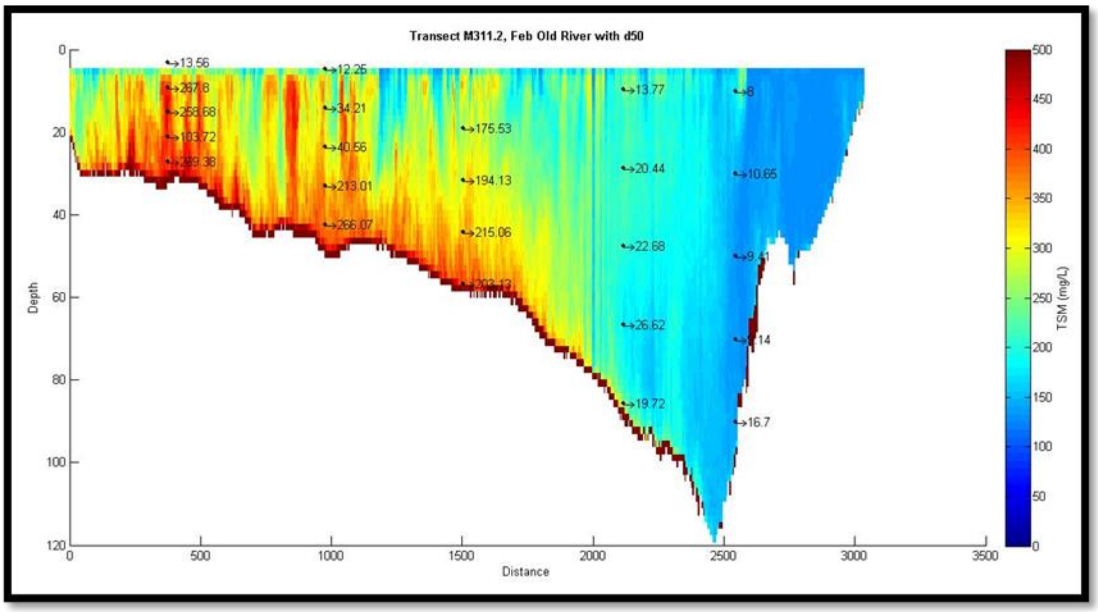


Figure 3-9. Lateral and vertical distribution of suspended-sediment concentration, lower Mississippi River; colors indicate concentrations and numbers indicate mean grain size in microns from point samples

d. Sediment Discharge.

(1) Sediment discharge is the quantity of sediment per unit of time passing a cross section. It is expressed as tons/day. The conversion from concentration (mg/l) to sediment discharge (tons/day) is given in the following equation (Porterfield 1972, Gray 2008):

$$Q_s = 0.0027\, Q_w\, C_{ppm}\, k \qquad \text{Equation 3-17}$$

where:

Q_s = sediment discharge (tons per day)
0.0027 = convert cfs to tons/day/1,000,000 parts
Q_w = mean daily water discharge, cfs
C_{ppm} = mean daily sediment concentration, ppm
k = conversion of ppm to mg/l = 1.0 for concentrations less than 16,000 ppm; otherwise, see Table 3-7 or Table 2 of Porterfield (1972)

(2) In the above equation concentration in mg/l is used regardless of the unit system. Otherwise, all other variables must be stated in only U.S. or metric units.

(3) Sediment discharge in tons per day can be converted to cubic feet per second using the following equation:

$$Q_{s,cfs\,=} = 0.02315\ {}^{Q_{s,tons/day}}\!/_{\gamma_s} \qquad \text{Equation 3-18}$$

where γ_s is the specific weight of the sediment in pounds per cubic feet (pcf).

e. Hyperconcentrated Flows. Hyperconcentrated sediment flows can be initiated by numerous causes, including intense rainfall, rapid snowmelt, post-fire runoff, volcanic activity, and manmade activities. Typical floods transport mostly fine sediments in relatively small quantities (as a percentage of the total flow volume), where the suspended sediment has little effect on flow behavior. In contrast, a hyperconcentrated flow can be defined as a fluid in movement in which a high percentage of solid material is transported and flow is non-Newtonian. Hyperconcentrated flows are considered an intermediate phase between low-density fluvial flows and high-density debris flows.

(1) Non-Newtonian fluids have flow properties that are not described by a single, constant viscosity (Pierson 2005). In non-Newtonian fluids, the relationship between shear stress and the shear rate (the viscosity) is different and can vary with time. This varying relationship means that a constant coefficient of viscosity cannot be defined. The non-Newtonian behavior of hyperconcentrated flow (as well as debris flow and mudflow) is highly dependent on the characteristics of the sediment particles. Beverage and Culbertson (1964) provided an early definition for the bounding sediment concentrations for hyperconcentrated flow—at least 20% by volume (40% by weight) but not greater than 60% by volume (80% by weight)—but gave no objective criteria that could be transferred to various site conditions.

(2) Various values are stated in literature regarding the initiation of hyperconcentrated flows. Table 3-7 provides guidance for concentration by volume and mass to define the onset of hyperconcentrated flow. Wannanen et al. (1970) stated that sediment concentrations generally less than 4% by volume (or 10% by weight) are characteristic values, and the behavior is Newtonian. A challenge to the development of a uniform classification system is the physical properties of hyperconcentrated flows vary over a wide range in the field.

(3) O'Brien and Julien (1985) provided further classification of hyperconcentrated flows according to the properties controlled by sediment concentrations, as water floods, mud floods, mudflows, and landslides.

(a) Mud floods are typically hyper concentrations of noncohesive particle (sand) that display fluid behavior for a range of sediment concentration by volume (C_v) between 20% and 45% (Garcia et al., 2008). Mud floods have hydrodynamic characteristics similar to turbulent flow with flow resistance a function of channel roughness.

(b) Debris flows and mud flows have sediment concentrations that are often greater than 60% by volume (80% by weight) (Costa 1984, 1988; Pierson and Costa 1987) which significantly influences the water-sediment fluid behavior. Garcia et al. (2008) state that mudflows are characterized by a high concentration of silts and clays (sediment size <0.0625 mm) by volume of 45% to 55% to behave as a highly viscous fluid mass capable of rafting boulders near the flow surface. Debris flows usually are mixtures with high coarse particle contents including boulders and woody debris and are generally less fluid than mud floods.

Chapter 4
Sediment Measurement Techniques

4-1. General. Satisfactory resolution of problems associated with sediment transported in streams requires both an understanding of sedimentation processes and a knowledge base of physical data. This chapter presents techniques for measuring bed material properties, suspended and bedload discharges, and particle analysis.

a. Currently, the field of sediment measurement is in the middle of a digital data revolution, with rapid development of new data collection and analysis methods. Practitioners are met with multiple data collection techniques, the results of which are often not compatible. It remains important to plan sediment data collection efforts carefully. Data collected by new techniques should be compared with other methods to ensure reliability and comparability.

b. Figure 4-1 illustrates the content of this chapter.

4-2. Federal Interagency Sedimentation Project. The FISP was initiated in 1939 to standardize sediment sampling equipment and measurement techniques across the federal agencies.

a. FISP was initially located at the Institute of Hydraulic Research at the University of Iowa. In 1948, it was moved to the St. Anthony Falls Hydraulic Laboratory, at the University of Minnesota. In 1992, it was relocated to the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi (WES), and in 2005, it was relocated to its current position at the U.S. Geological Survey (USGS) Hydrologic Instrumentation Facility, Stennis Space Center, Mississippi.

b. USACE has always been a major contributor to FISP, and has benefitted greatly both from the use of the standardized equipment and procedures developed by the project, and from the reliable database generated by other agencies. Each federal agency that provides financial support to FISP has one member on a technical subcommittee that guides the work of the project. Additional information related to FISP is available on the website, currently hosted by the USGS at http://water.usgs.gov/fisp/.

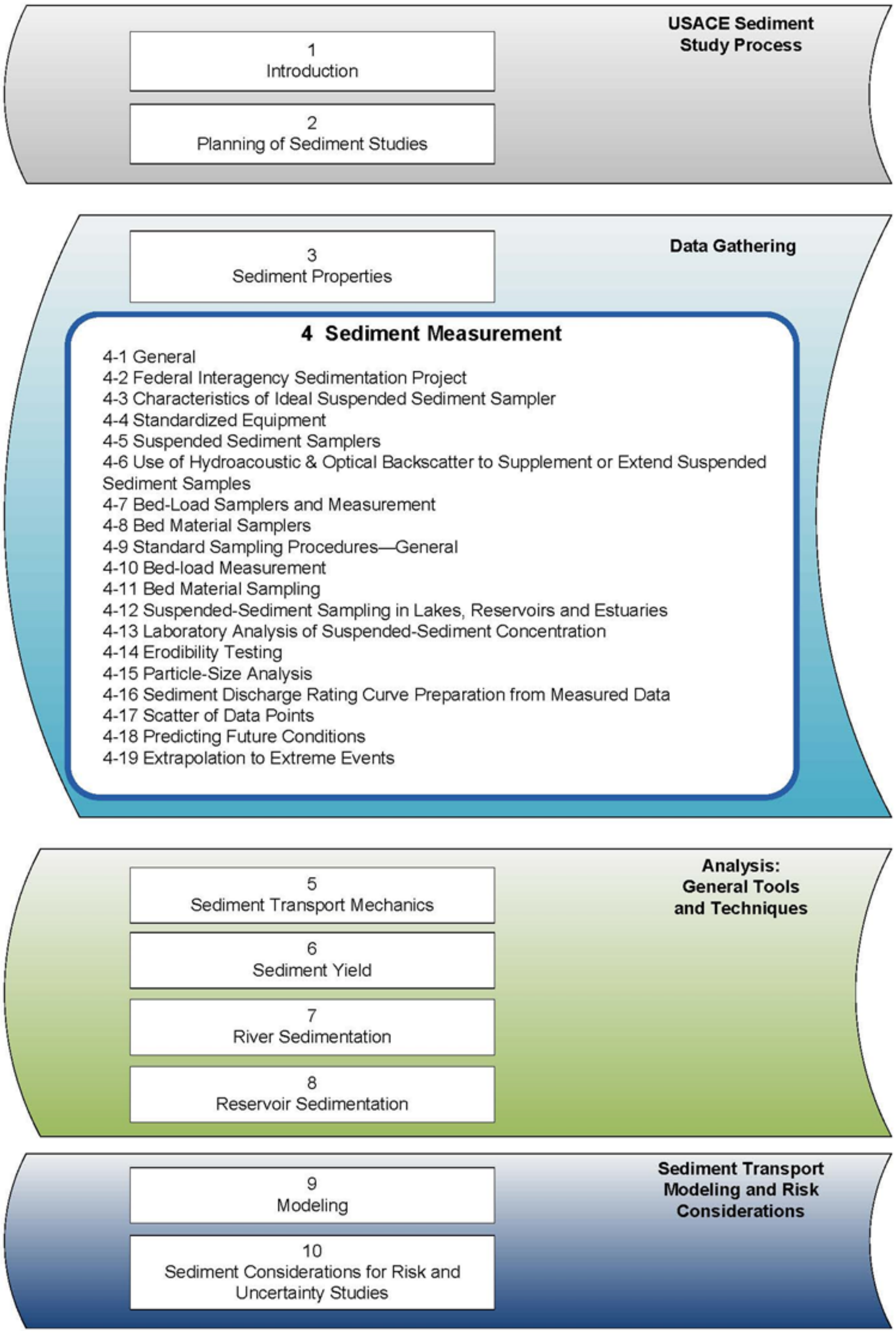


Figure 4-1. Chapter 4 content and general document structure

c. The FISP has developed a federally approved suite of physical sediment and water quality samplers that are available to collect accurate sediment samples. Individual samplers ordered from the FISP are tested and quality ensured before shipment. The US-series of suspended-sediment samplers developed by FISP embody most of the required and desirable features for an ideal sampler. In particular, all US-series integrating samplers provided by FISP are designed and calibrated to sample isokinetically: that is, the water-sediment mixture moves with no acceleration from the ambient flow into the sampler's nozzle intake. This isokinetic property is critical to obtaining an accurate representation of sediment concentration.

d. In addition to providing physical samplers and guidance, FISP sponsors and conducts research on emerging tools and technologies for measurement and analysis of sediment properties. Recent research is often focused on indirect (surrogate) methods of measuring sediment characteristics with improved resolution, accuracy, and cost.

4-3. Characteristics of Ideal Suspended-Sediment Sampler.

a. Nelson and Benedict (1951) summarized the following requirements of an ideal time-integrating suspended-sediment sampler:

(1) The velocity at the entrance of the intake tube should be equal to the local stream velocity (isokinetic).

(2) The intake should be pointed into the approaching flow and should protrude upstream from the zone of disturbance caused by the presence of the sampler.

(3) The sample container should be removable and suitable for transportation to the laboratory without loss or spoilage of the contents.

b. Furthermore, the sampler should:

(1) Fill smoothly without sudden inrush or gulping.

(2) Permit sampling close to the streambed.

(3) Be streamlined and of sufficient weight to avoid excessive downstream drift.

(4) Be rugged and simply constructed to minimize the need for repairs in the field.

(5) Be as inexpensive as possible, and consistent with good design and performance.

c. Samplers developed and used before 1940 were tested by FISP, and the results indicated that none met the criteria stated above. This resulted in problems with data reliability and comparability that should be considered when evaluating historic data. For example, measured decreases in suspended-sediment load in the Colorado River Basin were attributed at the time to changes in land use or other causes, but were probably due to sampling errors (Gray et al., 2008). For this reason, caution should be taken when using sediment data collected before 1940.

4-4. Standardized Equipment.

a. FISP-approved sediment samplers fall into three categories: suspended sediment, bedload, and bed material. Davis (2005) includes a description of FISP-approved samplers through 2005, along with useful flowcharts to aid in selecting a sampler (Figure 4-2, Figure 4-9).

b. Consult the FISP website for updates regarding approved samplers and guidance. The FISP website includes operator manuals for many samplers, along with relevant publications and technical committee notes. (This information is particularly useful for newer sampling techniques, such as bedload traps, that are not covered in earlier publications.)

c. Chapter 5 in the ASCE Manual No. 110 (Gray et al., 2008) has excellent summaries of sampling equipment and methods, including an update by Gray et al. (2008) of suspended-sediment samplers and sampling methods. Edwards and Glysson (1999) has information on samplers developed before 1988.

d. The samplers developed by FISP are designated based on their function and the year designed. For example, with a US DH-75 sampler, "D" signifies depth-integrating, "H" signifies handheld, and "75" indicates the sampler was designed in 1975. A US P-61 is a point (P) integrating sampler designed in 1961. Except in unique circumstances when specialized equipment is required, standardized equipment, provided and calibrated by FISP, should be used for data collection for USACE projects.

e. The current FISP catalog can be viewed at the FISP website (FISP 2016). Sales of FISP equipment to U.S. Government agencies are handled by the USGS Hydrological Instrumentation Facility (HIF). The HIF is authorized to sell only to Federal Governmental agencies of the United States.

f. The following sections describe some of the commonly used samplers and is not a comprehensive list. For more information on additional FISP samplers and older, discontinued samplers, refer to Davis (2005) and the FISP website.

4-5. Suspended-Sediment Samplers. Figure 4-2 shows a flowchart of common FISP-approved suspended-sediment samplers. Each sampler has an operating range for depth, velocity, and other critical parameters. Selecting a sampler should be based on the most extreme conditions (depth and velocity) under which the samples are likely to be collected. Consult the user manual, available on the FISP website, to verify sampler suitability for field conditions.

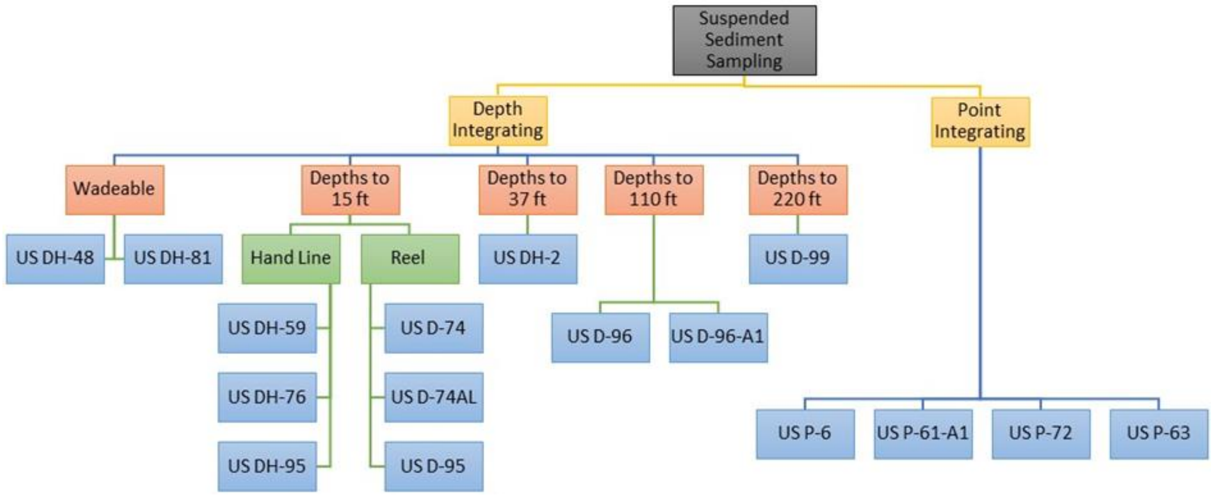


Figure 4-2. Flowchart of FISP-approved suspended-sediment sampling equipment (redrawn from Davis 2005)

a. Depth-Integrating Samplers. Depth-integrating samplers are designed to collect a continuous water-sediment sample through the water column at a uniform rate (known as the transit rate). Since it is a continuous measurement, the nozzle, with a diameter of either 1/8, 3/16, 1/4, or 5/16 in. is always open.

(1) The size of particles that can be collected is determined by the nozzle opening and range from clays through sands, although use of the 1/8-in. nozzle is discouraged because it tends to plug easily and surface roughness in the bore may affect the sampling rate (flow of water into the sampler). This nozzle is generally used only when conditions do not permit use of larger nozzles.

(2) Sampling depths are dictated by the volume of the collection container, nozzle opening, and transit time. The current set of FISP-approved samplers handles sampling depths of up to 220 feet. Numerous sampler options are available based on expected sediment characteristics, flow conditions, and site accessibility. Depending on site conditions, depth-integrating samplers can be deployed by hand or using a cable and reel system.

b. Handheld Samplers. Where streams can be waded or where a low bridge is available, lightweight handheld samplers can be used to obtain depth-integrated suspended-sediment samples.

(1) The US DH-48 (Figure 4-3a) is a streamlined aluminum sampler that weighs 4.5 lb, collects samples in a pint bottle, and can sample to within 3.5 in. of the bed.

(2) The US DH-59 (Figure 4-3b) and US DH-76 are bronze cast samplers, collect samples in pint and quart size bottles, respectively, and were designed to be suspended from a handheld rope in streams too deep to wade.

(3) The lightweight US DH-59 and US DH-76 weigh about 22 and 25 lb, respectively; applicability is limited to cases where the velocity is less than 5 feet per second (fps). These lightweight handheld samplers are the most commonly used for sediment sampling during normal flow in small and intermediate sized streams.

(4) The US DH-75 was designed for use in subfreezing winter conditions. The US DH-75 sampler may be used with a pint or a quart plastic bottle and most of the working parts are made of plastic.

(5) Other handheld samplers include the DH™-95, DH-2, and DH-81.

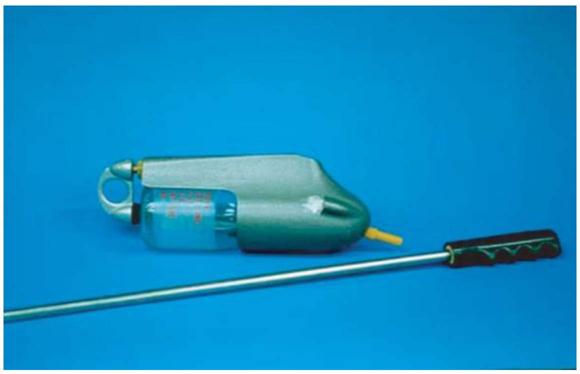
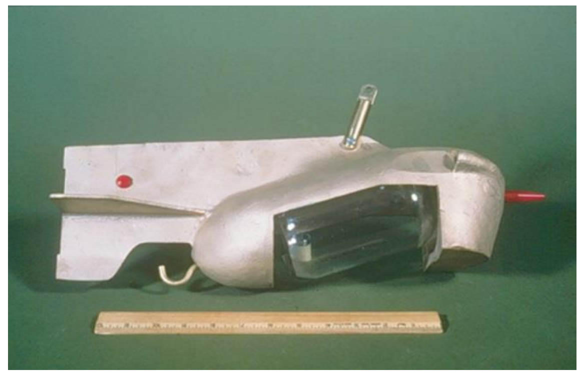



Figure 4-3. US DH-48 handheld (left), and DH-59 handline suspended-sediment sampler (right) (Davis 2005)

c. Cable and Reel Samplers. When streams cannot be waded, but are less than 15-foot deep, a US D-74 (Figure 4-4) sampler can be used. The US D-74 is a 62-lb bronze cast sampler. Samples are collected in a pint or quart bottle and the US D-74 can sample to within 4 in. of the streambed. Maximum calibrated velocity for the US D-74 is 6.6 fps. The US D-77 was designed to collect large volume depth-integrated samples. This sampler is used extensively in water quality sampling because all components that contact the sample are made of plastic or Teflon. The US D-77 was phased out in 2002 and has been replaced by the D-95™ sampler (USGS 2002). Other cable and reel type samplers include the D-74AL, D-95™, D-96, D-96A1, and D99.

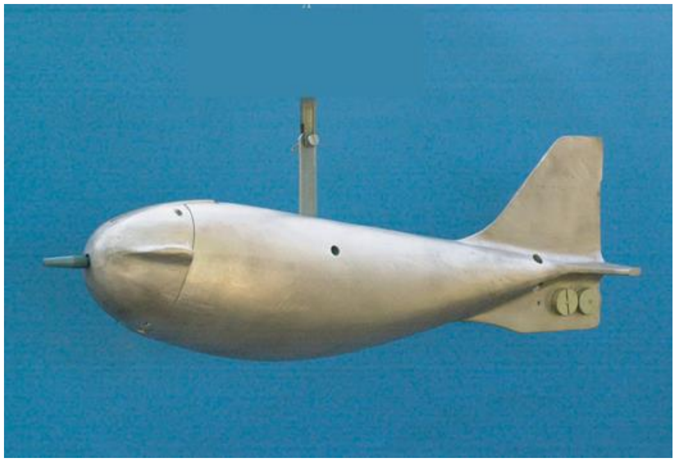


Figure 4-4. D-74 cable and reel suspended-sediment sampler (Davis 2005)

d. Point-Integrating Samplers. Point-integrating samplers use an electrically operated value to open and close the intake nozzle. As such, they can be used to collect either discrete or continuous samples, which makes them more versatile but also more complicated, than depth-integrated sampler types.

(1) Point-integrating samplers are more versatile than the simpler depth-integrating types. They can be used to collect a sample at any selected point in the water column, or they can be used to sample continuously over a range of up to 30-foot deep. The depth limit results from the requirement to maintain ambient pressure in the sample bottle as the sample is collected. In general, because of their greater mass, point-integrating samplers can be used in streams too deep or swift for the standard depth-integrating samplers.

(2) Point-integrating samplers contain an air compression chamber that allows for pressure equalization in the sample bottle up to depths of 180 feet when a pint-sized sample bottle is used. With a quart-sized bottle, depths up to 120 feet can be sampled. Sampling is controlled by a rotary valve, which is operated electrically by the operator. By positioning the sampler at the streambed before opening the valve, and sampling while transiting upward to the surface, a depth-integrated sample can be collected through a 30-foot deep water column. In deeper rivers, a depth-integrated sample can be collected by partitioning the total depth into segments, up to about 30 feet each, and by using a constant transit velocity throughout.

(3) The US P-61, which weighs 105 lb, is the classical point-integrating sampler. The distance between the nozzle and the sampler bottom is 4.3 in. A lightweight version of the US P-61 is the aluminum cast US P-72 (Figure 4-5), which weighs about 41 lb. For swifter streams, the 200-lb US P-63 can be used. The US P-63 can sample to within 5.9 in. of the streambed. The US P-50, weighing 300 lb, is a special point-integrating sampler developed for and used on large rivers such as the lower Mississippi. Other point-integrating samplers include the P-61-A1 and P-6.

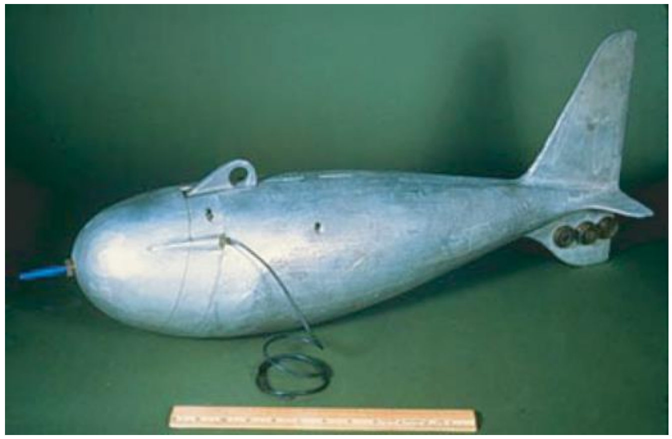


Figure 4-5. P-72 point-integrating suspended-sediment sampler (Davis 2005)

e. Auxiliary or Automatic Sampling Equipment.

(1) Single-stage samplers were developed as an aid in obtaining information on flashy streams. The most severe limitation of single-stage samplers is that they collect samples of the water-sediment mixture at a fixed point in the stream and, therefore, are most effective in streams carrying predominately fine sediments. Other limitations are presented by Edwards and Glysson (1999). More information on this sampling approach can be found in FISP (1961).

(2) The single-stage sampler may be a static sampler such as the US U-59A, U-59B, U-59C, or U-59D, which consists of a pint bottle filled from a vertical or horizontal intake tube using siphonic action or using a pump. In the case of the pump, the velocity in the intake is not usually equal to the stream velocity, and the intake does not usually point into the flow.

(3) Whereas silt and clay sizes collected in such samplers may be representative, since they are equally distributed throughout the water column, pumping samplers generally underestimate the concentration of sand sizes in the flow field (Hall and Fagerburg 1991), as they are not uniformly distributed in the water column. Sediment samples collected from automatic sampling equipment must be calibrated to samples collected from cross-section depth-integrated or point-integrated samples for reliable results.

4-6. Use of Hydroacoustic and Optical Backscatter for Suspended-Sediment.

a. Optical backscatter sensors and, more recently, acoustic backscatter sensors are indirect suspended sediment concentration monitoring techniques. The continual monitoring capability is desirable for the highly temporally variable suspended sediment concentration. The technologies of both types of instrument have improved markedly recently and they have been successfully applied to quantifying the suspended sediment transports in many fluvial environments. The more recently introduced ADCP have the advantage of being able to collect vertical profiling of sediment concentration and water velocity variation as compared to optical backscatter.

b. The conversion of backscatter data to suspended sediment concentration may be complicated due to the site-specific variability in sediment physical properties. Co-deployment of optical and acoustic backscatter sensors is one option that allows for partial validation of site-specific calibration. Landers et al. (2016) provide standard techniques for sediment acoustic index methods to help ensure accurate and comparable documented results.

c. Hydroacoustic data, particularly the echo intensity of acoustic backscatter of an ADCP unit, can be calibrated to concurrently collected concentration samples. This allows sparse data to be extended throughout a river section for a more complete qualitative and quantitative assessment of the suspended sediments in a cross section. Sediment flux measurements are the results of this process; if grain size data are also collected, then flux by size class result from the calculations. Figure 4-6 provides an example from the Missouri River at Kansas City, Missouri. Optical backscatter can be similarly calibrated to concentration.

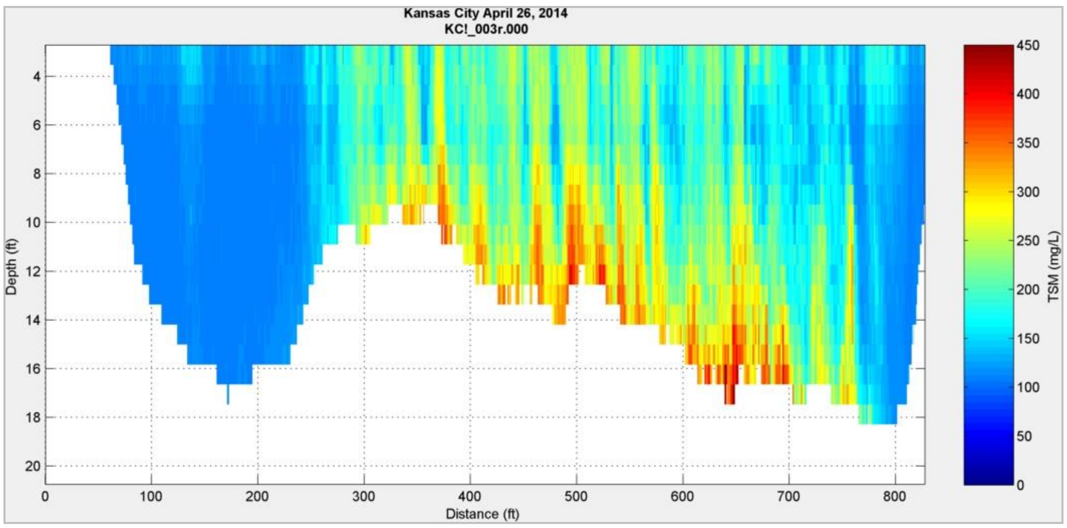


Figure 4-6. Suspended-sediment concentration computed via correlation with ADCP backscatter (Kansas City gage, April 26, 2014)

d. Turbidity can also be calibrated to suspended-sediment measurements to create a turbidity/concentration rating curve. Permanent turbidity meters installed at flow/sediment gaging stations can provide continuous measurements (typically at 15-minute intervals) that can be used to compute suspended-sediment concentrations at a fine temporal scale. Adding continuous turbidity monitoring to gages with continuous flow monitoring and intermittent concentration sampling improves the daily, monthly, and annual sediment load estimates over what is possible with flow and concentration sampling alone. Methods and consideration for measurement turbidity are described in the USGS Field Manual (USGS 2005), Chapter 6.7.

4-7. Bedload Samplers and Measurements.

a. Figure 4-7 shows a flowchart of FISP-approved bedload sampling equipment.

b. Bedload is difficult to measure for several reasons. Any mechanical device placed on the bed disturbs the flow and hence the rate of bedload movement. In addition, bedload is characterized by extensive spatial and temporal variability. For this reason, the sampling technique is just as important as the sampling equipment.

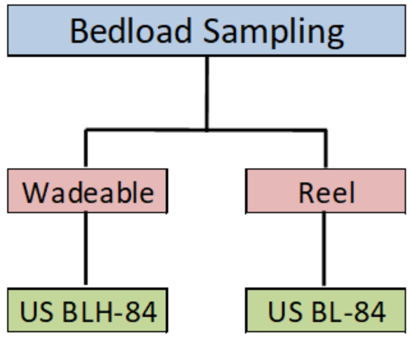


Figure 4-7. Flowchart for the selection of FISP-approved bedload sampling equipment (redrawn from Davis 2005)

c. All bedload samplers are designed with a flared entrance. This creates a pressure difference that keeps the entrance clear. In rivers with a sandy bed, this can result in oversampling the fine component of the bedload. This is generally not a problem in streams with gravel or coarser beds. Childers (1999) compared the sampling characteristics of six pressure-difference bedload samplers in high-energy flows of the Toutle River near Silver Lake, Washington, and found that ratios of bedload transport rates between measured bedload pairs ranged from 0.40 to 5.73, or more than an order of magnitude in differences of sampling efficiencies.

d. The Helley-Smith bedload sampler is the most commonly used bedload sampler in the United States. However, there are known problems with both oversampling and undersampling in gravel-bed streams (Bunte and Abt 2009). FISP recommends a bedload sampler with a nozzle flare angle that is different from that on the Helley-Smith sampler. The FISP-approved samplers are US BLH-84 for wadeable streams and US BL-84 (Figure 4-8) for non-wadeable streams. In general, the overall sampling efficiency of a specific sampler is not constant, but varies with size distributions, stream velocities near the bed, turbulence, rate of bedload transport, and the degree of filling of the sampler.

e. The bedload trap is a portable FISP-approved sampler designed specifically for collecting gravel and cobble bedload in wadeable streams. The bedload trap consists of an aluminum frame with a 12x8-in. opening attached to a 4-foot-long meshed net. The bedload trap is installed on ground plates that are anchored to the stream bottom with 3.5-foot-long metal stakes. Since the traps are not handheld and are attached to the stream bottom, sampling can be done over a long period of time, generally a 1-hour sample period. Guidelines for use are contained in U.S. Department of Agriculture (USDA 2007a). The approval of the device as an official FISP sampler is documented in FISP 2009. Bunte and Abt (2009) compare bedload trap sampling to using the Helley-Smith sampler. Bedload traps can be purchased from the HIF or rented for short-term use.

Figure 4-8. US BM-84 cable suspended bedload sampler (Davis 2005)

f. Various methods have been developed that compute the volume of bedload transport, which can be transformed to a mass transport rate via density and porosity. One such method is the Integrated Surface Section Difference Over Time, version 2 (ISSDOTv2) method. This method uses repeated multi-beam sonar to calculate the volume of scour on sand dunes to compute the bedload transport. More information on the ISSDOTv2 method is provided later in this chapter.

4-8. Bed Materials Samplers. Figure 4-9 shows a flowchart of FISP-approved bed material sampling equipment.

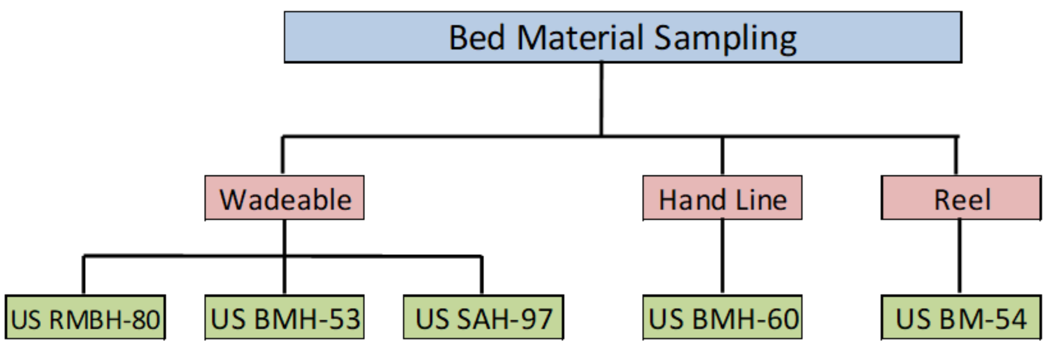


Figure 4-9. Flowchart for the selection of FISP-approved bed material sampling equipment (redrawn from Davis 2005)

a. FISP-Approved Bed Material Samplers.

(1) Bed material samplers approved by FISP are limited to collecting samples from relatively firm beds; they are not designed to collect samples from unconsolidated deposits of silt or clay. The US BMH-53 is a handheld piston-type sampler for sampling the bed of wadeable streams as shown in Figure 4-10. The collecting end of the sampler is a stainless steel, thin-walled cylinder with a diameter of 2 in. and length of 8 in. A handle is used for pressing the cylinder into the bed. The suction created by the piston holds the sample in the cylinder.

Sediments composed primarily of sands are difficult to sample with the US BMH-53 because the material tends to fall from the barrel when the cutting edge is lifted above the streambed.

Figure 4-10. US BMH-53 handheld piston-type bed material sampler (Davis 2005)

(2) For wadeable sand-bed and gravel-bed streams, the US RBMH-80 bed material sampler is available. It is a handheld rotary scoop sampler with a semi-cylindrical bucket for collecting the sample. Simple operation consists of placing the sampler on the streambed and the manual level closes the bucket to collect the sample. The bucket closure is sufficiently sealed to prevent loss of the sample while the instrument is lifted through the water column.

(3) The beds of deeper streams or lakes can be sampled with the US BMH-60. This is a hand-line, streamlined sampler with a spring-driven rotary bucket. It weighs 32 lb and is easiest to use in any reasonable depth when stream velocities are under 3 fps. The weight of the sampler limits its use to tranquil streams and moderate or slightly compacted sand and pebble bed materials. The rotary bucket penetrates the bed to about 1.7 in. and holds about 175 cc of sample. The US BM-54 (Figure 4-11) is a cable and reel suspension sampler with a design similar to the US BMH-60, but weighing 100 lb. It is used to collect sand and gravel samples from the bed of a stream, lake, or reservoir. The extra weight allows for sampling at any reasonable depth and in swifter streams.

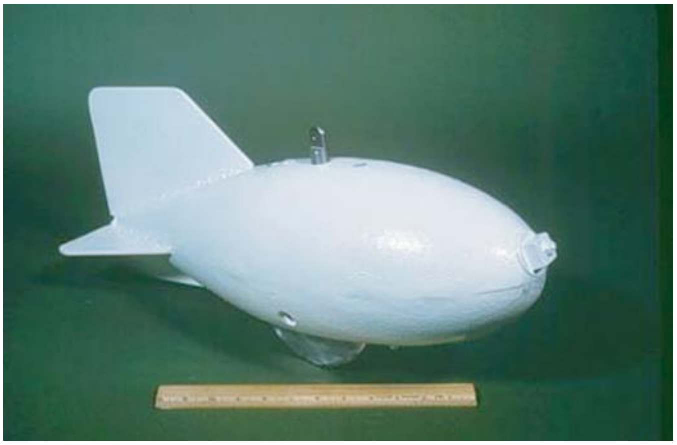


Figure 4-11. US BM-54 cable and reel bed material sampler (Davis 2005)

(4) The US SAH-97™ (also known as a gravelometer) is a handheld device made of aluminum alloy used to grade or measure gravel and small cobble-size bed sediments in the field, in wadeable streams. As shown in Figure 4-12, the US SAH-97™ has 14 square holes of common sieve sizes (1/2-phi unit classes) ranging from 2 to 180 mm. There is also a scale along one side that can be used to measure up to 310 mm. For field measurements, the particles in each size class are counted, using the pebble count method (Bunte and Abt 2001).

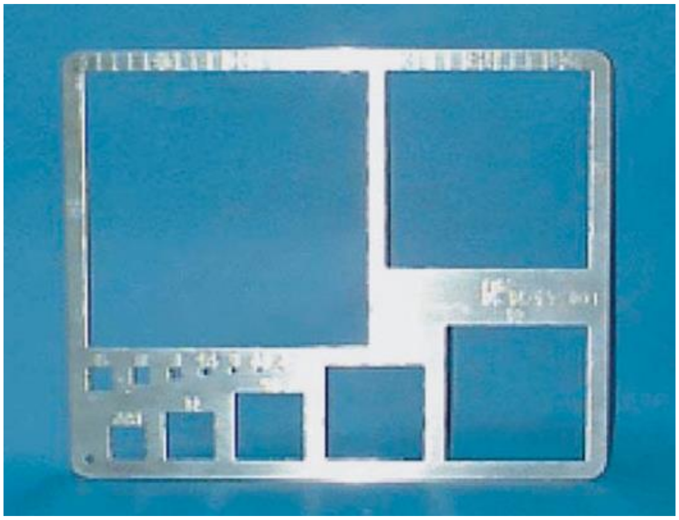


Figure 4-12. Gravelometer, US SAH-97™ analyzer (Davis 2005)

b. Nonstandard Bed Samplers.

(1) Nonstandardized bed samplers are frequently used for special applications or when the advantages of standardized equipment are considered unnecessary. Drag bucket, pipe samplers, clamshell samplers, and scoop samplers simply collect a sample into an open container by dragging or scooping. The disadvantage of these sampler types is that material, especially fine material, may be washed out of the container as the sample is brought to the surface. Clamshell samplers can be used when stream velocity is low. These have the disadvantage of frequent nonclosure if gravel is present in the sample.

(2) The Engineer Research and Development Center (ERDC) has developed the Ellis Sampler shown in Figure 4-13. This sampler uses a lead fish to deliver the drag bucket to the river bottom. The drag bucket is mounted on a pivot point below the fish, so that when it strikes the bottom, the bucket opens and is filled as the fish is dragged along the bed. Once the fish is lifted off the bottom, the bucket pivots and closes the mouth off so that fine material is not winnowed out as the fish ascends to the surface. Air pressure is applied to the bucket on the back side and the sample is pushed out into a bag or jar. This method returns excellent samples representing the entire size class on the bottom with no visible loss of fine materials. The bucket size can be adjusted for sampling in gravel-bed systems.

Figure 4-13. ERDC-Ellis bed material sampler

c. Core Samplers. When the purpose of the sampling program is to obtain information on the vertical composition of deposits to determine density and compaction, an undisturbed sample is required. These samples are collected using core samplers or piston core samplers that have removable sample-container liners.

(1) Coring deep into sediment generally requires drilling equipment or special pile-driving equipment, which may produce samples that are highly disturbed or compacted. Several deep-core samplers are described in American Society for Testing and Materials (ASTM) Standard D-4823 (published annually), and ASCE Manual No. 54 (Vanoni 1975, 2006).

(2) Samplers for obtaining short cores in shallow water in gravel- or cobble-bed streams are described in ASTM Standard D-4823 (ASTM, published annually). These include a barrel sampler with a serrated cutting edge that is driven into the bed. Once the sampler is in place, sediment is excavated, by hand, layer by layer.

d. Acoustic Techniques. Recent advances in hydroacoustics have resulted in the development of geophysical methods to assess the characteristics of bottom and sub-bottom sediments. Specifically, the engineering properties of sediments (density, mean grain size, soil classification, etc.) have been empirically related to the measured acoustic impedance of different sediment types. McGee et al. (1995) presents a detailed discussion of the application of acoustical techniques for the assessment of in situ sediment properties. Ground-truthing of acoustic techniques is recommended.

e. Visual Techniques. Digital photography offers a rapid means of computing grain size gradation. Buscombe (2013) documents an efficient method that was implemented into an online tool. Updates to available online grain size calculator tools are frequent and the practicing engineer should perform a thorough search to select appropriate software. These techniques have their limitations, and the practicing engineer must understand where these photographic techniques are applicable.

4-9. Standard Sampling Procedures – General.

a. Where and how a sampler is deployed in the field is as important as the selection of a sampler and its fundamental design. Information on the proper use of FISP-approved samplers can be found in the following references:

(1) Operator manuals on FISP website.

(2) Edwards and Glysson (1999) for samplers developed before 1988.

(3) Davis (2005) for samplers developed before 2005.

(4) Other references on the FISP website.

(5) ASTM Standard D-4411 (published annually).

b. The following sections outline a brief summary of standard sampling procedures.

c. Depth Integration. The procedure for collecting depth-integrated samples is to lower the sampler to the water surface so that the nozzle is out of the water and the tail vane is in the water, until the sampler is properly aligned with the flow. Depth integration is achieved by lowering the sampler to the streambed at a uniform transit rate and then immediately raising the sampler at a uniform rate until the nozzle clears the water surface.

(1) Each transit must be at a uniform rate, but the raising and lowering transits may be at different rates. To minimize the effect of nonhorizontal flow entering the nozzle, transit rates should not exceed four-tenths of the mean velocity. Other factors may limit the transit rate to significantly lower values. Transit depths are limited by the rate of air compression in the sample bottle. In addition, transit rates should be such that at the end of sampling, the sample bottle is about two-thirds full. If the bottle is overfilled (filled to within 1.5 in. of the top), the sample should be discarded.

(2) Graphs for determining transit rates as a function of nozzle diameter, mean velocity, and depth of integration are provided in Edwards and Glysson (1999). When the stream is shallow or the velocity is low, several transits may be made to obtain the appropriate sample volume, and several sample verticals may be included in a single sample bottle.

(3) Single Station Vertical. Streams with a stable cross section and insignificant lateral variation in the suspended-sediment load may be sampled using a single vertical, rather than the typical sampling strategy of multiple verticals.

(a) The same vertical is usually used for all discharges. The best location for the single vertical is determined by trial when the station is established. Detailed sediment discharge measurements using several verticals across the entire width of the stream at a range of discharges must be conducted at a new gaging site to determine the location of the most appropriate single vertical sampling point. The vertical should be located at least 10 feet from any supporting pier.

(b) The results of the fixed vertical should be compared with frequent cross-sectional sampling to verify an adjustment factor for the total sediment concentration. This adjustment factor should especially be checked after major flood flows that alter the channel shape.

(4) Multiple Station Verticals, Equal Discharge Increments (EDI) and Equal Width Increments (EWI). Lateral variation in depth, velocity, roughness, and grain size may make it unrealistic to relate sediment concentration for the entire cross section to concentration at a single vertical.

(a) A realistic sampling program may require sampling at two to five or more verticals. Verticals may be located by one of two methods: the method of the centroids-of-EDI across the stream, where the channel cross-sectional area is divided laterally into a series of subsections, each of which conveys the same water discharge; or the method of equally spaced verticals across the stream and an EWI at all verticals (sometimes referred to as equal-transit-rate (ETR)). The EDI method is usually limited to streams with stable channels where discharge ratings change very little during a year.

(b) The EWI method is most often used in shallow and/or sand-bed streams where lateral flow distribution is unstable. On the order of 20 verticals are usually ample for the EWI method. A nomograph to determine the number of sampling verticals is required to obtain results within an acceptable relative standard error based on the percentage of sand in the sample, the average velocity, and the depth is given in Edwards and Glysson (1988, p 68). The EDI method requires some knowledge of the streamflow distribution before the sampling verticals can be selected, but this method can save time and labor over the EWI method, especially on larger streams because fewer verticals are required.

(c) Samples collected using the EDI method may be composited to obtain total concentration if sample bottles contain equal, or nearly equal, quantities of sample. Samples collected using the EWI method can be composited regardless of the volume in each sample.

d. Point Integration.

(1) Point-integrating samplers provide additional options for sampling not available with the simpler depth-integrating types. They can be used to collect a suspended-sediment sample representing the mean sediment concentration at any point from the surface of a stream to within several centimeters of the bed, as well as to integrate over a range in depth.

(2) Point-integrating samplers are used in streams where the combination of depth and velocity cause the sample bottle to overfill at the maximum allowable transit rate (during a continuous round-trip integration with a depth-integrated sampler) (Gray et al., 2010). Both the EWI and EDI methods are applicable to point-integrating samplers when they are used for depth integration. Stream depth increments up to 30 feet can be measured with point-integrating samplers by integrating the depth in only one direction. When depth integration is used in only one direction, at least two samples should be taken and composited at each vertical: one by downward integration and one by upward integration.

(3) Point-integrating samplers are sometimes used to obtain sample concentrations at several points or levels in the vertical from which the distribution of sediment concentration in the vertical can be computed. This method is slower and more labor-intensive than depth integration and should be reserved for special studies.

(4) The advantage of using point samples in the vertical is that the data (when taken simultaneously with ADCP acoustic backscatter data) can be calibrated to suspended-sediment concentrations. ERDC has developed a methodology for calibrating point suspended-sediment sample for total suspended material analysis and laser diffraction particle size analysis in conjunction with acoustic backscatter data (Sharp et al., 2013).

4-10. Bedload Measurement.

a. The quantification of bedload transport rates can be accomplished by physical sampling to measure the mass rate of transport per time or by (typically indirect) methods to measure the volume change per time. Typically, smaller streams and rivers are more amenable to direct, physically sampling, while larger rivers are more amenable to indirect volumetric methods.

b. No single bedload transport relation has been shown to have general applicability, which highlights the importance of making direct measurements. These methods are challenging for a number of reasons: (1) size and quantity of bedload varies greatly in space and time, (2) collecting enough samples to get an average transport rate, and (3) difficulty in deploying the sampler during high flows.

c. Gomez (2006) observed that the collection of high-quality bedload transport data with physical samplers is an expensive and time-consuming task. Gray and Simões (2008) present a number of factors that impinge on the reliability of estimates from such bedload transport formulas in three categories: data issues, sediment supply issues, and other technical issues. They concluded that using formulas to estimate bedload transport rates, particularly in gravel-bed rivers, remains problematic and is the focus of ongoing research.

d. Bedload Physical Sampling Procedure. Bedload can move sporadically or continuously as a series of bed forms of various sizes or as sheetflow, and usually does so with significant lateral variability across the stream.

(1) Due to the significant temporal and spatial variation in bedload transport, when using any of the various types of mechanical samplers, many repetitive measurements must be made at a number of different lateral locations. Initially, the cross section should be divided into 10 to 20 sampling verticals or bins. The sampling sequence must be long enough to include the passage of several bed forms to account for the temporal variation in transport rate.

(2) Consideration must be given to the variation in hydraulic forces through a reach that may cause certain size classes to move primarily as bedload in one reach, but as suspended load in another reach. Thus extensive sampling must be made over the entire range of stream discharges to obtain a reliable bedload transport rating curve.

(3) The suggested technique for bedload sampling is to sample at 20 lateral locations initially to define the active bedload transport zone, then sample at 10 or more lateral locations within that zone on subsequent transects. At least four transects should be taken. If it is apparent that temporal variations are more significant than spatial variations, then a smaller number of lateral locations may be sampled (about five), but many replications at each location should be conducted.

e. Bedload Surrogate Technologies. Sustained, international research and development is underway on developing advanced surrogate technologies for use in bedload monitoring with methods that may be categorized as either active or passive sensor technology. Bedload surrogate technologies are still in the development phase and recent literature should be consulted.

(1) Active sensor surrogate technologies include devices that sense, either by light or sound, characteristics of the riverbed to produce estimates of sediment motion. A number of active sensor devices are in development for use as bedload surrogate technologies. These include ADCP, active sonar, radar, and “smart” tracers (Gray et al., 2010).

(2) Passive sensor surrogate technologies rely on natural signals to produce estimates of sediment motion. The advantage of passive sensors is that the monitoring system uses the active nature of bedload transport to report the unknown variables in a way that can easily be recorded. Passive sensors include geophones (impact pipes, plates, and columns with the respective sensors: microphones, accelerometers, piezoelectric sensors), hydrophones, and magnetic detectors. These sensors are used either in stand-alone mode, such as a hydrophone recording the acoustic energy of rock collisions, or in combination with an impact device, such as an air-filled pipe or plate on the riverbed (Gray et al., 2010).

f. Bedload Volumetric Measurement. In large rivers, physical sampling is impractical, and may yield unreliable results. In the mid-1990s, advances in hydroacoustic equipment enabled the acquisition of accurate and detailed 3D bathymetric data from moving vessels. Together with improvements in computer speeds and memory, these technology advancements opened new possibilities for measuring the volumetric transport rate by tracking either the size and migration or the volumetric rate of scour or deposition of individual dunes and dune fields.

(1) The practitioner should be aware that these different methodologies may give disparate results, and that the results may not be consistent with historical physical measurements.

(2) The practitioner must check the current status of development and understand the limitations of these new methodologies and potential sources of error.

(3) Different computational methods can result in significant differences in results, even when the same data set is used.

g. Bedload Field Measurement Using ISSDOTv2.

(1) As mentioned previously, the ISSDOTv2 is a method developed at ERDC for computing bedload transported by dune movement in large sand-bed rivers. This method relates the scour computed from time-sequenced 3D bathymetric data to the average transport in a sand dune. When multiple swaths cover a river from bank to bank, the values for each swath can be summed to provide the total bedload transport at a section.

(2) Abraham et al. (2011) validated ISSDOTv2 in a flume study. Field applications show consistent results (Abraham et al., 2016) and indicate that ISSDOTv2 quantifies not just the bedload transport at a given river section, but also its lateral variation (Abraham et al., 2015). With sufficient measurements, bedload sediment rating curves can be developed as shown on the Missouri River (Abraham et al., 2017). Shelley et al. (2013) provides a procedure for removing a source of systemic error. Abraham et al. (2016) compares ISSDOTv2 measurements to the Meyer-Peter and Muller and Toffaleti bedload predictors. Proper data collection and analysis procedures are critical to obtaining reliable data. Refer to Case Study 4A (Appendix N) application of the ISSDOTv2 method.

(3) Nittrouer, Allision, and Campanella (2008) present a very similar method to ISSDOTv2 for computing bedload transport. Three key differences from ISSDOTv2 are:

(a) The depositional volume rather than the scour volume is used to compute transport.

(b) The method assumes that the deposition volume per time is the downstream transport rate. However, by evaluating geometry for triangular waves, it can be shown that the deposition (or scour) rate is twice the downstream transport rate (Shelley et al., 2013). For this reason, caution is needed when using data processed with the Nittrouer et al. (2008) method.

(c) The method adjusts for systemic underprediction by assuming a sinusoidal wave shape and extrapolating missing area. In contrast, the ISSDOTv2 method uses the missing area rate of growth computed from additional passes for extrapolation.

(4) Evaluation of field bedload measurement techniques should consider the above-stated issues when evaluating new methods and applications.

4-11. Bed Material Sampling.

a. General. Deposited sediment is sampled to provide information on such things as size, specific gravity, shape, and mineralogy of the particles that make up the bed; stratigraphy, density, and compaction of the deposits; and the quantity and distribution of contaminants. For some of these purposes, a sample can be disturbed; others require undisturbed sampling. Different samplers and sampling procedures are available for different environments. Limited information for bed material sampling is provided below. Appendix E provides additional information for sampling bed material that was adapted from that previously reported by Copeland et al. (2001). Bunte and Abt (2001) provide detailed guidelines for sampling surface and subsurface particles.

b. Sediment Transport Studies. Streambed samples are used to determine the potential for sediment transport. For this purpose, undisturbed samples are not required. When sampling for sediment transport studies, do not sample over long distances. Instead, collect samples along cross sections to characterize the reach. Then proceed to the next sampling cross section and repeat the procedure.

c. Samples from Dry Beds. Sampling in the dry is preferred because there is less opportunity for fine-size classes to be lost from the sample during collection. Samples from dry beds are typically collected with a shovel or scoop. If there is an obvious layer of fine material on the surface of a dry bed, this should be removed before the sample is taken.

d. Samples from Streams with Flowing Water. To obtain satisfactory samples in flowing water, the bed sampler should enclose a volume of the bed material and then isolate the sample from the water currents while the sampler is being lifted to the surface.

(1) The sampler should disturb the flow field as little as possible while taking a sample. These criteria are met with standardized FISP US BM-54 and US BMH-60 samplers. Under certain flow conditions, simple drag bucket and pipe samplers have been shown to produce bed gradations similar to those obtained with the US BM-54.

(2) A comparison with standardized samplers should be conducted for each case. Open-ended drag bucket and pipe samplers are typically used from a boat. One technique is to lower the sampler to the bed and allow the boat to drift with the current. The sample is dredged up as the boat moves downstream. As the boat continues to drift, the sampler is hoisted back to the surface.

e. Sampling Surface Layers. Bed material particle size variation with defined layers exist in many stream systems. Representative samples can usually be taken from the upper few inches of the bed surface in sand-bed streams. However, vertical bed coarsening can be a significant factor in limiting degradation during flow events that is critical to capture when applying numerical models. In coarse bed streams, separate samples of the armored layer and the subsurface layers should be collected. The sample depth and quantity for the subsurface depends on the size of sediment and the equipment being used. It is generally necessary to obtain separate gradations of both the coarse surface layer (the armor layer) and the subsurface layer.

f. Lateral Variations. Lateral variation in the bed gradation is significant, especially in sand and gravel-bed streams and at channel bends. Lateral variation in gradation can be much more significant than longitudinal. The section should be divided into at least three sample locations taken across the cross section to account for lateral variations. In streams with variable depths, more samples are required.

(1) Taking bed samples at crossings where flow distribution is typically more uniform reduces the lateral variation in the samples. However, at low flow, crossings may become coarser than the average gradation and should not be selected as a sampling location for sediment transport studies. This is especially true of steep streams that develop riffle and pool planforms.

(2) Samples collected on point bars or alternate bars may exhibit considerable variation. Figure 4-14 shows a typical bed gradation pattern on a point bar. Note that although the typical grain sizes found on the bar surface form a pattern that varies from coarse to fine, there is no one location that always captures the precise distribution to represent the entire range of processes in the field.

(3) There is no single rule for locating sampling sites. The general rule is "always seek representative samples." That is, carefully select sampling locations and avoid anomalies that would bias either the calculated sediment discharge or the calculated bed stability against erosion. A good practice is to take samples at a crossing and at a point or alternate bar just above the low water level to establish a range of uncertainty for the bed gradation. Dead water areas behind sandbars or bridges should be avoided.

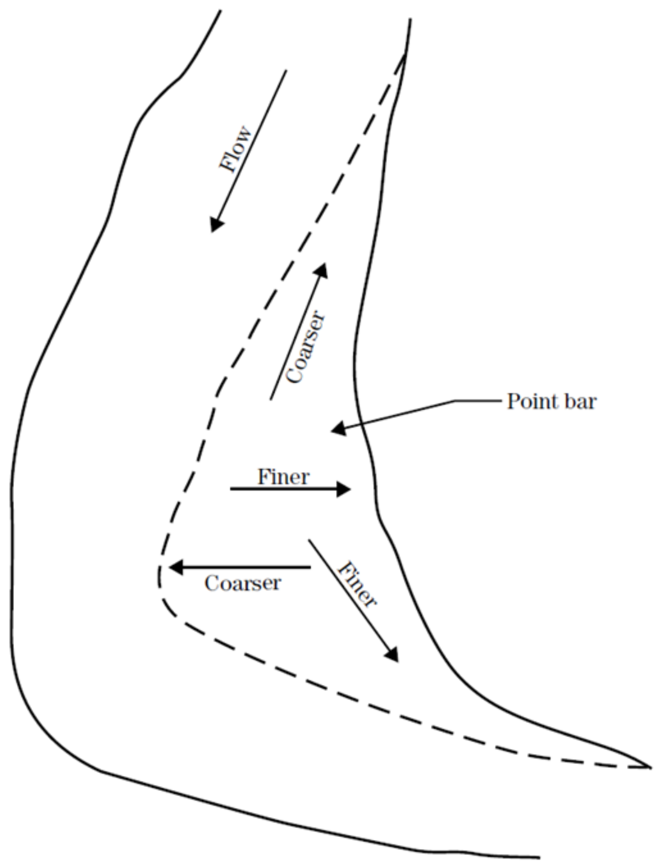


Figure 4-14. Gradation pattern on a bar (Copeland et al., 2001)

g. Coarse Beds (Pebble Count).

(1) When bed particle size is too large to obtain a manageable quantity of sample for sieve analysis, a pebble count (Wolman 1954, Bunte and Abt 2001, Kondolf et al., 2003, USDA 2007b) may be conducted where individual particles are collected at random by hand and the intermediate (b) axis is measured. This method requires that the stream be dry or wadeable. At

least 100 particles should be included in the sample. One method for choosing the particles is a random walk laterally across the stream or longitudinally along a point bar. Another is to set up a grid and measure particles at the intersection of grid points. Further detail on performing a field pebble count is provided in Appendix E.

(2) The gradation curve developed from these data is based on the number of particles in each size class, not their weights. Conversions are given in Chapter 5 of ASCE Manual No. 110 (Gray et al., 2008). This reference also describes sources of error in detail. One significant source of error in the pebble count method is the truncation of smaller size particles. Daniels and McCusker (2010) and Bunte and Abt (2001) provide a discussion on bias when performing pebble counts.

4-12. Suspended-Sediment Sampling in Lakes, Reservoirs, and Estuaries. Sediment measurement in low-velocity environments requires different equipment and techniques than in streams. As flow velocity approaches zero, movement, if any, results from complex circulation patterns, density currents, or tidal flow. Cross-sectional areas are usually very large; and instantaneous water discharges are rarely known. Projects in these environments will require a detailed sampling assessment plan to meet objectives.

a. Sampling Techniques. Sampling techniques need to be evaluated for accuracy and pertinence to the objective of the sampling program. Most samplers used in low-velocity environments are point or trap samplers that are oriented vertically and do not sample isokinetically. In lakes and reservoirs, sampling in the different thermally stratified zones is generally required. Frequently, samples are collected using pumping samplers. Due to continuous changes in sediment concentration in estuaries, neither of the previously discussed EDI or EWI methods that are used for sampling in flowing rivers are appropriate. General practice is to sample continuously through a tidal cycle at a number of locations to define temporal variation at each location.

b. In Situ Fall Velocity Measurement Methods. The Particle Imaging Camera System (PICS) is a fall velocity approach to sampling. In situ fall velocity measurements for flocs and individual grains can be achieved with the PICS, which is an ERDC-developed system for in situ measurements of cohesive sediment settling velocities.

(1) PICS collects digital video of particle settling within a small settling column and can be deployed in the water column or in a laboratory setting. Sample collection, optical and lighting design, and image acquisition were designed to produce high-quality, in situ image sequences. PICS consists of a 1-m long, 5-cm inner diameter settling column with a megapixel digital video camera and strobe light emitting diode (LED) lighting. The settling column is equipped with two pneumatically controlled ball valves at the column ends that permit sample capture, and a third pneumatic actuator for rotating the column from horizontal to vertical orientation for image acquisition.

(2) Image sequences collected by PICS are analyzed with automated particle tracking software to produce size, settling velocity, and density (inferred) distributions of particles suspended at the sampling location.

(3) Additional instrumentation may be deployed with PICS, such as Laser In Situ Scattering and Transmissometry (LISST-floc), Conductivity-Temperature-Depth (CTD), Acoustic Doppler Velocimeter (ADV), and pumping sampler. Smith and Friedrichs (2011) provide additional details of PICS, including system configuration and measurement uncertainty.

4-13. Laboratory Analysis of Suspended-Sediment Concentration. Evaporation and filtration are the two most frequently used methods for determining the sediment concentration.

a. The filtration method is faster if the sediment concentration in the sample is small and/or relatively coarse-grained. In addition, if the quantity of sediment is small, the evaporation method requires a correction if the dissolved-solids concentration is high. The filtration method can be used only on samples containing sand concentrations less than about 10,000 mg/l and clay concentrations less than about 200 mg/l. The sediment need not be settleable because filters are used to separate water from the sediment.

b. The evaporation method is usually best for high concentrations of sediment (>2,000 mg/l), such as those encountered in many arid-region streams. The wet-sieving filtration test method covers concentration measurements of two particle size fractions. The term fine fraction refers to particles small enough to pass through a sieve with 62 or 63-µm apertures; coarse fraction refers to particles large enough to be retained on the sieve. The fine fraction need not be settleable. This test method is useful when large samples must be collected in the field, but only small subsamples, typically 300 to 500 mL, can be shipped back to the laboratory.

c. The USGS follows guidance for selection of a particle size analysis as given in ASTM Standard D-3977. Suspended-sediment concentration (SSC) analysis is the approved method used in all USGS sediment laboratories in which the entire sample is analyzed. Total suspended solids (TSS) is a method used outside the USGS. This method requires the sample to be shaken, an aliquot taken, and only the aliquot analyzed, not the entire sample. If samples have heavy sands, there can be a misrepresentation in the data results (Office of Water Quality Technical Memorandum 2001). Laboratory procedures are well documented in ASCE Manual No. 54 (Vanoni 1975, 2006); Guy 1969; Knott et al., 1993).

4-14. Erodibility Testing. Cohesive sediment erosion differs significantly from coarse-grained, noncohesive (sand fraction and coarser) erosion phenomena. Noncohesive sediment erosion can generally be quantified based on applied shear stress and grain size distribution. Cohesive sediment erosion is related to these factors and to sediment bulk properties, including bulk density, mineralogy, pore water chemistry, and organic content. Although it is known qualitatively how many of these parameters may affect erosion, a priori quantitative methods of relating cohesive sediment erosion to these bulk properties are not available. Measurement methods have been developed to determine a site-specific relationship between erosion rate and shear stress.

a. Sediment Flume.

(1) Several instruments are available to measure cohesive sediment erosion using extracted cores: sediment flume (SEDflume) (McNeil 1996) has been applied as part of several

USACE dredging, navigation, and contaminated sediment/water quality studies to measure critical shear stress and erosion rates of sediments as a function of depth. The similar USACE SEDflume was used by ERDC for an extensive investigation on the Passiac River, New Jersey (Borrowman et al., 2006). SEDflume is designed to measure erosion rates for cohesive sediments in a laboratory environment. For SEDflume analysis, sediment samples (cores) are extracted from the bed of the reservoir. Great care is taken to minimize disturbance of the sediments in the core because this could alter erosion characteristics. Figure 4-15 shows a schematic for SEDflume.

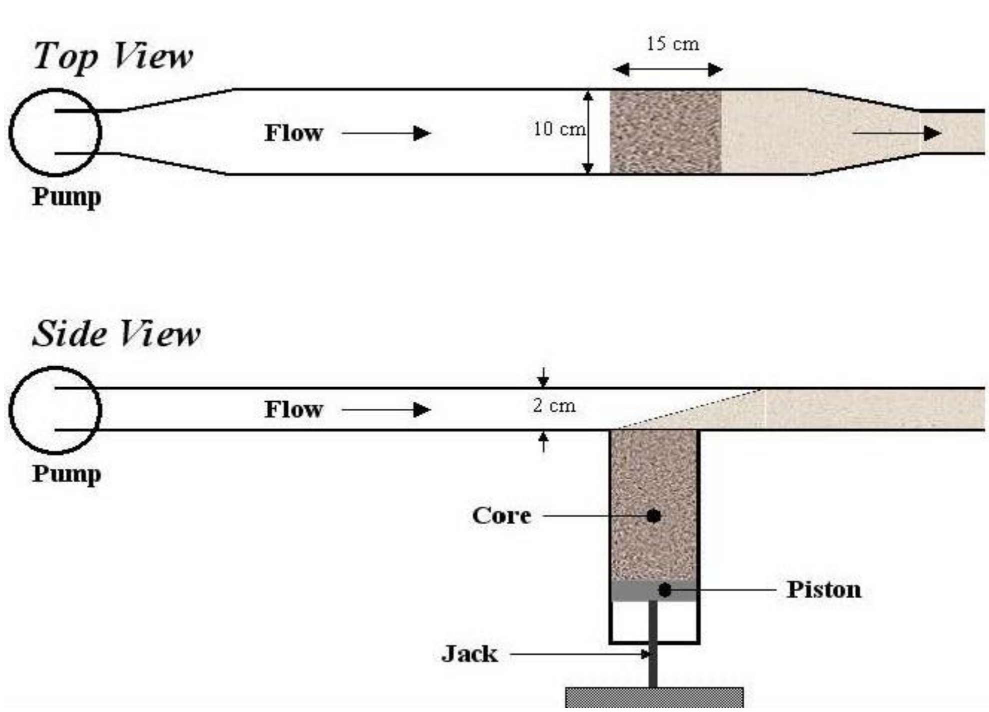


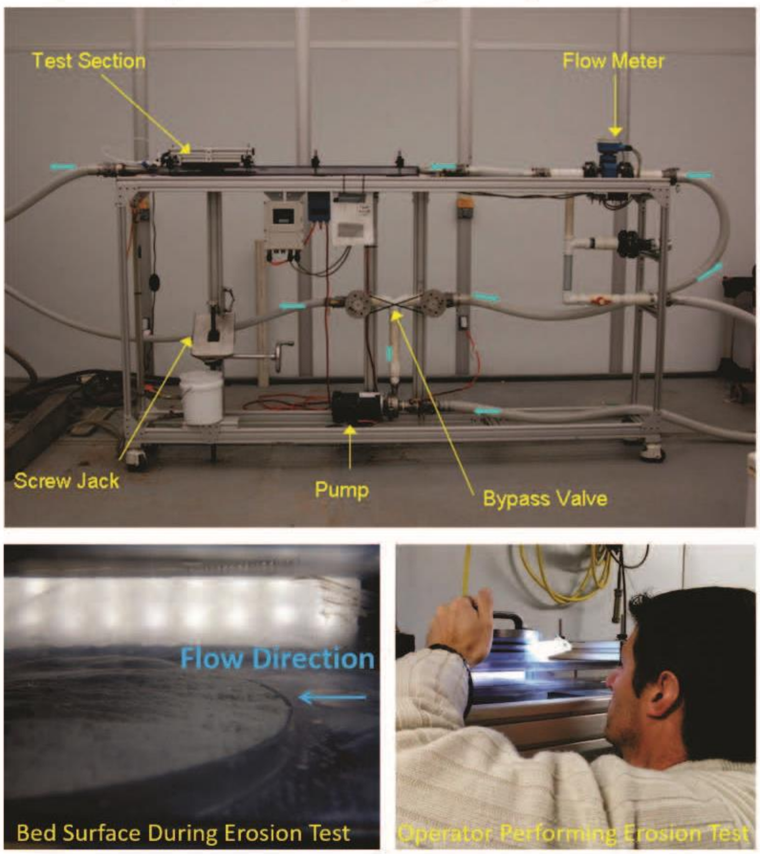
Figure 4-15. SEDflume apparatus schematic (Borrowman et al., 2006)

(2) The USACE SEDflume has been used to conduct erodibility testing at USACE projects. For example, SEDflume test data were collected at Lewis and Clark Lake on the Missouri River throughout the reservoir delta. Collecting test data in older areas (such as 50 years or more), and newly deposited (less than 5 years), can provide informative sediment properties. At Lewis and Clark Lake, results were used to evaluate erosion of deposited delta sediments during a theoretical drawdown flush. The SEDflume apparatus (Figure 4-16) has been used at multiple locations (Perkey et al., 2020; Smith et al., 2020).

(3) Roberts et al. (2001) at Sandi National Laboratories modified the SEDflume design to further test the transport mode and ratio of suspended load to bedload in the eroded sediment. Their apparatus was named the ASSET (Adjustable Shear Stress Erosion and Transport) flume. Briaud et al. (2001) at Texas A&M University developed an erodibility measurement device called the EFA (Erosion Function Apparatus). The mechanisms for these devices are similar. A cored or reconstructed (lab-created) sample is extruded into the test section at a controlled rate as

flow is passed over the to measure erodibility. Since the sample is introduced into the test section as a plane bed section, the effect of bed forms is not accounted for in the testing.

(4) Figure 4-17 shows the relative erodibility of different size fractions under similar velocities (Briaud et al., 2001). At a cross-flow of 1 m/sec, the clean coarse sand sample eroded at 10 meters per hour. The core from a 13 m depth on the Brazos River had a scour rate of 50 millimeters per hour, and the clay sample from the Trinity River was still stable at the 1 m per second velocity. Erosion curves can be concave, straight, or convex, depending on the erosion process at work.



Shear Stress (Pa)	Flow Rate (GPM)
0.1	5.2
0.2	7.9
0.4	11.5
0.8	17.8
1.5	25.8
1.0	20.3
1.5	25.8
2.0	30.6
2.5	34.9
3.0	38.8
4.0	46.0
5.0	52.4
6.0	58.3
8.0	69.1
10.0	78.8
12.0	87.7

Figure 4-16. USACE SEDflume and standardized shear stress and flow rate relationship (Perkey et al., 2020)

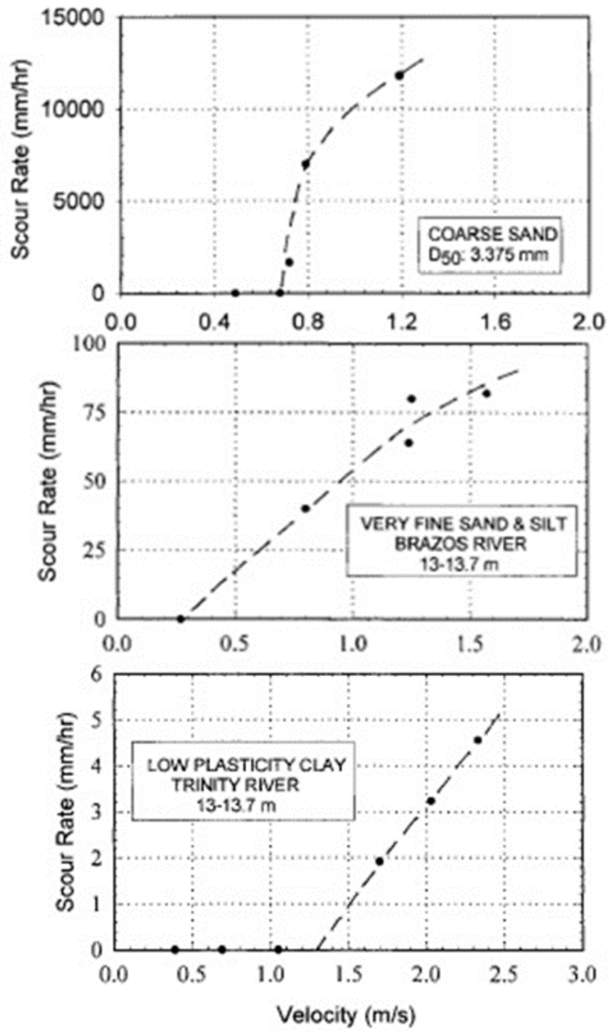


Figure 4-17. Comparison of erosion rates for sands, silt, and clays (from Briaud et al., 2001)

(5) SEDflume cores collected from the upper region of Lewis and Clark Lake exhibited layering that was generally associated with grain size and vegetation. This layering was likely driven by event-specific inflows of fine sediments. In some instances, layers of vegetation were dense enough to prohibit the ability to observe sediment erosion. It was found that the cores collected from the berms and shoals in the upper reservoir were dominated by sandy sediments. Thick (≥10 cm) lenses of silt and vegetation were commonly encountered throughout the cores. Critical shear stress for erosion increased significantly in these regions of the cores (Perkey et al., 2020).

(6) The results of the analysis were summarized and used in HEC-RAS analysis of reservoir management options. Table 4-1 summarizes the shear and erosion rates from the SEDflume samples that can be used to define bed material in HEC-RAS reservoir modeling.

Table 4-1
Erosion Parameterization for Use in HEC-RAS Numerical Modeling (Perkey et al., 2013)

Cohesive Sediment Erosion Parameterization for HEC-RAS								
	Shear Threshold		**Erosion Rate**		**Mass Wasting Threshold**		**Mass Wasting Rate**	
Layer ID	Pa	lb/ft^2	kg/m^2/s	lb/ft^2/hr	Pa	lb/ft^2	kg/m^2/s	lb/ft^2/hr
ETop-L1	0.20	0.0042	0.0357	26.3	0.40	0.0084	0.418	308
ETop-L2	0.20	0.0042	0.0649	47.9	0.40	0.0084	0.399	294
ETop-L1&2	0.20	0.0042	0.0555	40.9	0.40	0.0084	0.406	299
G-L0	0.19	0.0040	0.0503	37.1	3.0	0.063	0	0
G-L1	0.40	0.0084	0.1049	77.3	3.0	0.063	0	0
G-L2	1.49	0.0311	0.0298	22.0	12.0	0.251	0	0
G-L3	2.99	0.0625	0.0721	53.2	12.0	0.251	0	0
G-L4	0.40	0.0084	0.0431	31.8	3.2	0.067	0	0
I_L0	0.39	0.0082	0.0826	60.9	2.0	0.042	0	0
I_L1	0.35	0.0073	0.0850	62.7	2.0	0.042	0	0
I_L0&1	0.39	0.0082	0.0968	71.4	2.0	0.042	0	0
I_L2	0.25	0.0052	0.0774	57.1	2.0	0.042	0	0
I-All	0.34	0.0071	0.0881	65.0	2.0	0.042	0	0
J-L0	0.19	0.0040	0.0291	21.5	3.0	0.063	0	0
J-L1	0.70	0.0146	0.1600	118.0	3.0	0.063	0	0
J-L3	1.60	0.0334	0.1090	80.4	10.0	0.209	0	0
J-L4	0.64	0.0134	0.0052	3.83	15.0	0.314	0	0
J-L5	2.90	0.0606	0.0903	66.6	20.0	0.418	0	0
J-L6	0.66	0.0138	0.0726	53.5	6.0	0.125	0	0
J-L7	0.33	0.0069	0.1480	109.1	3.0	0.063	0	0
K-L0	0.40	0.0084	0.2900	214	2.0	0.042	0	0
K-L1	0.76	0.0159	0.0351	25.9	9.0	0.188	0	0
K-L2	1.60	0.0334	0.0517	38.1	12.0	0.251	0	0
K-L3	2.90	0.0606	0.0762	56.2	15.0	0.314	0	0
K-L4	Dense Vegetation							
K-L5	1.19	0.0249	0.0622	45.9	9.0	0.188	0	0
K-L6	2.68	0.0560	0.0675	49.8	12.0	0.251	0	0
K-L7	1.50	0.0314	0.0839	61.9	9.0	0.188	0	0

b. Jet Erosion Test Testing.

(1) A jet erosion test (JET) impinges a jet of water on the soil bottom/surface, which scours a small hole into the soil surface. After a short period of time, the jet is shut off and the depth of the hole is measured. The applied shear stress is calculated from the scour depth, available head, nozzle dimensions, and frictional losses as the jet travels through the water to the bottom of the scour hole. The jet of water is then allowed to continue scouring the soil material for another short time interval. As the scour hole deepens, frictional losses increase, the rate of erosion decreases, and longer jetting times are needed to register measurable change. Hanson and Simon (2001) reported application in the Loess Hills. Refer to Case Study 4B (Appendix N) for an example jet test conducted on the Missouri River.

(2) The JET developed for testing of materials in the laboratory and in situ (ASTM 1995, Hanson 1990).

(a) The JET testing differs from SEDflume in that it can be accomplished in the field for non-submerged sediment cores. In the case of submerged sediment cores, neither methodology can be completed in situ. Hanson (1991) developed a soil-dependent jet index based on the change in maximum scour depth caused by an impinging jet. This development included an empirical relationship between the jet index and erosion. In an attempt to remove empiricism and to obtain direct measurements of the excess stress parameters, Hanson and Cook (1997) developed analytical procedures for determining soil k_d based on the diffusion principles of a submerged circular jet and the corresponding scour.

(b) Caution should be applied when using data collected with the JET. This method applies a fluid stress normal to the sample. Most environmental flows will be tangential to this surface, not normal. Moreover, the results from a JET and those from a SEDflume are not comparable. Lastly, the ASTM for jet testing was withdrawn in 2016. Despite its limitations, JET is still used due to its simplicity and low cost.

(3) The results of erosion testing by either SEDflume or JET can be a useful indicator of possible changes in river and reservoir morphology and are often useful in predicting the erosion and resulting sediment transport with flow changes.

4-15. Particle Size Analysis.

a. Sediment particles range greatly in size, from grains of clay too small to be distinguished with the naked eye, to boulders the size of a car. Given this wide variation, it should come as no surprise that different particle size analysis methods are appropriate for different grain sizes. Only the most common techniques are discussed below.

b. Before conducting a study on sedimentation processes that relies on both historic and newly collected particle size data, verify consistency of data collection and lab testing methods. Variation in particle size due to testing methods has been noted on the Mississippi River by USACE in data collected and analyzed for the Mississippi River Geomorphology and Potamology Program. Particle size analysis methods have been extensively discussed in the literature (Lu et al., 2000; Rodriquez and Uriate 2009; Beuselinck et al., 1998; Buurman et al., 2001; Etzler and Sanderson 1995; and Syvitski 1991, 2010).

c. Recent developments that have augmented standard methods are discussed as newer technologies following the presentation of traditional methods in the following sections. Manual particle counting methods of coarse bed material were previously discussed (paragraph 4-11g).

d. Sediment particles vary not only in size, but in shape and specific gravity. Particles of a given size will behave as if they were larger or smaller, depending on how their shape and specific gravity compare with standard values. Due to the wide range in sediment characteristics, particle size is defined in terms of the method of analysis used to determine the size. Methods for determining sediment gradations are grouped into fine sediment methods and coarse sediment methods.

(1) The most commonly used methods for determining the gradation of fine sediment are the hydrometer, the bottom withdrawal tube, and the pipet. The X-ray method is a new method for determining fine sediment gradation.

(2) Two generally accepted methods for determining the size distribution of sand are the sieve and visual accumulation tube methods. The sieve method measures physical diameter, whereas all other methods measure sedimentation diameter.

e. A given sediment sample may require more than one method of analysis because of the broad range of particle sizes. In some instances, USACE offices may choose to perform only sieve analysis and ignore determination of gradation of the silt and clay size fractions. This decision should be based on a thorough understanding of study needs. For instance, silt and clay size fractions are an important factor in determining the susceptibility of fine-grained soils to frost action, which can be critical when evaluating bank failure mechanisms.

f. Table 4-2 lists recommended quantities of sediment sample, the desirable range in concentration, and the recommended particle size range for the most frequently used methods of particle-size analysis. Additional guidance for selection of a particle size analysis is given in ASTM Standard D-4822 (published annually) and for sieve analysis in ASTM D-6913.

(1) Many suspended-sediment samples will not contain sufficient sediment for any of these methods, in which case the analysis may be limited to simply determining the percentage of sands and fines. In these instances, the laser diffraction method is very valuable for obtaining a particle size distribution.

(2) A greater quantity of sediment may be obtained by using larger bottles in samplers or by compositing samples. Sometimes samples require splitting to obtain a reasonable quantity for analysis. Splitting should be done with care prevent sample bias. This is particularly a concern when the sample contains sand.

Table 4-2
Recommended Quantities for Particle Size Analysis

Method	Size Range, mm	Analysis Concentration, mg/l	Quantity of Sediment, grams
Sieve	0.062–64	–	0.07–64,000*
Visual accumulation (VA) tube	0.062–2.0	–	0.05–15.0
Pipet	0.002–0.062	2,000–5,000	1.0–5.0
Bottom withdrawal (BW) tube	0.002–0.062	1,000–3,000	0.5–1.8
Hydrometer	0.002–0.062	40,000	30.0–50.0

*See ASTM for limitations.

g. Laboratory Selection. Laboratory testing should be performed only at accredited facilities as determined and granted according to ER 1110-1-261 defining the requirement for the quality assurance of testing procedures and made by the authority of ER 1110-1-8100. The materials testing center (MTC) sediment laboratory located at the ERDC is the only agency authorized to validate commercial laboratories to work for USACE in the continental United States. Coordination with the lab before collecting field samples is recommended.

h. Hydrometer Method. Laboratory procedures for conduction of the hydrometer method are contained in EM 1110-2-1906. This method has been used extensively in the study of soils. Additional guidance for laboratory analysis of fine-grained material, silts, and clays, is given in ASTM Standard D-7928.

(1) Hydrometer analysis of an individual sample is generally more labor intensive and costly than sieve testing.

(2) The sedimentation or hydrometer method is used to determine the particle size distribution (gradation) of the material that is finer than the No. 200 (75-µm) sieve and larger than about 0.2-µm. The test is performed on material passing the No. 10 (2.0-mm) or finer sieve and the results are presented as the mass percent finer vs. the log of the particle diameter.

(3) The hydrometer method can be used to evaluate the fine-grained fraction of a soil with a wide range of particle sizes by combining the sedimentation results with a sieve analysis resulting in the complete gradation curve. The method is generally used when there are no coarse-grained particles or when the gradation of the coarse-grained material is not required or not needed.

(4) Consult recent developments for technologies that are supplanting the hydrometer test. Hydrometer analyses may still be needed when comparison and trends from older data is critical.

(5) Analysis of laboratory testing data has noted clay fractions settle more slowly than predicted due to particle shape effects and thermal convection. This gives an overestimation of the percent clay fraction and may introduce inconsistent results.

i. Bottom Withdrawal Method. The bottom withdrawal method requires specially constructed and calibrated tubes. It is not used extensively. This method is more accurate for very low concentrations of fine materials than the pipet method; however, it is more time-consuming. The bottom withdrawal method is described in ASCE Manual No. 54 (Vanoni 1975, 2006).

j. Pipet Method. The pipet method is the most routinely used method for fine sediment (clay and silt) analysis. The sample initially is dispersed uniformly throughout the pipet apparatus. Concentrations of the quiescent suspension are determined at predetermined depths and times based on Stokes law. The primary disadvantage with this method is its high labor intensity. The pipet method is described by Vanoni (1975, 2006), and Guy (1969). The pipet procedure yields the required grain size data, but the technique is slow, expensive, and in need of modernization. Similar to the hydrometer, the pipet method may also have concerns with clay fractions.

k. X-Ray Methods. The U.S. Geological Survey has recently approved use of X-ray grain size analyzers to determine fall diameter for clay and silt mixtures. The sample is dispersed uniformly in the instrument, which measures decreasing concentration with time. Cumulative mass percentage distributions are determined automatically.

(1) X-ray analysis requires less time than the pipet method and is therefore less expensive. The X-ray unit is a viable contender for replacing the pipet although comparisons of pipet and X-ray methods have shown that X-ray methods tend to produce slightly finer gradations. Fall diameter measurements are performed automatically.

(2) The unit is considered to have better repeatability than the pipet. Refer to Report OO Pipet and X-Ray Grain Size Analyzers: Comparison of Methods and Basic Data (FISP 2000) for further details. When the X-ray method is employed, duplicate samples on at least 10% of the samples at a site should be taken until a relationship between the X-ray and pipet results can be established (FISP 2000).

l. Sieve Method. Sieve analysis is a relatively simple method for obtaining a gradation for sediment larger than 0.0625 mm. Unfortunately, U.S. standard sieves do not correlate exactly with the AGU size class classification system.

(1) A set of U.S. standard sieves range between 3 in. and 0.074 mm. As discussed in Chapter 3, sediment diameters determined from sieve analysis do not necessarily correspond to equivalent spherical diameters. Sieve analysis does not account for variations in particle shape or specific gravity.

(2) Procedures for application of sieve analyses are found in EM 1110-2-1906 and described in ASTM Standard D-6913. The required sample size is a function of the maximum particle size. The data in Table 4-3 provide a guide for obtaining a minimum-weight sample. Note that for streams with maximum sizes larger than 3 in., the required sample weight should be at least 100 times the weight of the maximum size.

Table 4-3
Sample Size for Sieve Analysis

	Minimum Weight of Sample	
Maximum Particle Size, inches	**grams**	**pounds**
3.0	64,000	140
2.0	19,000	42
1.5	8,000	18
1.0	2,400	5.3
0.75	1,000	2.2
0.5	300	0.66
0.375	150	0.33
0.187	50	0.11
Particle Size Range, mm	–	–
16.0–1.0	20	0.044
2.0–0.25	0.5	0.0011
0.5–0.062	0.07	0.00015

m. Visual Accumulation Method. Typically no longer preferred, the visual accumulation (VA) method was used for much historic data. The VA method is used to determine the fall diameter of sands. Sediment finer than 0.062 mm is removed from the sample and analyzed by either the pipet or bottom withdrawal methods. Particles larger than 2 mm must be removed and measured by sieve analysis. In the VA method, sediment is added at the top of a settling tube and the deposited sediment is stratified according to the settling velocities of the various particles in the mixture. A continuous trace of the deposited sediment at the bottom of the VA tube is produced by the analysis. The VA apparatus and manual may be obtained from the FISP.

n. Newer Technologies.

(1) Newer technologies include optical methods such as laser diffraction and photographic imaging. Laser diffraction methods were reviewed in a workshop (USGS-CUAHSI 2012). The PICS is used with automated image processing routines to directly measure fall velocity and compute gradations (Smith and Friedrichs 2011). These methods are currently under development. Surrogate techniques are reviewed in recent publications and workshop presentations (USGS-CUAHSI (2012); Gray and Gartner (2010)).

(2) Increasingly, laboratory determination of particle size distributions is performed using a combination of sieve analysis and laser diffraction or related technologies.

(a) Typically, a sediment mixture containing sands and finer classes will be sieved to remove and classify the portion of the mixture containing coarse sand or larger particles. Then, the finer portion of the mixture will be classified via laser diffraction or traditional sedimentation methods.

(b) Some care must be exercised in interpretation of the resulting particle size distributions and in comparisons to historical measurements. Eshel (2004) indicated that laser diffraction results would typically yield coarser sizes than would be obtained with conventional sieve analysis. Obtaining a full grain size distribution across all size classes requires use of multiple methodologies. Dinis and Castilho (2012) provide a means of merging particle size distributions as does Gaines and Priestas (2016).

(3) Sieve analysis and other traditional approaches generate a particle size distribution by weight. Laser diffraction and other optical methods generate a particle size distribution by volume.

(a) These distributions may vary significantly if particle-specific gravity varies with particle size in a given sediment mixture. Laser diffraction techniques typically report an equivalent spherical diameter based on the volume of a particle, that is, the nominal diameter.

(b) For non-spherical natural sediments, the nominal particle diameter will usually, but not always, be larger than the sieve size. (This assumes that the b and c axis are nearly equal and that the a axis is larger, such as an ovoid. For particles with a relatively small c axis, such as disc shapes, laser diffraction may underestimate the sieve diameter.) Analytical corrections are feasible if particle shape is measured. While these differences are potentially significant, sieving is a discrete measurement technique that typically offers less resolution than laser diffraction.

o. Laser Diffraction Particle Size Analysis Method.

(1) Laser diffraction particle size analysis can be performed on isokinetically collected samples up to sand size. This method can be performed in the laboratory, and to a limited extent, in the field. The size range for a laboratory-grade instrument is from 0 to 2,000 microns, whereas the field version is from 0 to 800 microns. These methods return percent by volume rather than percent by weight as with the other laboratory techniques. Wen et al. (2002) discusses extensively the pros and cons of this method, as well as the standard hydrometer and sieve techniques. The results of their analysis indicate that the laser diffraction method is by far more repeatable.

(2) However, a recent study performed under the USACE Mississippi Valley Division Potomology Program found significant differences in results (Gaines and Priestas 2013).

(a) The laser diffraction method produced different results than did the pipet method, which is significant when comparing data from different times and analysis methods. Older data analyzed by gravimetric methods yielded different particle size distributions than with laser diffraction methods. Data from the two methods are not directly comparable and create difficulties when attempting to assess changes in particle size distributions over time.

(b) Therefore, when the study intent is to assess change in particle size distribution over time, the use of similar methodology for all datasets should be considered. Gaines (2016) provides a discussion of methods in the context of particle size distribution of bed sediments along the Mississippi River.

(3) To evaluate the consistency of results, the following protocol should be followed when the laser diffraction method is employed: duplicate samples for at least 10% of the samples at a site should be taken (and tested using both methods) until a relationship between laser diffraction method and historic data analysis results can be established. As standard practice, this protocol of duplicate analysis should be considered for use with all new methods that have not been fully tested.

(4) The FISP issued a memorandum in 2013 to allow the use of laser diffraction analyzers for environmental measurement of suspended-sediment volumetric concentration (SSCV) and volumetric particle size distribution (PSDV), with important qualifications (FISP 2013).

4-16. Sediment Discharge Rating Curve Preparation from Measured Data.

a. Success in developing sediment discharge rating curves will depend on the foresight in establishing an adequate sediment measuring program before the need for data. Sediment discharge rating curves are prepared from measured data. Historic data collections were often performed by USACE personnel and were available in USACE publications. Other areas used USGS staff with measurements available in annual USGS Water Resource Publications for each state. Recently, the USGS has greatly expanded the historic information available online.

b. Suspended-sediment measurements to develop these curves must be collected over a range of flow rates and hydrodynamic conditions (rising and falling limbs of the hydrograph, for example). In addition, seasonal variation due to a variety of factors such as bed forms, vegetation, and sediment source may also influence results. During each discharge measurement, surface bed grain size distribution should be measured. Suspended-sediment samples are collected at various elevations in the water column and analyzed for total mass and grain size distribution.

c. Development of sediment discharge rating curves typically starts with field data collection followed by analysis. Steps involved in this process are also reported in this manual in paragraphs 4-5, 6-3, and 9-3. Refer also to Gray and Simões (2008) for a presentation of USGS prescribed methods.

d. Data Analysis of Suspended-Sediment Measurements. The USGS follows a prescribed methodology using a power function to develop suspended-sediment transport curves (see the relationship previously presented, paragraph 3-4).

(1) A thorough discussion of the methodology used by the USGS (which is fundamentally unchanged since the 1940s) is presented by Gray and Simões (2008). Calculated mean daily sediment discharges are available from the USGS at measured gage sites; these are calculated values and should not be used to develop a sediment discharge rating curve.

(2) An example data set retrieved from an online USGS sediment gage, the Missouri River at Omaha, after formatting is shown in Figure 4-18. Note that sieve diameters are reported in columns 7–9 and fall diameters in columns 10–19. Data is presented from multiple years for

comparison. Data retrieved at:
https://nwis.waterdata.usgs.gov/usa/nwis/qwdata/?site_no=06610000

(3) Laboratory analysis will vary by site. For example, at some locations, sieve analyses may be conducted only for samples with low sediment concentrations, where there were insufficient quantities available for VA analyses. For many sample sites, only a fines/sand break will be determined.

e. Separation by Sediment Load Type (Bed Material Load and Wash Load). Sediment discharge rating curves should be prepared for the total measured load and the measured bed material load.

(1) The sediment discharge rating curve for the total measured suspended load can be developed from data in columns 3 and 6 in Figure 4-18 (although a much larger data set is required for a reliable rating curve).

(2) Total suspended-sediment load alone is not sufficient to analyze the sediment discharge characteristics. It is also important to separate the wash load from the bed material load because their transport is governed by different relationships: wash load is dependent on upstream supply, and bed material load is dependent on the availability of the sediment in the streambed.

(3) The size class break between wash load and bed material load is frequently assumed to correspond to the break between sand and silt (0.0625 mm); however, this assumption is not always valid. Bed gradations at the gage site are required to distinguish the wash load from the bed material load. The bed gradation should account for lateral variations across the cross section using an appropriate averaging technique.

(4) Einstein (1950) recommended using only the coarsest 90% of the sampled bed gradation for computations of bed material load. He reasoned that the finest 10% of sediment on the bed was either trapped material or a lag deposit and should not be included in bed material load computations.

(5) Once the division between wash load and bed material load is determined, the percent finer data from the appropriate column in Figure 4-18 can be used with the total concentration in column 5 and the discharge in column 3 to calculate wash load.

(6) If sufficient data are available, separate sediment discharge rating curves should be developed for each size class in the bed material load.

(7) For studies involving inflow to reservoirs, separate sediment discharge rating curves should be developed for each size class in the wash load, too. To accomplish this type of analysis, it is necessary that adequate numbers of particle size analyses are conducted on the collected sediment concentrations. Unfortunately, particle size data are frequently insufficient to develop sediment discharge rating curves. In such cases, a minimum requirement is to develop separate curves for the fines (clays and silts) and the sands.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Date	Time	Discharge Instant. (cfs)	Water Temp. Degrees Celcius	Suspended Sediment Concentration, mg/L	Suspended Sediment Discharge, Tons/day	Suspended Sediment, Sieve Diameter, % Finer 0.0625 mm	Suspended Sediment, Sieve Diameter, % Finer 1 mm	Suspended Sediment, Sieve Diameter, % Finer 2 mm	Suspended Sediment, Fall Diameter, % Finer 0.002 mm	Suspended Sediment, Fall Diameter, % Finer 0.004 mm	Suspended Sediment, Fall Diameter, % Finer 0.008 mm	Suspended Sediment, Fall Diameter, % Finer 0.016 mm	Suspended Sediment, Fall Diameter, % Finer 0.031 mm	Suspended Sediment, Fall Diameter, % Finer 0.0625 mm	Suspended Sediment, Fall Diameter, % Finer 0.125 mm	Suspended Sediment, Fall Diameter, % Finer 0.25 mm	Suspended Sediment, Fall Diameter, % Finer 0.5 mm	Suspended Sediment, Fall Diameter, % Finer 1 mm
USGS Parameter Code		00061	00010	80154	80155	70331	70335	70336	70337	70338	70339	70340	70341	70342	70343	70344	70345	70346
4/10/1975	12:20	38600	4	1140	119000				27	27	34	38		65	75	100		
5/2/1975	10:45	42700	11	1130	130000				29	30	36	42		73	82	99	100	
8/18/1975	11:30	70600	22	1390	265000				21	22	29	38		74	80	99	100	
2/17/1976	10:30	31200	2	925	77900				26	28	31	43		60	70	99	100	
10/13/1982	12:40		13	1290										21	29	72	98	100
5/9/1984	12:38		10.5	2560										30	34	78	99	100
6/6/1984	10:50		20	557										70	75	96	99	100
7/30/1984	11:25		25	436										40	51	88	99	100
7/30/1984	12:15		25	740										24	37	86	100	100
8/26/1996	12:00	55900	24.5	272	41100	37												
9/9/1996	11:15	57000	24	407	62600	47												
12/2/2004	11:30	14100	3	298	11300	32												
1/27/2005	11:00	17400	2	143	6720	55												
2/11/2005	11:00	12500	1	107	3610	61												
7/8/2011	9:17	192000	25.5	778	404000		99	100						19	22	54	96	
7/18/2011	9:57	191000	27.2	356	184000		100							40	45	72	95	
8/1/2011	10:57	186000	28.9	237	119000		100							44	48	72	98	
8/15/2011	10:07	169000	24.8	493	225000		100							18	19	40	97	
8/29/2011	15:37	131000	24.1	207	73300		100							44	49	70	97	
9/12/2011	10:17	102000	23	150	41300									54	63	81	100	
10/31/2011	10:07	47200	12	161	20500									43	56	81	100	
4/21/2015	11:33	32800	12	384	34000				27	32	36	44	60	75	79	96	100	
4/23/2015	13:33	30200	11.5	324	26400		100		30	34	38	46	60	70	75	93	99	
5/8/2015	14:23	38000	18.1	913	93700				42	48	53	60	78	91	93	98	100	
5/27/2015	13:18	39300	17.8	895	95000		100		34	39	46	56	78	88	90	96	98	
6/16/2015	10:38	40000	22.9	921	99500				37	44	50	62	81	91	92	99	100	
7/7/2015	13:38	38000	25.7	434	44500		100		28	34	37	48	73	79	83	96	99	

Note: Available data and lab tests vary by location.

Figure 4-18. USGS-measured sediment data, Missouri River at Omaha, Nebraska (data from USGS at https://nwis.waterdata.usgs.gov/usa/nwis/qwdata/?site_no=06610000)

f. Approximations by Calculation. When measured data are insufficient to develop a sediment discharge rating curve for each size class, then sediment transport equations must be employed to develop rating curves for individual size classes. The percentage of each size class in the suspended load will vary with discharge (the percentage of fines will be greater at lower discharges). Therefore, it is inappropriate to develop sediment discharge rating curves for mixed size classes using the average of measured size class fractions.

g. Adjustment for Unmeasured Load. Sediment discharge rating curves developed from measured suspended-sediment data need to be adjusted to account for the unmeasured load.

(1) This can be accomplished using the Modified Einstein Equation (Vanoni 1975, 2006), if the hydraulic parameters, concentration data by particle size, and bed material gradations are available. The Bureau of Reclamation Automated Modified Einstein Procedure (BORAMEP) is described by Holmquist-Johnson et al. (2009). The process for adjustment and development of total load employed by the USGS is also discussed by Gray and Simões (2008).

(2) If data are not available, the unmeasured load may be assumed to be a percentage of the measured load equal to the percentage that the bedload is of the total load. Bedload percentage for a stream can be determined using the Einstein or Toffaleti sediment transport equation. These are computerized in HEC-RAS (HEC 2016b and 2016c) and in Sediment Analysis Model (SAM) (Thomas et al., 2002.)

h. Bedload.

(1) Developing sediment discharge rating curves from measured bedload data is more difficult. Bedload moves in pulses and varies laterally across the stream. Therefore, significantly more measurements are necessary to obtain a reliable average condition. It has been demonstrated in gravel-bed streams and flumes that the percentage of each size class in the bedload closely corresponds to its percentage in the subsurface layer (Andrews and Parker 1987, Kuhnle 1989, and Wilcox and McArdell 1993). If a given gravel-bed stream is in equilibrium, it is reasonable to assume that the percentage of each size class in the bedload equals the percentage in the bed substrate.

(2) With the advent of bedload measurement techniques such as ISSDOTv2, it is becoming possible to quantify bedload in sand-bed streams where most of the bedload transport occurs due to dune movement. Therefore, it will be possible to begin building databases for the development of rating curves and to begin quantifying the relationship of bedload, suspended bed material load, and total bed material load.

4-17. Scatter of Data Points. At most sediment gage sites, a relatively good correlation between flow discharge and sediment discharge can be developed. However, sediment discharge depends on other variables as well, such as upstream supply, water temperature, roughness, and downstream stage. Therefore, data scatter is expected in sediment discharge rating curves. At some gages, separate curves need to be developed for the rising and falling limbs of flood hydrographs and/or for different seasons on the year.

a. Wash Load. Wash load is determined by its supply from upstream sources and is relatively independent of flow discharge, although flow discharge may be a good surrogate parameter because greater runoff from the watershed and greater bank erosion usually accompany higher flow discharge. Wash load is almost always greater on the rising limb of a flood hydrograph when finer sediment stored in the system is re-suspended, as shown in Figure 4-19. Typically, considerable scatter occurs about the average sediment discharge curve for wash load. Wash load is affected by watershed-wide processes that vary with multiple factors including season, land use, and rainfall.

b. Bed Material Load. Bed material load is very dependent on the hydraulic variables, which, in turn, are closely related to flow discharge; therefore, less scatter about the average sediment discharge curve is expected. In comparison to wash load, the bed material load is primarily a function of the relation between river power and the availability of transportable bed sediments. This is another reason to develop separate sediment discharge curves for wash load and bed material load.

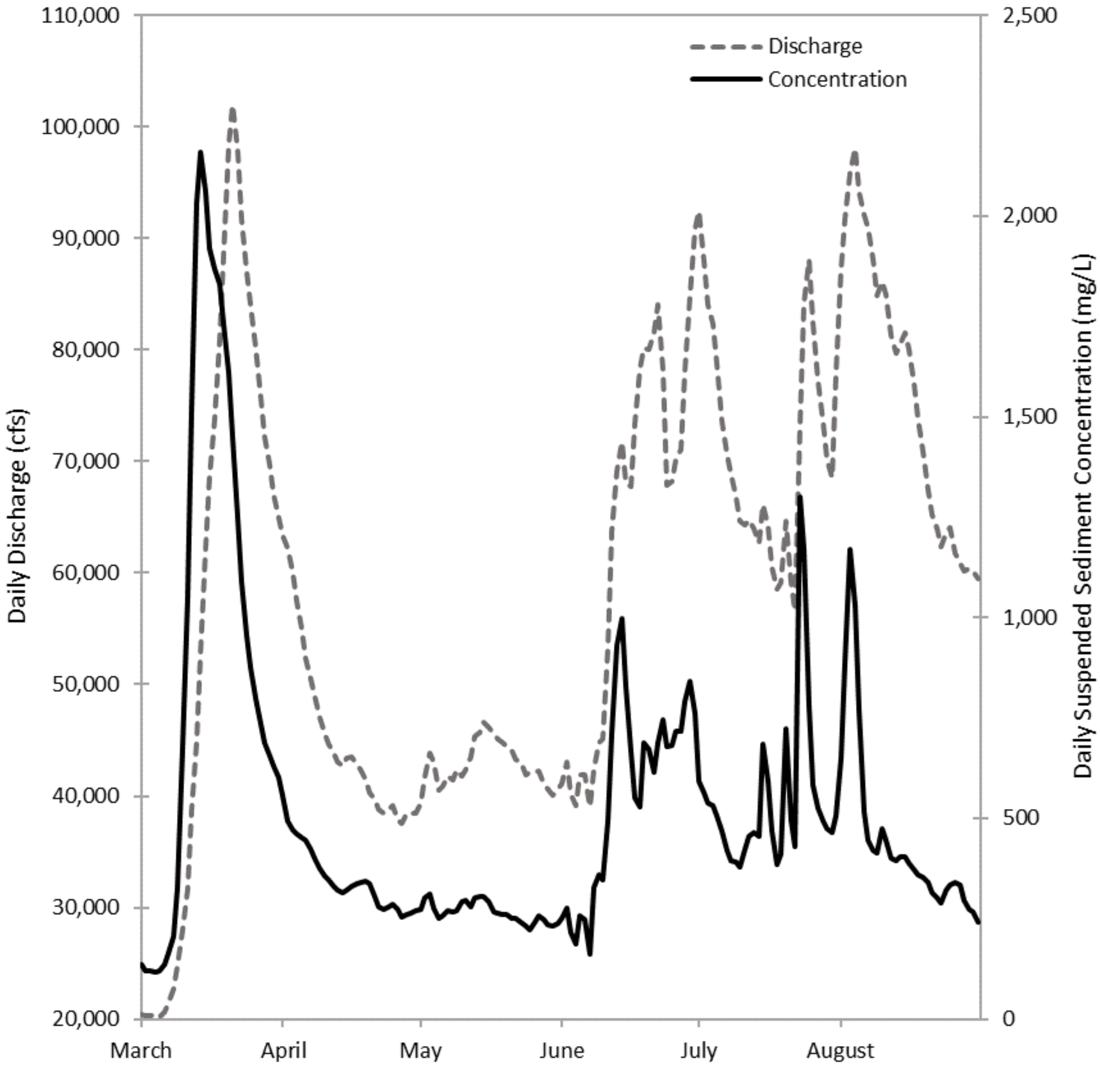


Figure 4-19. Mean daily discharge and SSC (Missouri River at Omaha, Nebraska)

4-18. Predicting Future Conditions. The sediment discharge rating curve may vary with time. This can be due to changes in land use or land management methods, construction of upstream reservoirs that trap sediment, construction of channel stabilization works that decrease bank erosion, or channel improvement work that increases channel conveyance. A significant downward trend in the average annual sediment discharge of the Missouri River at Omaha is shown as an example in Figure 4-20. At this location, the influence of upstream dam construction is pronounced. Although difficult to predict, the possibility of changes in the sediment discharge rating curve over the project life should be considered.

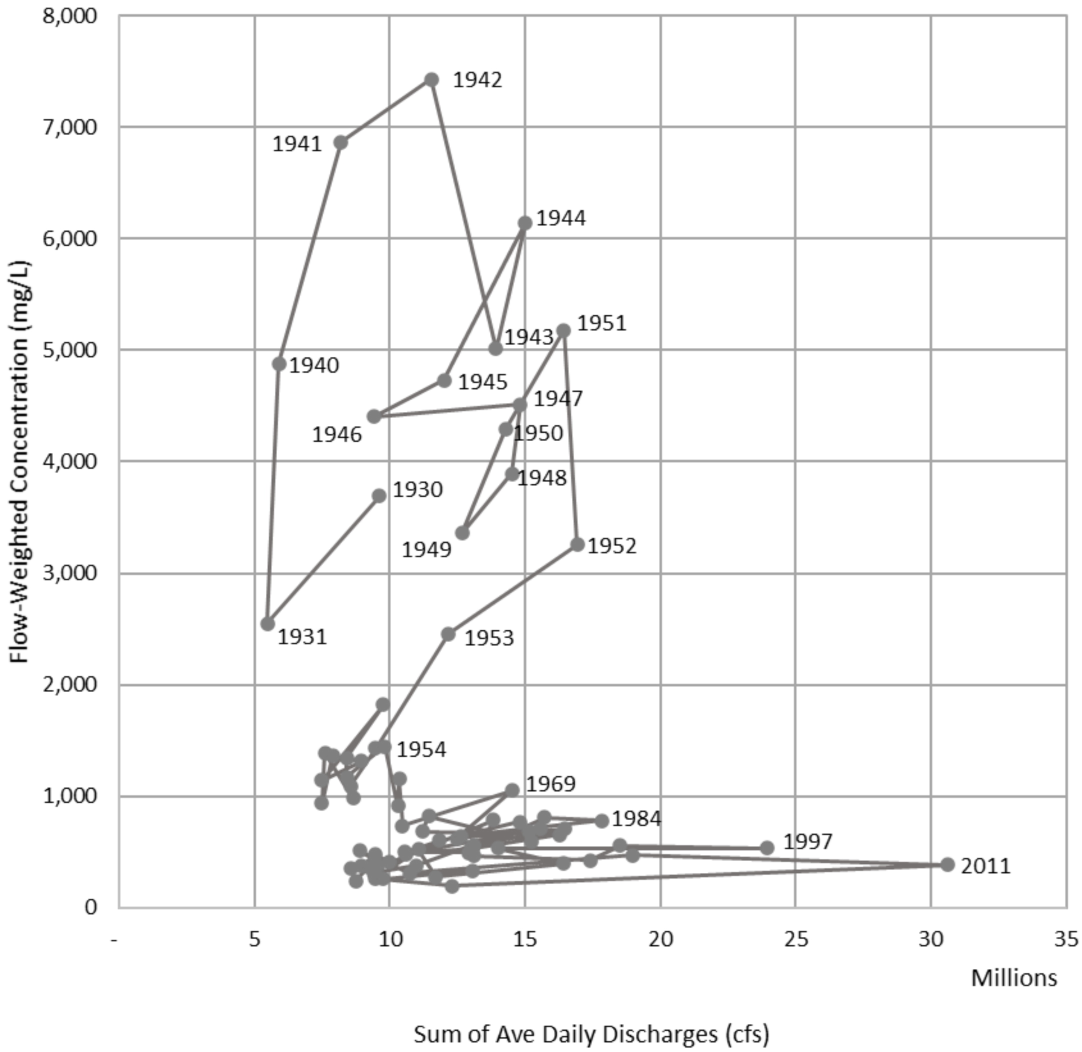


Figure 4-20. Average annual sediment concentration (Missouri River at Omaha, Nebraska)

4-19. Extrapolation to Extreme Events. Sediment data are seldom available for extreme events. This is due both to the infrequency of occurrence and the difficulty in obtaining sediment samples at high flows. Therefore, it is usually necessary to extrapolate the sediment discharge rating curve developed from measured data.

a. Typically, the rate of increase in sediment discharge with water discharge will decrease with an increase in the water discharge, especially for the finer size classes. The decline in rate of increase is more obvious when sediment concentration is plotted against discharge, as shown in Figure 4-21. The decline in rate of increase occurs in the sand sizes as well, as shown in Figure 4-22.

b. A more reliable extrapolation of the measured data for extreme events can be made if the extrapolation is based only on the high-flow measured data. In the absence of measured data at high discharges, extrapolation of the sediment discharge rating curve can be accomplished by calculating a sediment discharge rating curve for each size class in the bed material load and using the shape of the calculated curve to approximate the shape of the extrapolated curve. Expect a high degree of uncertainty for any given grain size that comprises less than 10% of the bed (wash load).

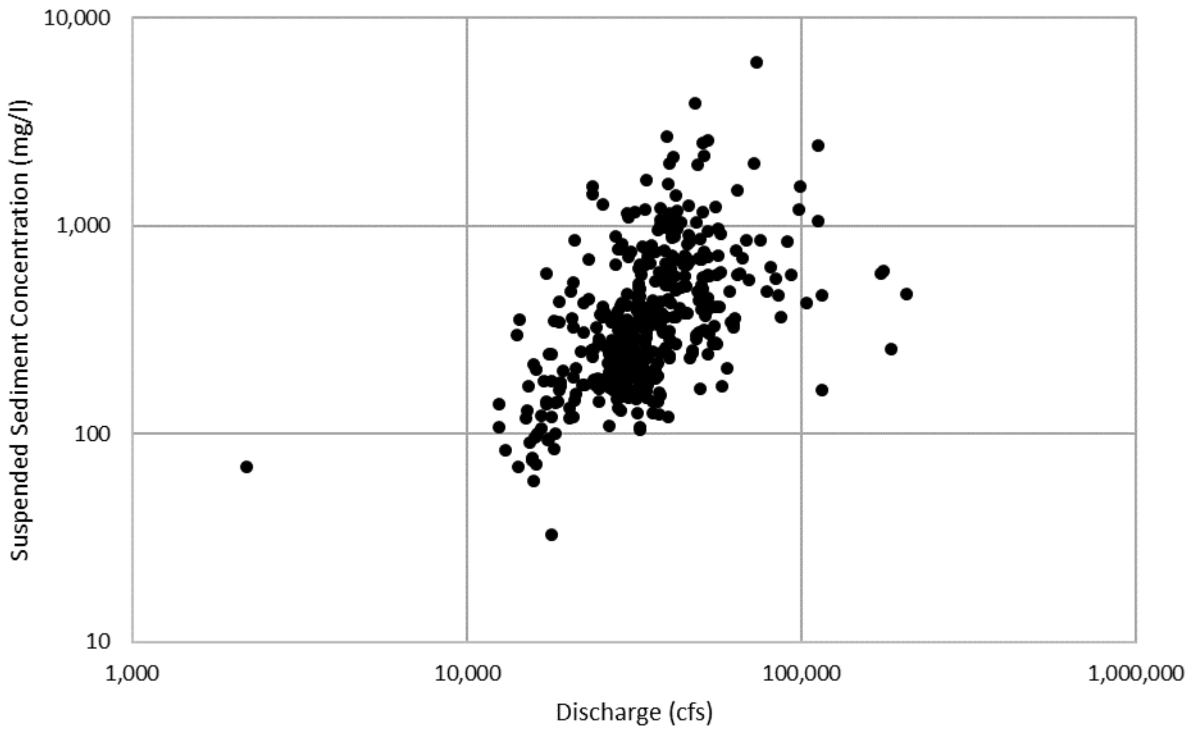


Figure 4-21. Average daily sediment concentration (Missouri River at Omaha, Nebraska)

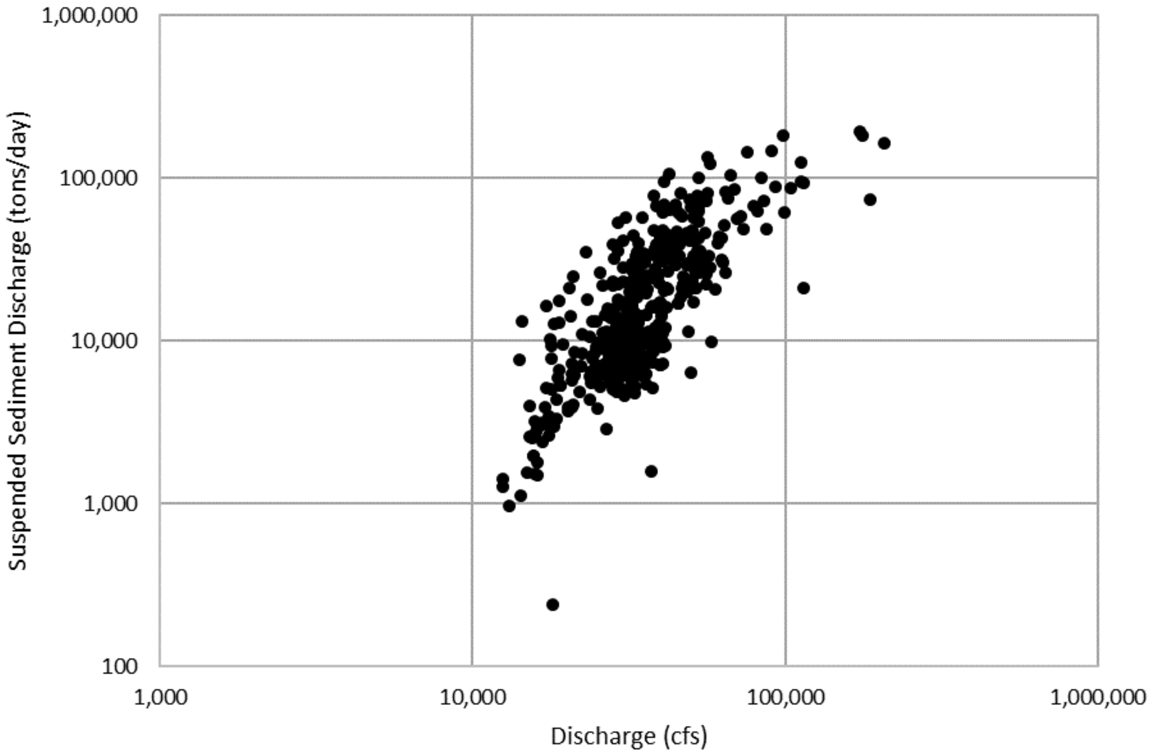


Figure 4-22. Very fine sand sediment transport (Missouri River at Omaha, Nebraska)

Chapter 5
Sediment Transport Mechanics and Analytic Transport Functions

5-1. Introduction.

a. Definition.

(1) Sedimentation embodies the processes of erosion, entrainment, transportation, deposition, and compaction of sediment. These are natural processes that have been active throughout geological times and have shaped the present landscape of our world. The principal external dynamic agents of sedimentation are water, wind, gravity, and ice. Although each may be important locally, only hydrodynamic forces are considered here. The topic of sediment transport mechanics, as typified by Einstein (1950), refers to theories and experiments concerning the physical factors that determine sediment displacement and transport and the methods of estimating quantities transported.

(2) This chapter focuses on sediment erosion, entrainment, and transport. These processes are critical to understanding when a sediment bed movement initiation occurs.

(3) This chapter addresses erosion mechanisms in open water and does not address the more complex local scour or deposition processes that exist near structures in energetic flow. Local scour, as compared to general erosion/deposition, refers to the scour hole that forms around a bridge pier, downstream from a hydraulic structure, or along the outside of a bend, etc. It involves fluid forces from multidimensional flow accelerations, pressure fluctuations, and gravity forces on the sediment particles. The complexity of local scour processes relegates analysis to empirical equations or physical model studies. This chapter does not address local scour. Figure 5-1 illustrates the content of this chapter.

b. Sediment Transport Basics.

(1) Noncohesive Transport Theory. Noncohesive bed sediments have generally been defined as those sediments having grain sizes greater than 0.0625 mm. In addition, they are considered to consist of particles that do not have significant electrochemical forces associated with them.

(a) As flow begins to move over a bed of noncohesive particles, these sediments are free to roll, slide, and saltate over one another unimpeded. This movement of particles is called bedload. Moving water is the agent providing the force on the particles to cause such movements.

(b) As flow increases, when the force vectors are of sufficient magnitude and have a large enough component in the upward vertical direction, then the particles can hop and jump over long distances, even becoming completely suspended. Bed sediments can and do move as bedload and as suspended load simultaneously, often called bed material load. They do exist in the river bed in appreciable quantities. This is because there is a continual interaction between particles at rest in the bed and those moving in the water.

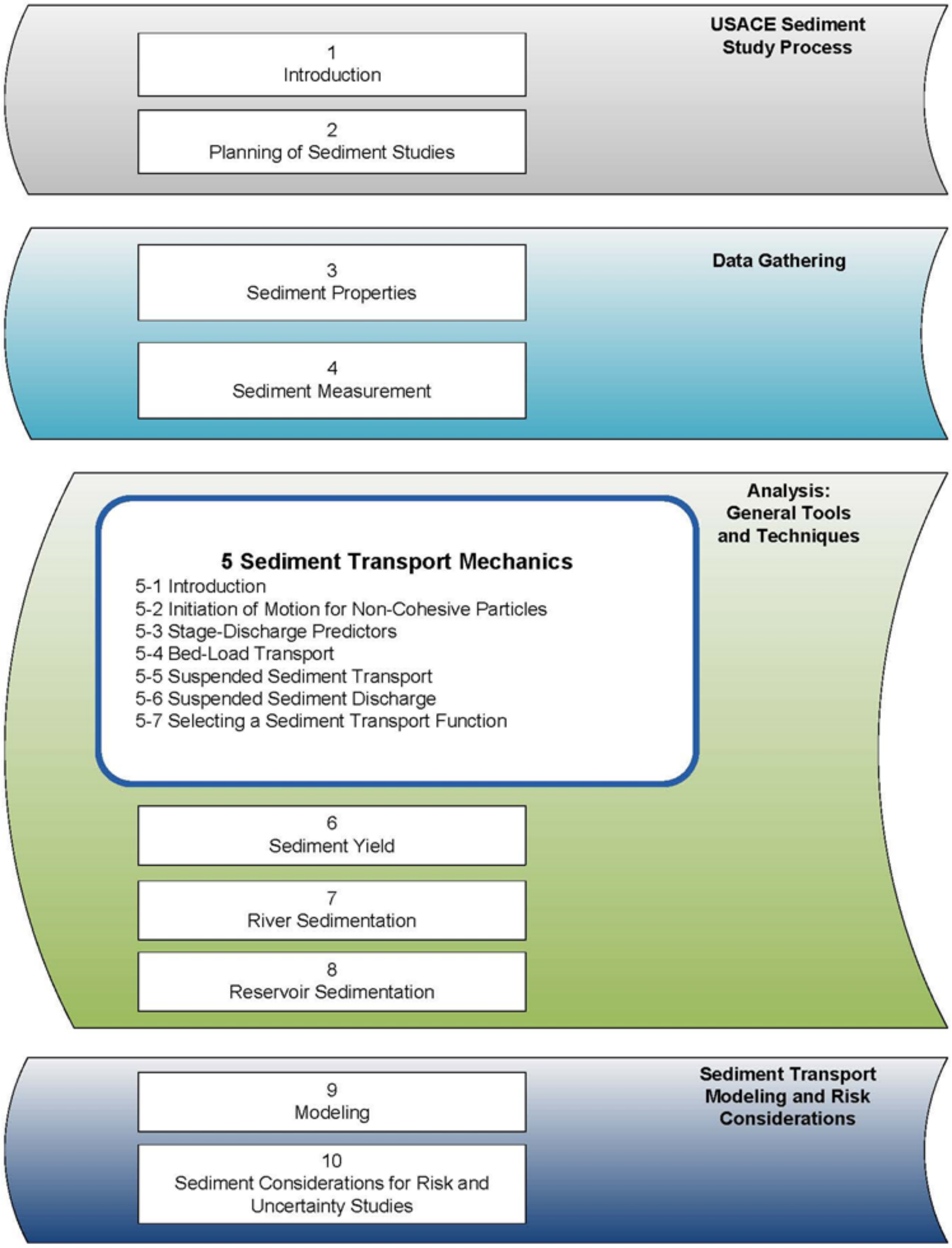


Figure 5-1. Chapter 5 content and general document structure

(c) With regard to bed sediment transport, when the number of mobilized particles is equal to the number of particles coming to rest in the bed, the river is said to be in an equilibrium transport condition. This is equivalent to the water moving bed sediments at a balanced rate. When the amount of mobilized bed sediments is greater than the amount coming to rest, the bed material load will be increasing, and the river bed will be in an erosional state. When the amount of bed sediments coming to rest in the bed is greater than the amount mobilized, the bed material load will be decreasing, and the river bed will be in a depositional state. Therefore, in order to quantify the bed material load, the factors that influence how the material is mobilized (eroded), entrained, transported, and deposited must be understood.

(d) Bedload and suspended load are both influenced by bed forms which are dictated by the transport of bedload and suspended load. This interaction makes the resolution of these processes difficult. Researchers have pursued both analytic and experimental investigations in attempting to solve the transport problem, with a wide range of results. A brief description of the progress is discussed in the following sections.

(2) Cohesive Transport Theory. Cohesive sediment beds include a significant number of fine-grained particles (<0.0625 mm). The dominant forces preventing cohesive particles from being eroded are electrochemical forces, not gravity forces. That is, when cohesive particles come in contact with the bed, they are likely to adhere to it and resist re-entrainment. Cohesive sediment erosion theory is briefly covered (paragraph 5-7h) of this chapter.

(a) Cohesive beds include both individual cohesive (for example, clay) and noncohesive (for example, sand) particles. Cohesive particles in a sediment bed can alter the entrainment and transportation processes of noncohesive particles also in the bed.

(b) Cohesive transport theory covers beds that behave in a cohesive manner, including (1) muddy beds and (2) mixed sediment beds composed of a mixture of cohesive and noncohesive particles.

(c) The bed is considered as cohesive if inter-particle cohesive forces are sufficiently strong to noticeably influence response of the sediment bed to hydrodynamic forces. Specifically, cohesive forces are not small relative to other forces working on the sediment bed (lift and drag).

(d) Settling rates of suspended sediment depend on flocculation of cohesive particles in the water column. Probability of deposition for settling particles is also a function of the cohesive forces in the bed and depositing sediments.

(e) There are analytical techniques for calculating the erosion, entrainment, transportation, deposition, and consolidation of cohesive sediments. However, it is a basic requirement to quantify site-specific sediment properties and processes by testing samples.

(f) Two fundamental processes are: (1) the shear stress for the initiation of erosion and deposition, and (2) the erosion rate as a function of applied shear stress to the sediment surface. The shear stresses for initiation of erosion/deposition are called erosion and deposition thresholds. Deposition thresholds vary with particle size, cohesion, floc state, and other properties. Erosion rate is expressed as a function of bed shear stress. These relationships are needed for the full range of hydraulic conditions expected at the site.

(g) Finally, settling velocities are required to predict transport and geomorphologic evolution. Cohesive sediments flocculate in the water column. Floc settling velocities are a function of sediment properties, concentration in the water column, internal shear, and differential settling. In the case of cohesive particles, the electrochemical bonds, related primarily to mineralogy and particle coating, are the most significant sediment properties.

5-2. Initiation of Motion for Noncohesive Particles.

a. General. Thresholds for noncohesive particle motion can be calculated, using average values for hydraulic parameters, if the fluid and sediment properties are known.

(1) The significant fluid properties are density (or the related properties of specific gravity and specific weight) and viscosity. For most applications in natural systems, density is calculated as a function of temperature and salinity.

(2) Significant sediment properties include particle size, shape, density, and position in the matrix of surrounding particles. In addition, bed roughness (grain and form roughness) influence threshold of motion.

(3) Significant hydraulic forces are bed shear stress, lift, pressure fluctuations related to turbulence, and impact from already suspended particles.

b. Shields Parameter.

(1) Velocity and grain size are often used to calculate initiation of motion for noncohesive sediment bed particles. This approach is not valid with cohesive sediment. These relationships are used for gross approximations of sediment stability because the more advanced relationships are dependent on bed and hydrodynamic properties that are often unknown.

(2) Figure 5-2 provides an approximation, derived from Shields (1936) relationship, for critical mean (vertically averaged) velocity as a function of particle diameter for quartz (the primary mineral in river sand). Velocity and quartz particle size are used because these quantities are generally available. These simplified relationships are dependent on some basic assumptions, such as non-stratified flow and typical quartz particle shapes. Critical velocity is minimum for particles between 0.1 and 0.2 mm. Critical velocity increases for particles smaller than this because of decreased bed roughness and cohesive particle coatings on naturally occurring quartz particles. Cohesive forces are small relative to drag forces for larger particles.

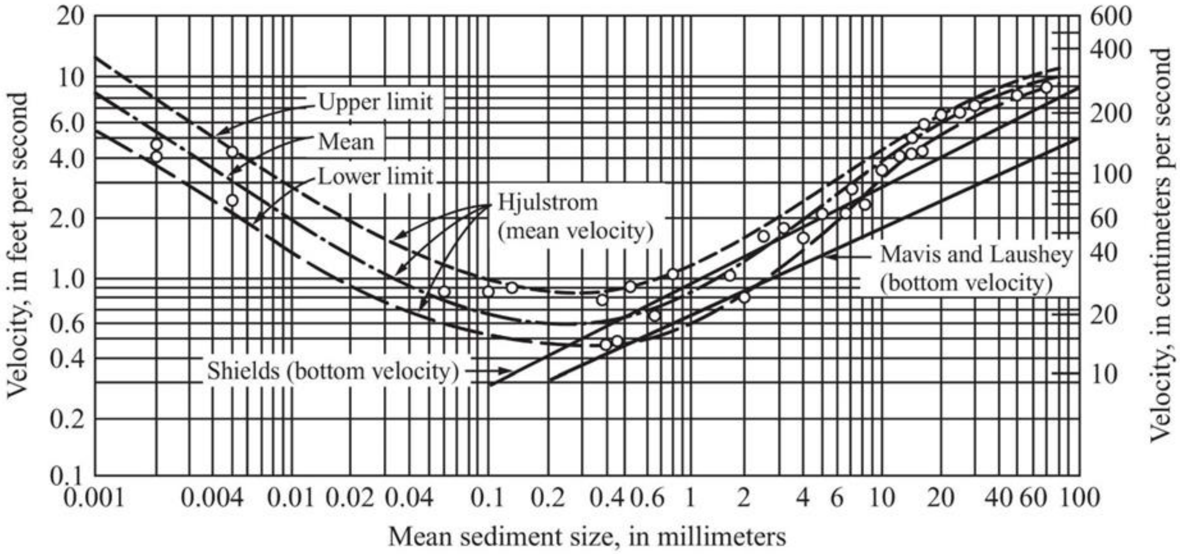


Figure 5-2. Critical water velocities for quartz sediment as a function of mean grain size (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

(3) More accurate relationships for quantifying initiation of motion from velocity include additional complexity that is addressed in Figure 5-2 only through uncertainty bounds. Additional complexities include issues such as grain size variability and water properties. Note that Figure 5-2 is not appropriate for use with cohesive particles.

(4) Shields relationship between dimensionless shear stress (also stated as Shields parameter), τ^*, and grain Reynolds number, R*, are used to develop the functions on which most applied stress/initiation of motion relationships are based. Shields parameter and grain Reynolds number are dimensionless (see Figure 5-3), so that any consistent units of measurement may be used in their calculation. Although the experimental work and analysis were performed by Shields, the curve termed the Shields Curve, which is shown in Figure 5-3, was proposed by Rouse as related in ASCE Manual No. 54 (Vanoni 1975, 2006). Equations for dimensionless shear stress and grain Reynold's number are provided in Figure 5-3. In a format suitable for program or spreadsheet application, Shields curve is defined by the Equations 5-1 and 5-2:

$$\tau^* = 0.22\beta + 0.06 \times 10^{-7.7\beta} \qquad \text{Equation 5-1}$$

$$where\ \beta = \left(\frac{1}{\nu}\sqrt{\left(\frac{\gamma_s - \gamma}{\gamma}\right) g d^3}\right)^{-0.6} \qquad \text{Equation 5-2}$$

where:

τ^* = dimensionless shear stress (Shields parameter)

γ_s = particle specific weight
γ = fluid specific weight
υ = kinematic viscosity of the fluid
g = acceleration of gravity
d = particle diameter
u = shear velocity = $(\tau_b/\rho)^{0.5}$
ρ = fluid density
τ_b = near-bed shear stress

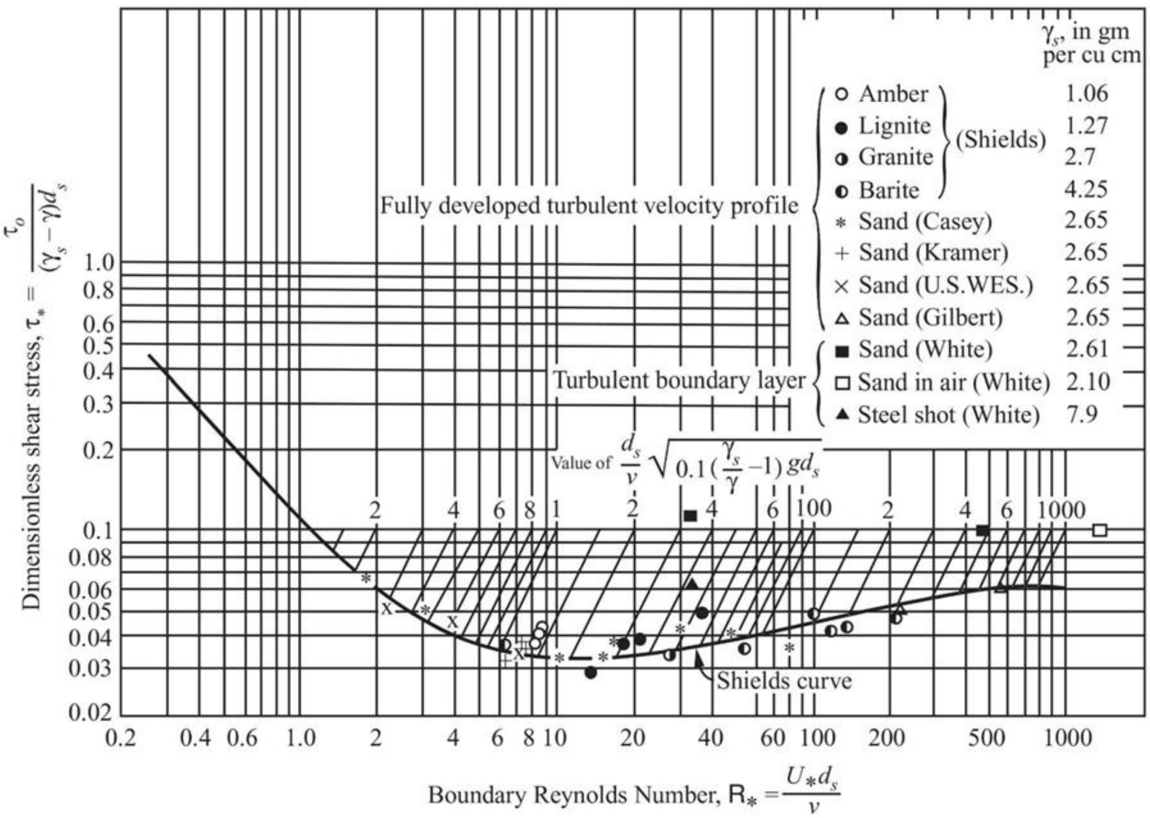


Figure 5-3. Shields Diagram with White data added (Vanoni 1975, 2006)

(5) Figure 5-3 shows a modified version of the well-known Shields diagram for initial movement or scour of noncohesive uniformly graded sediments on a flat bed. The diagram is applicable theoretically to any sediment and fluid. It plots the Shields number (or mobility number), which combines shear stress with grain size and relative density, against a form of Reynolds number that uses grain size as the length variable.

(6) For sediments in the gravel size range and larger, the Shields number for initiation of motion is independent of Reynolds number. The constant is shown as 0.06 in Figure 5-3, but it is often taken as 0.045, or even as low as 0.03 (as discussed in the following sections) if absolutely

no movement is required. For widely graded bed materials, the median grain size by weight (D_{50}) is generally taken as the representative size, although some writers favor a smaller percentile such as D_{35}.

(7) The critical shear stress, τ_c, for stability of a particle having a diameter, d, is calculated from the following equation:

$$\tau_c = \tau^*(\gamma_s - \gamma)d \qquad \text{Equation 5-3}$$

c. Adjusted Shields Parameter. Shields obtained his critical values for τ^* experimentally, using uniform bed material, and measuring sediment transport at decreasing levels of bed shear stress and then extrapolating to zero transport. There are three problems associated with the critical dimensionless shear stress as determined by Shields.

(1) First, the procedure did not account for the bed forms that developed with sediment transport. A portion of the total shear is associated with the bed form roughness; therefore the calculated dimensionless shear stress was too high. Gessler (1971) reanalyzed Shields' data so that the critical Shields parameter represented only the grain shear stress, which determines sediment transport and entrainment (Figure 5-4) where R^* is computed from shear velocity, kinematic viscosity, and the grain diameter, k . This curve is more appropriate for determining critical shear stress in plane bed streams with relatively uniform bed gradations. With fully turbulent flow ($R^* > 400$), typical of gravel bed streams, τ^* is commonly taken to be 0.047 using Gessler's curve (Chapter 8, NRCS 2007).

(2) Second, the critical dimensionless shear stress is based on the average sediment transport of numerous particles and does not account for the sporadic entrainment of individual particles at very low shear stresses. This becomes very important when transport of gravels and cobbles is of interest in low energy environments, and in the design of armor protection. This departure from Shields' curve was demonstrated by Paintal (1971) and is shown in Figure 5-5. Note that the extrapolated critical Shields number was about 0.05, but the actual critical Shields number was 0.03.

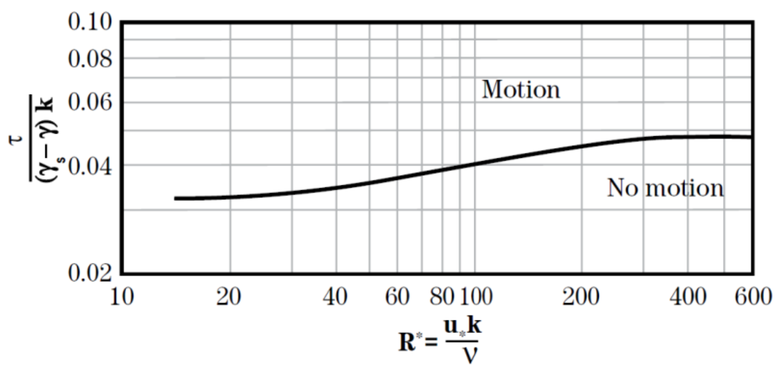


Figure 5-4. Gessler's reformulation of Shields diagram (NRCS 2007)
Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

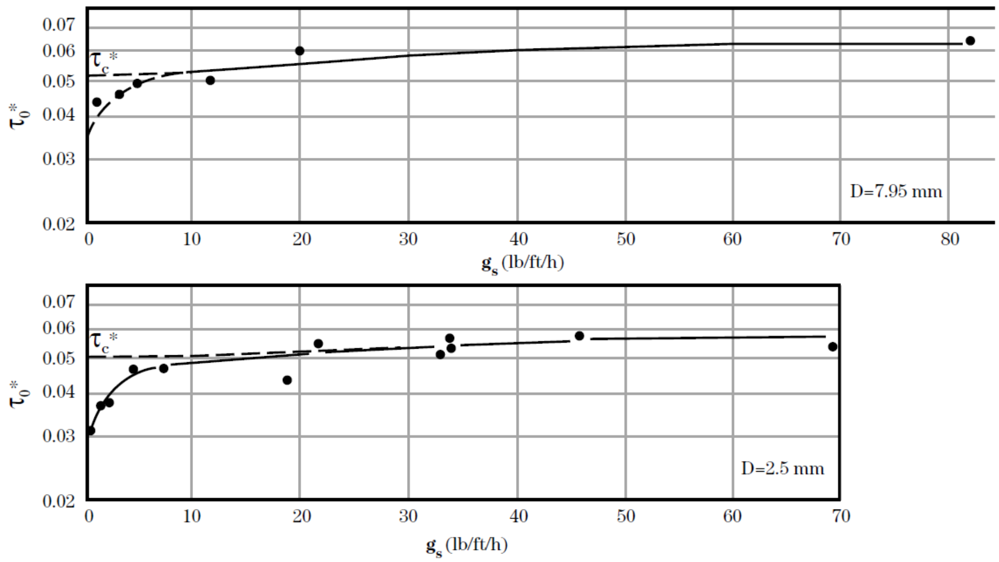


Figure 5-5. Variation in Shield parameter with decreasing sediment load (NRCS 2007)

Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

(3) Third, critical Shields number for particles in a sediment mixture may be different from that for the same size particle in a uniform bed material. Natural streambeds seldom have uniform bed gradations. Meyer-Peter and Muller (1948) and Gessler (1971) determined from their data sets that the critical Shields parameter for sediment mixtures was about 0.047. From his data, Neill (1968) determined that, in gravel mixtures, most of the particles become mobile when τ* for the median grain size was 0.030. Andrews (1983) found a slight difference in τ*, for different grain sizes in a mixture, and presented the following equation:

$$\tau^*{}_i = 0.0834\left(\frac{d_i}{d_{50}}\right)^{-0.872} \qquad \text{Equation 5-4}$$

where the subscript, *I*, indicates size class *I*, and d_{50} is the median diameter of the subsurface material.

(4) The minimum value for τ^*_I in the above equation was found to be 0.020. According to Andrews, the critical shear stress for individual particles has a very small range; therefore, the entire bed becomes mobilized at nearly the same shear stress.

d. Gessler's Concept for Particle Stability.

(1) Critical Shear Stress. Critical shear stress is difficult to define because at low shear stresses entrainment is sporadic, caused by bursts of turbulence. It is even more difficult to define for particles in a coarse surface layer because the critical shear stress of one size class is affected by the presence of other size classes in the bed mixture.

(2) Gessler (1971) developed a probabilistic approach to the initiation of motion for sediment mixtures. He reasoned that, due to the random orientation of grains on the bed and the

random strength of turbulence on the bed, for a given set of hydraulic conditions, part of the grains of a given size will move while others of the same size may remain in place. Gessler assumed that the critical Shields parameter represents an average condition, where about half the grains of a uniform material remain stable and half move. It follows then that, when the critical shear stress was equal to the bed shear stress, there was a 50% chance for a given particle to move.

(3) Using experimental flume data, Gessler developed a probability function, p, dependent on $\tau c/\tau$ where τc varied with bed size class (Figure 5-6). He determined that the probability function had a normal distribution and that the standard deviation (slope of the probability curve) was a function primarily of turbulence intensity and equal to 0.057. Gessler found the effect of grain size orientation to be negligible. The standard deviation also accounts for hiding effects (that is, no attempt was made to separate hiding from the overall process).

(4) Gessler's analysis demonstrates that there can be motion of particles even when the applied shear stress is less than the critical shear stress, and that not all the particles of a given size class on the bed will necessarily be entrained until the applied shear stress exceeds the critical shear stress by a factor of 2.

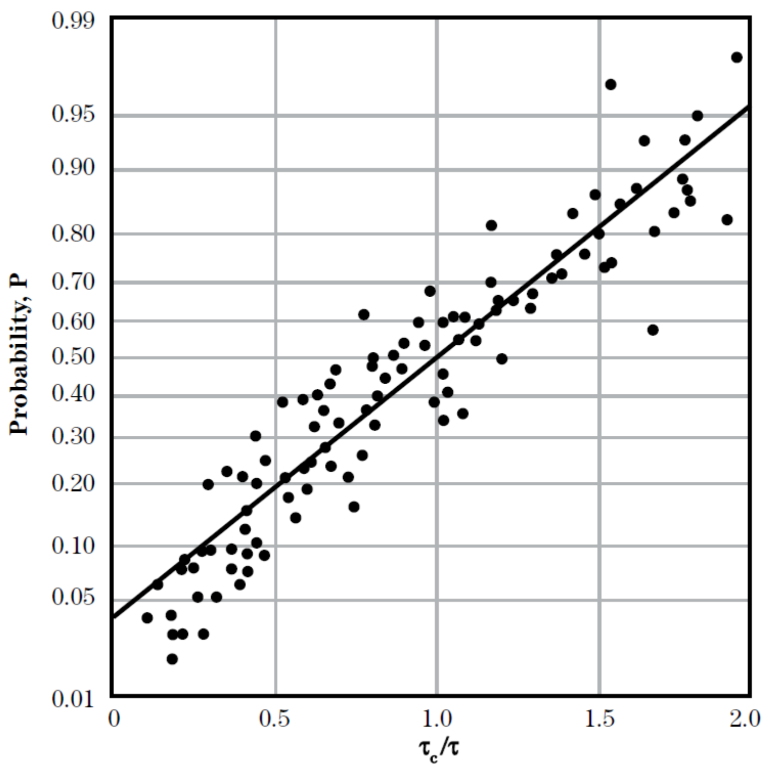


Figure 5-6. Probability for grains to remain on bed (NRCS 2007)
Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

(5) Mean Value of the Probabilities for the Bed Surface.

(a) Gessler suggested that the mean value of the probabilities for the bed surface to stay should be a good indicator of stability:

$$\bar{P} = \frac{\int_{i_{min}}^{i_{max}} P^2 f_i di}{\int_{i_{min}}^{i_{max}} P f_i \, di} \qquad \text{Equation 5-5}$$

where P is the probability function for the mixture and depends on the frequency of all grain sizes in the underlying material, and f_i is the fraction of grain size *i*.

(b) Gessler suggested that, when P > 0.65, that the surface layer of the bed would be unstable.

e. Grain Shear Stress and Total Bed Shear Stress. The total bed shear stress may be divided into that acting on the grains and that acting on the bed forms. Bed forms are defined as ripples, dunes, and other physical features of the sediment bed that are larger in size than individual grains. Entrainment and sediment transport are a function only of the grain shear stress. Grain shear stress thus must be determined to make sediment transport calculations. Einstein (1950) determined that the grain shear stress could best be determined by separating total bed shear stress into a grain component and a form component, which are additive. The equation for total bed shear stress is:

$$\tau_o = \tau' + \tau'' = \gamma RS \qquad \text{Equation 5-6}$$

where:

τ_o = total bed shear stress
τ' = grain shear stress
τ'' = form shear stress
R = hydraulic radius
S = slope

f. Hydraulic Radius. Einstein (1950) suggested that the hydraulic radius can be divided into grain and form components that are additive. These equations for grain and form shear stress become:

$$\tau' = \gamma R' S \qquad \text{Equation 5-7}$$

$$\tau'' = \gamma R'' S \qquad \text{Equation 5-8}$$

where *R'* and *R''* are hydraulic radii associated with the grain and form roughness, respectively.

(1) The total bed shear stress can be expressed as:

$$\tau_o = \gamma R' S + \gamma R'' S \qquad \text{Equation 5-9}$$

(2) Slope and the specific weight of water are constant, so that the solution becomes one of solving for one of the *R* components. The Limerinos (1970) equation can be used to calculate the grain roughness component:

$$\frac{V}{U_*'} = 3.28 + 5.66 \log_{10} \frac{R'}{d_{84}} \qquad \text{Equation 5-10}$$

$$U_*' = \sqrt{gR'S} \qquad \text{Equation 5-11}$$

where *V* is the average velocity and d_{84} is the particle size for which 84% of the sediment mixture is finer.

(3) Limerinos developed his equation using data from gravel-bed streams. Limerinos' hydraulics radii ranged between 1 and 6 feet; d_{84} ranged between 1.5 and 250 mm. This equation was confirmed for sand-bed streams without bed forms by Burkham and Dawdy (1976). The equation can be solved iteratively when average velocity, slope, and d_{84} are known.

g. Bed Form Shear Stress.

(1) Einstein and Barbarossa (1952) used data from several sand-bed streams to develop an empirical relationship between bed form shear velocity, U_*'', and a dimensionless sediment mobility parameter, ψ′. Figure 5-7 illustrates the relationship.

$$\Psi' = \left(\frac{\gamma_s - \gamma}{\gamma}\right)\frac{d_{35}}{R'S} \qquad \text{Equation 5-12}$$

where d_{35} is the particle size for which 35% of the sediment mixture is finer.

(2) R'' can be solved for directly using the following equation:

$$R'' = \frac{(U_*'')^2}{gS} \qquad \text{Equation 5-13}$$

The value of V/U_*'' is determined from the bar resistance curve and U_*'' can be calculated because average velocity is known (refer to Equation 5-11 for a similar form of U_*').

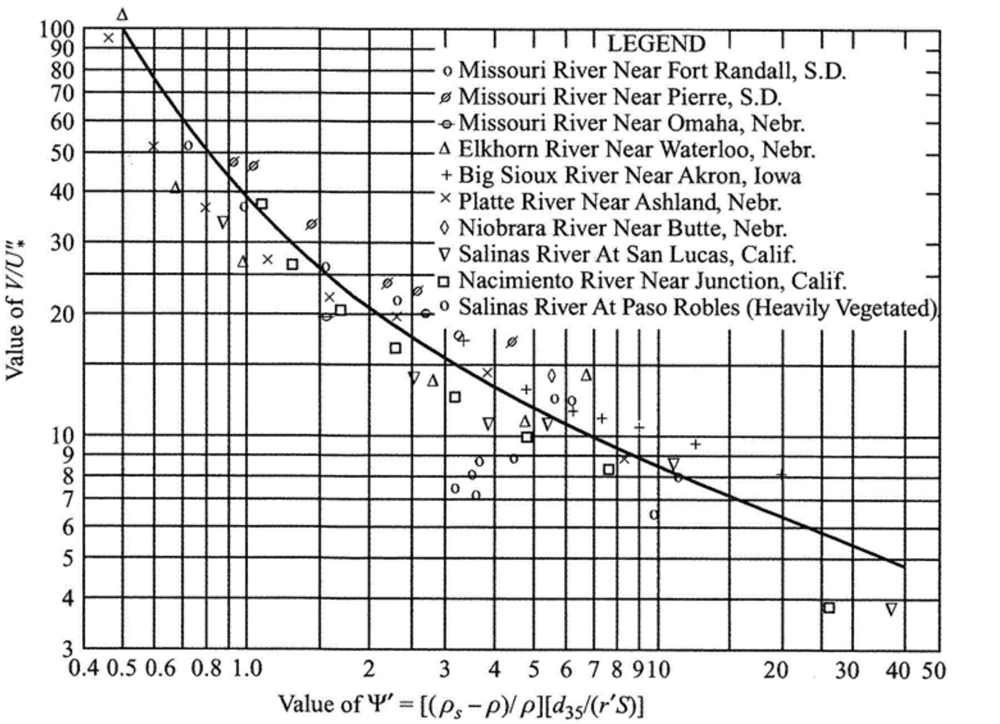


Figure 5-7. Einstein-Barbarossa bar resistance graph (Vanoni 1975, 2006 after Einstein and Barbarossa 1952)

(3) Typically, either the grain or the form hydraulic radius is calculated directly, and the other hydraulic radius component is determined to be the difference between the total hydraulic radius and the calculated component.

h. Bank or Wall Shear Stress.

(1) Whenever the streambanks contribute significantly to the total roughness of the stream, the shear stress contributing to sediment transport must be further reduced. This is accomplished using the side-wall correction procedure that separates total roughness into bed and bank roughness and conceptually divides the cross-sectional area into additive components. The procedure assumes that the average velocity and energy gradient are the same in all segments of the cross section.

$$A_{total} = A_b + A_w \qquad \text{Equation 5-14}$$

$$A_{total} = P_b R_b + P_w R_w \qquad \text{Equation 5-15}$$

where *A* is cross-sectional area, *P* is perimeter, and subscripts *b* and *w* are associated with the bed and wall (or banks), respectively.

(2) Note that the hydraulic radius, *R*, is not additive with this formulation as it was with *R'* and *R"*. The Manning equation has been applied extensively to evaluate open channel flow velocity:

$$V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2} \qquad \text{Equation 5-16}$$

where *V* is the mean velocity in ft/sec, *R* is the hydraulic radius in feet, *S* is the slope of the energy line, and *n* is the coefficient of roughness specifically known as Manning's *n*.

(3) Many sources for estimating the roughness value are available including EM 1110-2-1601 and standard open-channel hydraulic textbooks such as Chow (1959). In rare circumstances at field locations, all values in the above equation other than Manning's *n* may be measured and the equation rearranged to solve for the roughness value. Regardless of source, a value for Manning's *n* is necessary. Rearranging the Manning equation, with a known average velocity, slope, and roughness coefficient, the hydraulic radius associated with the banks can be calculated:

$$\frac{V}{1.486 S^{1/2}} = \frac{R^{2/3}}{n} = \frac{R_w^{2/3}}{n_w} \qquad \text{Equation 5-17}$$

$$R_w = \left(n_w \frac{V}{1.486 S^{1/2}}\right)^{3/2} \qquad \text{Equation 5-18}$$

where Velocity is in feet per second and hydraulic radius *R* is in feet.

(4) The side-wall correction procedure is outlined using the Darcy-Weisbach equation in ASCE Manual No. 54 (Vanoni 1975, 2006). Total hydraulic radius and shear stress considering grain, form, and bank roughness can be expressed by the following:

$$R_{total} = \frac{P_b(R' + R'') + P_w R_w}{P_{total}} \qquad \text{Equation 5-19}$$

$$\tau_{total} = \gamma S \left(\frac{P_b(R' + R'') + P_w R_w}{P_{total}}\right) \qquad \text{Equation 5-20}$$

5-3. Stage-Discharge Predictors.

a. General. There are several stage-discharge predictors that have been developed for alluvial channels and these are presented in Chapter 2 of ASCE Manual No. 110, Chapter 2 (Garcia 2008b), ASCE Manual No. 54 (Vanoni 1975, 2006), and by USACE (1994b). The Limerinos (1970) equation is suggested as a stage-discharge predictor for gravel-bed streams. The Einstein-Barbarossa (1952) method was the first stage-discharge predictor to account for variability in stage due to bed form roughness by calculating separate hydraulic radii for grain and form contributions. More recently, Brownlie (1981) developed regression equations to calculate a hydraulic radius that accounts for both grain and form roughness in sand-bed streams.

b. Brownlie Approach.

(1) Database. Brownlie's resistance equations are based on regressions from over 1,000 records from 31 flume and field data sets. The data were carefully analyzed for accuracy and consistency by Brownlie. The resistance equations account for both grain and form roughness, but not bank roughness. Brownlie is most applicable where the form roughness, bed roughness from bed froms, is much more important than grain roughness. The data covered a wide range of conditions: grain size varied between 0.088 and 2.8 mm, and depth ranged between 0.025 and 17 m. All of the data had width to depth ratios greater than 4, and the gradation coefficients of the bed material were equal to or less than 5.

(2) Regression Equations. Brownlie developed separate resistance equations for upper and lower regime flow. The equations are dimensionless, and can be used with any consistent set of units:

(a) Upper Regime:

$$R_b = 0.2836 d_{50} q_*^{0.6248} S^{-0.2877} \sigma^{0.0813} \qquad \text{Equation 5-21}$$

(b) Lower Regime:

$$R_b = 0.3742 d_{50} q_*^{0.6539} S^{-0.2542} \sigma^{0.1050} \qquad \text{Equation 5-22}$$

where:

$$q_* = \frac{VD}{\sqrt{g d_{50}^3}} \qquad \text{Equation 5-23}$$

R_b = hydraulic radius associated with the bed
d_{50} = median grain size
S = slope
σ = geometric standard deviation of the sediment mixture (EM 1110-2-1601, Figure 5-4)
V = average velocity
D = water depth
g = acceleration of gravity

(c) To determine if upper or lower regime flow exists for a given set of hydraulic conditions, a grain Froude number, F_g, and a variable, F_g' were defined by Brownlie:

$$F_g = \frac{V}{\sqrt{g d_{50}\left(\frac{\gamma_S - \gamma}{\gamma}\right)}} \qquad \text{Equation 5-24}$$

$$F_g' = \frac{1.74}{S^{0.3333}} \qquad \text{Equation 5-25}$$

(d) According to Brownlie, upper regime flow occurs if S > 0.006 or if F_g > 1.25 F_g', and lower regime flow occurs if F_g < 0.8 F_g'. The zone between these upper and lower regime limits is the transition zone. Within the transition zone, there are two valid solutions (a lower and an upper regime solution). A general rule of thumb is to use the upper regime for the rising end of the hydrograph and the lower regime for the falling end of the hydrograph (HEC 2016a).

5-4. Bedload Transport.

a. General. Bedload transport occurs when noncohesive sediment rolls, slides, or jumps (saltates) along the bed. Any particle size can move as bedload, depending on hydraulic forces. As flow increases, some of the sediment moving as bedload will usually be entrained by vertical turbulent mixing into the water column and be transported for extended periods of time in suspension. Thus, it takes more energy for the flow to transport sediment in suspension than as bedload. Sediment transported in suspension is referred to as suspended load. The total load is the sum of the bedload and suspended load as previously listed in Table 3-7. Bedload is typically between 10% and 25% of the total load, though for beds with a high fraction of coarse sediment, the percentage will normally be higher.

b. DuBoys' Concept of Bedload.

(1) Between 1879 and 1942, much of the work in sediment transport was influenced by DuBoys. He proposed the idea of a bed shear stress and visualized a process by which the bed material moved in layers. The significant assumptions in the DuBoys approach were that sediment transport could be calculated using average cross-section hydraulic parameters and that transport was primarily a function of the excess shear stress (the difference between hydraulically applied shear stress and the critical shear stress of the bed material). The general form of the DuBoys equation is:

$$q_B = K\tau_o(\tau_o - \tau_c)^m \qquad \text{Equation 5-26}$$

where:

q_B = bedload transport rate in weight per unit time per unit width
τ_o = hydraulically applied shear stress
τ_c = critical, or threshold shear stress, for the initiation of movement
K and m = constants

(2) The functional relationship between K, τ_c, and grain size was determined experimentally and is presented in ASCE Manual No. 54 (Vanoni 1975, 2006). In DuBoys' equation m = 1.0. No movement occurs until the bed shear stress exceeds the critical value.

c. Einstein's Concept of Particle Movement. Einstein (1950) proposed a major change in the approach to predicting sediment transport when he presented a bedload formula based on probability concepts in which the grains were assumed to move in steps of average length proportional to the sediment size. He describes bed material transportation as follows:

> "The least complicated case of bedload movement occurs when a bed consists only of uniform sediment. Here, the transport is fully defined by a rate. Whenever the bed consists of a mixture, the transport must be given by a rate and a mechanical analysis or by an entire curve of transport against sediment size. For many years, this fact was neglected, and the assumption was made that the mechanical analysis of transport is identical with that of the bed. This assumption was based on observation of cases where actually the entire bed mixture moved as a unit. With a larger range of grain diameters in the bed, however, and especially when part of the material composing the bed is of a size that goes into suspension, this assumption becomes untenable.
>
> The mechanical analysis of the material in transport is basically different from that of the bed. This variation of the mechanical analysis will be described by simply expressing in mathematical form the fact that the motion of a bed particle depends only on the flow and its own ability to move, and not on the motion of any other particles." –Einstein 1950.

(1) Equilibrium Condition. Einstein's hypothesis that motion of a bed particle depends only on the flow and its own ability to move and not on the motion of any other particles allowed him to describe the equilibrium condition for bed material transportation mathematically as two independent processes: deposition and erosion. He proposed an equilibrium condition and defined it as the condition existing when the same number of a given type and size of particles must be deposited in the bed as are scoured from it.

(2) Bedload Equation. In his formulation for bedload transport, Einstein determined the probability of a particle being eroded from the bed, *p,* to be:

$$\frac{p}{1-p} = A^*\Phi_i^* \qquad \text{Equation 5-27}$$

$$\Phi_i^* = \frac{i_B}{i_b}\frac{q_B}{\gamma_s}\left(\frac{\gamma}{\gamma_s-\gamma}\right)^{1/2}\left(\frac{1}{gd_i^3}\right)^{1/2} \qquad \text{Equation 5-28}$$

where:

A* = constant
Φ_i^* = bedload parameter for size class *i**
i_B = fraction of size class *I* in the bedload
i_b = fraction of size class *I* in the bed material
q_B = bedload transport in weight per unit time and width
d_i = grain diameter of size class *i*

(3) He then reasoned that the dynamic lift forces on a particle are greater than particle weight when the probability to go into motion is greater than unity. Assuming a normal distribution for the probability of motion yields:

$$p = 1 - \frac{1}{\sqrt{\pi}} \int_{\eta_o}^{\eta} e^{-t^2}\, dt \qquad \text{Equation 5-29}$$

$$\eta_o = -B^* \Psi_i^* - 2.0 \qquad \text{Equation 5-30}$$

$$\eta = B^* \Psi_i^* - 2.0 \qquad \text{Equation 5-31}$$

where:

B^* = a constant
Ψ_i^* = dimensionless flow intensity parameter
t = variable of integration

(4) Ψ_i^* is a function of grain size, hydraulic radius, slope, specific weight, and viscosity. Correction factors are applied to account for hiding and pressure variations due to the composition of the bed material mixture. Setting the probability of erosion equal to the probability of motion yields the Einstein bedload function:

$$1 - \frac{1}{\sqrt{\pi}} \int_{\eta_o}^{\eta} e^{-t^2}\, dt = \frac{A^* \Phi^*}{1 + A^* \Phi^*} \qquad \text{Equation 5-32}$$

(5) The equation can be transformed into the following and solved for sediment transport rate, q_B:

$$i_B q_B = i_b \Phi^* \gamma_s d_i \sqrt{g d_i \left(\frac{\gamma_s - \gamma}{\gamma}\right)} \qquad \text{Equation 5-33}$$

where Φ^* is a function of Ψ^*which is determined using empirically derived graphs provided by Einstein (1950) or Vanoni (1975, 2006).

(6) Limitations. The dependence of the Einstein method on these empirical graphs, which were derived from limited data, limits the applicability of the method. The important contributions of this work were the introduction of the probability concept for bedload movement, the identification of processes influencing entrainment and transport of sediment mixtures, and a formulation of the interactions. Einstein was aware of the limitations of his method and did not intend that it should be considered as a universal one.

5-5. Suspended-Sediment Transport.

a. The most important process in maintaining sediment in suspension is flow turbulence. In steady turbulent flow, velocity at any given point will fluctuate in both magnitude and direction. Turbulence is greatest near the boundary where velocity changes are the greatest. When dye is injected instantaneously at a point in a turbulent flow field, the cloud will expand as it is carried downstream at the mean velocity. This process is called diffusion and is the basis for the analytical description of sediment suspension. The 1D sediment diffusion equation balances the upward flow of sediment due to diffusion with the settling of the sediment due to its weight:

$$C\omega + \varepsilon_s \frac{\delta C}{\delta y} = 0$$ Equation 5-34

where:

- C = sediment concentration
- ω = settling velocity (see paragraph 3-2g)
- ε_s = sediment diffusion coefficient
- y = depth

b. For boundary roughness dominated flows, it is common practice to assume that the sediment diffusion coefficient is equal to the momentum diffusion coefficient, ε_m, which can be described by:

$$\varepsilon_s = \varepsilon_m = \kappa U^* \frac{y}{D}(D - y)$$ Equation 5-35

where:

- κ = Von Karman constant
- U* = shear velocity
- D = total water depth

c. Integration yields the Rouse equation:

$$\frac{C_y}{C_a} = \left(\frac{D-y}{y}\frac{a}{D-a}\right)^z$$ Equation 5-36

$$z = \frac{\omega}{\kappa U^*}$$ Equation 5-37

where:

- a = reference elevation
- C_a = concentration at reference elevation
- C_y = concentration at y elevation above the bed

d. The equation gives the concentration in terms of C_a, which is the concentration at some arbitrary level y = a. This requires foreknowledge of the concentration at some point in the vertical. Typically, this point is assumed to be close to the bed and C_a is assumed to be equal to the bedload concentration. One problem with this equation is that concentration approaches infinity as *y* approaches zero. Therefore, the equation cannot be used to calculate the total sediment load from the bed to the surface without additional assumptions and approximations. Figure 5-8 shows a graph of the Rouse suspended load distribution equation.

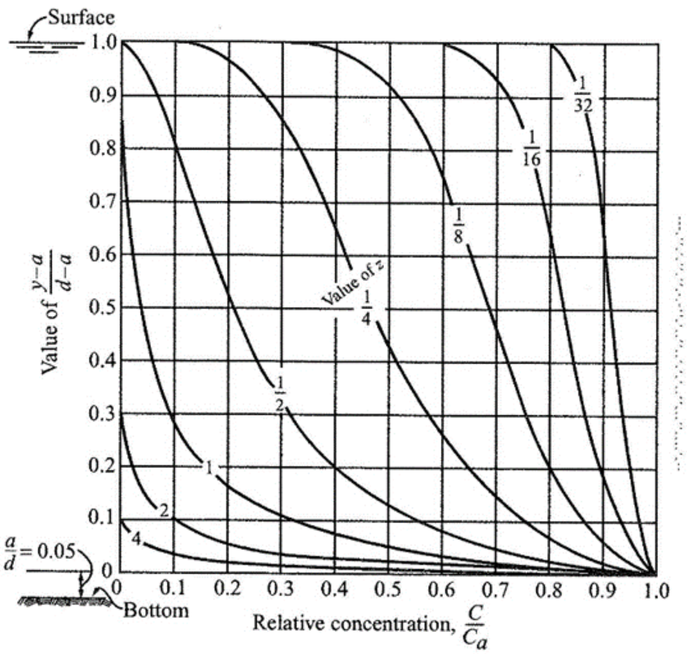


Figure 5-8. Rouse's SSC distribution for a/d = 0.05 and several values of Z (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

5-6. Suspended-Sediment Discharge. Suspended-sediment discharge, q_s, is calculated from the concentration profile using the following equation:

$$q_s = \int_{y=y_o}^{D} C_y \, u \, dy \qquad \text{Equation 5-38}$$

where u is the local velocity. Solution of this equation requires an analytical description of the vertical velocity distribution.

a. Einstein's Approach. Einstein (1950) assigned the lower limit of integration, $y_o = 2d_i$, and called this the thickness of the bed layer. He assumed that C_a was equal to the bedload concentration. He used Keulegan's logarithmic velocity distribution equations (Keulegan 1938) to determine velocity. Since this work was done before the common use of computer, Einstein prepared tables for the solution of the integral. These are found in Einstein (1950) and Vanoni (1975, 2006) as well as other sediment transport texts.

b. Brooks' Approach. Brooks (1963) developed a graph that can be used to calculate suspended-sediment transport if the sediment concentration at mid-depth is known. Using the Rouse equation, Brooks assigned a = 0.5 D. The lower limit of integration, y_o, was determined to be the depth where u = 0. Brooks used a power law velocity distribution equation and numerical integration to develop the curve shown in Figure 5-9. This figure can be used to determine total

SSC when the concentration at mid-depth, the average velocity V, and the shear velocity U are known.

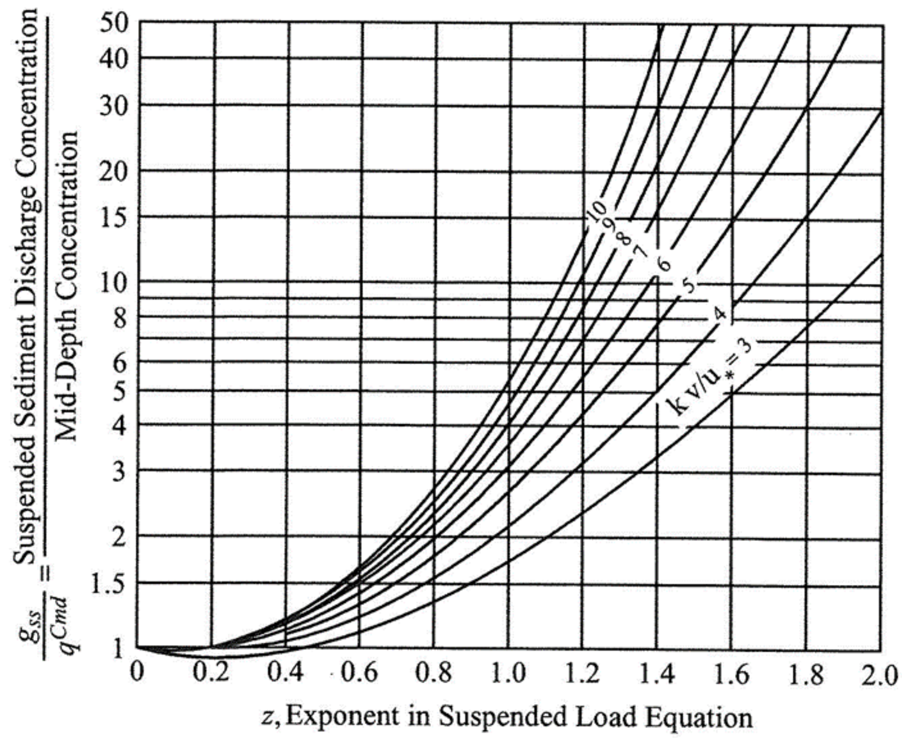


Figure 5-9. Brooks curve for determining SSC, g_{ss} from known values of kV/u*, exponent z, and mid-depth concentration C_{md} (Vanoni 1975, 2006)

5-7. Selecting a Sediment Transport Function.

a. General.

(1) Most sediment transport functions predict a rate of sediment transport for a given set of steady-state hydraulic and bed material conditions. These functions estimate sediment load passing through a specific cross section of the river or stream. Some sediment transport equations were developed for calculation of bedload only, and others were developed for calculation of total bed material load. Other functions quantify both bed and suspended load. This distinction can be critical in sand-bed streams, where the suspended bed material load may be orders of magnitude greater than the bedload.

(2) Another important difference in noncohesive sediment transport functions is the manner in which grain size is treated.

(a) Some sediment transport functions were developed as single-grain size functions, usually using the median bed material size to represent the total bed. Single-grain size functions are most appropriate in cases where equilibrium sediment transport can be assumed; that is, when (1) the project will not significantly change the existing hydraulic or sediment conditions, (2) bed sediments are well sorted with a fairly tight distribution around the median grain size, and (3) hydraulic conditions do not vary rapidly when moving from upstream to downstream.

(b) When studies evaluate project impacts on sediment transport characteristics (the USACE project may create nonequilibrium conditions), then a multiple-grain size sediment transport equation should be used. Multiple grain-size functions are very sensitive to the grain size distribution of the bed material. Extreme care must be exercised to ensure that the fine component of the bed material gradation is representative of the bed surface for the specified discharge. This is very difficult without measured data. For this reason, Einstein (1950) recommended ignoring the finest 10% of the bed material sample for computation of bed material load with a multiple-grain size function. This method is acceptable if that 10% does not increase cohesive forces sufficiently to influence transport and fate.

(c) Frequently, single-grain size functions are converted to multiple-grain size functions simply by calculating sediment transport using geometric mean diameters for each size class in the bed (sediment transport potential) and then assuming that transport of that size class (sediment transport capacity) can be obtained by multiplying the sediment transport potential by the bed fraction. This assumes that each size class fraction in the bed acts independently from other size classes in the bed, thus ignoring the effects of hiding, which can produce unreliable results.

b. Testing. It is important to test the predictive capability of a sediment transport equation against measured data in the project stream or in a similar stream before its adoption for use in a sediment study. Different functions were developed from different sets of field and laboratory data and are better suited to some applications than others. Different functions may give widely differing results for a specified channel. Experience with sediment discharge formulas can be summed up in Figure 5-10.

c. Sediment Transport Equations.

(1) A generalized noncohesive sediment transport equation can be presented in a functional form:

$$Q_s = f\left(V, D, S_e, B, d_e, \rho_s, G_{sf}, d_s, i_b, \rho, T\right) \qquad \text{Equation 5-39}$$

where:

B = effective width of flow
D = effective depth of flow
d_e = effective particle diameter of the mixture
d_s = geometric mean of particle diameters in each size class i
Q_s = total bed material discharge rate in units of weight divided by time
G_{sf} = grain shape factor

i_b = percentage of particles of the i*th* size class that are found in the bed expressed as a fraction
S_e = slope of energy line
ρ = density of fluid for other than temperature effect
ρ_s = density of sediment particles
T = water temperature
V = average flow velocity

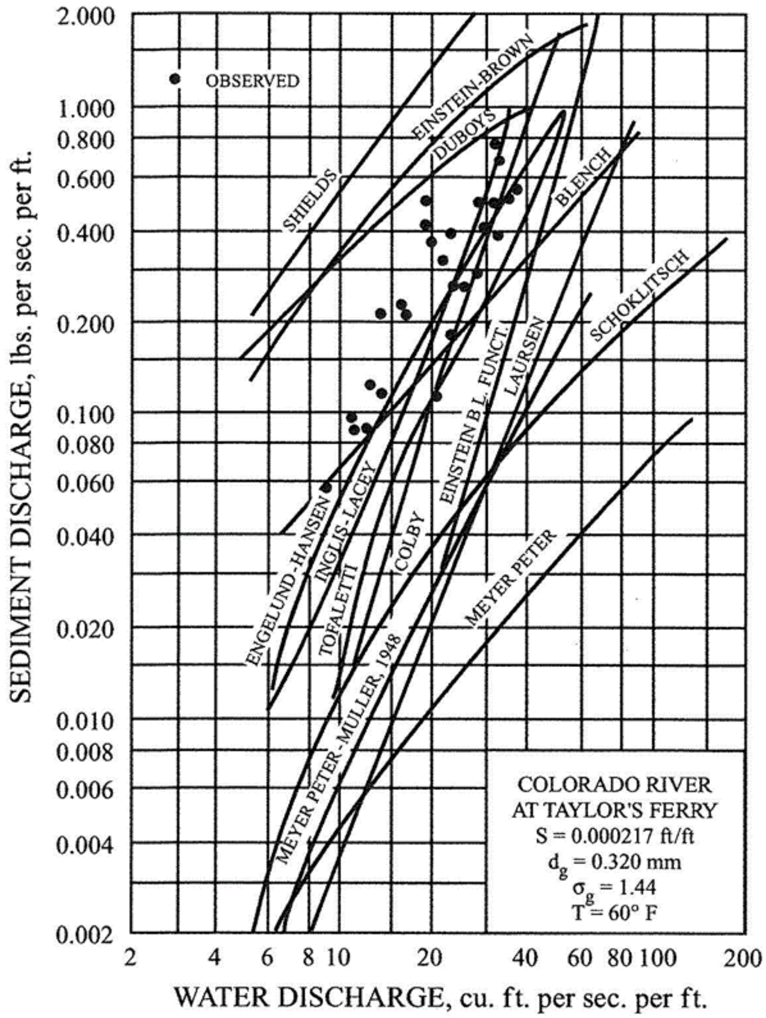


Figure 5-10. Sediment discharge rating curve, Colorado River at Taylor's Ferry (Vanoni 1975, 2006)

(2) Of particular interest are the groupings of terms: hydraulic parameters (V, D, S_e, B), sediment particle parameters (d_e, ρ_s, G_{sf}), sediment mixture parameters (d_s, i_b), and fluid properties (ρ, T).

d. Processes. Einstein's (1950) classic work presents a complete view of the processes of equilibrium sediment transportation and is useful for understanding those processes. However, it is less useful for application to actual sedimentation analyses. Many researchers have contributed sediment transport functions developed from various field data sources that are too numerous to name. However, no single function has been proven superior in all cases. The following general guidelines are given to aid in selection of a transport function. Project site data must be used to confirm selection. When data is not available for confirmation, the scatter between calculated values, similar to that shown in Figure 5-10, may be used in establishing a sensitivity range or a risk and uncertainty factor.

(1) Ackers-White (1973). The Ackers-White transport function is a total load function developed under the assumption that fine sediment transport is best related to the turbulent fluctuation in the water column and coarse sediment transport is related to the net grain shear with the mean velocity used as the representative variable. The transport function was developed in term of particle size, mobility, and transport. The function is based on flume data for relatively uniform gradations ranging from sand to fine gravels. Hydrodynamics were selected to cover a ranged of bed configurations including ripples, dunes, and plane bed conditions. Consult Ackers-White (1973) for the details in applying this approach.

(2) Colby (1964). The Colby equation has been used successfully on a limited class of shallow sand-bed streams with high sediment transport. The Colby function was developed as a single-grain size function for both bedload and suspended bed material load. Its unique feature is a correction factor for very high concentrations of fine sediment. This correction factor may be used with other sediment transport equations and has been incorporated into the HEC-6 numerical model where it is used with all sediment transport equations.

(3) Einstein (1950). The Einstein equation has application for both sand and gravel-bed streams. It is a multiple-grain size sediment transport function that calculates both bedload and suspended bed material load. The hiding factor in the original equation has been modified by several investigators (Einstein and Chien 1953, Pemberton 1972, and Shen and Lu 1983) to improve performance on specific studies.

(4) Engelund-Hansen (1967). The Engelund-Hansen function is a total load predictor that gives adequate results for sandy rivers with substantial suspended load. It is based on flume data with relatively uniform sand sizes less than 1 mm. Application should be restricted to sand systems. It is a relatively simple function of channel velocity, bed shear, and the material D_{50}. It has been found to be fairly consistent with field data.

(5) Laursen-Copeland (Copeland and Thomas 1989). The Laursen (1958) sediment transport equation is a multiple-grain size function for both bedload and suspended bed material load. Initially based on flume data, it was expanded in 1985 by Madden (1993) to include data from the Arkansas River and other sand-bed rivers. Copeland (1989) generalized the equation for gravel transport. It is based on excess shear and a ratio of shear velocity to the fall velocity. It is applicable to large and intermediate size sand-bed rivers with a wide size range from sand to medium gravels. A distinctive feature is that the function development included material in the

silt range. Many other functions use extrapolation to compute silt sediment transport potential. Without a hiding factor, application is questionable in streams with a wide range of grain sizes.

(6) Meyer-Peter and Muller (1948). This equation was developed from flume data as a multiple-grain size function, although it is frequently applied as a single-grain size function. Sediment was transported as bedload in the Meyer-Peter and Muller flume. The transport rate is a simple excess shear function, proportional to the difference between the mean shear stress acting on the grain and the critical shear stress. It is strictly a bedload equation developed for sand and gravel under plane bed conditions. Its applicability is for bedload transport in gravel-bed streams. Recently Wong (2003) and Wong and Parker (2006) demonstrated that this function overpredicted transport by approximately a factor of two based on a reanalysis of Meyer-Peter Muller's original results.

(7) Toffaleti (1968). Toffaleti is a total load function developed primarily for sand transport. This multiple-grain size function has been successfully used on many large sand-bed rivers. It calculates both bedload and bed material suspended load and is based on extensive sand-bed river and flume data. While formulation follows Einstein, there are significant differences.

(a) The function is not heavily dependent on shear velocity or bed shear. Instead, it was formulated from regressions on temperature and an empirical exponent that described the relationship between sediment and hydraulic characteristics.

(b) This method breaks the suspended-load distribution into vertical zones, replicating the vertical variation in transport rates. Therefore, this approach is most appropriate for transport with significant suspended load. The function has been used successfully on large systems like the Mississippi, Arkansas, and the Atchafalaya Rivers.

(c) The Toffaleti equation generally underestimates the transport of gravel size classes. However, it has been combined with the Meyer-Peter and Muller equation in HEC-6 to provide an equation with more potential to transport a wider range of size classes.

(8) Yang (1973, 1984). Yang is a total load function that bases transport on stream power, the product of velocity and shear stress. Yang developed two regression equations, one for sand and one for gravel, from flume and extensive measured data on a wide variety of streams.

(a) This is a single-grain size equation, and when applied as a multiple-grain size function in HEC-6, it is done so without a hiding factor. The function is not as sensitive to grain size as other functions and, therefore, is less likely to produce wide variations in calculated sediment transport.

(b) Yang tends to be very sensitive to stream velocity, and it is more sensitive to fall velocity than most other functions. It is most applicable to intermediate to small sand-bed streams with primarily medium to coarse sand size bed material. It would not be appropriate for use with analysis of applications when significant armoring or hydraulic sorting of the bed are expected.

(9) Van Rijn (1984). Using a shear stress approach similar to Shields, van Rijn (1984a) developed a dimensionless form of the bedload transport. To predict the noncohesive suspended-sediment load in a water body, it is necessary to determine whether, for a given particle size and flow regime, the sediment is transported as bedload or as suspended load. Van Rijn (1984b) compared the bed shear velocity, u_*, to the critical shear velocity, u_{*cs}, to distinguish between bedload and suspended load.

(a) When the bed shear velocity exceeds the critical shear velocity for a given particle size, erosion of that size (and smaller) sediment from the bed surface is assumed to occur. Therefore, if the following inequality is true, sediment will be transported as bedload (and not as suspended load): $u_{*cs} < u_* < w_s$. Under this inequality condition, deposition of suspended sediment occurs when the bed shear velocity is less than the critical shear velocity.

(b) If the bed shear velocity exceeds both the critical shear velocity and settling velocity for a given particle size, then that size sediment (and any smaller) is assumed to be eroded from the bed and transported as suspended load, and any sediment of that particle size (and smaller) already moving as bedload is assumed to be subsequently transported in suspension.

(10) Wilcock and Crowe (2003). Wilcock and Crowe (2003) is a bedload equation designed for well-graded beds containing both sand and gravel. Developed from surface gradations of flumes and rivers, it is a river-bed surface transport method based on the theory that transport is primarily dependent on material in direct contact with flow.

(a) It has a hiding function that reduces the transport potential of smaller particles based on the premise that they are nestled between larger gravel clasts. The central theory is that gravel transport potential increases for greater sand content.

(b) For increasing sand content, the reference shear decreases, the excess bed shear increases, and the total transport is increased. Accordingly, the Wilcock function is sensitive to the sand content. It is most appropriate for bimodal systems and tends to diverge from other equations for unimodal gravel or sand transport.

e. Comparison and Selection of a Transport Function.

(1) Many comparisons have been made between the different transport functions. The following is a short list of some documents containing summaries, tests, comparisons, and general guidance on their selection and use (Mahmood 1980; Stevens and Yang 1989; Nakato 1990; Simons and Sentürk 1992; Chien and Wan 1999; Thomas et al., 2002; Thomas and Chang 2008). This list is not meant to be exhaustive, but provides coverage of the subject by a range of authors.

(2) A guidance module was included in the SAM hydraulic design package (Thomas et al., 2002) to aid in the selection of a sediment transport function. The significant hydraulic and sediment variables of slope, velocity, width, depth, and median grain size applicable to a given stream are provided to the computer program. The program then checks the given data against 17 sets of field data collected by Brownlie (1983) and looks for a river with similar characteristics.

Ten sediment transport equations were tested with each of the 17 data sets and the best three were determined. The program then reports to the user the three best sediment transport equations for each of the data sets with hydraulic characteristics that matched the given stream.

(3) Guidance in HEC-RAS (HEC 2016b and 2016c) presents the transport functions, including a discussion of the range of parameters used in flume test when developing the transport function including material diameter, flow velocity, depth, width, and temperature. Transport functions in HEC-RAS are a limited subset of those included in SAM. Refer to the latest sediment users guide for current options. Procedure for Calculating Sediment Discharge by Transport.

f. Procedure for Calculating Sediment Discharge by Transport.

(1) In addition to the sediment discharge curve derived from measured data (previously discussed in paragraph 4-16), the sediment discharge rating curve may be calculated using sediment transport theory. This calculation can be made only for sand and gravel-bed sediment using equilibrium sediment transport functions. The calculation should be made by particle size for the full range of water discharges in the study hydrograph. Whenever possible, calculated sediment transport rating curves should be compared to values developed from measured data.

(2) The basic steps are as follows:

(a) Assemble field data (cross sections and bed gradations). Select the channel location carefully to represent the river slope, velocity, width, and depth.

(b) Develop a representative bed gradation from field samples. There is no simple rule for locating samples. Carefully select sampling locations and avoid anomalies that would bias the calculated sediment discharge. Uniform bed samples may be composited by sampling from three to five sections within the cross section.

(c) Avoid using average geometry and bed gradation over extended lengths of a river to maintain the river hydraulic and sediment transport relationships.

(d) Calculate the stage-discharge rating curve accounting for possible regime shifts due to bed form change.

(e) Calculate the bed material sediment discharge rating curve using hydraulic parameters from the stage-discharge calculation. The calculation should be made by particle size for the full range of water discharges in the study hydrograph.

(f) Compare to measured values when available. When comparing total load transport function values to suspended-load measurements, correction for unmeasured load (Table 3-7) is necessary. Collected data for bedload will vary with stream cross-section location. Similarly, suspended concentration data will vary vertically and laterally. Advances in bedload measurement techniques (paragraph 4-10) can provide accurate measured data for comparison.

Alternatively, bedload fluxes at various stream discharge rates can be estimated using available equations, surface grain size distribution, bottom roughness, and near-bottom velocity.

(3) Computation of sediment transport capacity has been automated in SAM (USACE 2002) and HEC-RAS (HEC 2016b). In HEC-RAS, the sediment transport capacity option is accessed from the stable channel design menu. Computations may be performed for multiple cross sections, transport functions, flows, and by grain size class. Output may be viewed as sediment transport rating curves and tabulated output.

g. Illustration of Transport Modes for Noncohesive Sediment.

(1) Multiple equations and methods for estimating transport mechanisms of noncohesive sediment have been shown in previous sections. Transport modes can be summarized using transport mechanisms of easily identified parameters. Two dimensionless parameters, the bed shear stress (Shields Figure 5-3) for uniform flow and the particle Reynolds Number, Re_p, provide an effective delineator between sand and gravel-bed rivers. Whether a sediment under a given flow condition behaves as bedload or as suspended load depends on the relationship between the entrainment function and the dimensionless grain size as examined by Ackers (1972).

(2) Parker also applied this concept with a schematic of transport modes for noncohesive sediment (Figure 5-11). The dimensional axis values as used in Figure 5-11 are defined as:

$$\tau^* = \frac{\tau_b}{\rho g R D} = \frac{HS}{RD} \qquad \text{Equation 5-40}$$

where:

τ_b = near-bed shear stress
g = gravitational acceleration
ρ, ρ_s = water and sediment density, respectively
R = $\frac{\rho_s - \rho}{\rho}$ submerged specific gravity of the sediment
D = mean sediment diameter
H = flow depth
S = stream slope, for steady uniform flow is equivalent to energy gradient

(3) The particle Reynolds number Re_p is defined as:

$$Re_p = \frac{\sqrt{gRD}\,D}{v} \qquad \text{Equation 5-41}$$

where v is the kinematic viscosity of water.

(4) In Figure 5-11, there is no sediment motion below the curved heavy line, which is the Shields Curve and represents the conditions under which incipient motion occurs. Figure 5-11 can be used to evaluate the prevailing mode of transport, suspended load, or bedload, after motion occurs. Sediment generally moves as bedload immediately after the onset of motion.

A limiting size for the wash load is found where the curves for initiation of motion and suspension intersect. The figure also shows that, in gravel-bed rivers, bed material is transported mainly as bedload. Finally, in sand-bed rivers, both suspension and bedload transport of bed material coexist, particularly at high flows.

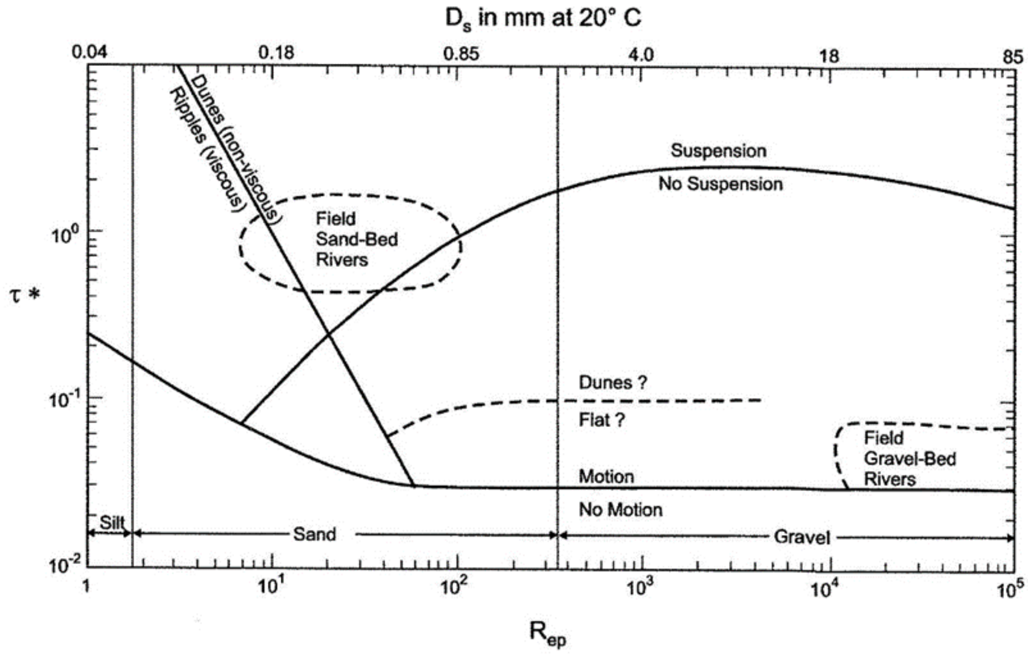


Figure 5-11. Parker's transport modes for noncohesive sediment (Garcia 2008)

h. Cohesive Sediment Transport.

(1) Before discussing cohesive sediment transport, it is informative to contrast the differences between noncohesive and cohesive sediment characteristics. Noncohesive (sand and gravel) particles resist entrainment due to gravitational and grain-related resistance forces. Gravitational forces are derived from particle mass. Resistance forces are related to particle location in the sediment matrix (a sand particle resting on a perfectly smooth bed would, in theory, have no resistance force). Inter-particle cohesive (electrochemical) forces, which bind particles together, are negligible in noncohesive sediment beds.

(a) If sufficient cohesive sediment particles exist in a bed, then an additional force of similar magnitude to the other forces will act on resisting particle motion. This is the cohesive force. Cohesive forces, if sufficiently large, will influence initiation of motion for both cohesive and noncohesive grains in the bed. Specifically, cohesive sediment can act as a bonding agent that holds both cohesive and noncohesive particles in a matrix with cohesive bonds that must be broken to permit transport.

(b) Cohesive particles are generally much smaller than noncohesive particles. The most cohesive particles are clay size (<0.004 mm). Other particles, particularly silt size, can be cohesive. Even fine sand can include natural coatings that produce cohesive forces, even though quartz, the primary mineral in most river sands, is not a cohesive substance.

(c) For beds that include sand, silt, and clay, clay particles fill the interstitial space between individual sand particles. The clay particles are attracted both to each other and to the silt and sand. These clay particles that congregate between sand particles form the bonds that produce the cohesive forces in the bed.

(d) Most of the sediment transport equations were developed from sand and/or gravel data. Therefore, most silt and all clay particles are outside the range of applicability of the transport function. In most systems, the particles are wash load with only trace amounts found in the bed because transport capacity always exceeds supply. For some USACE studies, it may be reasonable to assume that these fines are throughput load that never interact with the bed, the model is insensitive to them, and that including fines adds unnecessary complexity. However, silt and clay particles can be very important in reservoirs and backwater or low-energy zones when deposition can affect project function.

(e) Suspended fine sediments entering reservoirs may exist as flocs/aggregates within the water column. The larger size of these aggregates results in settling velocities up to several orders of magnitude greater than that of the primary particles of which they are composed. Numerical models that do not represent fine sediment as aggregates often have to coarsen the sediment size distribution entering a reservoir to match the measured reservoir trapping efficiency (Floyd et al., 2016).

(f) Additional guidance on cohesive sediment transport can be found in EM 1110-2-1607. Mehta and McAnally (2008) provide detailed discussion on fine-grained sediment transport. Modeling cohesive sediment transport is further discussed in Chapter 9 of this manual.

(2) For cohesive sediment beds, the hydrodynamic forces working on an individual particle or aggregate of particles must be greater than the cumulative gravitational, grain-related resistance, and cohesive forces working to keep the particle in the bed. For cohesive sediment beds, critical shear stress is generally used to identify initiation of particle movement. Noncohesive sediment bed critical shear stress can be approximated as a function of hydrodynamic conditions, particle size and bed roughness. Cohesive forces are more difficult to approximate.

(a) Mineralogy, organic content, grain size distribution, density/consolidation process, pore water chemistry, structure of the flow-deposited bed and other bed properties will influence cohesive bond strength and subsequent critical shear stress for initiation of motion. Cementing agents, such as oil or iron oxide can also significantly influence erosion potential of a cohesive sediment bed.

(b) Sediment properties influencing critical shear stress and erosion potential are often interdependent. While it is qualitatively understood how individual sediment bed properties influence cohesive forces, there are no ways to quantify critical shear stress as a function of measurable sediment bed properties. Recall that the Shields threshold is valid only with noncohesive sediment.

(3) Cohesive sediment bed erosion occurs whenever the shear stress induced by water flowing over the sediment is sufficient to break the electrochemical interparticle bonds (Partheniades 1965, Paaswell 1973). Under these conditions, cohesive sediment bed erosion occurs as both individual particles and small aggregates. These aggregates are tightly bound sand, silt, and clay clumps that have a density similar to the sediment bed. This type of erosion is time-dependent and is defined as surface erosion or resuspension.

(4) Another erosion mechanism exists for cohesive sediment beds and typically occurs at shear stresses much greater than those required to remove individual particles or small aggregates. This second erosion mechanism is instantaneous and occurs as removal of relatively large pieces of the sediment bed. This process is referred to as mass erosion or redispersion and occurs when the bed shear stress exceeds the bed bulk strength along deep-seated planes and fissures. This type of erosion generally occurs during high-energy events.

(5) Aggregates produced during cohesive bed erosion are tightly bound sand, silt, and clay clumps that have a density similar to the sediment bed. These aggregates can move as both suspended and bedload. Suspension and bed transport of aggregates is often quantified using particle transport theory, but instead of mineral density, particle density is defined as the bed density. However, in reality, little is presently known about bed aggregate transport mechanisms.

(6) Critical shear stress for erosion and the determination of erosion rate as a function of applied shear are site-specific for cohesive sediment beds. No formulations exist to associate sediment bulk properties with erosion processes. Several commonly applied formulas have been developed. Each requires site-specific parameterization. Parameterization will change with varying bed properties. In addition, the erodibility of a cohesive bed typically varies with depth. Density increases with depth due to self-weight consolidation. Increasing density permits more frequent and stronger inter-particle bonds as pore space between particles is reduced.

(7) Several of the most common cohesive sediment erosion relationships are presented in the following sections. All formulas quantify erosion rate as a function of applied shear stress and include variables developed from site-specific measurements and parameterization. There is a high level of uncertainty in parameterization. Naturally occurring beds are variable in bulk properties. Therefore, parameterization could vary over short spatial spans within a bed. Parameters include general bounding values that can be used to represent erosion potential for the majority of the bed. Parameterization provides guidance to bed behavior and should not be considered exact values for critical shear or erosion rate.

(8) Applying the cohesive methods successfully requires estimating the shear thresholds and the erosion rates well. These parameters are site-specific and can differ by five orders of magnitude between sites. Therefore, the parameters can be determined experimentally (see Chapter 4 for methods) or are calibration parameters adjusted to replicate measured bed change.

Briaud (2001) summarizes that no widely accepted correlation could be found between cohesive erodibility and bulk soil parameters after extensive literature review.

(9) Ariathurai and Arulandandan (1978) proposed the following general relationship (modified from Krone (1962)) for erosion rate from consolidated, cohesive beds.

$$\varepsilon = M\left(\frac{\tau_b - \tau_c}{\tau_c}\right) \qquad \text{Equation 5-42}$$

where:

ε = erosion rate (g/cm^2)
M = site-specific parameter developed from measurements
τ_b = applied shear stress due to hydrodynamic forces (dynes/cm^2)
τ_c = critical shear stress for initiation of cohesive sediment bed erosion (quantified through site-specific measurements) (dynes/cm^2)

(a) Erosion is zero for τ_c less than τ_b. Later research produced a similar form of Equation 5-42, which permits for nonlinear relationships between shear stress and erosion rate. The revised equation is (simplified from Gailani et al., 1991):

$$\varepsilon = M\left(\frac{\tau_b - \tau_c}{\tau_c}\right)^n \qquad \text{Equation 5-43}$$

where n = site-specific parameter developed from measurements.

(b) As noted in Gailani et al. (1991), the value of M will decrease with increasing density/consolidation (increasing depth below the original sediment/water interface). Therefore, the sediment bed is often represented by a layered model. Each layer has different values of τ_c and M. Values of τ_c increase with depth below the original sediment/water interface and values of M decrease with depth below the original sediment/water interface.

(10) Erosion experiments in SEDflume (1996) permitted Jepsen et al. (1997) to perform closer analysis of density effects on otherwise similar sediment samples. They produced an erosion equation that includes density effects:

$$E = A\tau^n\rho^m \;\; for\; \tau > \tau_c \qquad \text{Equation 5-44}$$

$$E = 0 \quad for\; \tau < \tau_c \qquad \text{Equation 5-45}$$

where:

E = erosion rate (cm/s)
A, n, and m are site-specific parameters developed from SEDflume data
ρ = sediment bed bulk density (increasing with depth below the initial sediment/water interface) (g/cm^3)

(a) Values of n are typically between 1 and 3. Values of m are typically negative and vary greatly due to the sensitivity of erosion rate to sediment bed density. In general, bulk density profile of the sediment bed is not known in sufficient detail to use Equation 5-44 as a function of continually changing density with depth.

(b) Since the bed is discretized into layers, each having a specified density derived from field data. Equation 5-44 and Equation 5-45 can then be rewritten as:

$$E_i = A_i \tau^n \quad for\ \tau > \tau_{ci} \qquad \text{Equation 5-46}$$

$$E = 0 \quad for\ \tau < \tau_{ci} \qquad \text{Equation 5-47}$$

where:

E_i = erosion rate (cm/s) of layer i
A_i and m_i are site-specific parameters developed from SEDflume data for each layer i
τ_{ci} = critical shear stress for layer I (dynes/cm^2)

(11) Cohesive transport within HEC-RAS includes three methods: applying the standard transport equation or two different implementations of the Krone and Parthenaides methods (HEC 2016a). Standard transport equations will extrapolate well outside its derived range and usually compute enormous (often unreasonable) transport potential. When study objectives make modeling cohesive erosion and deposition important (such as reservoir and low energy backwater applications), then the standard equations are not sufficient. Carefully consider USACE study objectives and project area sediment processes when selecting the cohesive modeling approach.

Chapter 6
Sediment Yield

6-1. Introduction. USACE projects require development of sediment yield estimates as an integral component of project analysis. Sediment production and transport in a watershed are influenced by complex geomorphic processes that vary in time and space. Figure 6-1 illustrates the content of this chapter.

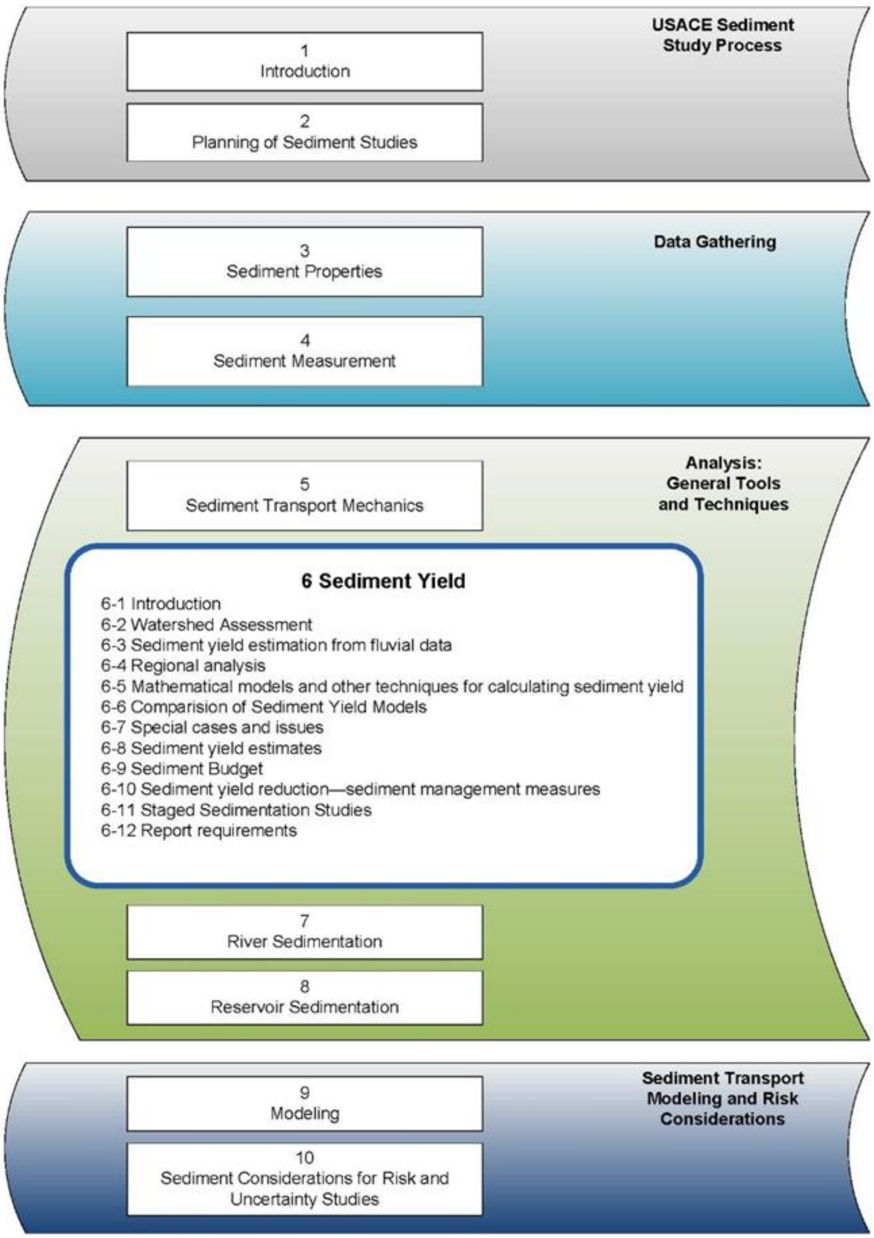


Figure 6-1. Chapter 6 content and general document structure

a. Purpose and Scope.

(1) This chapter presents guidance on the selection and application of procedures for calculating sediment yield, along with the underlying assumptions in these methods. This chapter also discusses field methods to validate these calculations and examines the effects of natural and anthropogenic disturbances on sediment yield.

(2) Key Principles.

(a) Rain droplets impacting on the landscape surface have the potential to dislodge soil particles.

(b) Water moving across and/or through the landscape has the potential to move soil particles.

(c) Eroded soil particles may be redeposited a few inches from where they were dislodged.

(d) Soil erosion or soil loss includes any movement of soil particles from their original location.

(e) Soil erosion or soil loss is not the same as sediment yield.

(f) Sediment yield estimates the amount of soil particles that mobilize due to water movement and that continue their movement either for the duration of the event or until they reach a distant point downstream.

(g) Sediment yield estimates provide a means to quantify the volume of sediment moving within a watershed river/stream system.

(h) The ratio of sediment yield at a point of interest to the gross soil erosion upstream of that point is the sediment delivery ratio (SDR).

(i) Sediment yield is influenced by many factors including slope, soil type, particle size and density, intensity and duration of rainfall, climate, and type of land cover.

(j) USACE study efforts on climate change have investigated potential links to sediment yield through storm intensity, depth, and duration. Additional possible effects include changes in the type of precipitation event (rainfall vs. snow) and vegetative cover conditions.

(k) Due to the spatial and temporal diversity of physical processes and available data, sediment yield estimates often require application of a weight of evidence approach whereby multiple methods (both qualitative and quantitative) are used to inform a framework for estimating baseline magnitudes and uncertainty ranges and identifying short- and long-term trends.

(l) Determination of sediment yield is not normally the end product for sediment investigations for USACE projects.

(m) Sediment yield estimates are an intermediate step in broader studies related to:

- Reservoir sedimentation.
- Local flood risk management (FRM) projects.
- Navigation projects.
- Project alternative formulation.
- Assessing future project conditions.
- Other USACE projects.

b. Major Limitation of Sediment Yield. While sediment yield estimates are a critical component of evaluating sedimentation processes, they are not based on exacting methods. Estimates may be obtained by direct measurements or by empirical methods.

(1) Direct measurement methods are the most rigorous as they are based on quantifying volumes of material deposited or moved over a given time period. During this time period, land use, rainfall, and runoff are known. However, the real need in almost every case is to predict future conditions including long-term trends over extended timescales.

(2) Prediction of future conditions requires the use of empirical methods, or models, to evaluate changes in sediment yield due to altered land use, rainfall, and/or runoff that must be estimated. An additional complication in estimating future sediment yield comes from the need to develop two different predictions: (1) a long-term average to provide results over the project life and for maintenance, and (2) a short-term single event to provide results for local feature analysis.

c. Background and Need for Sediment Yield Studies.

(1) Sediment is produced either when rocks undergo physical or chemical weathering, or when previously weathered particles are dislodged from an existing deposit, such as a soil. If this sediment is produced at an elevation higher than the surrounding terrain, it will begin moving downslope by way of gravity. The rate at which it moves may be fast, such as in a rock fall or debris flow, or may be slow, such as off of an agricultural field in fairly flat terrain.

(2) The sediment movement rate is influenced by many factors, including slope, particle size, particle density, intensity and duration of rainfall, type of land cover, and other factors. Eventually these particles will be delivered to a stream, where they will be transported downstream to a receiving body of water, such as the ocean or a reservoir. Stream bed/banks may also function as sediment sources or sinks that can be a significant sediment source. Once

deposited in the ocean or reservoir, the particle will remain in this sink as part of the sea or reservoir bed. Chapter 8 of this manual further discusses reservoir sedimentation processes.

(3) The elevation difference between the point at which the particle was dislodged from its parent material and that of the receiving body of water represents the potential energy stored in this particle. Consequently, flat landscapes have little potential energy stored in them and, all other things being equal, deliver smaller quantities of sediment to a stream, as compared to steeper terrains.

(4) In summary, the pathway for sediment to move from source to sink consists of sediment production upslope by dislodging from the parent material (or an existing sediment deposit), entrainment by overland runoff and delivery to a stream, where it is subsequently transported to a receiving body of water. The sediment yield processes discussed in this chapter are limited to the production of sediment and its delivery to a stream.

(5) USACE projects are often to construct engineering works in the pathway of this flowing sediment. These works may consist of dredged channels, dams and their accompanying reservoirs, locks, and harbors. These works can be significantly affected by the delivery and transport of sediment. Accurate quantification of sediment supplies is critical when assessing the lifespan and functioning of these engineering works.

d. Fluvial Response to Changes in Sediment Supply. Over long periods of time, rivers evolve to be in balance with the upstream supply of sediment and water (Figure 1-4). While erosion and deposition will always occur in a river, a state of dynamic equilibrium can be achieved in which the dimension (width, depth, cross-sectional area), pattern (sinuosity, belt-width, meander wavelength, and radius of curvature), and profile (slope) of a stream are maintained over time.

(1) If the upstream sediment supply (sediment yield) changes due to land use changes or other watershed disturbances, the river will respond by adjusting its dimension, pattern, and profile. These resulting adjustments will manifest themselves as either erosion or accretion and have the potential to affect the performance of engineering projects.

(2) The ability to predict sediment yield is critical in assessing the performance and lifespan of engineering works constructed in a river. Chapter 7 of this manual further examines these geomorphic principles.

(3) Reservoirs. Any structure built across a river with the purpose of impounding water has the potential to intercept and store fluvial sediment. These dams and their resulting reservoirs are often built for use in hydropower generation, water supply, and flood storage and for recreational purposes.

(a) The interruption in the flow of sediment can affect the performance of the reservoir and may also have downstream impacts. Some of the potential downstream impacts consist of increased erosion of the stream bed (channel incision) and the reduction or loss of fluvial features such as mid-channel and transverse bars, which may be important to the ecological function of certain species. The most significant upstream impact is the loss of storage volume in the reservoir. Additional environmental impacts may also result from the warming of the water in the reservoir, loss of fluvial riparian habitat, and other effects.

(b) A sediment yield analysis will result in the quantification of sediment delivered to the reservoir over time. Depending on methods, a sediment yield model may be used in conjunction with a sediment transport model to deliver the sediment eroded from the watershed to a point of interest, such as a proposed reservoir. Some sediment yield models have the ability to transport sediment to the point of interest; however, these sediment transport routines are usually less sophisticated than those typically used in a true sediment transport model (see Chapter 9 of this manual). Chapter 8 of this manual further discusses reservoir sediment processes.

(4) Local Flood Risk Reduction Projects (Channels and Levees). While reservoirs provide flood risk reduction to downstream areas by reducing peak flows, local projects modify channel or floodplain hydraulics to reduce flood risk.

(a) The performance of local flood risk reduction projects can be adversely affected without knowledge of the upstream sediment supply and project impacts on downstream supply. In general, when a channel is enlarged to accommodate a larger volume of water, its ability to transport sediment is reduced. An engineer must have knowledge of the upstream sediment supply to properly assess the performance of a proposed flood reduction project. The existence of natural levees along the river bank has been observed in many systems that are typically formed when the larger sediment particles deposit as flow exits the channel and enters the floodplain with a corresponding velocity reduction.

(b) The response of a riverine system to levee construction varies due to many factors. Levees constructed near the channel bank may sufficiently constrict flow such that channel velocities are significantly increased and scour results. In other systems, setback levees may result in greater floodplain sediment deposition rates as flood flows are confined to a narrower corridor.

(c) Knowledge of the upstream sediment supplies can come from the application of simple empirical equations, sophisticated numerical models, or the collection of bedload and suspended load in the field, or a combination of these. The quantification of the upstream sediment supplies should include both the quantity (mass/time) and the gradation. Unlike reservoirs, which function as a sediment sink, levees and constructed channels can be both a source and a sink for sediment. Chapter 7 of this manual further discusses river sediment processes.

(5) Channel Projects for Navigation. Although the water-sediment behavior is similar to that in flood risk reduction project channels, the question being addressed is different. A flood project seeks to reduce the stage. A navigation project seeks to provide reliable navigation channel dimensions, width, and depth, at the minimum navigation channel flow rates. These objectives may be complementary or competitive. A significant difference between the two projects is the resolution required to locate problem areas. Also, navigation channel issues generally occur during normal or low within-channel flow periods. Maintenance of navigation

channel dimensions requires consideration of sediment distribution within the channel to avoid issues such as localized shoals.

(6) Ecosystem Restoration Projects. Most ecosystem restoration projects are either in line with the river or immediately adjacent. Projects built in line with the stream will have to be designed to pass the sediment supplied from upstream or aggradation will result. As with the assessment of channel and levee projects, this analysis will require the application of either a sediment transport capacity analysis and/or a sediment transport routing model in conjunction with a sediment yield analysis. The results of the sediment yield analysis will be used as input to the sediment transport analysis or model, which will assess the state of aggradation or degradation throughout the ecosystem restoration project. Copeland (2001) presents more information on sediment impacts to restoration projects.

(7) Watershed Sediment Management. Management actions in a watershed that affect sediment yield are largely due to changes in land use. The other major factors driving sediment yield are slope, rainfall, and the particle’s resistance to erosion; these are only minimally influenced by management actions in a watershed.

(a) In the absence of human disturbances, watersheds in humid regions would be covered with a mature, closed forest or prairie grasses. These undisturbed watersheds produced very low sediment yield. Over time, many of these watersheds were cleared, plowed, and converted to agricultural use. As urban centers began to sprawl, the adjacent agricultural land was incrementally converted to urban or suburban uses. Each of these land use changes would be expected to result in a different sediment yield.

(b) Previously constructed water development and sediment reduction projects that have experienced significant sediment deposition may have reduced storage capacity in the future. The reduced storage capacity will alter sediment yield to downstream points.

(c) Stream bank erosion and floodplain processes can also be a significant factor in watershed sediment yield. Natural disturbances such as wildfires or volcanic eruptions also significantly impact sediment yield. Discerning the effects of anthropogenic watershed activities and natural processes is an important factor to understanding historic and future sediment yield. A combination of sediment yield modeling and field measurements can be used to quantify changes in watershed sediment yield. Additionally, these models can be used to predict the future sediment yield under varying watershed management scenarios.

(8) Other. There are a variety of instances where a sediment yield study will provide useful information that will lead to a more successful federal project. Examples include dam removal studies, evaluating the effects of climate change on navigation channels, and developing a sediment budget. When a project is being considered, sediment studies should forecast a future condition without the project in place to establish trends for how stream stability is changing through time as hydrology and sediment supply adjust to changes in land use, climate, and to other projects in the basin.

e. The Formation, Loss, and Importance of Soil.

(1) Soil erosion is a major problem in the United States and around the world, resulting in a variety of impacts. Annually, 1.7 billion tons of soil are eroded from the 360 million acres of U.S. cropland (NRCS 2010). The physical, biological, and chemical damage caused by sediment is estimated to range from $16 billion (Osterkamp et al., 1998) to $44 billion annually (Pimentel et al., 1995). Greater demand in the future for food and agricultural-based energy sources will create greater pressure on these soils, resulting in even greater loss of this vital and limited resource.

(2) The entire world's population relies on a thin veneer of organic soil for their food supply. The slow loss of this soil through water and wind erosion gradually reduces its ability to support mankind. Soil erosion played a role in the collapse of many prominent societies, including the ancient Greek, Roman, Incan, and Aztec empires, and many others (Montgomery 2007). Loss of soil fertility is often a slow process and can be difficult to perceive on human time scales; however, over a few to several generations, the soil can be irreversibly degraded. Soils develop slowly, requiring a couple to a few hundred years to build an inch of soil (Hunt 1972 and Hall et al., 1982).

(3) The loss of soil affects not only agricultural fertility, but also impacts fish and their spawning beds, increases flood risks, reduces the useful lifespan of reservoirs, causes eutrophication in downstream receiving water bodies, and physically impairs navigation.

(4) This chapter provides information for quantifying sediment yield, processes, and understanding potential impact on USACE project design and operations.

f. Field Reconnaissance of Watershed. An initial reconnaissance visit of the watershed should be made to locate the sources of sediment to the river and, if possible, quantify these sources. The goal of the site visit is to assess the relative magnitude of sediment from the following sources: bed, bank, and overland runoff (sheet/rill erosion and gully erosion).

(1) The equations and models for quantifying these sediment sources are different. Consequently, the engineer must have reasonable confidence in the source of the majority of the sediment to apply the correct sediment yield equation. The site visit should include an examination of the likely sources of upland erosion as well as sources from the bed and banks. An examination of topographic maps, soil maps, and aerial photography can help guide the engineer to the locations of likely sediment sources.

(2) The engineer should particularly note the following features as they may indicate potential sources of sediment: steep stream or valley sections on the topographic maps, confined valley walls creating entrenchment of the river, evidence of headcuts in the topography, areas of highly erodible soil, tight meanders, the presence of a high bank on the outer (cutting) bank, the location of tributaries that may be significant sediment sources, and anthropogenic disturbances such as road crossings, mining activities, urbanization, timber harvesting activities, dams, and similar occurrences.

(3) The engineer should make every effort to make these observations away from bridge crossings as the bridge, with its abutments, wing walls, piers, and scour countermeasures, can significantly influence morphologic and sediment observations at the sub-reach scale. It is easy to arrive at an incorrect impression of the state of aggradation/degradation in a river if the engineer only makes observations at road crossings.

(4) If possible, the engineer should traverse a significant portion of the river by motor boat/canoe/kayak to properly identify the sources and sinks of sediment and to assess the state of aggradation, degradation, and overall channel stability. For larger watersheds, consideration should be given to conducting an aerial reconnaissance to identify processes on a reach scale that might otherwise be overlooked in ground-based investigations. Geo-referenced digital video of the system can be collected as part of the aerial investigation to support subsequent geospatial analyses.

(5) During field assessment, it is recommended that sediment samples be taken of the active bed (surface and subsurface) and of any eroding banks. The gradation of these samples will help characterize the bed and banks in any sediment transport models subsequently developed and in the development of a sediment budget.

(6) If sedimentation is a critical issue in the federal project, then a rigorous sediment yield analysis is recommended early in the project planning process.

g. Methods for Determining Sediment Yield. The large variety of sediment yield methods can be placed into two broad categories: methods based on direct measurement and methods based on relationships developed between hydraulic parameters and sediment transport potential (Garcia 2007).

(1) Direct measurement methods may consist of watershed assessments, yield based on fluvial data, and yield based on reservoir data. Relationship methods may consist of fluvial sediment transport models, models of upland erosion, and models of watershed sedimentation.

(2) Only those based on direct field measurements are considered a rigorous approach; mathematical methods of estimating sediment yield, particularly when applied without direct measurements, may be best applied as trend indicators and techniques for alternative comparisons. However, application of both direct measurements and models are often required when the evaluation of multiple future scenarios with altered historic sediment yield is required.

6-2. Watershed Assessment.

a. Sediment Sources, Transport, and Sinks (by Grain Size).

(1) Concepts of Sediment Erosion, Yield, and Delivery Ratio. Erosion of sediment from upland sources starts when raindrop impacts detach individual particles from the soil matrix. These particles are randomly splashed in a ring around the impact site and may be transported as far as a few feet. While the impact of one raindrop may not be significant, millions of drops on bare soil can mobilize many tons of sediment per acre during a single rain event. When this

erosion (sometimes called gross erosion) occurs on a flat landscape, very little sediment actually leaves the area. As the slope of the landscape increases, more sediment is exported from the site. Sediment exported from a site per unit time is the sediment yield. Typical units of sediment yield are tons/day, tons/year (yr) and yd^3/yr. Sediment yield may be normalized by drainage area and is then tons/year/acre (tons/yr/ac) or a similar unit.

(2) Sediment yield at a point may be calculated as the product of the gross sediment yield upstream of a point and the SDR. SDRs represent the fraction of the gross erosion that is expected to be delivered to a point of interest.

(a) SDRs are typically fairly low and are a function of drainage area, gradation of eroded material, stream length, relief, and other factors. Osterkamp et al. (1998) and Meade and Parker (1985) reported a continental scale SDR for North America to be approximately 0.1. In these cases, 90% of the eroded sediment remained stored on hillslopes, in bottomlands, on riverbeds and in lakes and reservoirs.

(b) SDR tends to decrease with increasing drainage area (Figure 6-2). In this figure, SDRs range from approximately 4% to 60%, and is specific to the Piedmont watersheds examined in Meade (1982). Moreover, SDRs vary greatly across the United States and even within a watershed. As such, the engineer should not assume that the SDR in their study area is the same as reported above. Numerical modeling, field measurements or previously published SDRs should be used to determine the SDR at a particular study area.

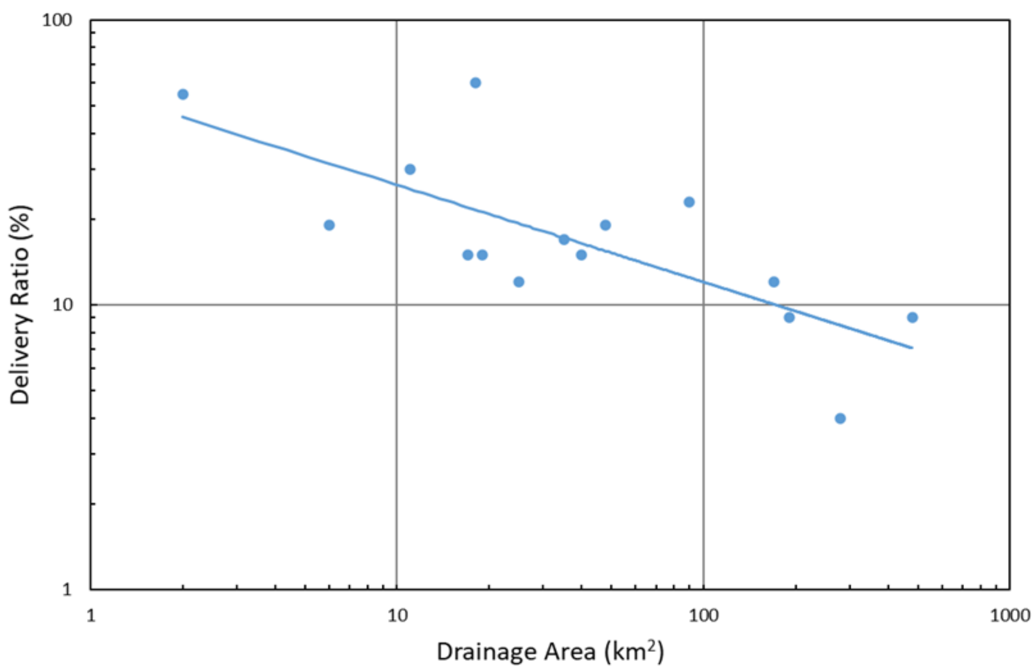


Figure 6-2. Relationship between SDR and drainage basin area in select basins in the Piedmont (redrawn from Meade 1982)

(3) Upland vs. In-Channel Sources. Sediment that is delivered to a federal project, such as a dam or federal navigation channel, typically came from either upland sources, in-channel sources, or both. Upland sources include material eroded from the landscape in the form of sheet, rill, and gully erosion. In-channel sources include material eroded from the bed and banks of the river. When constructing a sediment budget, the engineer must quantify the amount of sediment coming from both of these sources. Riverine bank erosion estimating methods are available using the Bank Stability and Toe Erosion Model (BSTEM) process in HEC-RAS (Chapter 8 and 10; HEC 2016b).

(4) Sediment Fingerprinting. Sediment fingerprinting is a technique in which the source of a particular sediment is identified based on differences in mineralogy, chemical composition, presence and type of organic matter, color, and particle size. Sediment fingerprinting is useful during the development of a sediment budget and can help identify the sub-watershed that a sediment is sourced from, or whether the sediment came from bank erosion or from overland sources. For further information, see Davis and Fox (2009).

(5) Wash Load vs. Bed Material Load.

(a) River sediments will either be in suspension in the water column, moving close to the bed, or lying in storage on the bed. Wash load typically consists of fine material (silts and clays) that remain in suspension in the water column even during low flows. Wash load is the portion of the suspended load that is not found in appreciable quantities in the bed of a river and has been defined by some as material finer than the d_{10} of the active bed (Einstein 1950).

(b) Bed material load may be transported as suspended load (material in the water column) or as bedload (material rolling, sliding, or saltating along the bed). The proportion of the bed material load being transported in suspension will depend on the flow conditions at the time; hence, material that is wash load in one reach may transition to be bed material load in a reach with different hydraulic conditions. Sediment load terms are further explained in paragraph 3-4b of this manual.

(c) Bed material load is typically capacity limited; that is, the channel can only convey a certain amount. When the ratio of bed material supply significantly exceeds its transport capacity, bed material load can be computed based on the hydraulics of flow. If the local sediment transport capacity exceeds locally available sediment load (for bed material load) the river may erode material off the bed and banks to compensate. Within a transport reach the presence of localized bed material source and sinks is common. Conversely, wash load is supply limited. The channel can transport a virtually unlimited quantity, and it is not hydraulically controlled. Refer to paragraph 9-3 for additional discussion.

(6) Floodplain Storage.

(a) A floodplain is the swath of land adjacent to the stream that is inundated during flow events greater than the bankfull event. Floodplains are generally depositional environments but can also act as a sediment source if the flow, and resulting bed shear stress, becomes large enough. Sediment is stored on the floodplain in several geomorphic forms.

(b) Natural levees are ridges of relatively coarse-grained sediment deposited parallel to the channel that are generally near locations where flow first exits the channel and enters the floodplain. When flow exceeds channel capacity and overflows the natural levees, crevasse channels may be cut into these levee features, leaving a fan-shaped deposit (crevasse splays) behind them.

(c) Further landward, sheet deposits of finer sediment are left on the floodplain. In many river systems, these natural levee and floodplain processes can be quite extensive in size and encompass large scale agricultural operations, such as those observed on the Mississippi and Missouri Rivers. Note that abandoned levees, material disposal sites, and other floodplain construction activities are easily mistaken as evidence of ongoing large scale natural processes.

(d) Deposition of sediment on the floodplain may be a significant portion of the sediment budget and can be quantified with field measurements. One method to quantify deposition on a floodplain is to distribute a tracer layer of contrasting sediment (such as feldspar clay, which is often a bright color) on the floodplain and measure the thickness of the deposit above this marker. Other techniques using radionuclides, luminescence dating, fallout, and environmental nuclides are discussed in Kondolf and Piegay (2003). A detailed review of floodplain dynamics can be found in Bridge (2003) and Anderson et al. (1996).

b. Regional and Spatial Variations in Sediment Yield.

(1) Sediment yield varies greatly across the United States. In regions with a lot of relief, the yield is expected to be high. In regions that experience a lot of precipitation, the potential to produce and deliver sediment to a stream is also great. Interestingly, arid regions also have great potential to mobilize sediment due to the lack of protective vegetation. Langbein and Schumm (1958) discussed this interplay between the erosive and resistive forces in humid versus arid regions and concluded that the least yield comes from regions that get enough precipitation to sustain protective vegetation, but not so much that excessive overland runoff is created. Figure 6-3 shows a graphical representation of this argument (Langbein and Schumm 1958).

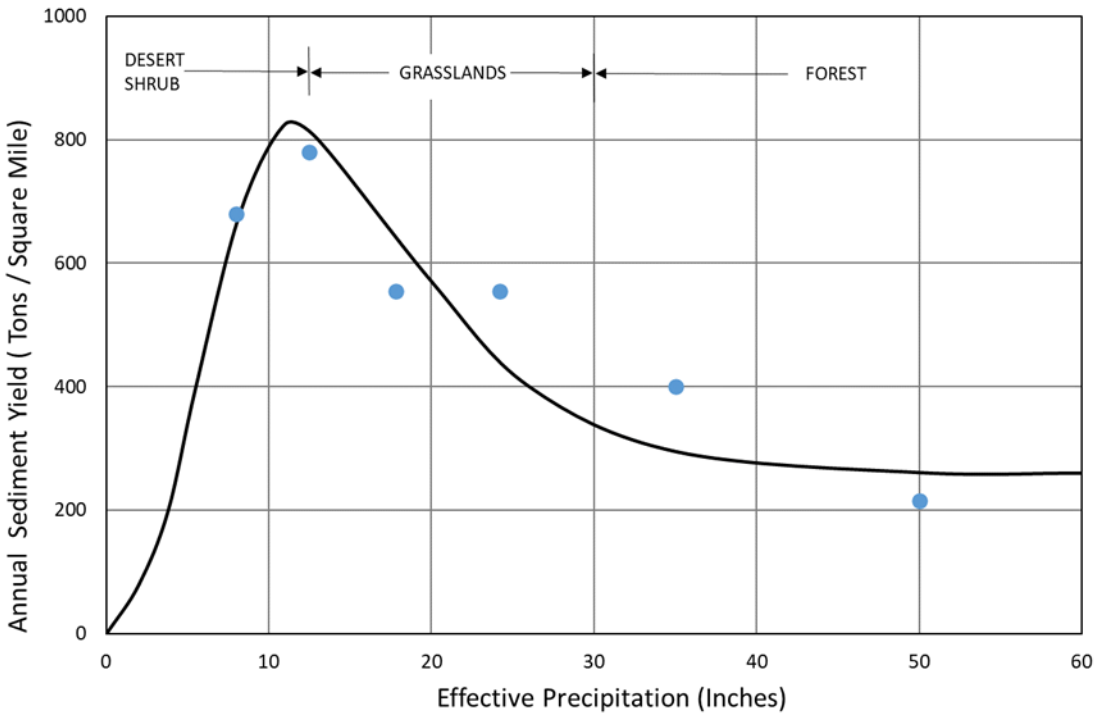


Figure 6-3. Climatic variation of yield of sediment (redrawn from Langbein and Schumm 1958)

(2) The Natural Resources Conservation Service (NRCS) regularly produces maps that estimate erosion rates on cropland in the United States. Figure 6-4 shows an example that illustrates the spatial variability in sediment yield for 2010. Other forms of sediment yield maps are also available that focus on particular areas of emphasis such as ecoregions (Simon 2006). Erosion rate maps depict a combination of many factors, such as climate, soil texture, and soil management that all play a role in the erodibility of croplands.

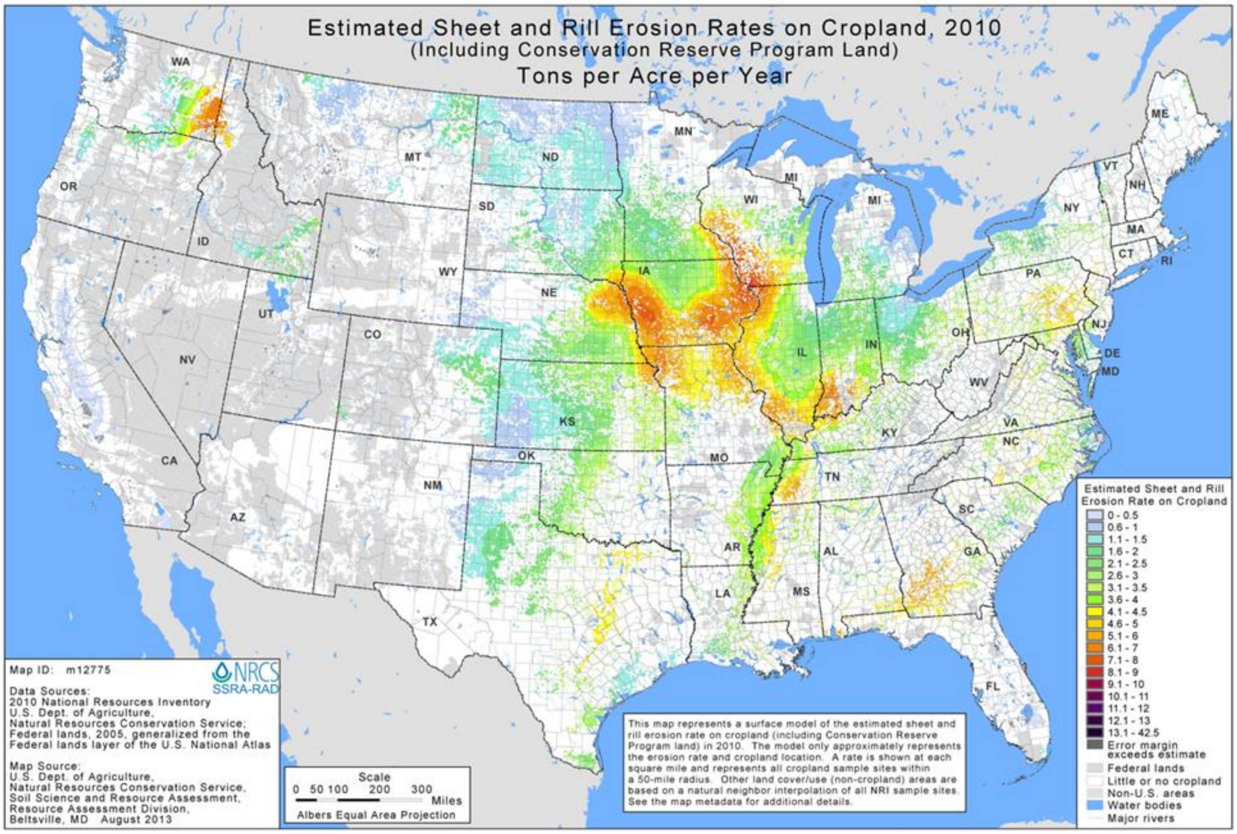


Figure 6-4. Typical erosion rate map (NRCS 2010)

Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

c. Temporal Variation in Sediment Yield.

(1) It is common to find orders of magnitude of variance in a sediment rating curve. The sediment rating curve derived from USGS data at Judy's Branch in East St. Louis, Illinois, illustrates this variability (Figure 6-5).

(2) All of these data points were collected at different instances in time. While some of the variance may be due to sampling error in the field, the remainder represents the temporal variance in sediment yield inherent in rivers.

(3) The temporal variability in sediment yield occurs on multiple scales. At the scale of a single storm event, it is common for the SSC to vary for the same flow, depending on the rising or falling limb of the hydrograph. Refer to paragraph 9-3 for a further discussion of variability in sediment flow-load rating curves.

(4) A supply limited system may run out of available sediment to be transported before the hydrograph passes, resulting in variation at the same flow. Other short-term variations have to do with the nonuniform nature in which precipitation is distributed across a watershed and the timing of flows and sediment supplies from its tributaries.

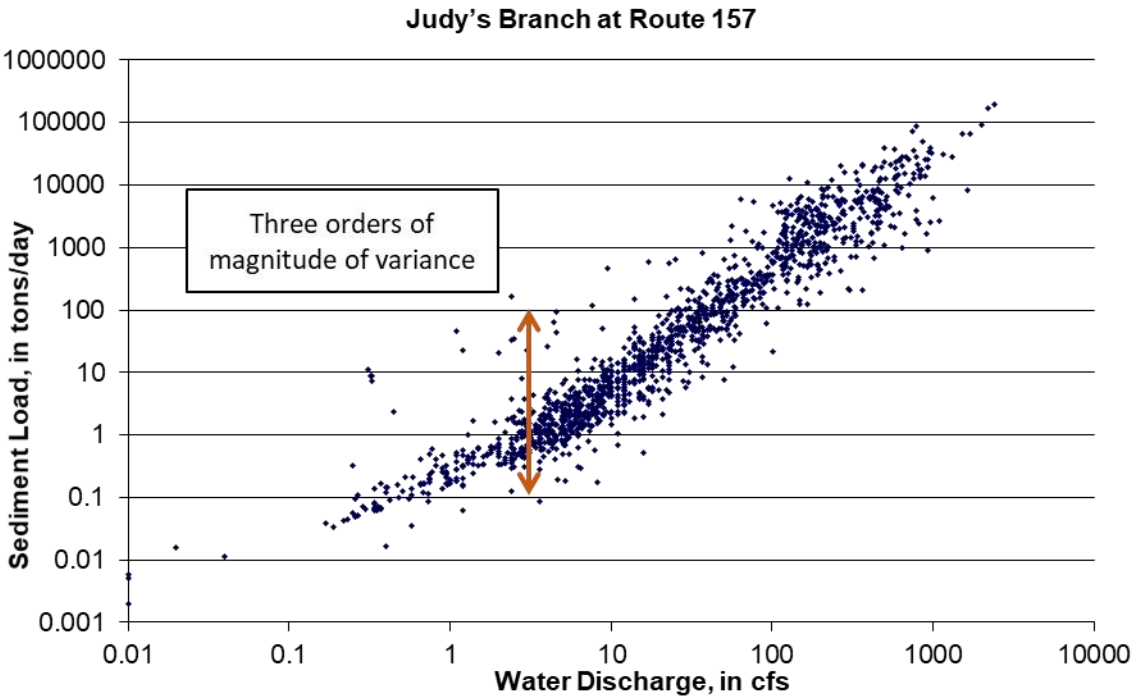


Figure 6-5. Sediment rating curve, Judy's Branch in East St. Louis, Illinois

(5) Long-term, seasonal variations may be observed in regions where winter conditions result in frozen soil. A 3-in. precipitation event on frozen soil will result in a vastly different sediment yield than the same event in the spring or summer.

(6) The timing of precipitation events can add variation in the sediment yield. For example, over time, burrowing organisms such as worms, gophers (Yoo et al., 2005), and others "load" up the landscape with easily mobilized sediment. After a precipitation event, these deposits are transported away. One can see how the same precipitation event occurring on consecutive days is likely to result in greatly differing sediment yields due to different antecedent conditions in the landscape.

(7) Land management and vegetative cover are also factors with seasonal variation that affect sediment yield. For instance, the sediment yield off of a recently plowed field may be much greater than that from a field with a mature crop present.

d. Reliability of Sediment Yield Estimates. Reliability of sediment yield estimates varies widely by methodology, data sources, and data accuracy. In general, developing sediment yield estimates using multiple methods should be considered when feasible. Sediment yield reliability should be considered as a factor inherent in further sedimentation processes evaluations.

e. Geomorphic Assessment of Watershed.

(1) Sediment production and transport in a watershed are influenced by a complex set of geomorphic processes that vary in time and space (HEC 1995). These processes range from

detachment of soil by raindrop at the particle size scale to channel degradation and bank erosion at a reach scale. Other geomorphic processes affecting watershed sediment yield include rill and gully erosion, landslides and hill slope failures, wind erosion, land use changes, and wildfires. Geomorphic process impacts on watershed sediment yield can vary significantly with long-term cycles of watershed development, rejuvenation, and channel geometry evolution (headcuts, channel migration/avulsion). The spatial and temporal variability in sediment production, transport, and deposition within a system makes estimating watershed sediment yield a challenging task.

(2) Valuable insight into the morphologic processes that influence sediment yield of a watershed can be gained through a geomorphic assessment of the system. A geomorphic assessment provides the process-based framework to define past and present watershed dynamics (Biedenharn et al., 1997) that influence sediment production and transport in the system. The understanding gleaned through a geomorphic assessment provides a foundation for proper application of sediment yield estimation techniques.

(3) In terms of estimating sediment yield of a watershed, the geomorphic assessment helps in identifying sediment sources, pathways, and sinks. Schumm (1977) presented a generalized fluvial system characterized by three physiographic zones: (1) a sediment production (source) zone, (2) a sediment transfer (pathway) zone, and (3) a sediment deposition (sink) zone (Figure 6-6).

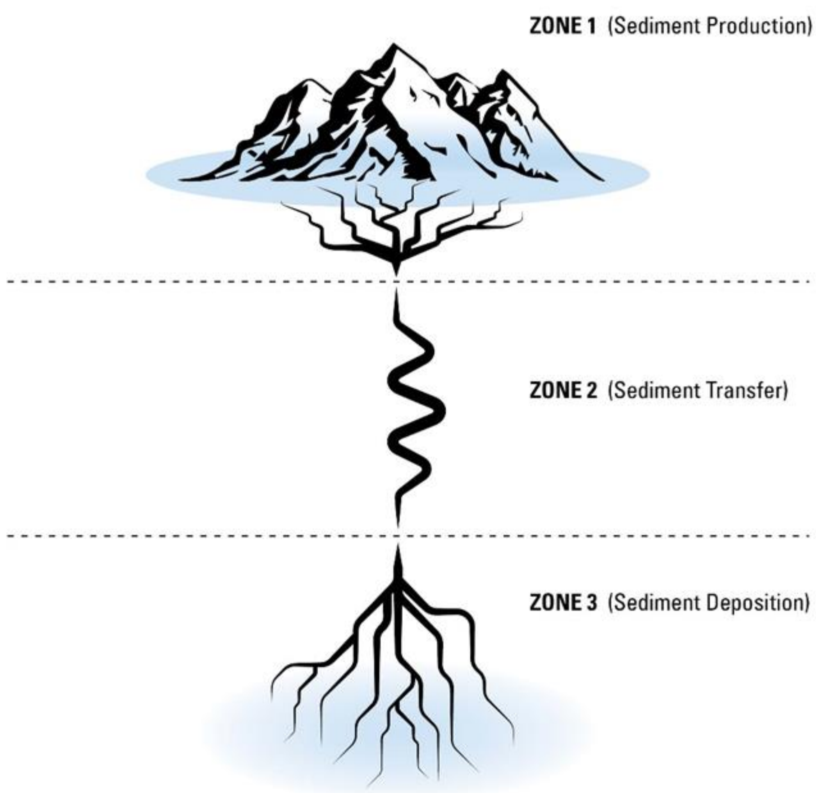


Figure 6-6. Generalized fluvial system (adapted from Schumm 1977)

(4) Sediment production zones are typically located in the upper portions of the watershed, with sheet/rill erosion and gulley erosion being the dominant erosional processes. Sediment production in this zone is generally dependent on soil type, vegetative cover, hydrologic conditions, and land use.

(5) Sediment transfer zones are characterized by fluvial transport processes that can vary with time. Hydraulic capacity of the channel and the frequency and duration of transport competent flows have a major impact of the morphology of the reach. It is important to note that sediment may be temporarily stored in this reach in the form of point bars, mid-channel bars, or braided channels. In addition, sediment production can occur in this reach through channel bed and bank erosion and channel avulsions. This is particularly true in the case of streams that have been disturbed through channel incision and are no longer in a state of dynamic equilibrium.

(6) Sediment deposition zones are generally located in the lower portion of the watershed and are characterized by mild channel slopes, broad flood plains, and deltas or alluvial fans. This zone is dominated by sediment deposition, with the median grain size of deposited material becoming generally finer in the downstream direction. Sediment deposition is this zone is often detrimental to flood control channels and navigation channels and harbors, as well as ecologically sensitive areas.

(7) A geomorphic assessment integrates historic data, field investigations, and analytical techniques to form a comprehensive understanding of the stability of the system and helps to identify the dominant fluvial processes in the watershed. Biedenharn et al. (1997) provide guidance on a generalized methodology for conducting a geomorphic assessment. The method is divided into three components: (1) data gathering, (2) field investigation, and (3) channel stability assessment.

(a) Data Assembly. A comprehensive assembly of historic data is essential to a geomorphic assessment and enables identification of trends and rates of change in the system. The types of problems in the system and the overall project objectives can dictate the types of data required. In general, data relevant to a geomorphic assessment for support of a sediment yield study might include channel and reservoir surveys, stream gage and discharge records, suspended sediment measurements, channel bed and bank material samples, flood history, topographic maps and aerial photography, land use data, and geologic data. Storing data in a geographic information system (GIS) database is desirable.

(b) Field Reconnaissance of Watershed. A field investigation of the watershed is essential in identifying stream characteristics that are indicators of geomorphic processes occurring in the basin. An aerial reconnaissance of the watershed, if practical, can provide valuable insight into basin-wide processes. Ground-based investigations provide the means to determine problem areas of channel bed and bank instability, identify sediment sources, and locate structures and other channel controls. Additional information related to field reconnaissance is presented in Appendix C. A site visit to investigate study area conditions is a critical foundation for USACE sediment yield studies.

(c) Channel Stability Assessment.

- The channel stability assessment can use various tools and analytical methods ranging from simple empirical methods to complex numerical sediment transport models. These tools can include specific gage analyses, comparative geometry of channel dimension, pattern and planform, tractive force and permissible velocity methods, regime methods, equilibrium slope by reference reach, and 1D hydraulic and sediment transport models.

- The final geomorphic assessment of the watershed integrates the information collected from the data gathering, field investigation, and channel stability tasks. Results from various analyses may often be contradictory, requiring sound engineering experience and judgment to arrive at an accurate resolution. Ideally, the final geomorphic assessment will provide a description of historic channel stability based on trends identified in the assessment. In terms of watershed sediment yield, the geomorphic assessment should provide a comprehensive understanding of how sediment processes in the watershed have evolved from a spatial and temporal aspect.

- Additional information regarding channel stability assessment and typical geomorphic process encountered in typical USACE river engineering and analysis is presented further in Chapter 7 of this manual and a stability assessment in Case Study 7A (Appendix N).

6-3. Sediment Yield Estimation from Fluvial Data. In-stream sampling techniques are documented in Edwards and Glysson (1999) and the USGS (1978). This is the most reliable approach, and the several methods presented in the following subparagraphs are listed in the order of preference. Refer to Chapter 3 and 4 of this manual for additional information related to sediment sampling methods and a definition of critical concepts including measured, unmeasured, bed material, and wash load.

a. Published Long-Term Daily Discharge Records.

(1) The most accurate historical sediment discharge is that calculated from a long-term sediment gage record. The standard procedure used by the USGS is to plot the daily water discharge hydrograph and the daily sediment concentration graph, then integrate them as illustrated in Porterfield (1972) and also Gray et al. (2008). These records usually express sediment concentrations in milligrams per liter, and those units can be converted to tons per day, as previously discussed in paragraph 3-4.

(2) Usually, only the measured load is published; however, suspended samplers do not measure the entire height of the water column. Due to physical limitations of the sampler and site conditions, the typical measurement does not capture the bottom portion of the water column nearest to the bed where suspended concentrations and bedload are generally highest. The sediment concentration in this unmeasured zone is commonly estimated based on ratios for available data, and that value is added to the suspended load to estimate the total load.

(3) For large sand-bed rivers, estimates of the unmeasured zone have been made using 5% to 15% of the measured load, but this is dependent on site conditions and may be a good

approximation only for the annual average load. Using the ISSDOT technique, discussed in Chapter 4, for measuring bed material load from multi-beam bathymetric data on the Mississippi and Missouri Rivers, results show that bedload transport can account for 40% of the total bed material load at low to moderate flows (Abraham et al., 2016; Abraham et al., 2011).

(4) Since the bedload fraction typically decreases at higher flows, the 5% to 15% estimate may be a reasonable estimate of the annual average unmeasured load. A further factor for consideration is that the wash load fraction of the suspended load can vary widely depending on the characteristics of the watershed and channel. Therefore, adoption of the 5% to 15% rule of thumb is often not applicable to total annual load and should not applied in some river systems (also refer to paragraph 3-4, paragraph 4-16, and Case Study 4A (Appendix N).

(5) Before comparing annual sediment yields, the period-of-record data should be examined for homogeneity. Adjustments for upstream reservoirs, the hydrologic record, land use changes, and farming practices may be necessary before the correlation between sediment yield and water yield can be established.

(6) Sediment measurement and calculations are published over a variety of time scales, ranging from instantaneous to daily to annual. The engineer must consider how the data is going to be applied and choose an appropriate time scale. For example, a small or highly impervious watershed may respond to a precipitation event so rapidly that using a daily average will not provide enough information about the actual concentration during the event or about the peak to be useful. In addition, the variability of sediment-loading patterns and potential for seasonal hysteresis as illustrated in Figure 8-9 should also be considered when interpreting sediment measurements.

b. Period Yield Sediment Load Accumulation. This technique is used by the USGS to calculate monthly and annual suspended-sediment yield after the long-term mean daily values have been computed. Summations generally use the average daily sediment discharges, but they may be hourly for smaller streams. Yearly sediment yield summations may be computed with monthly or weekly loads in some instances, such as rivers downstream of a major reservoir with little tributary contribution or reaches of major rivers where the discharge is fairly constant for long periods of time. Determine an appropriate time interval based on project reach conditions.

c. Flow Duration Sediment Discharge Rating Curve Method.

(1) The flow duration sediment discharge rating curve method for computing sediment yield is a simple integration of a flow duration curve with a sediment discharge rating curve for the outflow point of the basin to generate an effective discharge curve. It is a commonly used USACE method and is discussed in more detail in Case Study 6D (Appendix N).

(a) Both the flow duration curve and the sediment discharge rating curve are process-based and can be changed from the historical values needed for hind-casting to values needed for forecasting water and sediment runoff in the future.

(b) These curves can be developed to reflect specific components of the sediment runoff process (a sediment discharge rating curve can be calculated for bed material load composed of sand and gravels when those are the types of sediment of most interest to project performance) as well as seasonal (vs. annual) flow duration curves.

(2) The sediment discharge rating curve is a relationship between water discharge and sediment discharge; an example is provided for the Elkhorn River at Waterloo, Nebraska, (Figure 6-7) illustrating the commonly applied log transforms to both the flow and sediment discharge measurements.

(3) A known limitation of this approach is that least-squares regression generates bias in logarithmic relationships. Log-transform regressions return the geometric mean instead of the arithmetic mean, which will underestimate loads dramatically. Figure 6-7 includes trend lines from both the measured data least-squares regression and after applying an unbiased corrector developed from methodology proposed by Duan (1983). Refer to paragraph 9-3 for a further discussion regarding the use of unbiased correctors with measured sediment data. Figure 6-8 shows the flow duration curve of mean daily water discharges at that same location. A thorough discussion of the effective discharge calculation is presented in Case Study 6A (Appendix N).

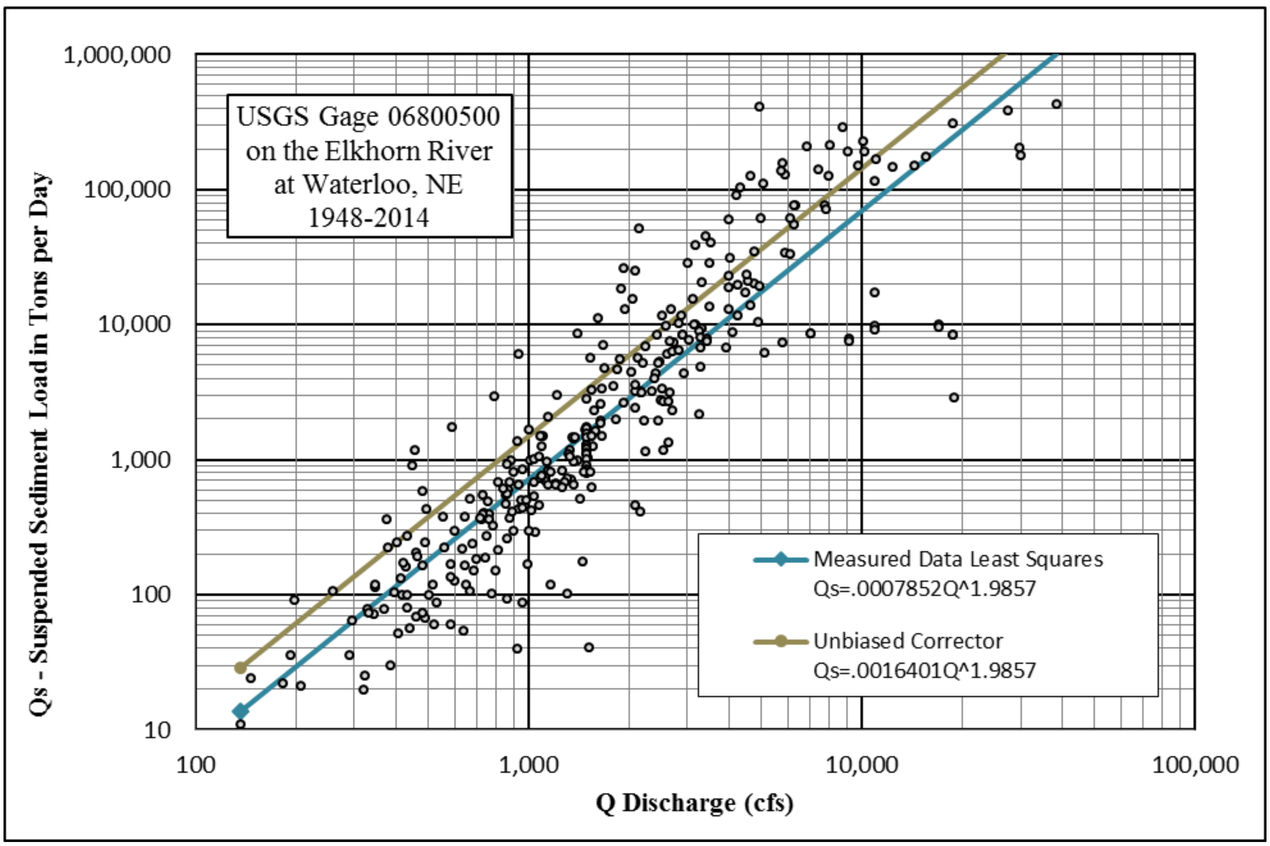


Figure 6-7. Sediment discharge rating curve, Elkhorn River at Waterloo, Nebraska, using least-squares and unbiased corrector

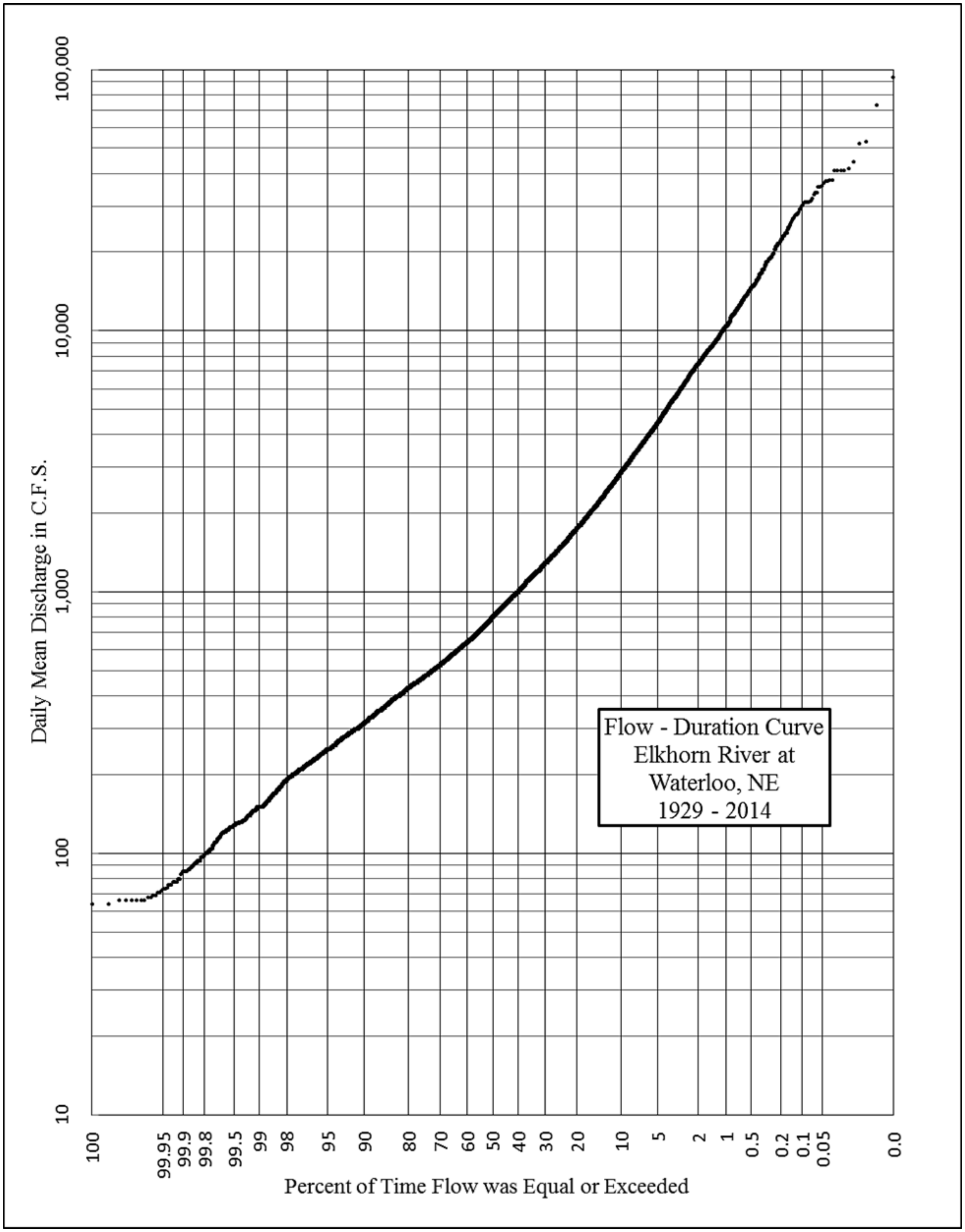


Figure 6-8. Flow duration curve, Elkhorn River at Waterloo, Nebraska

(4) Points of caution regarding the flow duration sediment discharge rating curve method for computing sediment yield are as follows (also see paragraph 9-3):

(a) The sediment discharge rating curve is plotted as water discharge (Q) vs. suspended-sediment discharge (Qs) on a logarithmic scale. However, the typical amount of scatter in such plots indicates that sediment discharge is not a simple function of water discharge. Consequently, the user should investigate and evaluate any regional and watershed characteristics that might contribute to variability in the data. For example, the water discharge (Q) should be plotted vs. the SSC in ppm to avoid the autocorrelation of having Q in both the dependent and independent variables of the sediment discharge rating curve.

(b) Tests for data homogeneity with respect to seasonal effect, systematic changes in land use, type of sediment load, and type of erosion mechanisms may need to be conducted. Use a multiple correlation approach coupled with good engineering judgment to establish the dominant factors influencing historical concentrations. Predict how those factors might change in the future and how such changes will impact sediment concentrations and particle sizes. Gray and Simões (2008) discuss the application of seasonal separation, and other causes of scatter in sediment discharge records.

(c) For flashy streams, peak discharge values may be significantly more than mean daily values. Use of mean daily discharges in these situations may grossly underestimate basin sediment yield. In these cases, the use of a smaller discharge time interval may be warranted for development of the flow duration curve.

(d) Most studies of riverine channels focus primarily on the bed material portion of the entire sediment yield, as the bed material load is commonly more important in terms of understanding and predicting channel morphology.

(e) The amount of wash load in the sediment discharge may increase the amount of scatter in the data because wash load is a function of supply rather than hydraulic transport. Also, as the concentration of fines increases above 10,000 ppm, the transport rate of sands and gravels is increased (see discussion of sediment transport functions, paragraph 5-7). Wash load can be a significant sediment volume contributor in reservoir studies as further detailed in Chapter 8.

(f) Water temperature can cause a significant variation in transport capacity of the bed material load. Separate warm and cool weather sediment discharge rating curves may be required to capture the seasonal variability in sediment yield.

(g) Sediment samples should be separated according to population for later analysis. For example, land surface erosion caused by sheet and rill processes is strongly correlated with rainfall impact energy. Therefore, the correlation of in-stream sediment concentrations with water discharge from rainfall-runoff, (which has different erosive mechanisms than the snowmelt-runoff process), may show an improvement when compared with the correlation of the entire data set. Artificial floods, such as the pond break-out that occurred on the avalanche formed by the May 1980 eruption of Mt. St. Helens, will contain yet another discrete population of erosive mechanisms. Data from such events should be analyzed separately from both snowmelt- and rainfall-runoff events.

(h) Use caution when extrapolating the sediment discharge rating curve to water discharges well above the range of measured data. First consider extrapolating concentrations (SSCs), rather than suspended-sediment discharge rates. Lines of constant SSC (SSC = 1,000; 10,000; 100,000; and 1,000,000 ppm) can be plotted along with the measured data to be used as guides for rating curve extrapolation. The maximum possible concentration is 1 million ppm, which is solid rock, so be careful not to extrapolate beyond a realistic range. As a final step, convert the relationship back to a sediment discharge rating curve and check the appropriateness of the resulting sediment rating curve.

(i) Extrapolating the relationship for the total SSC does not guarantee the proper behavior of individual grain size classes. Check each size class of interest before accepting the results.

(j) Hyperconcentrated flows are generally defined as having SSC > 100k ppm where the sediment volume exceeds 5% of the total volume, while viscous mud-flows can average 50% or more of the total volume (SSC ~700k ppm). Note that the water-sediment fluid behavior changes with high concentration levels and that stated threshold concentrations vary by reference. At high concentrations, the water-sediment fluid mixture no longer behaves as a Newtonian fluid but becomes non-Newtonian. Extrapolating data to very different sediment processes, such as hyperconcentrated flows, should be avoided. See paragraph 3-4 for a discussion on sediment concentrations and related references for additional details.

(k) The magnitude of variation in SSC between events can be as great as that which occurs from one discharge to another within a single event. Developing an SSC rating curve for a single event analysis should account for any variability. Statistical methods with bias correction should be used to develop 95% probability curves as well as the mean value curve. Sediment yield computations should be conducted using the mean value curve as well as the probability curves to address the sensitivity of the results to the uncertainty in the rating curve.

(l) The flow duration sediment discharge rating curve method is considered to give a reliable estimate of sediment yield. However, historical data from long-term records should always be used where available to check the calculated yield results. Adjustment of the sediment rating curve may be required to reproduce historical values.

(m) Western regions of the United States can experience pronounced wet and dry periods that may require separate suspended-sediment rating curves for the early rainy season from the remainder. Aeolian mechanisms are particularly active during the dry season, which may leave an abundance of easily erodible material for the beginning of the subsequent rainy season. As that supply of material is exhausted by the early precipitation events, the sediment runoff can shift from one having a very high concentration due to abundant supply to one having a lesser concentration based on transport capacity. Variation can be expressed by using seasonal curves for suspended-sediment discharge and flow duration.

(n) Caution should be used in applying this approach to ice-affected rivers. Sediment data are very seldom collected under these conditions, even though the large flows associated with ice cover breakup often result in the largest sediment transport of the year (Ettema 2008). In general, ice cover will reduce the influx of overland sediment to the stream, but may increase contributions from the bed and banks due to scour from both constrained flow and interactions of the ice and banks.

d. Dredge Data and Other Surrogates.

(1) Other methods may also be available to calibrate/validate sediment yield calculations. The application of a sediment yield equation or model will predict a quantity of sediment eroded from the landscape and delivered to a stream, and in some models, the sediment will be transported downstream to a point of interest. The measurement of SSCs and bedload transport rates in the stream during the time period simulated by the model will allow assessment specific model accuracy. However, it is not always possible to make field measurements due to project schedules or budgetary constraints.

(2) Indirect or surrogate measurements are sometimes used to validate a numerical model. For instance, if the volume of sediment stored behind a dam is known, a comparison can be made between the observed volume and that predicted by the model or equation. Similarly, federal navigation channels often act as sediment traps allowing the volume deposited in them to be compared to the predicted yield. Caution should be applied when using this approach for model validation as the trapping efficiency of a navigation channel will vary among and within rivers and will rarely ever have a 100% trapping efficiency. It may be necessary to apply a sediment transport model (Nairn and Selegean 2014) to better understand the trapping efficiency in a navigation channel.

(3) Dredging operations may be constrained by budgetary and logistical considerations. Therefore, quantities produced by individual dredging events or annual quantities may not represent recent deposition rates. Inconsistencies may be identifiable by comparing dredging quantities to estimated sediment yield. One of the benefits of using dredge quantities as validation for sediment yield models is that the dredging history can usually be reconstructed going back decades, a rare quality in sediment validation data.

(4) Municipal water treatment plants often draw their intake water from rivers and may have a useful long-term record of SSC of their influent.

(5) Turbidity, while not universally convertible to SSC, is a qualitative measure of the suspended and dissolved material in the water column. Where possible, suspended-sediment samples should be taken in conjunction with the turbidity measurements so that a correlation specific to that site can be developed. In the absence of this correlation, the engineer can rely only on trends and peaks in the turbidity to gain insight into the validity of the predicted sediment yield.

e. Recommendations for a Limited Sampling Program.

(1) Sediment sampling is often limited by project schedules and budgets. Under such constraints, the engineer should keep in mind that the majority of sediment typically moves during the largest few events each year and subsequently target those events for sampling. Flexibility is needed to sample these events since they often occur during inconvenient times and during inclement weather. A limited sampling program should also be specifically targeted to address project objectives. However, budget or schedule decisions that result in the collection of poor quality or inconsistent data will be detrimental to overall study objectives and should be avoided.

(2) Points of Caution for Limited Sampling. The same points are appropriate that were discussed for the flow duration sediment discharge rating curve method (paragraph 6-3c(3)). In addition, consider the following because the short record provided by a limited sampling program will not necessarily provide a representative sample set.

(a) This yield estimate should be regarded as less reliable than values determined by the flow duration sediment discharge rating curve technique because the data may not be representative of the long-term sediment concentrations from the watershed. The absence of floods or the occurrence of one or two large events may bias the yield calculation.

(b) Since there is less confidence in yield estimates, sensitivity tests should be performed to evaluate the implications of shifts in the load curve for the alternative being analyzed. If doubling or tripling the sediment discharge does not greatly affect the alternative under study, additional sediment data may not be necessary.

(c) Since sediment discharge curves are often displayed as a straight-line relationship logarithmically against discharge, and often with a slope of about 2, anticipation of that “rule of thumb” slope is comforting when working with a limited amount of measured data. However, in sand-bed streams, it is recommended to use sediment transport functions to curve-fit and extrapolate the sand discharge data. In gravel-bed streams (with a relatively high energy regime), sand is more commonly transported in suspension behaving like wash load, but sediment transport functions are useful for curve fitting and extrapolating the gravel fraction discharge.

(d) There is no rule of thumb, nor is there a generalized transport function, for the amount of wash load in a stream. A correlation has been observed, at some locations, between the fraction of the bed material present in the suspended-sediment samples and the total concentration. If present, such a correlation allows the wash load to be provisionally extrapolated because the bed material discharge can be calculated using transport functions.

(e) Sediment yield estimates often rely upon a weight of evidence approach. As such, use a variety of methods when field data are inadequate. Always include sediment transport calculations for the sand and gravel loads, evaluating sensitivity when appropriate. Consider using numerical models to interpolate missing data by transposing existing records.

(f) Where a limited sampling program can be scheduled and funded before the start of detailed studies, this technique becomes quite valuable to supplement/modify the results of other methods. If such a program was not possible during the feasibility report stage, one is strongly recommended for the design phase.

f. Sediment Yield Estimates from Reservoir Sedimentation Data. Sediment yield estimates from reservoir sediment surveys can be an accurate method. Chapter 8 of this manual provides a detailed discussion of the computation methods to determine reservoir capacity, comparison of reservoir survey data to estimate sediment yield, and factors that can affect sediment yield including reservoir trap efficiency (TE), sediment size, sediment settling velocity, and sediment consolidation.

g. Transfer of In-Stream Data. A wide variation in sediment discharge curves will be seen at different locations along a stream because minor changes in hydraulics (such as velocity and shear stress) can produce a significant change in the sediment transported. Therefore, transfer of sediment discharge rating curves from one point in a watershed to another point is discouraged. However, converting the discharge curve data to an annual sediment yield curve will usually result in a consistent relationship with drainage area, when land use, topography, and soils are similar. A plot of annual sediment transported against annual discharge can be used to estimate yield at different locations using the technique presented in the next paragraph.

h. Transfer of Reservoir Deposition Data.

(1) Sediment yield data calculated at a specific reservoir site can be transferred to the study watershed provided the topography, soils, and land use, particularly the percentage of both basins in agricultural use, are similar. If these similarities exist, transfer can be made by techniques described in USDA (1975). NRCS, as described in National Engineering Handbook (Section 3, Chapter 8), uses the following practices in transferring reservoir data east of the Rocky Mountains:

(a) Direct transfer for study watersheds greater than 0.5 or less than 2.0 times the drainage area of the reservoir surveyed area.

(b) No transfer for study watershed less than 0.1 or greater than 10.0 times the drainage area of the reservoir surveyed area.

(2) Application of the following equation for study watersheds within these boundary limits:

$$Y_e = \mathrm{Y}_m \left(\mathrm{A}_e/\mathrm{A}_m\right)^{0.8} \qquad \text{Equation 6-1}$$

where:

Y_e = the total annual sediment yield estimated for the area under study, tons/year
Y_m = the total annual sediment yield measured at the reservoir site, tons/year

A_e = the contributing drainage area for the site estimate
A_m = the contributing drainage area for the reservoir measurement

(3) These guides do not apply to mountainous areas, which often show no consistent change in sediment yield for change in drainage area, or to streams where channel erosion may increase the sediment yield per unit area relationship with increasing drainage area.

6-4. Regional Analysis.

a. Regional analyses have been performed for some areas of the United States and sediment yield is shown on maps, by graphs, or with equations based on definable parameters. These methods should be regarded as suitable for preliminary procedures and are suggested as alternatives to support other, more detailed, methods. Regional methods should not be the only techniques used to calculate sediment yield. Regional methods are presented for reference use and to provide historic context. Webb et al. (2001) present an application of regional methods for estimating sediment yield of ungaged tributaries. For most USACE study needs, watershed modeling has replaced regional analysis for determining sediment yield.

b. In choosing a regional method, always justify that the regression parameters include the erosive mechanisms that are predominant in your particular area of the region. That is, drainage area is an adequate parameter for land surface erosion, but it should not be correlated with stream bank erosion or even gullying. If these latter two are the predominant erosive mechanisms in your specific problem area of the region, avoid a regional equation that only includes drainage area. A few regional methods are described below.

c. Dendy and Bolton Method (1976).

(1) This equation for sediment yield, developed by Dendy and Bolton (1976), has the widest potential application in the United States. Sediment yield from about 800 reservoirs throughout the continental United States was related to drainage area and mean annual runoff by the following two regression equations:

(2) For watersheds having a mean annual water runoff equal to or less than 2 in.:

$$s = 1280 * (Q^{0.46}) * (1.43 - 0.26 \log A) \qquad \text{Equation 6-2}$$

(3) For watersheds having a mean annual water runoff greater than 2 in.:

$$S = 1958 * [e^{0.055 * Q}] * (1.43 - 0.26 \log A) \qquad \text{Equation 6-3}$$

where:

s = unit sediment yield for the watershed, tons per square mile per year
Q = mean annual water runoff for the watershed, inches
A = watershed area, square miles
e = 2.73

(4) Since these equations were developed from average values of grouped data, they are appropriate for general estimates. A better estimate can be expected for the larger, more varied watersheds than for smaller site-specific areas. Do not use these equations for mountainous areas.

d. Pacific Southwest Interagency Committee Method. The Pacific Southwest Interagency Committee (PSIAC) method (Wischmeier and Smith 1978) was developed for planning purposes and is applicable for basins in the western United States greater than 10 square miles.

(1) Sediment yield is directly proportional to the total of the numerical values assigned to nine different factors: land use, channel erosion\sediment transport, runoff, geology, topography, upland erosion, soils, ground cover, and climate. Numerical values range from 25 to -10 for each factor.

(2) Sediment yield can range from 0.15 acre-feet per square mile per year for watersheds with low PSIAC factor (20) to more than 3 acre-feet per square mile per year for large factors (100 or more). The PSIAC technique has compared well with actual watershed data and is one of the few methods that can estimate changed sediment yield caused by local land use management changes.

e. Tatum Method for Southern California (1963). This is a special application used to determine the sediment yield into flood control debris basins in mountainous terrain that was originally developed by Tatum in 1963. Subsequent analysis by the USACE Los Angeles District has further refined the unit debris yield estimates (USACE 2000a). It was developed specifically to calculate sediment yield and debris volumes for the arid, brush-covered, mountainous areas of southern California. It was developed from observed debris volumes that reflected ground conditions influenced by prior rainfall-runoff conditions and areas subjected to partial or complete burning by wildfire. Estimates are from a nomograph using an equation with adjustment factors for size, shape, and slope of the drainage area, 3-hour precipitation, the portion of the drainage area burned, and the years between the burn and the event.

f. Other Regional Studies. Several other regional approaches are available for estimating sediment yield from various references. In addition, site-specific studies, conducted by USACE, other federal agencies, state agencies, universities, drainage districts, planning units, and other commissions and groups, may offer valuable sources of regional information for sediment yield. The engineer should perform a thorough literature search to determine what information may be available for the area under analysis.

g. Basin-Specific Regionalization. Most of the regional criteria available for sediment yield are applicable over a wide area, and may not give an acceptable yield estimate for a specific watershed within the region. Consider applying the regional concepts described above to the specific watershed of the problem area. This type of study could significantly improve the accuracy of yield calculations as compared to those obtained from generalized criteria. Procedures for performing regional studies are described in HEC 1975.

h. Great Lakes Dam Capacity Study. A study was commissioned in 2011 to assess the storage capacity remaining in reservoirs in the Great Lakes. There are over 140 federal harbors in the Great Lakes, most of which are situated at the downstream end of a watershed. These federal navigation channels receive their sediment supply in part from watershed sources. Most of these watersheds have tens to hundreds of dams in them that intercept and store fluvial sediment before reaching the federal navigation channel.

(1) The intent of this study was to assess the remaining storage capacity behind these dams, and to determine the time remaining before the federal navigation channels will receive a significant increase in sediment produced in the watershed that can no longer be stored behind dams. This assessment was made by applying a variety of different methods at 10 dams around the Great Lakes. The results are then to be extrapolated to the remainder of the Great Lakes through future studies.

(2) The methods employed consisted of an assessment of excess ^{210}Pb and ^{137}Cs tracers from cores retrieved from the impounded sediment to determine the sediment accumulation rates. These sediment accumulation rates were compared to three other methods: (1) bathymetric comparisons over the time, (2) a sediment yield curve developed from 61 other watersheds studied in the Great Lakes, and (3) a USGS flow and sediment gage upstream and downstream of each impoundment. This investigation (Baskeran et al., 2015) can serve as a model study for application to other dams both in the Great Lakes and other similar projects.

6-5. Mathematical Models and Other Techniques for Calculating Sediment Yield.

a. General.

(1) The second major grouping of methods for calculating sediment yield are mathematical methods that may be described as the application of analytical techniques to calculate sediment yield from a watershed based on the sediment and hydraulic parameters. There are several techniques which are grouped into four categories: sediment transport models, soil loss equations for small watersheds, bank/gully erosion, and watershed models. Most sediment yield studies use mathematical methods supplemented by whatever actual data are available, with the term mathematical methods here encompassing computer-based implementations. Chapter 9 of this manual presents general modeling information.

(2) Mathematical model results are usually not as reliable as the direct measurement methods presented in the previous section, especially when limited model input and calibration data are available. However, mathematical models do allow simulation of non-sampled events, perform a rigorous sensitivity analysis, account for changes in historic relationships, evaluate future trend scenarios, evaluation of alternatives, and provide a basis for decision-making to augment available physical data.

(3) A sampling program should be considered to refine estimates made with the techniques presented in this section.

(4) Sole reliance on a single method to provide quantitative estimates of sediment yield should be viewed with caution.

(5) The following sections present additional information on applicable models. These methods are not listed in order of reliability. In addition, models continue to evolve with time. Updated model description should be evaluated before selecting study methods. Case Studies 6B and 6C (Appendix N) illustrate the use of mathematical models to evaluate sediment yield.

b. Fluvial Sediment Transport Models. The focus of this chapter is on sediment yield, but it is important to recognize that the sediment delivered to a stream is not instantaneously delivered downstream in its entirety and must be routed through the downstream channel network.

(1) The proportion that is moved downstream can either be estimated using a long-term (typically annual) SDR (described in the section below) or modeled using a number of different software applications.

(2) One-dimensional numerical models that are commonly used to examine the transport of sediment in USACE projects include the SAM, Sediment Impact Assessment Method (SIAM) model, HEC-6T, and HEC-RAS. Multidimensional models such as Adaptive Hydraulics (AdH) or physical models may be necessary for more complex problems and systems. Chapter 9 of this manual gives detailed information on sediment transport modeling.

c. Models of Upland Erosion. Upland soil erosion consists mostly of sheet or inter-rill and rill erosion, the basic forms of erosion. Mathematical models of soil erosion are rarely applied directly; however it is useful to have a basic understanding of them as they form the basis of many modern computer models of upland soil erosion and delivery.

(1) Universal Soil Loss Equation (USLE).

(a) The USLE was originally developed to predict the long-term (annual or longer time scale) average soil loss from agricultural land. The USLE and its descendants (modified universal soil loss equation (MUSLE), Revised Universal Soil Loss Equation (RUSLE)) are some of the most widely used equations for estimating upland soil loss. A detailed explanation of the development of the USLE can be found in Wischmeier and Smith (1978).

(b) The equation was developed by the USDA using rainfall simulators to create erosive energy on 72-foot-long test plots of uniform slope. Surface erosion occurred in the form of rills and the quantity of eroded soil was measured at the outflow point and expressed as tons per acre per year. This limits the direct application of the USLE, although it may be useful for initial estimates in small watersheds when used in conjunction with an SDR.

(c) A revised version of the USLE, the RUSLE, was developed and documented by Renard et al. (1997) that included a more detailed consideration of topography and farming practice for erosion prediction. The equation itself is:

$$A = R\ K\ L\ S\ C\ P \qquad \text{Equation 6-4}$$

where:

A = soil loss per unit area per time period (for example, tons per acre per year)
R = rainfall erosion index
K = soil erodibility factor
L = slope-length factor
S = slope-steepness factor
C = cover and management factor
P = support practice factor

(d) A value is estimated for each of these variables using information gained through a field reconnaissance of the watershed to enter tables and nomographs provided in Wischmeier and Smith (1978) and more recently by Renard et al. (1997). NRCS personnel can often provide local C factors and should be consulted to ensure that values appropriate for the watershed have been selected.

(e) Guidance on adapting the equation to incorporate the effects of thaw, snowmelt, and irrigation on the area; on estimating erosion from construction sites; and on modification of the R-value to estimate sediment yield on a frequency basis through the 20-year recurrence interval event for individual hypothetical storms is presented in reference Wischmeier and Smith (1978) and Renard et al. (1997). A discussion of sediment yield computations using RUSLE is included in Borah et al. (2008).

(f) The following points are made to stress proper use of the RUSLE:

- Channel Projects. The RUSLE gives no information on gradation of the eroded sediment. Consequently, the equations would be of limited value in analyzing the effects of a channel project where sands and gravels are of primary interest.

- Construction Sites. The significance of selecting coefficients can be illustrated by looking at the soil erodibility factor, K. Published coefficients for cropland imply regular soil tilling, which disturbs the natural armor layer that forms during rain events. The significance of the soil erodibility factor, K, for a construction site is not the same as that published for crop land in the RUSLE manual. Soil in a construction area would be expected to exhibit similar erosion to agricultural land during the first rain event after the ground was disturbed, but successive rainfall events would erode that soil at a reduced rate because the construction site is not plowed regularly.

- Erosion Mechanisms. The channelization of surface water runoff due to construction or certain tilling practices may increase gully and channel erosion significantly and the RUSLE would not account for this because it is formulated for sheet and rill erosion. If this is a significant issue in the watershed, newer models, such as the Ephemeral Gully Erosion Model (EGEM) should be considered.

- Sediment Transport. There is no transport function in the RUSLE, and a watershed SDR or computer model must be applied to account for overland deposition. However, the validity of results is questionable when the RUSLE is applied uniformly to subareas in excess of a few square miles.

(2) Sediment Delivery Ratio.

(a) With the addition of an SDR (the amount of sediment output (yield) normalized by the amount of sediment input (erosion)), the USLE can be extended to areas of several square miles. The SDR is a factor, ranging from 0 to 1, to multiply times the annual soil loss to obtain the annual sediment yield for the watershed. SDRs have been calculated for specific areas, but no generalized equations or techniques are yet available to universally determine an SDR. The SDR is proportional to drainage area, and the available data indicates the ratio may vary with the 0.2 power, in the form of:

$$\left(\frac{SDR_2}{SDR_1}\right) = \left(\frac{A_1}{A_2}\right)^{0.2} \qquad \text{Equation 6-5}$$

where:

A_1 = reference drainage area, square miles
A_2 = desired drainage area, square miles
SDR_1 = sediment delivery ratio for reference drainage area
SDR_2 = desired sediment delivery ratio

(b) ASCE Manual No. 54 (Vanoni 1975, 2006) suggests using a reference drainage area of 0.001 and an SDR_1 of 1.0 in this equation, to represent the runoff from a small, experimental plot. Any arbitrary SDR selected solely on the basis of drainage area could be in considerable error; other factors (soil moisture, channel density, land use, conservation treatment, soil type, rainfall intensity, topographic relief, and so forth) will also influence the SDR.

(c) SDRs are also used in some computer models, typically as a surrogate for sediment routing. The upland erosion is first determined using a method such as USLE and this result is compared with a known sediment quantity at the downstream end of the watershed to estimate the proportion of sediment delivered to that point.

(3) MUSLE.

(a) Williams (1977) modified the Universal Soil Loss Equation with the resulting equation termed the Modified USLE (MUSLE). The MUSLE allows the estimation of soil losses for each precipitation event throughout the year, thereby becoming an event model rather than an average annual runoff model. As an event model, the MUSLE and similar techniques have more application to USACE analyses. The full equation defining the MUSLE is:

$$Y = 95 \cdot (Q \cdot qp)^{0.56} \cdot K \cdot C \cdot P \cdot LS \qquad \text{Equation 6-6}$$

where:

Y = sediment yield from an individual storm through sheet and rill erosion only (tons)
Q = storm runoff volume in acre-feet
qp = peak runoff rate in cubic feet per second
LS = slope length and gradient factor
K, C, P = as defined previously for the USLE

(b) Williams (1995) updated this equation to include factors based on area and the amount of coarse rock fragment, as well as conversion to the metric system. This version forms the basis of the Soil and Water Assessment Tool (SWAT) computer model.

(c) The MUSLE is simply the USLE with the rainfall erosion index replaced by the runoff rate term. Since erosion is computed for each event, an SDR is not necessary. The “Q” and “qp” terms would be obtained from the runoff hydrograph, with “Q” used in estimating the amount of soil detachment and “qp” used in determining the volume of soil transported. The sediment yield for each event is summed to obtain each year’s total, with average annual sediment yield being the average of all the yearly values. The points of caution given for USLE also apply to MUSLE:

- Application. Long-term simulation is normally required to obtain a representative estimate. While much additional information is gained from the use of the MUSLE and the necessity of determining an appropriate SDR is eliminated, this technique requires considerable data gathering and calibration effort to apply correctly.

- Runoff. A separate rainfall-runoff model is needed to calculate flood volume and flood peak runoff rate. Calibration is usually against measured water volume, with at least 3 years of data normally needed.

- Confirmation. Comparison and confirmation of sediment yield calculated with MUSLE should be made against that from other techniques. A report by Dyhouse (1982) describes a study in which sediment yield that had been calculated by a method similar to the MUSLE was calibrated using a flow duration sediment transport integration.

d. Gully and Streambank Erosion. When the drainage basin exhibits extensive stream bank erosion and gullying, either on the primary stream or on tributaries to it, sediment yield determined by the following methods should be added to the sheet and rill erosion predicted by the soil loss equations.

(1) Streambank Erosion.

(a) Soil losses through stream bank erosion and bank caving contribute significant quantities of the total sediment yield for most natural rivers. Estimates as high as 1,700 tons/year/mile of bank have been made at some locations. The causes of streambank erosion are many and varied, and the prediction of future losses at specific locations is difficult.

(b) No universally applicable analytical procedures have yet been developed to formally calculate sediment yield or specific bankline losses from stream bank erosion. Methods for predicting potential erosion and failure of streambanks include streambank erodibility relationships such as the U.S. Department of Agriculture-Agricultural Research Service (USDA-ARS) BSTEM or the Bank Assessment for Nonpoint source Consequences of Sediment (BANCS) method (Rosgen 2001) which pairs a categorical bank erosion hazard index with estimates of near-bank stress. The BSTEM method for predicting bank erosion has been incorporated into HEC-RAS (HEC 2016b; Gibson et al., 2015).

(c) The most successful methods for estimating erosion that has already occurred are based on aerial photography or LiDAR data in which successive overflights can be used to overlay bankline movement with time. By measuring the surface area between successive banklines and estimating bank heights from the field reconnaissance, quantities of sediment lost to erosion can be calculated between the surveys and average annual rates estimated. Other methods include annual monitoring of bank erosion pins or repeated profile surveys of representative banks.

(2) Gully Erosion. Soil loss from gullies is seldom sufficient to warrant inclusion in USACE studies because it makes up a very small percentage of the total sediment yield when the study area is more than 10 square miles. However, some parts of the country, such as in the Delta Headwaters Project area in northern Mississippi, experience major sediment losses from gullying. When significant gully erosion is suspected, repeated surveys, aerial photography, or LiDAR data can be used to estimate volumes as outlined above. The local NRCS office should also be contacted to obtain their estimates of gully erosion. Additional information can be found in Piest et al. (1975) and USDA (1966).

(3) Future Conditions. When the future or recent past includes changes to the watershed such as land use change, channelization, or the creation of reservoirs, do not accept historical bank erosion or gullying quantities without justification. In these cases, an assessment of the likelihood of changes in historical values should be made based on knowledge of river morphology and the reaction of rivers to similar changes in other watersheds.

6-6. Comparison of Sediment Yield Models.

a. Watershed models that simulate the hydrologic and sediment yield processes can provide watershed managers a better understanding of the watershed. Many watershed models can predict hydrologic runoff, sediment yield, upland soil and stream erosion, and transportation and deposition of sediment. An understanding of the sediment dynamics can assist in determining potential long-term impacts to navigation in a river. Many numerical modeling tools exist to simulate the movement of water and sediment across a watershed.

b. USACE has developed guidance for software validation for the Hydrology, Hydraulics, and Coastal Community of Practice and a list of approved software models that have been validated for use by USACE. In addition, the engineer should verify that the selected model is approved for use by USACE elements and District commands. Before initiating model development, either verify that the selected model has been validated or include the model validation process within the study scope.

c. Table 6-1 lists a number of models with sediment yield capability that are available (not all have been validated for use within USACE) as of 2017.

Table 6-1
Watershed and Sediment Yield Models Considered

Model Name	Acronym	Citation
Areal Nonpoint Source Watershed Environment Response Simulation	ANSWERS	Beasley et al., 1980
Precipitation-Runoff Modeling System	PRMS	Leavesley et al., 1983
Agricultural Nonpoint Source Pollution Model	AGNPS	Young et al., 1987
KINematic runoff and EROSion model	KINEROS	Woolhiser et al., 1990
Hydrological Simulation Program – Fortran	HSPF	Bicknell et al., 1993
European Hydrological System model	MIKE SHE	Refsgaard and Storm 1995
Soil and Water Assessment Tool	SWAT	Arnold et al., 1998
Annualized Agricultural Nonpoint Source model	AnnAGNPS	Bingner et al., 2018
Dynamic Watershed Simulation Model	DWSM	Borah et al., 2002
ANSWERS-Continuous	ANSWERS-Continuous	Bouraoui et al., 2002
Hydrologic Engineering Center – Hydrologic Model System 4.2	HEC-HMS	HEC 2016a
Gridded Surface Subsurface Hydrologic Analysis model	GSSHA	Downer and Ogden 2006

d. Technical Capabilities of Various Models.

(1) The models in Table 6-1 contain a range of complexity and functionality. Simple models are easy to build and use, but may not capture the various activities in a watershed, and may be incapable of providing desired detailed results. Complex models can be computationally demanding and challenging to construct and calibrate. Therefore, an appropriate model should be selected based on the question(s) to be answered by the model.

(2) The watershed models listed in Table 6-1 can generally be divided into either long-term continuous models or watershed-scale storm-event models. Long-term continuous models are useful for analyzing long-term effects of hydrologic changes and watershed management practices. A long-term continuous model can be applied to address the following types of questions: (1) the creation of an annual sediment budget, (2) assessing the effects of climate change, and (3) other watershed processes at work over times scales greater than an individual precipitation event. Storm-event models are useful for analyzing several actual or design single event storms and evaluating watershed management practices. Some listed models listed have both long-term and storm-event capability.

(3) The model capabilities have been compared based on the technical mechanisms to address various hydrologic and sediment processes. Borah et al. (2008) compared many of these models based on technical criteria and a summary of the findings are described in Table 6-2 and Table 6-3.

Table 6-2
Comparison of Hydrologic Properties of Models (revised from Borah et al., 2008)

Description/ Criteria	AnnAGNPS	ANSWERS-Continuous	HSPF	MIKE SHE	SWAT	GSSHA	HEC-HMS
Model components and capabilities	Hydrology, snow- melt transport of sediment, nutrient, and pesticides, precipitation and irrigation, source accounting capability, and user interactive programs including TOPAGNPS generating cells and network from DEM.	Daily water balance, runoff and surface water routing, drainage, river routing, ET[4], sediment detachment, sediment transport, nitrogen and phosphorous transformations, nutrient losses through uptake, runoff, and sediment.	Runoff and water quality constituents on pervious and impervious land areas, movement of water and constituents in channels and reservoirs, USEPA BASINS model system with interface and GIS platform.	Interception-ET, overland, channel flow, unsaturated\ saturated zone, snowmelt, aquifer-river exchange, solute advection and dispersion, chemical process, crop growth nitrogen process in the root zone, soil erosion, porosity, and irrigation.	Hydrology, soil temperature, weather, sediment, crop growth, nutrient, pesticide, agricultural, reservoir and channel routing, water transfer, USEPA BASINS model system with interface and GIS platform.	Spatially varying rainfall including radar estimates, rainfall excess and two-dimensional flow routing on over-land grids, soil moisture accounting, diffusive wave or full channel routing, upland erosion, sediment transport in channels.	Hydrology, transport of sediment, sedimentation, channel and reservoir routing.
Temporal scale	Long term; daily or sub-daily steps.	Long term; dual time steps: daily for dry days, 30 seconds for precipitation days.	Long term; variable constant steps (hourly).	Long term and storm event; variable time steps for stability.	Long term; daily steps.	Long-term and storm event; variable time steps for stability.	Long-term and storm event; variable time steps.
Watershed depiction	Homogeneous land areas (cells), reaches, and impoundments.	Square grids with uniform features, some with companion channel elements; 1D simulations.	Pervious and impervious land areas, stream channels, and mixed reservoirs; 1D simulations.	2D rectangular/ square overland grids, 1D channels, 1D unsaturated and 3D saturated flow layers.	Sub-basins for climate, hydrologic units (lumped areas with same cover, soil, and management), ponds, and groundwater.	2D square overland grids and 1D channels.	Sub-basins grouped based on hydrologic areas with same cover, soil, and practice, reservoirs, and main channel.
Rainfall excess on overland/ water balance	Water balance for constant sub-daily timesteps and two soil layers (8-in. tillage depth and user-supplied second layer).	Daily water balance, rainfall excess using interception, Green-Ampt infiltration, and surface storage coefficients.	Water budget considering interception, ET, and infiltration with empirically based areal distribution.	Interception and ET loss and vertical flow solving Richards equation using implicit numerical method.	Daily water budget; precipitation, runoff, ET, percolation, and return flow from subsurface and groundwater flow.	Interception and ET loss, infiltration using Green-Ampt method, and overland flow retention.	Water budget considering interception, ET, and infiltration with empirically based areal distribution.
Runoff on overland	Runoff curve number generating daily runoff following SWRRB[1]/EPIC[2] procedures and SCS TR-55[3] method for peak flow.	Manning and continuity equations (temporarily variable and spatially uniform) solved by explicit numerical scheme.	Empirical outflow depth to detention storage relation and flow using Chezy-Manning equation.	2D diffusive wave equations solved by an implicit finite-difference scheme.	Runoff volume using curve number and flow peak using modified Rational formula or SCS TR-55 method.	2D diffusive wave equations solved by explicit finite-difference scheme.	Several hydrograph functions including Clark Unit, kinematic wave, modClark, SCS Unit, Snyder Unit or User-specified.

[1] Simulator for Water Resources in Rural Basins (SWRRB).

[2] Erosion-Productivity Impact Calculator (EPIC).

[3] Technical Release 55 (TR-55), developed by the NRCS, presents simplified hydrologic procedures for use with small watersheds.

[4] Evapotranspiration (ET).

Table 6-3
Comparison of Sediment and Other Properties of Various Models (revised from Borah et al., 2008)

Description/ Criteria	AnnAGNPS	ANSWERS-Continuous	HSPF	MIKE SHE	SWAT	GSSHA	HEC-HMS
Runoff in Channel	Assuming trapezoidal and compound cross section, Manning's equation is numerically solved for hydraulic parameters and TR-55 peak flow.	Manning and continuity equations (temporarily variable and spatially uniform) solved by explicit numerical scheme.	All inflows assumed to enter one upstream point, and outflow is a function of reach volume or user-supplied demand.	1D diffusive wave equations solved by an implicit finite-difference scheme.	Routing based on variable storage coefficient and flow using Manning's equation adjusted for transmission losses, evaporation, diversions, and return flow.	Two options: 1D diffusive wave equations explicit finite-difference for channels or implicit finite-difference of 1D full dynamic equations for subcritical flow.	Assuming trapezoidal and compound cross sections, Manning's equation is numerically solved for hydraulic parameters.
Overland Sediment	RUSLE to generate sheet and rill erosion daily or user-defined runoff event, HUSLE[1] for delivery ratio, and sediment deposition based on size distribution and particle fall velocity.	Raindrop detachment by rainfall intensity and USLE factors, flow erosion by unit-width flow and USLE factors, and transport \deposition of sediment by modified Yalin equation.	Rainfall detachment and washing off the detached sediment by transport capacity as function of water storage and outflow plus flow scour using power relation with storage and flow.	No information.	Sediment yield based on MUSLE expressed in terms of runoff volume, peak flow, and USLE factors.	Soil erosion and sediment deposition are computed using modified Kilinc-Richardson equation with USLE factors and conservation of mass.	Sediment yield based on MUSLE expressed in terms of runoff volume, peak flow, and USLE factors.
Channel Sediment	Modified Einstein equation for sediment transport and Bagnold equation to determined transport capacity of flow.	Not simulated.	Noncohesive sediment transport user-defined relation with flow velocity, Colby, or Toffaleti method, cohesive transport based on critical shear stress and settling velocity.	No information.	Bagnold's stream power concept for sediment transport, degradation adjusted with USLE soil erodibility and cover, deposition based on particle fall velocity.	Sand-size total sediment load computed with Yang's unit stream power method.	Sediment continuity based on Exner equation. Sediment transport based on Ackers-White, Englund-Hansen, Laursen-Copeland, Muller, Toffaleti, Wilcock, Yang.
Reservoir Sediment	Sediment deposition based on constant detention discharge, zero transport capacity, and dilution with pool water.	Not simulated.	Same as channel.	No information.	Outflow using simple continue based on volumes and concentrations of inflow, outflow, and storage.	Not simulated.	Trap efficiency method based on grain size and critical velocity.
Best Management Practices Evaluation	Agricultural management.	Impact of watershed management practices on runoff and sediment losses.	Nutrient and pesticide management.	No information.	Agricultural management: tillage, irrigation, fertilization, pesticide applications.	No information.	Not directly simulated.

[1] Hydro-geomorphic Universal Soil Loss Equation (HUSLE)

e. Cost Considerations of Various Models. Most of the models under consideration are free, while a few require the purchase of a license or require pre- or post-processing software to edit the input and visualize the output.

(1) Research specifics regarding model licensing and cost prior to making a selection. Requirements for some models follows. MIKE SHE by the Danish Hydraulic Institute (DHI) is software that requires the purchase of a license to run; however, some agencies have an agreement with DHI to waive the license fee. Hydrologic Simulation Program-Fortran (HSPF), Soil and Water Assessment Tool (SWAT), and Gridded Surface Subsurface Hydrology Analysis (GSSHA) use an optional pre-processing software. Although the pre-processing software is optional to run the model, it is extremely difficult to process the large amounts of spatial data without using the assigned pre-processing software. HSPF uses a free GIS software (MapWindow); SWAT incorporates ArcGIS with Spatial Analyst; GSSHA uses WMS (Watershed Modeling System); and Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) 4.2 uses ArcGIS with Spatial Analyst and 3D Analyst.

(2) The cost to run and debug a numerical model can be significant and highly variable. These factors may have a large influence on model selection. Before initiating model development, verify that the selected model is approved for USACE use or include the model validation process, which will affect both study schedule and cost, in the study scope. Model selection and cost should also consider that model versions may change during the course of the study. Acquiring new versions with the desired model capabilities may add additional study cost due to fees or delays during the USACE model validation process.

f. Ease of Use. GSSHA and MIKE SHE are both physically based models using multidimensional flow-governing equations with approximate numerical solutions schemes. Therefore, these models tend to be more computationally intensive, subject to numerical instabilities, and more complex to use. The remaining models are empirical models that do not require approximate solutions to any partial differential equations. HSPF and SWAT are commonly used models and have abundant documentation, online support, and training opportunities. Annualized Agricultural Nonpoint Source (AnnAGNPS) and ANSWERS-Continuous are less commonly used models and have less documentation and online support. HEC-HMS 4.2, released in 2016, has significant sediment yield modeling capabilities.

g. Model Selection. One method to help quantify the functionality of each model and select the most appropriate one is to develop a weighting of the functionality of each that is specific to the watershed being studied and to the questions that are trying to be answered. The data listed in Table 6-4 provide an example of this.

Table 6-4
Model Comparison

Alternative*	Criteria			
	Technical Basis	Cost Basis	Ease of Use	Total
AnnAGNPS	4	3	2.5	9.5
ANSWERS-Continuous	0	3	2.5	5.5
HSPF	2	3	4.5	9.5
MIKE SHE	2	0	0.5	2.5
SWAT	4	2	4	10
GSSHA	5	5	1	11
HEC-HMS 4.2	4	5	6	15
Sum of the Criteria	21	21	21	63

(0 = worst; 6 = best. Rankings are normalized to add up to 21 total points per category)

* Tabulated data are from a single watershed. Similar comparison should be performed for each USACE study. Although values are totaled, avoid assuming that this factors are equally weighted.

h. It is important to note that the above scoring is specific to one watershed and the questions that the model is being asked to resolve. If this approach is used, a table specific to the study watershed should be constructed. Moreover, other criteria may be added if deemed important to the study. For example, if a certain process is important to resolve, such as surface water-groundwater interaction, then a column can be added to rank each of the alternative models against each other with respect to that process.

6-7. Special Cases and Issues.

a. Urban Sediment Yield. Urbanization has enormous impacts on every aspect of watershed behavior. In particular, the increased peak flows from urbanization normally cause channel incision throughout the watershed.

(1) Jonas (2010) discussed channel evolution in urbanizing watersheds. Channel incision increases the sediment yield dramatically. It was formerly predicted that, as the upland watershed areas were stabilized, that watershed sediment yield in urbanized areas would drop to lower values than pre-development conditions. This prediction has proven to be incorrect, and studies document yields from urban watersheds that exceed pre-urbanization conditions (Lee 2009). Instead, impervious surface area increases have led to more runoff and flashier hydrographs. This, in turn, has caused in-channel sources of sediment to become dominant.

(2) Channel incision and bank erosion become the largest sources of sediment (with estimates of 75% or more) and the contribution from the upland areas (which are mainly paved or stabilized) becomes much smaller (Fraley and Miller 2009; Allmendinger et al., 2007; Wallbrink 1998). Additionally, channel beds and banks in urban areas may be lined with steel

sheet pile, concrete, or rip rap to protect infrastructure and quickly evacuate flood waters. This can have the effect of moving sediment consequences downstream of the urban area.

b. Dam Removal.

(1) The management of the sediment impounded behind a dam can be a very challenging and expensive aspect of a dam removal project. Dams are often removed to improve the fishery, however, careless management of sediment during a dam removal can have a significant adverse effect on the same fishery that the project is trying to improve.

(2) The engineer will first need to quantify the volume, size class, and spatial distribution of sediment stored in the reservoir. Collected sediment information will be used to assess its potential for mobility, the quantity likely to be mobilized, and the downstream consequences. The volume of sediment can be determined in a number of ways.

(3) The preferred method to determine the sediment volume is from a comparison between bathymetric surveys (as built bathymetry vs. current bathymetry). Additional methods consist of collecting borehole data in the impoundment, identifying the contact between the original sediment and the impounded sediment, and creating a sediment thickness map.

(4) A sediment yield model can also be applied and will provide yet another estimate of the amount of sediment delivered to the reservoir, usually in the form of an annual load. By applying the estimated annual load over the lifespan of the dam, the engineer can determine the amount of sediment that has been historically delivered to the reservoir. It is important to note that not all of the sediment delivered to a reservoir will be retained, as some may pass the dam as wash load or suspended load depending on the TE.

(5) In some instances, a push-probe (or depth-of-refusal) survey can be performed to approximate the quantity of impounded sediment. This will only work if the quantity of sediment behind a dam is fairly small, and the water depths are shallow as the push probes tend to buckle when the unbraced length is too long. Additionally, there must be some recognizable contrast between these two layers, such as the presence of a cobble or gravel layer overlaid by finer reservoir sediment.

(6) Geophysical techniques such as an acoustic sub-bottom profiler or ground-penetrating radar have been used to determine the quantity of impounded sediment. In instances where greater certainty is needed, more than one method can be applied.

(7) The dam removal project should be designed and phased such that sediment transport and concentrations are acceptable for the downstream reach. The quantity of sediment that is acceptable will vary by location and should be determined in consultation with fisheries biologists, fluvial geomorphologists, and regulatory agencies. Impounded sediments can also have raised heavy metal levels or other water quality concerns that should be considered. If available, a Sediment Evaluation Framework (SEF) developed for dredge material disposal can also be applied to dam removal projects to facilitate interagency coordination on potential sediment impacts (RSET 2016).

(8) The current and future sediment yields will have to be determined to assess the potential for post-dam removal aggradation and degradation. These quantities include both the sediment from the upstream watershed and any material that is likely to be eroded due to the base-level lowering resulting from the dam removal (for example, deltas at tributary confluences).

(9) In some instances, a channel is designed and constructed through the formerly impounded sediment. If the channel is improperly sized and cannot pass the upstream sediment supply, aggradation may occur, resulting in flooding and planform instability of the channel. If the channel is designed with too much sediment transport capacity, degradation and incision may occur.

(10) Knowledge of the upstream sediment supplies, either through numerical models or field measurements (or both if erosion is expected to occur due to base-level lowering), is necessary if the engineer is to design a stable channel. In some instances, however, the river has been allowed to cut its own channel through the impounded sediment. Knowledge of the upstream sediment supply is less important in these situations; however, care should be exercised such that the new river does not become entrenched with limited access to a floodplain, otherwise future sediment supplies from bank erosion may significantly increase. Chapter 7 provides additional details regarding river processes.

c. Reservoir Operation Impacts. Reservoirs operate as part of a system and often have other reservoirs located upstream or downstream. Changes in operation (to both water and sediment releases) will affect other reservoirs. Chapter 8 provides additional details on reservoir operation impacts.

(1) Reservoir Life. Due to the age of the U.S. reservoir population, it is anticipated that multiple reservoirs in a basin will be reaching a threshold in the next few decades where sedimentation affects reservoir operation. This means that a reservoir that is facing its own sedimentation problems may also have to deal with increased sediment inflow from an upstream reservoir that has initiated sediment management measures such as sediment bypassing or flushing.

(2) Sediment Management Techniques. Reservoirs that implement techniques such as flushing or bypassing will increase sediment loads downstream, potentially impacting downstream reservoirs. This has already happened with at least one USACE reservoir, where inflowing sediment loads have increased due to sediment flushing activities at an upstream reservoir.

(3) Changes in Trapping Efficiency as Reservoirs Fill. As reservoirs fill, their trapping efficiency gradually decreases. This will result in larger sediment loads delivered to reservoirs downstream. When a reservoir is substantially full, it reaches steady-state, where almost all inflowing sediment is passed through the reservoir and delivered downstream. Another serious consequence is that a reservoir full of sediment becomes a sediment source during floods. Sediment is scoured out of the reservoir by high flows and carried downstream. Conowingo Reservoir, at the mouth of the Susquehanna River, is a documented example of this process. The

computation of contributing drainage area will need to be revised as upstream reservoirs reach capacity with regard to sediment.

(4) Sustainability Planning. Reservoir sustainability planning must account for the anticipated future operation of upstream reservoirs since sediment yield may increase significantly as reservoirs fill or initiate sediment management strategies.

d. Climate Change.

(1) Sedimentation investigations involve consideration of the hydrologic cycle that may be influenced by climate change. Any change in climatic conditions can result in one or two principal changes in basin hydrology.

(2) Storm intensity, depth, and duration may be different from that indicated in past data. Typically storm intensity increases with climate change, which will directly impact sediment yield computations. Changes in sediment yield impact the quantity and characteristics of sediment entering streams, which consequently affects in-channel sedimentation.

(3) The seasonality of storms may be different from historical patterns. This change in seasons of when storms occur may also impact sediment yield calculations and in-channel sedimentation rates. Examples of this include more rainfall in the early spring that could combine with snowmelt for higher runoff events, or winter precipitation trending from snowfall to rain events. Seasonal variation can also affect vegetative cover conditions with an associated impact to sediment yield.

(4) USACE guidance is currently evolving. Following current guidance, most sedimentation investigation studies will use a sensitivity analysis to adjust hydrologic inputs and other basin changes as appropriate. For example, historic period of record flows may be adjusted for future conditions based on climate models and non-stationarity trend analysis. The adjustment should be consistent with the expected degree of climate change impacts to runoff and streamflow as available in peer-reviewed literature and USACE reports. An evaluation of sensitivity to climate change through the hydrologic inputs should be included in all sediment investigations (for sediment yield, in-channel sediment routing, or reservoir sedimentation).

(5) Forecasts of anticipated climate change vary dramatically throughout the United States, and thus will have different impacts at USACE projects. For example, predicted climate change trends in the Midwest consist of increased temperatures that are likely to result in earlier spring snowmelt, decreased snowmelt season duration, and decreased peak snow water equivalency. Extremes in climate will also magnify periods of wet or dry weather resulting in longer, more severe droughts, and larger more extensive flooding. Refer to Case Studies 6C, 6D, 8C, and 8D (Appendix N) for illustrations of different aspects of sediment yield evaluation with climate change.

e. Wildfires. Extensive and frequent wildfires have exposed watersheds to erosion (Goode and Buffington, 2010). Widespread wildfire followed by regional storms of even moderate intensity can drastically increase short-term sediment yield and disturb the drainage system

(1) Land use and forest management practices employed in the United States have significantly reduced the number of annual fires from the early 1900s. As the United States experienced economic development and population growth for the past century, fire suppression tactics sought to protect regional resources and residents by extinguishing fires immediately after ignition. Despite the good intentions of this policy, its implementation has increased the chance of catastrophic fires and has been linked to increases in wildfire size and severity (Miller et al., 2009).

(2) The relationships between climate, wildfire, and sediment yield are important to consider when planning USACE project evaluations and strategy to meet extreme sediment inflows and estimate long-term sediment yields. Cosmogenic analysis of sediment yield in the mountains of central Idaho by Kirchner et al. (2001) strongly suggests that such extreme events define the long-term sediment balance of the Lower Snake and Clearwater basins, and that current sediment yield rates, which have not been impacted by wildfires, are less than long-term rates by an order of magnitude.

(3) The magnitude of watershed hydrologic and sediment yield changes following wildfire depend on many factors, including burn severity, landscape susceptibility to erosion, and the timing and magnitude of storms that follow the fire.

(a) Wildfire effects on vegetation include less rainfall interception, shorter time to initial runoff, less surface friction, and less evapotranspiration. Other effects include increased surface connectivity, reduced runoff routing time, and possibly higher soil moisture and groundwater levels.

(b) Wildfire effects on snowpack include a change in accumulation, albedo (black carbon on snow changes energy balance), and the timing of melt dates.

(c) Wildfire effects on soil and sediments include soil structure changes (soil aggregates break down and fracture, may have surface sealing) and a reduction in soil permeability. Soil burn severity measures characteristics including char depth, organic matter loss, and reduced infiltration.

(d) Peak discharge measured following wildfire have been shown to increase by ~1.5 to 1,000 times (plus) over pre-fire rates (Moody and Martin 2001; Cannon and Gartner 2005).

(e) A wildfire in the contributing watershed, particularly a mountainous watershed, can increase the sediment flow-load curve dramatically (Ryan et al., 2011). Runoff from wildfire impacted watershed can take on non-Newtonian fluid characteristics. The increased sediment erosion and transport can rapidly impact USACE projects by decreasing channel capacity and depositing material in downstream reservoirs.

(f) The Forest Service, an agency of the USDA, deploys Burn Area Emergency Response (BAER) teams to perform post-fire assessments and determine values at risk. BAER teams develop estimated potential increases in postfire runoff and sediment that place downstream values at risk or threaten human life and natural resources. The BAER teams typically use empirical, event-based models to accommodate rapid assessment (Foltz et al., 2009). BAER team assessment results can provide a valuable resource when initially determining wildfire potential impacts to USACE projects.

(4) Effect of the burnt landscape can be translated to sediment yield estimates by means of a watershed hydrologic model with interdependent components to simulate hydrology, snowmelt and rainfall-runoff, soil erosion, stream flow routing, groundwater interactions, and vegetation life cycles. To simulate postfire conditions, model parameters are adjusted to reflect changes in watershed properties. Refer to Case Study 6D (Appendix N) for an illustration of a sediment yield evaluation that includes the effect of wildfire.

(a) HEC-HMS includes a sediment transport module (STM) with simulation components for surface erosion, channel sediment transport, and reservoir sediment routing (HEC 2016a). Ongoing developments within HEC-HMS continue to expand model capability for use with post-fire assessments.

(b) While precise quantification of wildfire impacts is challenging, reasonable quantitative estimates of watershed sediment yield are possible (Miller et al., 2011; NRCS 2016). Kinoshita, Hogue, and Napper (2014) evaluated five of the most common models used by post-fire assessments of watersheds in the western United States. Comparing the models, they found that discharge estimates are highly variable for the studied watersheds, heavily influenced by climatology (location), geophysical properties, and soil burn severity, and that no single model appeared best suited across the range of systems studied.

(c) The NRCS (2016) provides guidance for hydrologic analysis of burned watersheds. The document discusses specific impacts of wildfire on the runoff process, presents hydrologic models and analysis techniques for post wildfire area runoff simulation, and modeling to evaluate sediment yield and transport including bulking due to high sediment loads. The document includes five case studies that model actual burned watersheds.

(5) Impacts of wildfire on sediment loads for Little Granite Creek were evaluated by comparing pre- and post-wildfire sediment load measurements (Ryan et al., 2013). Despite relatively low flows during the first runoff season, the estimated sediment load was about five times that predicted from regression of data from the pre-burn record. In other environments, moderate- to high-intensity rainstorms caused significant flooding and widespread debris flows. The authors speculate that the sedimentation pattern and geomorphic response in Little Granite Creek may be fairly typical of stream responses to wildfire during times of continued drought and in the absence of significant rainfall.

(6) Wildfire has also been associated with increased risk for debris flow. Cannon (2008) demonstrated that frequently occurring rainfall intensities in the range of a 2 to 10-year return interval are capable of generating debris flows from burned areas. The National Landslide Hazards Program lists assessment methods and reference publications (https://www.usgs.gov/natural-hazards/landslide-hazards). Kean (2011) presented debris flow measurement results at multiple sites in southern California and noted substantial differences in debris-flow dynamics between sites and between sequential events at the same site. Refer to

paragraph 3-4 for a discussion of hyperconcentrated flows which includes mud floods, mudflows, and debris flows. Debris basin design is discussed in paragraph 8-14.

(7) Moody and Martin (2004) developed a wildfire impact index, W, as a relative estimator of potential sediment transport due to wildfires in the western United States. While the index does not provide quantitative predictions of sediment transport rates, it indicates regions where wildfires may have a greater impact on reservoir sedimentation relative to other regions. The index may be used as a predictive tool for identifying regions where USACE reservoirs are more susceptible to wildfire sedimentation impacts.

$$W = \left(\frac{F}{F^{max}}\right)\left(\frac{I_{30}}{I_{30}^{max}}\right)^{3.0}\left(\frac{S}{S^{max}}\right)\left(\frac{K}{K^{max}}\right) \qquad \text{Equation 6-7}$$

where:

F = fire frequency (as a return interval in years)
I_{30} = 30-minute rainfall intensity (in mm/hr)
S = channel slope
K = soil erodibility factor (m^{-1})
max = refers to the maximum value for the entire study area

(a) Moody and Martin concluded that the index variables (fire frequency, soil erodibility, channel slope, and rainfall intensity) are more important than the type of geologic terrain in determining post-fire sedimentation.

(b) The wildfire impact index for the mountainous western United States is highest in the southwestern states of Arizona, Utah, Colorado, and New Mexico.

(c) Other areas in the high category are the Sierra Nevada along the eastern side of California, the Transverse Ranges in southern California, the east-facing side of the Cascades in Washington, and the Black Hills of South Dakota.

6-8. Sediment Yield Estimates. The availability of sediment data and estimates varies tremendously from one watershed to another. Some watersheds have lengthy fluvial sediment gage records, while many have no measured fluvial sediment data. Grain size distributions may be totally absent. Sediment sources, sinks, and transport may have been extensively studied or not addressed at all.

a. General Considerations. Sediment yield can be computed at several levels of detail. The most basic and fundamental is an estimate of average annual sediment inflow in tons per year, at a point often the upstream end of a reservoir or the mouth of a tributary. Grain size distribution is helpful information, but not always available. It may be sufficient to estimate the sediment yield at a single point. Often, however, it is necessary to determine sediment yields for different sub-basins. As a further step, it may be necessary to estimate the sources of sediment that comprise the sediment yield. All these estimates may be made for present and future conditions, and for various alternatives.

b. Regional Evaluation.

(1) Sediment yield estimates should be compared with those from nearby basins to see if the results are consistent within the region.

(2) Spatial Review within the Watershed. The sediment yields at different points in the watershed should be reasonable when compared. Sediment sources and sinks should be reflected in results.

(3) Comparison by Grain Size. The sediment load for each size class should be reasonable when compared within the watershed. Comparison should account for source material and both hydraulic transport capacity and representative grain size of the channel boundary (surface and subsurface).

(4) Comparison by Source. The sediment yields from each source should be reasonable in the context of the total sediment yield and the contributions from other sources.

c. Estimating Sediment Yield Using Multiple Methods. Within a weight of evidence framework, multiple methods to estimate sediment yield should be considered when performing typical USACE sediment investigations. For instance, eight different methods were employed to estimate sediment yield for the Caliente Creek watershed (USACE 1990). Results determined an average annual sediment yield at the reservoir site of 0.75 acre-feet (ac-ft)/sq mi/yr with a range from 0.2 to 0.97 ac-ft/sq mi/yr for the various methods. Sediment yield estimates with a 500% variation in range complicate evaluating sediment yield impacts on USACE projects.

d. Inaccuracies in Sediment Yield Estimates. It is critical to realize that sediment data, computations, analyses, and estimates have a much greater potential for inaccuracy than hydrologic or hydraulic computations. Every method of sediment data collection has potential blind spots and variable ranges of uncertainty and bias, which should be considered by the engineer. The use of multiple methods to cross-check sediment yield estimates remains the best defense against serious inaccuracies.

(1) Examples of Inaccuracies.

(a) The predicted sediment inflow to Jennings Randolph Reservoir was based on fluvial sediment gage data. Actual deposition has been almost 10 times as great as that prediction. Reservoir sedimentation data from nearby reservoirs turned out to be a much better predictor, and corresponded very closely with the actual deposition in Jennings Randolph.

(b) In some cases, estimates of sediment yield from bank erosion, made using data from bank pins, have exceeded the sediment yield from all sources computed using other estimates.

(c) Estimates made using the USLE (or other methods that predict upland yield) may significantly under predict sediment yield in watersheds where in-channel sediment sources are dominant or other erosive process such as gullying are significant.

(d) Estimates made for sediment yield in the Susquehanna River Basin using the HSPF model varied significantly from estimates made using fluvial sediment gage data.

(2) Accuracy Assessment Method Proposed for USACE Sediment Yield Studies.

(a) The method developed by MacArthur and others is recommended for use in sediment yield studies (USACE 1990, Burns and MacArthur 1996). It consists of plotting all applicable measured and computed sediment yield data against drainage area for comparison (see Table 6-5, Figure 6-9, and Figure 6-10). This highlights potential inaccuracies and uses the strengths of several methods to compensate for the inadequacies of any single method.

(b) A similar comparison was made in the Battle Creek watershed in Michigan, as shown by the example data in Table 6-6 (Riedel et al., 2010).

Table 6-5
Sediment Yield Data for Caliente Creek (California) from Various Sources and Computational Methods (adapted from USACE 1990)

Data Source[1]	Drainage Basin, Reservoir or Computational Method Used	Drainage Area (sq mi)	Average Annual Yield (ac-ft/sq mi/yr)
SCS[2]	Blackburn	7.1	2.20
SCS	Antelope Canyon	4.4	1.50
CESPK	Isabella	2,074	0.37
CESPK	Pine Flat	1,542	0.20
CESPK	Success	393	0.76
CESPK	Terminus	560	0.75
SCS	SCS Yield Map of Western U.S. (HEC)	470	0.47
Computed	Integration of the Event Volume vs. Frequency Curve (HEC)	470	0.55
Computed	Flow Duration Method (HEC)	470	0.90
Computed	Dendy and Bolton Method (HEC)	470	0.71
Computed	PSIAC Method (HEC)	470	0.75
Computed	Kern County Water Agency Study (Simons, Li, & Associates)	470	0.97

[1] Refer to USACE (1990) for additional information regarding cited references and performed calculations.

[2] Soil Conservation Service (SCS).

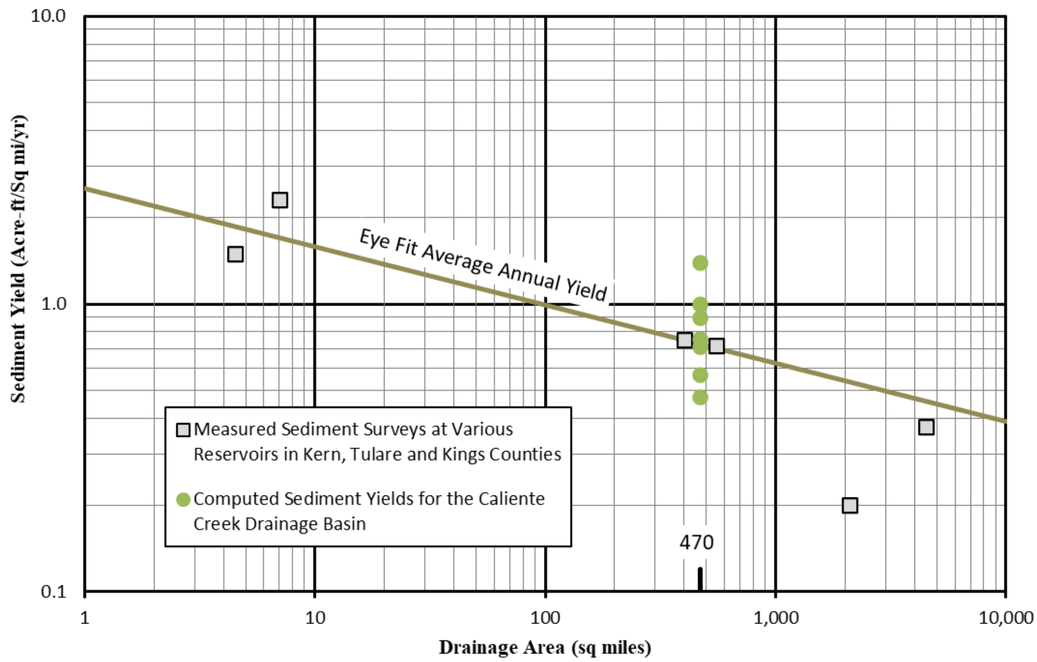


Figure 6-9. Sediment yield from measured and computed values for Caliente Creek, California (USACE 1990)

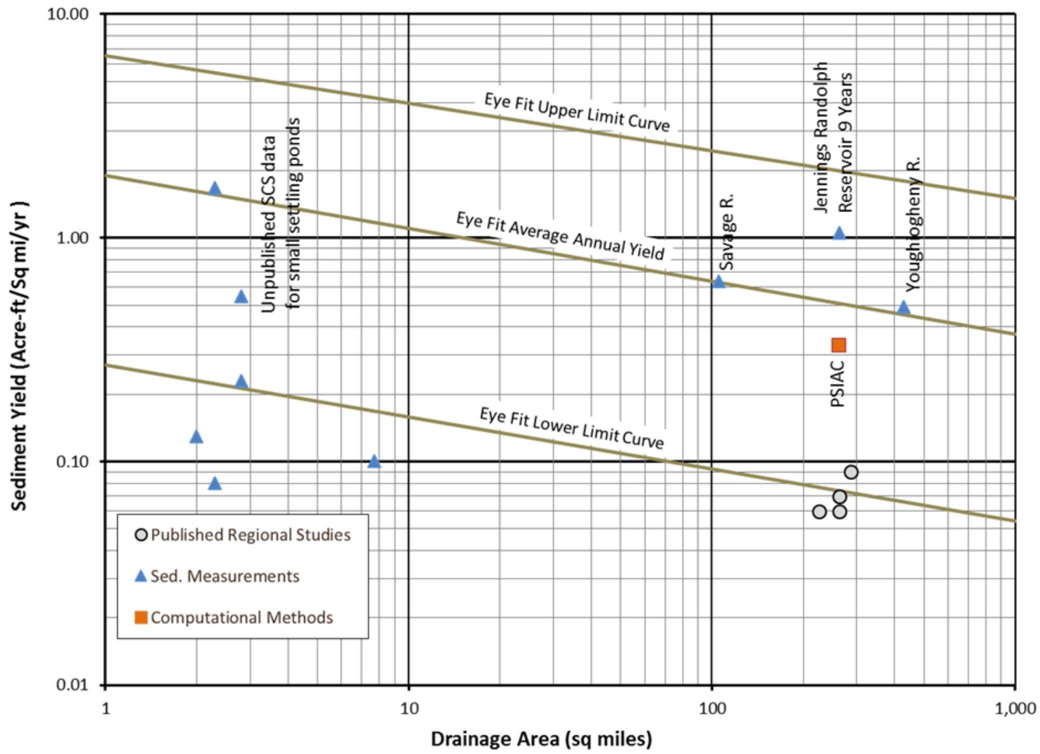


Figure 6-10. Sediment yield from measured and computed values for Jennings Randolph Reservoir, West Virginia and Maryland (Burns and MacArthur 1996)

Table 6-6
Comparison of Sediment Yield Methods in Battle Creek Watershed, Michigan (adapted from Riedel et al., 2010)

	Sediment Yield	
Source	**T/mi^2/yr**	**t/ha/yr**
U.S. Water Resources Council 1968	10–800	0.35
Leopold et al., 1995, Corbel 1959	131	0.46
Brune 1951	1,514	5.30
Dendy and Bolton 1976	685	2.40
Syed, Bennett, and Rachol 2004	22	0.08
Ouyang, Bartholic, and Selegean 2005	25–49	0.09–0.17
Past 516(e) studies	154	0.54
SWAT model	240	0.84

6-9. Sediment Budget.

a. Sediment budgets are used to examine the entire range of sediment processes in a watershed. They represent an accounting of all sediment sources and sinks at the watershed outlet. A successful sediment budget will address the following elements:

(1) Identification of the contributing watershed and sediment discharge point.

(2) Literature and data review.

(3) Identification and quantification of sediment budget components.

(4) Modeling watershed hydrology and sedimentation.

b. Identification of the contributing area and sediment discharge point is often straightforward. Many times the sediment budget will be performed only for the area upstream from a project. It may be desirable, however, to extend the area downstream to a sediment gaging station or to include river reaches that are likely to be impacted by a project.

c. A literature and data review should gather all information related to sediment issues within the study watershed. This may include geospatial data, streamflow and sediment data, scientific publications, government agency reports, and local media sources.

d. Identification and quantification of sediment budget components can be the most difficult portion of the sediment budget. The source, transport, and sink areas should be identified and, where possible, quantified. This includes soil erosion, bank erosion, bed material load, mass wasting, and deposition. It is often desirable to include multiple estimates for both the individual components and the overall budget. Alternative estimates for sediment yield to the watershed outlet include regional sediment yield estimates and sedimentation surveys in reservoirs or navigation channels.

e. It may be necessary to model the sediment yield and transport to quantify the sediment transport processes through the watershed. Approaches range from coarse-level sediment impact analysis methods (SIAM), that establish sub-watershed sources and sinks based on continuity of transport potential, to more comprehensive sediment routing models based on transport rates and hydrologic timeseries for long-term forecasts. Appropriate models and techniques are described in more detail in Chapter 9 of this manual.

6-10. Sediment Yield Reduction – Sediment Management Measures. The reduction in sediment yield of a basin is a common goal of many watershed sediment management projects. Effective watershed sediment management requires identifying the sediment sources and sinks in the watershed and understanding the processes responsible for transporting the sediment along the pathways that link the sources and sinks at the reach and watershed scales. An understanding of the characteristics of the sediment sources along with the transport mechanisms will help guide the selection of the most appropriate and potentially effective sediment management practices for achieving sediment yield reduction.

a. Watershed Sediment Practices.

(1) Watersheds may vary considerably with respect to sediment yield and various treatment measures. For example, sediment management techniques that target upland sources in watersheds where SDRs are low due to available sediment storage capacity in valleys and minimum transport capacity in the channels may result in very little reduction in overall basin sediment yield.

(2) Trimble (1983) developed two sediment budgets for the Coon Creek watershed in Wisconsin. One was for a time period characterized by poor land management and excessive soil erosion, and the other was for a period of increased soil conservation measures. The sediment budgets indicated a significant reduction in upland soil erosion by approximately 26% due to the increased conservation measures, yet the overall sediment yield of the basin was essentially unchanged. Most of the soil eroded during the first time period was stored in the valley floor and channels, with only a small portion being transported to the basin outlet. Thus, the sediment reduction measures were successful in terms of reduced soil erosion but had little impact on the basin sediment yield.

(3) Conversely, other watersheds may contain channel systems that are highly efficient sediment transporters and may also serve as significant sediment sources via channel incision or bank erosion. Leech and Biedenharn (2012) provided extensive discussion of sediment management at the watershed level. They noted that watersheds where the channels are efficient conveyors of sediment may act as dominant sediment sources rather than sinks and provide examples from the Delta Headwaters Project (DHP) in north Mississippi. Using the average reported value from empirical studies, sediment management measures in the form of in-channel structures and bank stabilization in these streams had an average sediment yield reduction of 60%.

b. Sediment Reduction Measures.

(1) The sources, transport, and gradation of material will guide the selection of management measures. In general, watershed sediment management measures can largely be grouped in two broad categories: (1) upland measures, and (2) in-channel measures. A comprehensive discussion of these measures is beyond the scope of this manual, but a general introduction of each type of measure is presented below.

(2) Upland sediment management measures are typically conservation practices that prevent detachment of soil particles from the land surface and reduce the delivery of the eroded material into the receiving water courses. These measures address protection from sheet and rill erosion to ephemeral gully erosion. Types of sediment management measures in this category include conservation cropping practices, contour terraces, vegetated buffer strips, filter strips, vegetated waterways, and vegetated surface treatment.

(3) Structural techniques such as debris basins, detention ponds, and floodwater retarding structures are also effective upland sediment management measures by providing peak runoff reduction and sediment retention. A comprehensive source for the conservation practice

standards of upland sediment management measures can be found at the USDA NRCS website[8] (also see internet search using: NRCS Conservation Practices).

(4) An example of a study to determine the sediment yield reduction potential from upland management measures is provided by Bingner et al. (2010) for the Cache River watershed in northeastern Arkansas (see Case Study 6B in Appendix N). The USDA-ARS AnnAGNPS model suite was applied to the Cache River watershed to evaluate sediment production from gully erosion and to assess the potential sediment reduction effectiveness of various upland sediment management measures.

(5) In-channel sediment management measures are typically more robust structural methods that provide erosion resistance to vulnerable channel bed and banks. The structures accomplish the erosion control/sediment management by providing a protective layer of erosion-resistant material on exposed areas, by increasing roughness and reducing velocities and erosive stress along the channel perimeter, and by retarding/reducing channel discharges. These types of countermeasures can be very effective in reducing sediment yield if the channel bed and banks are significant sediment sources.

(6) Typical in-channel sediment management measures include bank stabilization (both hardened and bio-engineered), grade control structures, flow retards, bendway weirs, drop inlets, and check dams. General application guidance for in-channel sediment management measures is presented by Biedenharn et al. (1997). Dendy et al. (1979) and Biedenharn and Watson (2011) presented an example of the sediment yield reduction that can be achieved with these types of sediment management measures for incised watersheds in the Mississippi Delta Headwaters Project (MDHP) in northern Mississippi.

(7) It is important to remember that the characteristics of the targeted sediment sources will not only guide the determination of the most appropriate sediment management measures to achieve sediment yield reduction, but will also influence the time frame over which sediment reduction is realized. Sediment management measures that control fine grain material that is typically wash load may produce immediate reductions in sediment yield. Management of coarse grain material in the bed material size classes may not reduce sediment yield at the basin outlet for years or even decades. In terms of coarse grain sediment, a sediment routing model is the most reliable means to estimate the timeframe required for management strategies to produce measurable reductions in basin sediment yield.

c. Timing of Sediment Reduction After Implementation. Agricultural, forestry, and urban best management practices (BMPs) are often applied in watersheds with the hope of reducing sediment delivery to federal navigation channels, among other benefits. Since federal navigation channels are often located far from the implemented BMPs, it may take a considerable amount of time before a reduction in sediment supply is observed in the navigation channel.

(1) The lag time before this reduction is realized depends on many factors, including the watershed slope, watershed shape, size of particle, precipitation regime, extent of storage in the

[8] http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/technical/cp/ncps/?cid=nrcs143_026849

watershed, and flashiness of the river. Many (Marutani et al., 1999; Clark and Wulcock 2000; Newson 2007) have examined this lag time and found it to be typically between a decade to a half century, or longer. While it may initially seem discouraging that an immediate reduction in sediment delivery is not likely to be realized, a long-term view of sustainable navigation must include BMPs to control sediment at its source.

(2) The cost to prevent sediment from entering the stream is significantly less (as much as an order of magnitude less) than dredging it from a downstream federal navigation channel (Pimintel et al., 1995). Many additional impairments are improved through the application of BMPs, such as loss of soil fertility, loss of fish habitat, increased flooding due to aggradation, reduction in aesthetics, etc.

6-11. Staged Sedimentation Studies. Chapter 2 provides a detailed discussion of staged sediment studies and the SSWP. Once study objectives have been identified, it is up to the engineer to select an appropriate evaluation procedure. ER 1110-2-8153 requires that a sediment impact assessment be prepared for all projects. A "staged sediment studies" approach should be followed in which contingency factors are assigned and revised as more data and analysis are available to decision-makers. Typical components of the staged study, as applied to sediment yield studies, are as follows:

a. Stage 1 – Sediment Impact Assessment (Reconnaissance).

(1) Often performed to support other hydrologic studies.

(2) If sediment problems are determined to be negligible, the sediment impact assessment can be the final stage.

(3) Stage 1 examples include:

(a) Regional sediment yield relationships.

(b) Sediment budget at coarse level.

(c) Sediment gage analysis.

(d) Sediment yield from reservoir capacity depletion trends.

b. Stage 2 – Detailed Sedimentation Study (Feasibility).

(1) Sediment impact assessment predicts a sedimentation problem.

(2) Similar, existing project is experiencing sedimentation problems.

(3) Computational analysis of existing/future conditions and with/without project.

(4) Stage 2 examples include:

(a) Detailed sediment budget.

(b) Flood risk reduction study project area with sediment issues.

(c) Sediment yield impacting dam safety.

c. Stage 3 – Feature Design Sedimentation Study (Preconstruction Engineering and Design).

(1) Extension of the detailed sedimentation study to increase accuracy.

(2) Seldom applicable to sediment yield studies.

6-12. Report Requirements. Sediment yield study reporting requirements vary widely with study methods and details. As a minimum, the following topics shown in Table 6-7 should be considered for inclusion in a typical study report.

Table 6-7
Typical Study Report Topics

• Essential. o Basin and study area map. * Aerials. ~ Current. ~ Historical. * Drainage area boundaries. * Urbanized area. * Land classifications. * Soil type map. o Stream profile. * Bed elevation vs. river miles. * Hydraulic controls and key slope breaks. * Structures. * Distributaries/tributaries. o Description of sediment-related issues. * Current. * Future. ~ With and without project. • Next Level. o Drainage area vs. river mile. o Annual water yield vs. river mile. o Flow duration. o Rating curves.

* Water.
o Hydrographs.
* Water discharge.
* Stage.

- Next Level.
- o Rating curves.
  - ▪ Sediment.
    - Total load ratings
    - Wash load and bed-material load ratings
    - Select ratings discretized by sediment size class
- o Sediment yield.
  - ▪ Average annual sediment yield.
    - By sub-basin.
    - Trends with time.
  - ▪ Fraction of average annual sediment yield.
    - By ranges of water discharge.
      - o Probability density function.
- Next Level.
  - o Sediment yield.
    - * Single events.
      - ~ Actual.
      - ~ Hypothetical.
    - * Class.
      - ~ Clay.
      - ~ Silt.
      - ~ Sand.
      - ~ Gravel.
    - * Fraction.
      - ~ VFS, FS, MS, etc.
  - o Sediment budget.
    - * Future conditions.
      - ~ Without project.
      - ~ With project.

Chapter 7
River Sedimentation

7-1. Purpose. The purpose of this chapter is to introduce fundamental concepts of fluvial geomorphology related to sedimentation to provide an understanding of river processes, identify potential river sedimentation problems using stability assessments, associate those problems with project purposes, and propose approaches for analyzing them. Figure 7-1 illustrates this chapter's content.

7-2. Scope.

a. This chapter presents typical sedimentation problems and offers guidance in selecting methods for their analysis. To diagnose sediment problems, it is necessary to distinguish between the independent or driving processes and the dependent or symptomatic processes. Fluvial geomorphology concepts and the analysis of sedimentation problems that are associated with flood risk management, navigation, and ecosystem restoration projects are presented in detail because of their importance to the USACE mission. River sedimentation investigation steps, data requirements, and maintenance needs are emphasized. Additional references are included that provide a more comprehensive presentation fluvial geomorphology fundamentals and the application to USACE projects.

b. Sedimentation problem analyses are conducted at varying levels of detail. Chapter 2 of this manual discussed the recommended staged sediment study approach. The appropriate level of detail for a particular evaluation depends on the status of the study, the perceived seriousness of potential problems, the scale of the project and the resources available (Copeland et al., 2001). Analysis detail levels appropriate for different study stages and complexity are discussed in Chapter 2 and shown in Figure 2-4, Figure 2-5, and Figure 2-6. Chapter 9 of this manual provides specific guidance on sediment modeling pertaining to river analyses.

7-3. Fundamentals of Fluvial Geomorphology.

a. Sedimentation Processes. Fluvial geomorphology is the study of the form and structure of the surface of the earth as shaped by water. A significant principle of fluvial geomorphology is the systems approach. This approach recognizes that the character and behavior of the fluvial system, at any location, reflect the integrated effects of a set of upstream and downstream controls. The five basic sedimentation processes active in fluvial systems are: (1) erosion/degradation, (2) entrainment, (3) transportation, (4) deposition/aggradation, and (5) compaction. Sedimentation engineering is concerned with all five of these processes. Common vernacular frequently uses the terms sedimentation and deposition interchangeably, however in this manual, deposition is only one process associated with the general term "sedimentation."

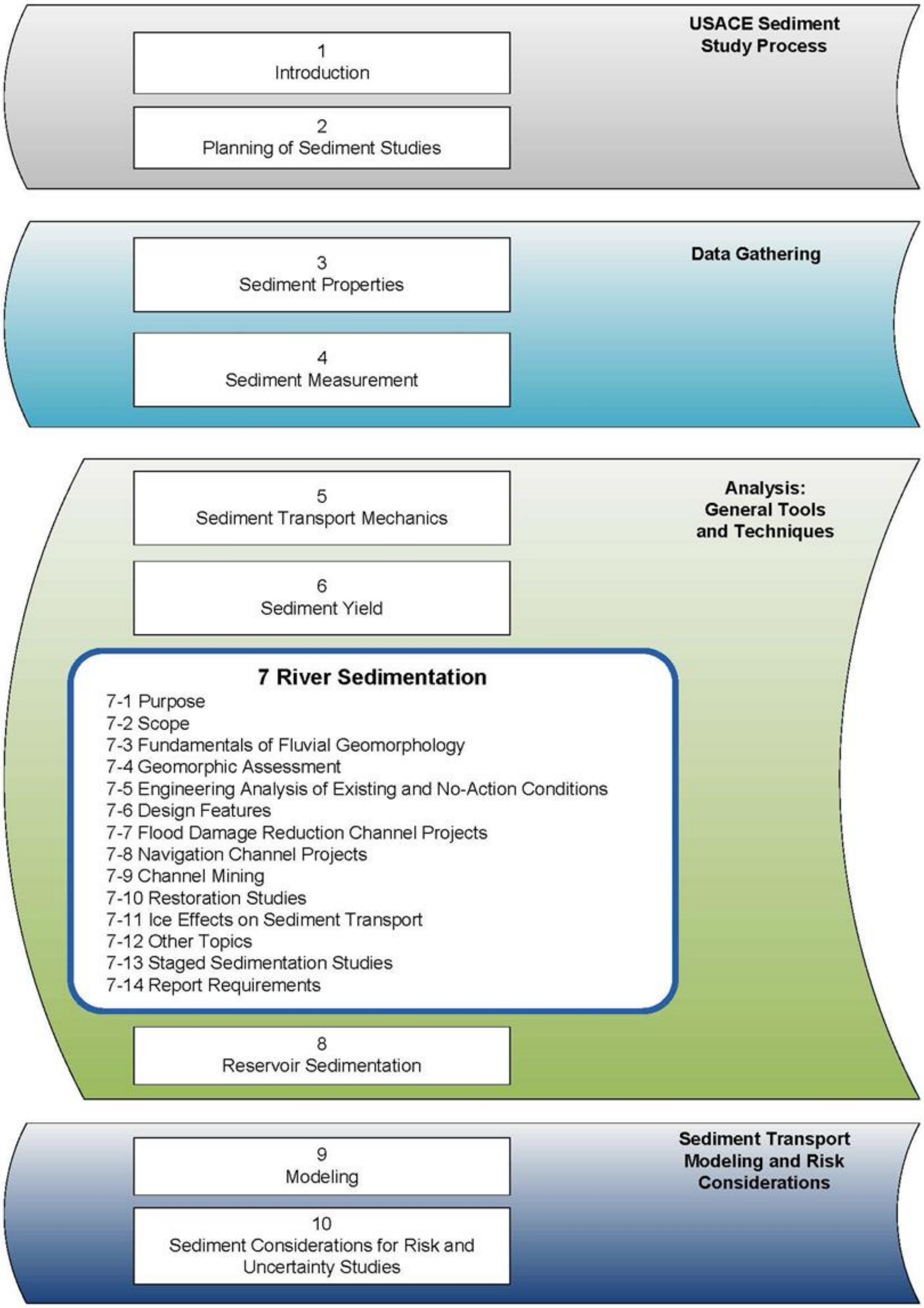


Figure 7-1. Chapter 7 content and general document structure

b. Basic Concepts. Six basic concepts have been identified for working with watersheds and rivers (Biedenharn et al., 1997; Biedenharn et al., 2008): (1) the river is part of a system, (2) the system is dynamic, (3) the system is complex, (4) geomorphic thresholds exist, and when exceeded, can result in abrupt changes, (5) system evolution and response are time-scale-dependent and engineering geomorphic analyses must provide a historical perspective, and (6) system evolution and response are space-scale-dependent and the physical size of the system must be considered in the engineering geomorphic analyses.

c. Idealized Fluvial System. Figure 7-2 shows an idealized sketch of a fluvial system (Schumm 1977). The upper zone in Schumm's fluvial system is the primary source of water and sediment. The dominant sedimentation process in the upper zone is erosion. The middle zone is the transport zone where the river is the most stable. The lower zone is the depositional zone, which may be a delta, wetland, lake, or reservoir. These zones are idealized because sediments can be stored, eroded, and transported in all zones. However, within each zone one process is usually dominant. Transition from one zone to the next is not necessarily continuous. The fluvial system may have perturbations such as an alluvial fan, which acts as a local depositional zone at the mountain to valley floor transition.

(1) The system is dynamic. The very nature of the fluvial system with different zones and different dominant sedimentation process suggests that change is occurring in the system and that the system is dynamic. The magnitude of sediment erosion in the erosional zone changes over the long term and annually with variation in the hydrograph. Consequently, changes occur in the transport and depositional zones to accommodate the variability in sediment supply. Dynamic equilibrium may occur in the transport zone where a stream has adjusted its width, depth, and slope such that the channel is neither degrading nor aggrading in the long-term. Within the idealized fluvial system, the depositional zone is dominated by sediment accumulation so that channel change and long-term storage and consolidation are dominant.

(2) The system is complex. Changes at one location in the fluvial system may result in activation or amplification of sedimentation processes that induce channel change both locally and throughout the system. Changes may occur over a prolonged period of time and at variable rates. An example is the downstream end of the Truckee River in Nevada, where the river flows into Pyramid Lake.

(a) Lake elevations vary with annual runoff and with the magnitude of irrigation diversions upstream. Lake levels dropped dramatically in 1905, when trans-basin flow diversions were initiated.

(b) Lowering the base level by about 80 feet resulted in channel incision and channel widening through the original delta and upstream through the river system. The degradation process increased the sediment supply to the downstream end of the river causing the rapid formation of a new delta at the lower lake elevation.

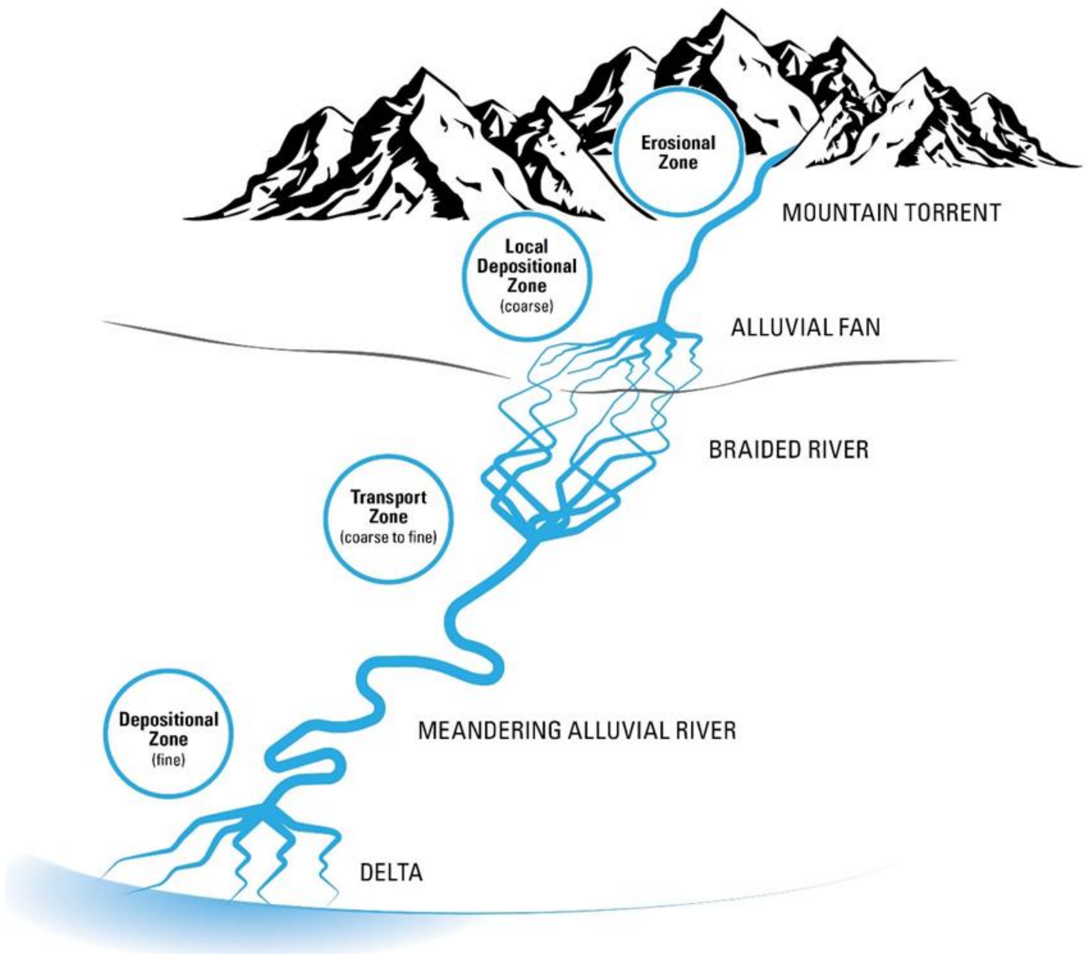


Figure 7-2. A fluvial system (redrawn following Schumm 1977 and EM 1110-2-1418)

(c) In 1967, diversions were reduced to halt the continuous drop in lake water surface elevations; however, lake levels continue to fluctuate with variations in annual runoff.

(d) A dam was constructed about four miles upstream from the normal water level of the existing lake in 1976 to halt continued headcutting. The dam temporarily decreased the sediment supply downstream. Storage has now filled so that sediment supply has essentially returned to pre-dam conditions.

(e) As a result of all these changes, the river's landforms include a complex pattern of both degradation and aggradation, which reflects the changes in dominant sedimentation processes over the years.

(3) Geomorphic Thresholds. Within the fluvial system progressive change in one variable may eventually bring about an abrupt change in the system. Channel systems have a measure of elasticity that enables some change to be absorbed by a shift in fluvial conditions. However, if the system is near a threshold condition, a minor change may result in a dramatic response.

(a) For example, eroding a few grains of soil from the toe of a river bank in single events may not be significant, but, over time, enough material may be eroded to cause the bank to fail. In another example, the cohesive river bed acting as a stabilizing hard point is slowly eroding. Once the cohesive layer is completely eroded and a much more erodible sand layer is exposed, channel degradation can progress rapidly.

(b) Erosion or deposition is not always progressive, and the crossing of thresholds in high-relief situations may lead to episodic erosion or deposition (Schumm 1979). In application, when fluvial system conditions (stream slope, meander pattern, etc.) are near a threshold, a major flood is likely to significantly alter the stream geomorphology.

(4) Time Scales. When describing geomorphic change, it is important to keep in mind the length of time associated with the geomorphic process. Geomorphic change time scales can be described as: (1) geologic, (2) modern or graded, and (3) present or steady.

(a) Geologic time is very long term—thousands or millions of years. Processes that take place in geologic time include mountain building, sea level change, and glaciation. Climate variability has affected some of these processes. Equilibrium concepts are not relevant when geologic time frames are considered.

(b) Modern time is long term, describing a period of tens of years to several hundred years. This is generally the time frame associated with river adjustment to some imposed condition such as land use change or major engineering works. During modern time, a fluvial system may adjust to a dynamic equilibrium where the river's geometric form has adjusted to the prevailing watershed conditions. However, recent scientific evidence shows that in some geographic locations, climate change is shifting the climatological baseline about which natural climate variability occurs (USACE 2014a). A climatological shift may alter the concept of a fluvial system reaching dynamic equilibrium during the modern time scale.

(c) Present time is short term, 1 to 10 years. A river system may respond quickly in present time to constructed features such as navigation dikes, dams, and grade control structures. Initial river adjustments can be evaluated to determine short-term effects in present time.

(d) Sedimentation investigations must consider both modern and present time. Understanding geologic time scale changes may also be relevant to sediment investigations in some systems, especially with respect to sediment. For example, the Upper Mississippi River is referred to as an "underfit" river following the glacial runoff period that drastically altered the river geomorphology during the last glacial age about 10,000 years ago (Wright 1990). Although in the geologic time scale, this continues to affect sedimentation processes in the present time.

(5) Physical Scale. The size of the fluvial system influences the length of time required for natural adjustments to occur. Large rivers will generally require more time to adjust than small streams.

(a) Howard (1982) holds that the times scale of change is affected by sediment and water volumes and also reports of very long time scales for gradient adjustments in large river systems

(>10,000 years). For this reason, sedimentation studies of large rivers should evaluate longer time periods when determining the effect of engineering works. For example, although over 10 feet of degradation had occurred since Gavins Point Dam closure in 1955 and degradation trends were nearly stable, several feet of additional degradation occurred over three months during the high reservoir releases of 2011 (USACE 2012d).

(b) The physical size of a stream also affects the appropriateness of various types of engineering works and study techniques. Root balls (wads) placed on a river bank will have a significantly different effect on a small stream than on a large one. A large river with a heavy suspended load will react differently to channel contraction works than a small stream with a gravel bedload.

d. Fluvial Process Concepts. Fluvial geomorphology and the presence of specific landforms can be used to help determine which of the five basic sedimentation processes are dominant and the existing system stability. Field reconnaissance is key to identify landforms and ongoing processes present in both the project reach and in the reaches upstream and downstream from the project. Some typical fluvial geomorphic concepts are discussed below. More detail can be found in Biedenharn et al., 2008; Biedenharn et al., 1997; EM 1110-2-1418, Chapter 4; and Thorne 1993.

(1) Channel Deposits. Channel deposits include the prevailing bed form, localized bar formations, and zones of aggradation.

(a) In alluvial sand-bed channels, the presence of bed forms is an indicator of bedload transport. As bed forms change with discharge, so can the influence of bed forms on the dominant sediment transport mode between bedload and suspended-load. Rivers also have widely varying characteristics when evaluating transport modes. For instance, the Missouri River has large dunes as confirmed by bedload measurements (paragraph 4-10g; Abraham et al., 2017). However, suspended load is the dominant transport mode in the range of 85% to 95% of the total load at multiple locations, time periods, and flows.

(b) Riffles and boulder steps are characteristic of a threshold condition and suggest suspended load is the dominant transport process across these features. Point bars are deposits found in both gravel-bed and sand-bed rivers on the inside of bends. These features are essential for the transport of sediment through a sinuous fluvial system where hydraulic conditions change with the annual hydrograph.

(c) The presence of bars in gravel-bed streams suggests that bed material transport is occurring across the bar and that it is a significant process. However, it is possible that there is no "dominant" transport process and that aggradation/degradation are also significant processes at work. The presence of point bars does not necessarily imply system stability (point bars can exist in both aggrading and degrading rivers). The presence of mid-channel bars often suggests a localized instability where deposition is the dominant process. Mid-channel bars may form when coarse bed material load cannot negotiate a meandering thalweg, in areas with channel widening, and may be aggravated by increased upstream sediment input. Vegetated mid-channel bars may indicate a previous non-equilibrium condition to which the river has adjusted.

(d) Channel deposits can occur in long reaches of aggradation in unstable rivers or in the deposition zone (Figure 7-2) of a fluvial system (Figure 7-3). Deposits can be large and result in significant stage increases over time that can impact typical USACE FRM projects.

Figure 7-3. Aggradation reach, Crooked Creek, Tennessee

(e) Temporal factors and the recent flow regime are important to consider when evaluating channel deposits. For instance, a recent extreme event can give the appearance of system instability when in fact the river is in the process of adjusting back to a more stable condition.

(2) Floodplain Deposits. Sediment deposition on the floodplain indicates that the river has a substantial suspended load and may indicate that the channel is undersized with respect to its dominant or channel-forming discharge. One of the most significant types of floodplain deposits are natural levees adjacent to alluvial streams. Natural levees are formed when the river overflows its banks and coarser suspended sediment immediately drops out, forming a high berm adjacent to the river (Figure 7-4).

(a) The presence of a natural levee indicates that the river is or has been in a deposition zone. The grain size of floodplain deposits provides insight into the composition of the sediment wash load during overbank flow conditions. The historical progression of meander migration may be determined from an investigation of floodplain deposits (Figure 7-5).

Figure 7-4. Natural levees of the Lower Mississippi River at Head of Passes (only natural levees are visible at this high water stage)

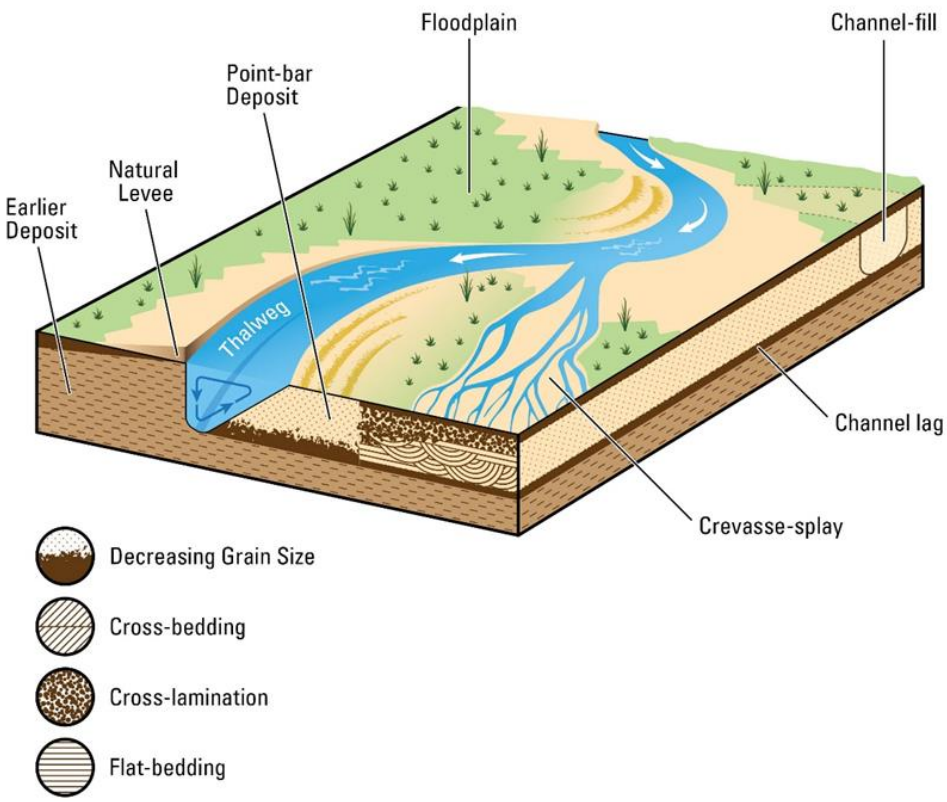


Figure 7-5. Landforms for a meandering river (redrawn from previous efforts including Collinson 1978, Allen 1970, and Biedenharn et al., 1997)

(b) Natural levees can provide habitat diversity in the river valley due to the formation of a barrier between flowing channels (lotic habitat) and off-channel (lentic or floodplain habitat).

(3) Channel Degradation. Channel degradation often called headcutting or scour, is a degradational process that indicates system instability.

(a) Channel degradation maybe system-wide or more localized. Degradation is characterized by channel-bottom erosion moving upstream through a basin, indicating that a re-adjustment of the basin slope and its stream discharge and sediment load characteristics are taking place. It can be initiated by base-level lowering channel straightening or as a result of natural down valley progression of meander bends resulting in channel cutoffs (Figure 7-6).

Figure 7-6. Channel incision due to headcutting on Crowder Creek, Mississippi (headcutting has been arrested by road culverts)

(b) The headcutting process frequently produces knickpoints and knickzones in streams with cohesive beds. A knickpoint is a location along the streambed where an abrupt change in elevation occurs. In noncohesive channels, knickpoint identification is more difficult, as there are no abrupt changes in channel bed elevation. A knickpoint is a transient feature that moves upstream as the cohesive material in the bed slowly erodes. Knickpoint migration rates can vary and affect sediment delivery for significant time periods. A knickzone is similar to a knickpoint, except that it extends over a significant length of the stream.

(4) Incised Channel. An incised channel is a channel that is not hydraulically connected to its floodplain due to degradation. Incised channels can be caused by base-level lowering associated with channelization or cutoffs. Another cause can be a reduction in the sediment supply caused by urbanization, dam construction, or introduction of relatively clear water from wastewater treatment plants. Considerable deterioration of habitat is typically associated with an

incised channel. The presence of an incised channel usually indicates system instability as opposed to local instability. However, some systems with unique geomorphic characteristics are incised and exhibit long-term stability.

(5) Terraces. Terraces are abandoned floodplains formed when the river flowed at a higher level. Terraces are produced as a consequence of channel incision. The river then may establish a new floodplain at a lower elevation (Figure 7-7).

(6) Berms. The formation of berms can indicate the tendency of an incised channel to re-establish stability and denotes the building of a new active floodplain within the incised channel (Figure 7-8).

(7) Conditions at Structures. When deteriorating conditions at structures are consistent over a long reach of the river, system instability can be inferred. Exposure of bridge piles, failure of outlet aprons, exposed pipelines, and undercutting of bank protection are all indications of instability due to degradation. Aggradation at these structures can be difficult to identify without original construction plans, although it can be obvious in extreme cases (Figure 7-9).

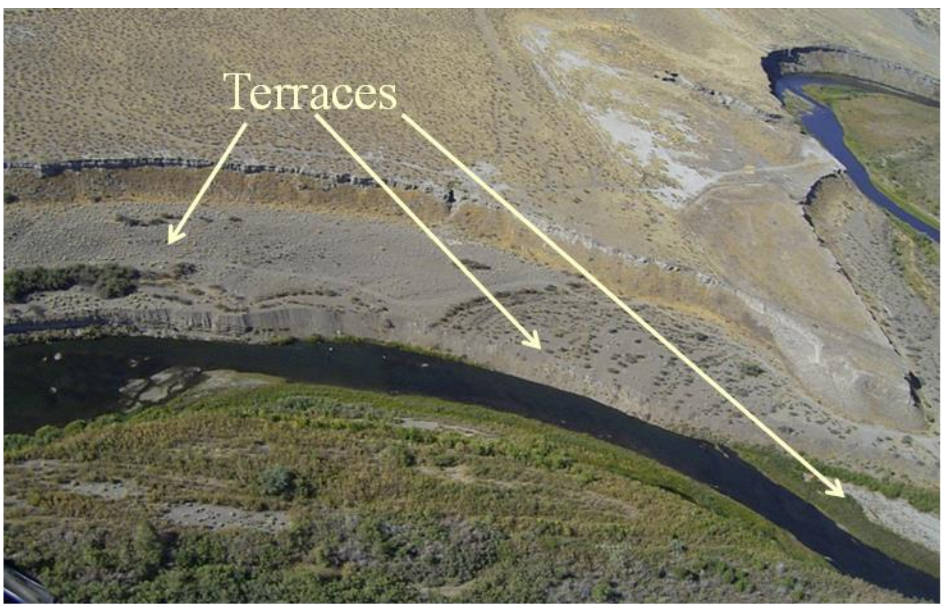


Figure 7-7. Terraces formed on Lower Truckee River, Nevada (photo illustration shows different stages of channel incision)

Figure 7-8. Berm formation on South Papillion Creek, Nebraska (photo illustrates new berms formed in an incised channel at new active floodplain level)

Figure 7-9. Aggradation at bridge, Whitewater River, Minnesota

e. Bed Forms.

(1) Bed forms are transient sedimentary features found in the stream bed. Bed form geometries are correlated to sediment transport characteristics. The bed form type transitions according to flow regime with lower regime bed forms such as ripples and dunes, followed by a transition to upper regime bed forms such as plane bed and antidunes (see Figure 7-10). Bed forms are wave-like features that have a measurable wavelength and height. An analog to bed forms are desert sand dunes.

(2) General observations of bed forms, adapted from Chapter II, ASCE Manual No. 54 (Vanoni 1975, 2006) are:

(a) Bed forms are the result of an orderly pattern of scour and deposition.

(b) Bed forms are a consequence of an instability phenomena.

(c) Bed forms are dependent on flow parameters and bed sediment properties.

(3) The bed forms shown in Figure 7-10 are an illustration of the classic forms of ripples, dunes, and antidunes (Simons and Richardson 1962). Dunes and ripples migrate downstream and are often viewed to approximate a wedge or triangular cross-sectional shape with a sharply sloping slip face in the downstream direction. Antidunes are associated with supercritical flow, and may migrate either upstream or downstream. Ripples are smaller bed forms that mostly occur with a viscous sublayer.

(4) Ripples and dunes are the most common bed forms observed for subcritical flow. Ripples are steeper and shorter than dunes, and their length depends on particle diameter. Dune height and length are generally considered to be a function of the flow depth (as shown by the Froude number in Figure 7-10) with a more complex dependency on particle size and fluid properties. Experimental observations have shown that ripples disappear, and dunes become the prevailing bed form when the flow transitions from hydraulically smooth to hydraulically rough conditions (Garcia 2008b).

(5) The progression of bed forms with changing flow conditions is often significant, as illustrated in Figure 7-10 (Simons and Richardson 1962). The bed is assumed to be initially flat at low velocity because no sediment is moved. As the velocity increases, ripples form (subset A). At higher flow velocity values, dunes form and coexist with ripples (subset B) or bed forms may exist only as increased size dunes (Subset C). In many systems, further increases in velocity reach a critical value and dunes may suddenly wash out resulting in a flatbed known as an upper regime (supercritical) flatbed (subset D and E). Further velocity increases lead to the formation of antidunes and finally to the chute and pool pattern (Garcia 2008b). The transition regions from dunes to flat bed and antidunes is defined in by varying Froude number values within Figure 7-10.

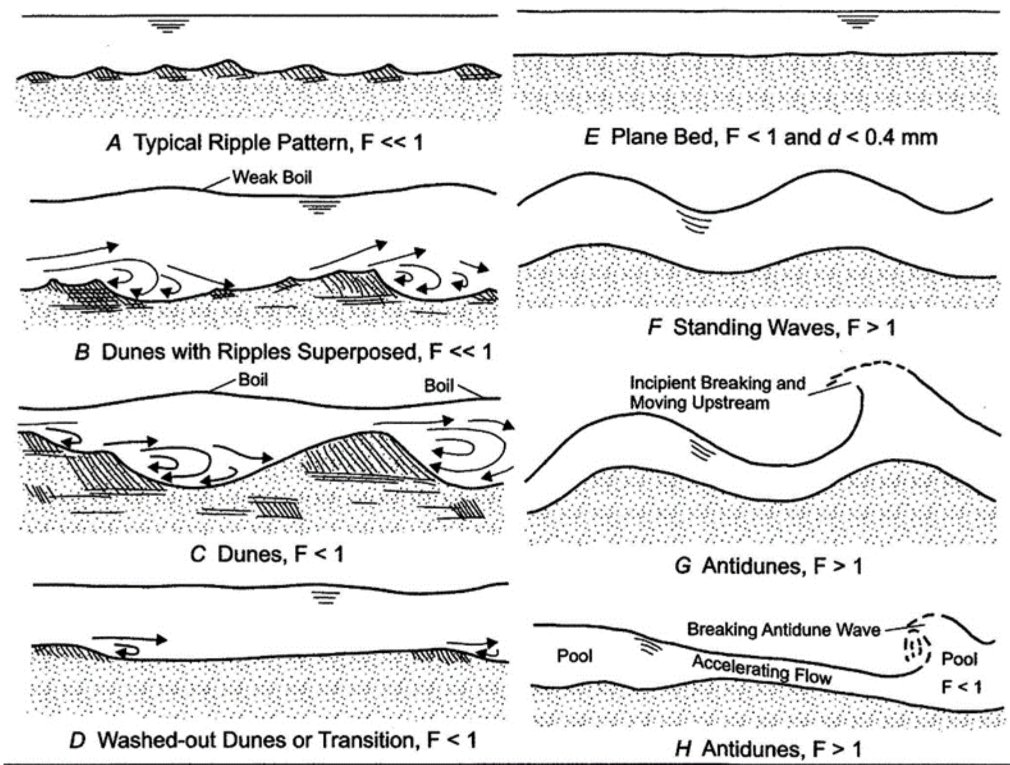


Figure 7-10. Forms of bed roughness in alluvial channels (Simons and Richardson 1962)

(6) Bed forms change the roughness of the stream bed to a much larger degree than that of individual grains alone. Refer to Simons and Richardson (1962) and ASCE Manual No. 110, Chapter 2 (Garcia 2008b), for a thorough discussion on bed forms, their prediction, and impacts to roughness and channel hydraulics. Recent advances in bed form measurement (paragraph 4-10) illustrates the potential for significant advances in bed form predictive equations.

(7) Bed form characterization using multiple parameters have been proposed by numerous investigators. Figure 7-11 shows two bed form discriminators. Refer to Garcia (2008b) for further illustration of multiple bed form discriminators that are available for a wide range of flow conditions.

(8) Knowledge of bed forms and how they affect the river system can be of vital importance for design and analysis of USACE projects. Bed forms in alluvial channels can have a significant influence on flow resistance and thus, on sediment transport with a corresponding impact on the stage-discharge relationship. A variation of 0.5 to 1.5 feet in stage at normal flow was observed on the Missouri River at Omaha due to variation in bed forms (USACE 1977). Refer to paragraph 5-3 for further discussion on bed form flow resistance and roughness prediction.

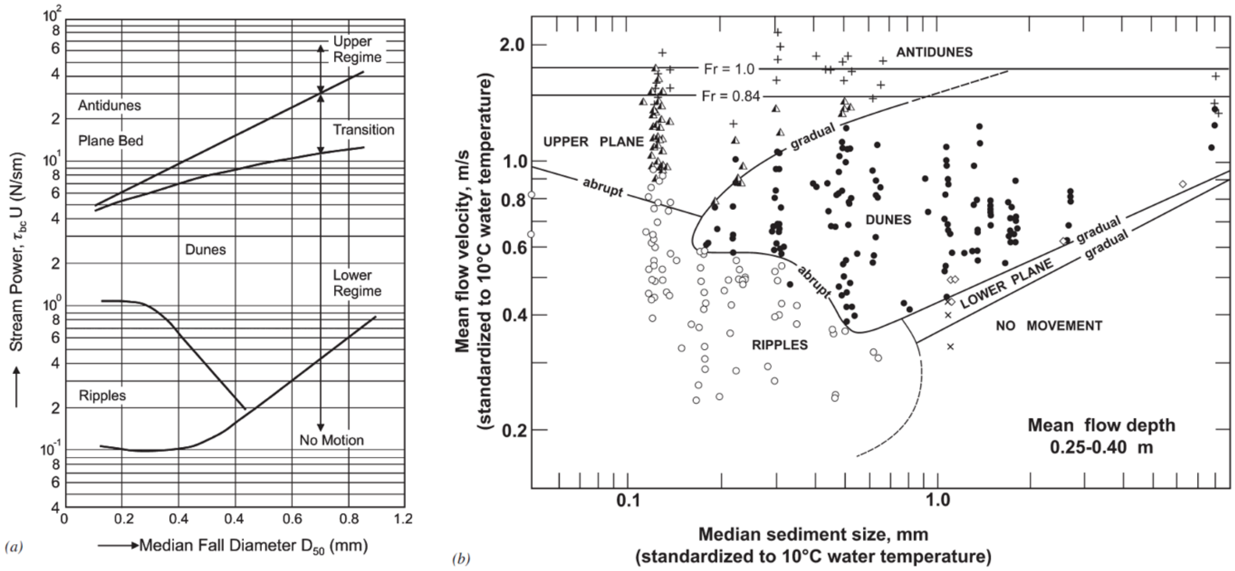


Figure 7-11. Bed form discriminators proposed by (a) Simons and Richardson and (b) Boguchwal and Southard; (Garcia 2008b)

Used with permission of ASCE, from Chapter 2, Sedimentation Engineering, Processes, Measurements, Modeling, and Practice, Manual No. 110, Garcia, M.H., 2008; permission conveyed through Copyright Clearance Center, Inc.

(9) Bed form geometry can also be influenced by changes in water temperature. Colder water temperatures are associated with a decrease in dune height with an associated decrease in flow roughness and water surface elevation (USACE 1976). Conversely, Simons and Richardson (1962) report that, if water temperature is lowered in a sand-bed covered with small ripples, the mobility of the particles is increased (because the sand fall velocity is decreased), larger ripples form, and flow resistance increases.

(10) In recent years, there has been great progress in the understanding and measurement of bed form dynamics that has been linked to significant advances in the ability to monitor flow and dune morphology in the laboratory and field. Paragraph 4-10 addresses field measurement of bed forms.

(11) Bed form processes are complicated with significant variation in transport rates laterally across the stream. Figure 7-12 shows a field illustration of lateral variation.

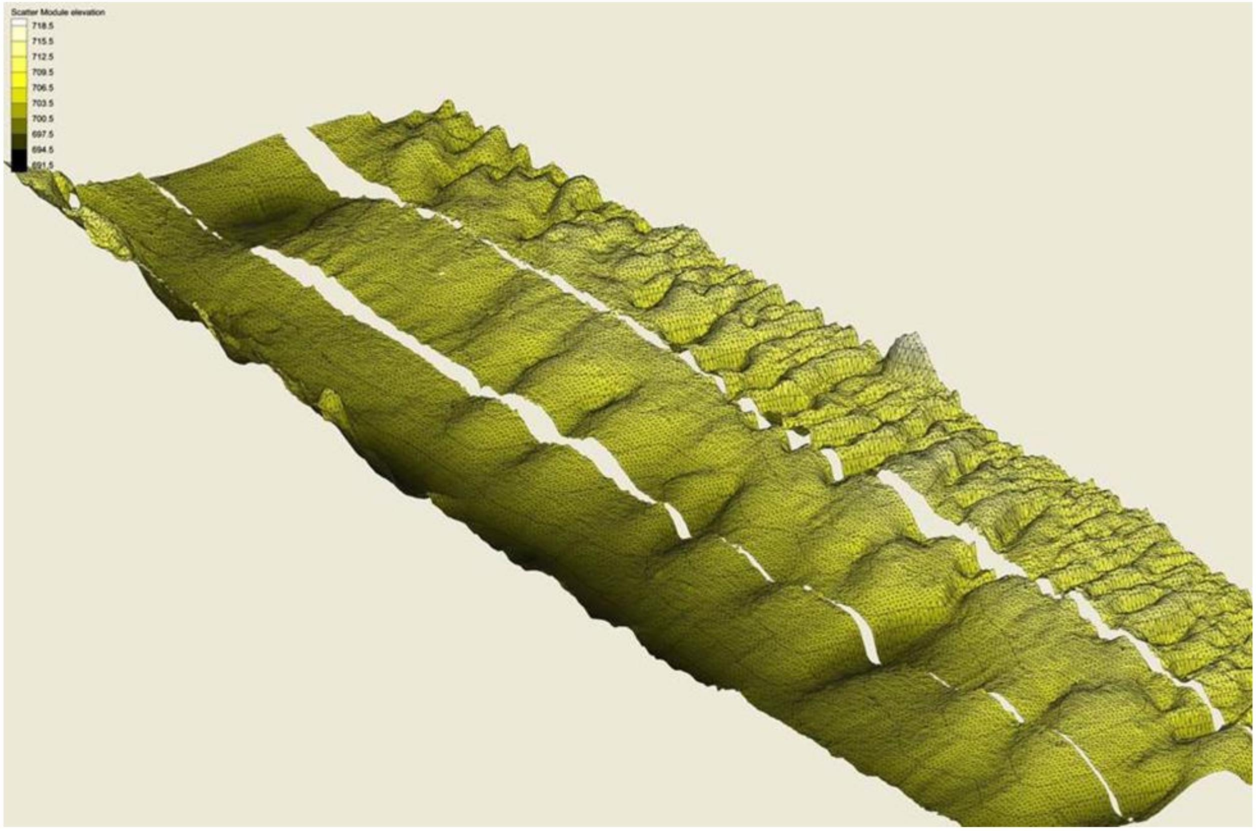


Figure 7-12. Lateral variation in bed forms, Missouri River near Kansas City

f. Common Channel Types. This section provides a brief overview of a number of common channel types and their characteristic stability problems (adapted from EM 1110-2-1418). Chapters 6 to 10 of this manual describe in detail the sedimentation processes and analysis methods appropriate for these sedimentation processes and channel types.

(1) Mountain Torrents. These are high-velocity streams on steep slopes, often exhibiting a sequence of drops and chutes controlled by large boulders, fallen timber, etc. Erosion and deposition are sometimes confined to severe flood events. Some mountain torrents on very steep slopes are subject to the phenomenon of debris flows or debris torrents whereby, under severe flood conditions, the bed becomes fluid, and a virtual avalanche of boulders and gravel runs down the mountainside. FRM projects in these areas require detailed analysis with means to mitigate the debris flow as discussed in paragraph 7-8.

(2) Alluvial Fans. Alluvial fans are depositional landforms developed over a geologic time scale, located at the base of mountain ranges where steep slope mountain streams transition to the lesser slope valley floor. They are usually conical, or fan-shaped in planview. They are depositional features usually characterized by coarse alluvial materials and unstable multiple channels subject to frequent shifts or avulsions. The main channel is often "perched" on the highest ground. Sometimes the alluvial fan is inactive depositionally, and the stream is eroding into earlier deposits. On topographic maps, alluvial fans appear as contour lines that are concentric around a river canyon mouth. In wooded country, they are not always easily

recognized on the ground. Stability problems on alluvial fans can be severe and catastrophic, as discussed in paragraph 7-8e.

(3) Braided Rivers. Braided rivers consist of a network of interlacing channels with unstable bars and islands. They generally occur in the upper and upper middle zones of a basin. Bed materials are usually gravels or cobbles, however, braided sand rivers are found occasionally. Bed material transport tends to be high, at least in flood periods. Stability problems include how to maintain the channel through transport of the bed material load and how to avoid serious disturbances of the longitudinal profile. Points that require consideration are the planned cross section, the alignment in plan, and provision for future shifting and erosional attack.

(4) Arroyos. Arroyos are streams in deserts and arid areas that are dry much of the time, but carry large discharges and heavy sediment loads under occasional flood conditions. The channel may be deeply incised into the terrain in some reaches and liable to frequent overspill in others. FRM projects typically include project features to address the heavy sediment loads. Project capacity can be reduced rapidly by sediment deposition if velocities are reduced by channel enlargement, weirs, or other works.

(5) Meandering Alluvial Rivers. These generally occur in the middle and lower zones of a basin. The single channel follows a characteristic sinuous planform and is normally eroding into the floodplain on one bank and depositing material on the opposite bank. Bed material is usually sand, or sand and gravel. In undisturbed natural systems, future channel shifting maybe predictable from comparison of sequential maps or aerial photographs. Numerous stability and sedimentation problems may arise from FRM works on meandering streams. Meandering systems are often sensitive to modest imposed changes. USACE FRM project planning requires consideration of past channel behavior and likely responses to modifications, including stabilization measures.

(6) Cohesive Channels. Channels in cohesive materials may be found in a variety of environments, including glacial till plains, coastal marine deposits, filled lakes, etc. Channels in till tend to have irregular planforms: the occurrence of an occasional sequence of regular meanders may indicate intersection with an infilled alluvial channel. In uniform marine clays, channels sometimes exhibit a series of uniform wide flat meanders easily distinguished from meanders in alluvial materials. The stability of channels in cohesive materials may vary widely, but it is generally greater than in alluvial materials.

(7) Underfit Streams. Underfit streams are common in glaciated regions such as the northern Great Plains and upper Midwest. They can be small streams or large rivers such as the Upper Mississippi River. In either case, the existing condition channel occupies a wide valley bottom that was formed and occupied by a much larger stream, usually the outflow from a glacial lake, near the end of the last glacial period. The slope along the valley bottom tends to be quite flat, and the underfit stream is usually of low velocity, relatively stable, with well vegetated banks. Sometimes the planform is highly contorted. Underfit streams are also found throughout the country in abandoned river courses and as a result of FRM projects such as reservoirs. Stability problems may occur due to an imbalance of sediment input and transport capacity.

(8) Deltas. Deltas occur on flat slopes where a river discharges into still water (reservoir or lake) and deposits its sediment load. Under natural conditions, the river splits into a number of distributaries, the bed levels of which rise over time as the delta extends into the water body. Deltas are similar to alluvial fans in that they both form because of a loss of sediment transport capacity resulting in sediment deposition. However, the term alluvial fan is specific to the stream transition from mountainous areas to the valley floor. FRM levees adjacent to deltas can require periodic raising, particularly if the river is confined artificially to a single channel. The potential for channel avulsions upstream of the works also is a possible occurrence.

(9) Modified Streams. In some regions, many streams have been modified in the past by human activity and resemble natural rivers. A common form of modification is straightening or enlargement for FRM. If changes occurred many decades ago, the details may be difficult to discover. Another form of modification is by FRM diversions or reservoirs with corresponding changes in stream geometry and sediment loads. Stabilization measures may reduce channel erosion rates and affect sediment supply. Floodplain modifications such as levee units and bridge embankment constrictions may have minimal impact on normal river flows, but highly impact high flows. The effects of USACE project modifications on sedimentation processes are complex and may require rigorous evaluation.

(10) Regulated Rivers. These are generally streams where the flood discharges have been reduced, sediments have been trapped, and the low flows increased by upstream storage reservoirs. Such streams often exhibit a reduction in morphologic activity compared with previous natural conditions, and the cross sections of their channels may have been reduced by deposition of sediment and encroachment of vegetation. However, if the stream carried substantial loads of bed material under natural conditions, trapping of sediment in reservoirs may initiate slope changes downstream. The effects of regulation on stability are complex and depend on the previous characteristics of the stream, as well as on the degree and mode of regulation.

(11) Navigation Channels.

(a) Modifications for navigation channels often have similar characteristics as those discussed for both modified streams and regulated rivers. Navigation dams, which typically provide minimal storage and flow modification, are constructed to maintain a minimum water surface elevation for commercial navigation. Their design and operation is performed to minimize effect on water surface profiles during flood conditions. However, geomorphic changes in the upstream pool may change the patterns of sediment erosion, transport, and deposition.

(b) Navigation channels include a high degree of stabilization structures to provide a reliable channel location. Features to minimize sedimentation and maintenance are also common. Some channels, such as the Missouri River, are designed as self-scouring. Bank armoring, in-channel structures, and maintenance dredging vary widely within USACE navigation channel projects, but generally have significant impact on sedimentation processes.

g. Channel Stability Concepts.

(1) A stable river channel is one in which the planform, cross section, and longitudinal profile are sustainable over time. Rivers may migrate laterally and longitudinally and still be stable in the geomorphic definition. However, the presence of highways and structures on the river bank may affect the perception of stability. Maintaining sediment continuity through a river reach is the key to channel stability. Alluvial river channels adjust their geometry to the range of flow and sediment inputs they experience. When these inputs fluctuate within a normal range, the mobile boundaries that form the stream dimensions fluctuate around a balanced condition known as dynamic equilibrium.

(2) Biedenharn et al. (1997) provides the following definition: "… a stable river, from a geomorphic perspective, is one that has adjusted its width, depth, and slope such that there is no significant aggradation or degradation of the stream bed or significant planform changes (meandering to braided, etc.) within the engineering time frame (generally less than about 50 years). By this definition, a stable river is not in a static condition, but rather is in a state of dynamic equilibrium where it is free to adjust laterally through bank erosion and bar building."

(3) Natural events such as volcanic eruptions, major floods, earthquakes, and changes in climate could lead to instability and dramatic trends in river morphology, even in the pre-development condition.

(4) The primary indicators of system instability are aggradation and/or degradation. Degradation is a progressive lowering of the streambed and can occur along a length of stream or throughout the entire system. Aggradation is a process by which streambeds are raised in elevation due to the deposition of material eroded from upstream areas. Both processes occur over a significant length of river. Changes in planform characteristics (meander wavelength and sinuosity) can also be an indicator of system instability. Local channel instability refers to local bank erosion and local scour or deposition that is not symptomatic of watershed dis-equilibrium but results from site-specific factors and processes. Examples of local instabilities are bank erosion caused by flow impingement, deposition at the mouth of a tributary, and scour at bridge piers.

(5) Lane's Balance.

(a) Lane (1955a) presented stable channel design concepts and (1955b) developed the widely used relationship known commonly as Lane's balance (Biedenharn et al., 1997, Figure 1-4). The relationship states that the product of water discharge (Q) and slope (S) is directly proportional to the product of bed material load (Q_s) and the median bed material size (D_{50}), as shown in the following equation, and previously illustrated in Chapter 1, Figure 1-4:

$$QS \propto Q_s D_{50}$$

Equation 7-1

(b) Based on the relationship, a change in any one variable will cause a change in some or all of the other variables until balance is restored. USACE projects, such as bank stabilization, may alter one or more of these factors. For instance, significant bank stabilization eliminates a potentially significant sediment source from the river. According to the Lane balance, when this occurs, the river will respond by seeking sediment from another source to replace the lost sediment, reducing its slope and transport capacity, or a combination of both. Note that the D_{50} sediment size may be different for bank and channel materials; therefore, the sediment contributions for these sources may not be equivalent to maintaining a balance.

h. Hydrology.

(1) Hydrologic computations are an integral part of the evaluation of sedimentation processes and USACE project design. Identification of multiple flows are required in a typical project. Base flow estimates are used to assess habitat conditions, channel-forming flows are used to determine channel dimensions, and flood flows are used to determine stability of structures and for scour depth predictions. Future flow condition estimates are needed to assess future project performance. The maximum design flow used for stability analysis should be based on project objectives and failure consequences. For example, the 100-year discharge might be used to design bank protection in a densely populated area, while a 10-year discharge might be appropriate in a rural stream (Copeland et al., 2001).

(2) Frequency and Flow Duration Analysis. A hydrologic peak frequency analysis is performed to relate the magnitude of a given flow event with the frequency of occurrence. The quantity of sediment transport depends on flow duration. A variety of hydrologic techniques are available for predicting flow frequency and flow duration. In general, the hydrologic analysis follows USACE standard practice procedures as outlined in EM 1110-2-1415 and the Guidelines for Flood Flow Frequency, Bulletin 17C. Copeland et al. (2001) illustrated incorporating flow duration analysis with sediment transport.

(3) Channel-Forming Discharge.

(a) Although not universally accepted, the concept of a channel-forming discharge is found in many sedimentation texts discussions on stability, including Dunne and Leopold (1978), Copeland et al. (2001), NRCS (2007), and Biedenharn et al. (2008). Channel-forming discharge refers to the concept that natural alluvial streams experience a wide range of discharges and adjust their shape and size during flow events that have sufficient energy to mobilize either the stream's bed or banks. This discharge, therefore, dominates channel form and process and may be used to make morphological inferences.

(b) While many techniques and methodologies are used to estimate a channel-forming discharge in stable alluvial channels, all can be characterized as one of four main types. These are: discharge based on bankfull indices, discharge based on drainage area, discharge based on specified statistical recurrence intervals, and discharge based on an effective discharge calculation.

(c) The recurrence interval for channel-forming discharge is often assumed to fall between an annual and 2.5-year event, with a mean of 1.5 years. However, Williams (1978) determined a wider range on 35 river studies in the United States, with variation from the 1- to

32-year recurrence interval with only about a third between a 1- and 5-year recurrence interval. A specified recurrence interval is often used as a first approximation of channel-forming discharge. Because of the noted range, field verification is recommended. In addition, streams that do not fall within the expected range may be responding to natural or anthropogenic disturbances (NRCS 2007).

(d) Wolman and Miller (1960) demonstrated that, in most rivers, over an extended period of time, the total amount of sediment transported by a discharge of a given magnitude depends on both its transport capacity and the frequency of occurrence. Although extreme events usually result in spectacularly high sediment loads, they happen so infrequently that their overall contribution to the total river sediment movement during a long period is relatively small. The frequently occurring small events also make a small contribution to the total river sediment moved because their high frequency is offset by their very low sediment transport. Therefore, it follows that flows of both moderate magnitude and moderate frequency are responsible for the greatest amount of sediment movement (Copeland et al., 2001; NRCS 2007).

(e) Methodologies for estimating channel-forming discharge present challenges. In practice, problems often arise when attempting to identify bankfull stage in the field. Recurrence intervals for channel-forming discharge have been shown to vary widely for different types of streams. Calculation of effective discharge requires hydrologic and sediment data. In light of these challenges, it is recommended that multiple methods are used and compared to reduce the uncertainty in the final estimate of the channel-forming flow (Copeland et al., 2001).

7-4. Geomorphic Assessment.

a. A geomorphic assessment is recommended for flood risk management, navigation, and environmental studies to assess past and present sedimentation processes including watershed dynamics, erosion, transport, deposition, and consolidation and sorting. This is an essential part of the design process, whether planning stream bank protection or attempting to develop a comprehensive plan for an entire watershed. Knowledge of the dominant fluvial processes determined with a geomorphic assessment allows prediction of the proposed project’s impact on river morphology and channel stability and the effect that natural processes will have on the functionality of the project.

b. Evaluation of Channel Stability.

(1) One of the first tasks in a geomorphic assessment is to determine the stability of the existing system and the project reach. A channel is considered stable when the prevailing flow and sediment regimes do not lead to aggradation or degradation or to changes in the channel cross-sectional geometry over the medium to long term. Short-term changes in sediment storage, channel shape, and planform are both inevitable and acceptable in natural channels with unprotected banklines.

(2) Evaluation of stability can be undertaken at various levels, ranging from geomorphic assessments based on qualitative methods, Appendix F, to quantitative techniques using numerical models and analytical techniques. The geomorphic assessment, which is the most

fundamental level of assessment, is empirically based and is the initial step in the planning process. Haring et al. (2020) and Haring and Biedenharn (2021) provide a rapid watershed assessment approach using high-resolution terrain data to assess channel stability.

(3) System Driving Forces. Independent boundary conditions imposed on a river system are the inflowing water and sediment hydrographs, the particle size distribution of the sediment load, the composition of the stream bed, and the downstream water surface elevations. The boundaries include both the main channel of the river upstream and tributaries in the study reach. The downstream water surface elevation, or tailwater, can be the normal river energy grade; a geometric control such as at a structure or valley contraction; another river, a lake, or the ocean; or a regulated boundary condition, like a reservoir.

(4) Dependent Variables. The dependent variables are width, depth, slope, roughness, planform, and bankline migration. The end product of a geomorphic assessment is the predicted reaction of each of those dependent variables in each reach of the channel to the aggregate of forces from the independent variables. The behavior of each reach depends on the reaction of the reach just upstream from it. This interaction is referred to as the “stream system concept.” The concept of independent boundary conditions and dependent variables also suggests that a constructed channel should not be expected to perform without maintenance unless there is a corresponding change in the forces being imposed on the system.

(5) System Behavior. Empiricism suggests that the six dependent variables listed in the previous paragraph change in system-like fashion as each reach of the river responds to the load being placed on it from the upstream reach, from tributaries, and from lateral inflows. Likewise, a reach of the river will modify the inflowing loads and pass a slightly different set of loadings to the next reach downstream. The concept of changes occurring with time is an important one. Rather than studying streams at only one fixed point in time, the engineer must view the stream system as one of dynamic equilibrium in which channel width, depth, slope, bed roughness, planform, and bankline migration are continually changing.

c. Geomorphic Assessment Components.

(1) The most basic form of stability analysis is the assessment of bed stability—the determination of whether the channel bed is aggrading, degrading, or stable. Other aspects of stability assessment are bank stability, planform stability, historic or future changes in hydrology or sediment inflow, and changes in channel width or cross section. This section discusses the steps for assessing channel stability. Additional information can be found in Biedenharn et al. (2008); Schumm et al. (2008); Copeland et al. (2001); Biedenharn et al. (1997); USACE 1994a; and Thorne (1993, 1998).

(2) Data Assembly. The first step in the geomorphic assessment is to gather and compile existing data. Historical data are used to identify trends, to provide information on rates of landform change in the watershed, and to help the engineer determine land use impacts on current conditions. Data requirements depend on project objectives and watershed characteristics. Chapter 2, Appendix B, and Appendix D of this manual provide guidelines for data collection.

(a) Geographic Information System Database. A GIS database is useful for compiling data and readily accessing information. Significant geomorphic and engineering features can be located by longitude and latitude on a digital map. Information relating to these features and to the study reach in general can be stored and accessed as needed. Information may include field notes, other text and reports, plans, and photographs.

(b) Aerial Reconnaissance. Aerial reconnaissance using geo-referenced photos or videos are a valuable tool for geomorphic studies. Repetitive photos at the same location over time indicate both magnitude and rate of change. They can provide latitude and longitude in a continuous fashion along the entire flight route, thereby allowing accurate location of all pertinent features along the river or stream. This technology allows investigators to acquire a broad perspective of the entire watershed and to determine the exact location of significant geomorphic and engineering features, which can be entered into the GIS database. Using aerial reconnaissance can significantly reduce field efforts.

(3) Field Investigations. Field reconnaissance is used to gather data and make observations leading to understanding the active processes and conditions of the stream. Experienced personnel from hydraulics, geotechnical, and environmental disciplines conduct the field inspection. Field reconnaissance is used to describe the geomorphological landforms of study reaches, and to identify potentially destabilizing phenomena based on reach-scale evidence of erosion, sediment storage, and deposition. Appendix D contains basic information on conducting field investigations to collect data for channel stability assessment. Biedenharn et al. (1997) contains a detailed discussion on field equipment and a description of features to look for in the field. Thorne (1993) provides guidelines for use of stream reconnaissance sheets in the field.

(a) Collection of field data can be aided by using appropriate field assessment data sheets. Appendix D and Copeland et al. (2001) provide example data sheets. These sheets are comprehensive and should be adapted to specific study needs. The level of effort required to conduct a field reconnaissance varies depending on conditions. Establish a consistent technique that is tailored to the watershed conditions and the study goals. Prior to initiating a large watershed-level effort, conduct a trial run to assess methodology, time requirements, and applicability to study goals.

(b) Field assessments are best made during low-water conditions and during the dormant season when the banks can be readily examined. However, it is important to recognize that conditions may be different at high flows. For safety and logistical reasons, field work is best accomplished by teams of at least two people. Field work (particularly in urban areas) may raise significant health and safety issues. Potential hazards include crime, needles, and exposure to raw sewage and waterborne pathogens such as hepatitis.

(c) Inspections at bridge crossings should be treated with caution since bridges are frequently placed at constrictions and/or at bedrock outcrops. Bridge locations may not be characteristic of the stream as a whole. However, valuable indicators of stream stability can be observed at bridges and other points where infrastructure crosses the stream. In assessing streams in the field, it is important to keep in mind that a channel typically has six degrees of freedom (the dependent variables): width, depth, slope, roughness, planform, and bankline migration.

(d) During the field reconnaissance, the following basic information (at least) should be collected:

- Descriptions of watershed development and land use, floodplain characteristics, channel planform, and stream gradient.

- Assessment of historical conditions, which can be obtained via interviews with knowledgeable landowners.

- Measurements of low-flow and bankfull channel dimensions and channel slope in critical reaches; identification of terraces and active floodplains.

- Characterization of the channel bed. Determine if it is bedrock, erodible cohesive material, armored, or alluvial. Determine the gradation of any armor layer (see paragraph 8-6c) and collect bed-material samples of the substrate layer. Appendix D of this manual gives guidelines for collection of bed material samples.

- Descriptions of river bank profiles and bank materials; evidence of bank instability.

- Descriptions of point bars, pools, riffles, and bed instability; evidence of sedimentation processes.

- Observations of impacts due to channel alterations and evidence of stream recovery.

- Descriptions of channel debris and bed and bank vegetation.

- Preliminary project alternatives should be identified so that information can be gathered on possible constraints such as access, utilities, and staging areas.

- Photographic records of critical stream and watershed characteristics.

(e) There are many possible indicators of stream stability. Table 7-1 lists a range of field indicators in a watershed. It is important to recognize that these are not absolutes, and that all items listed as possible indicators of instability may occur in natural and/or stable streams and vice versa.

Table 7-1
Possible Field Indicators of River Stability/Instability

Evidence Type	Indicators
Evidence of Degradation	• Terraces (abandoned floodplains). • Lack of floodplain connection. • Perched channels or tributaries. • Headcuts and knickpoints. • Exposed pipe crossings. • Suspended culvert outfalls and ditches. • Undercut bridge piers. • Exposed or air tree roots. • Leaning trees. • Narrow/deep channel. • Banks undercut, both sides. • Armored bed. • Hydrophytic vegetation located high on bank. • Width/depth ratios < 10. • Low channel sinuosity values.
Evidence of Aggradation	• Buried structures such as culverts and outfalls. • Reduced bridge clearance. • Presence of mid-channel bars. • Outlet of tributaries buried in sediment. • Sediment deposition in floodplain. • Buried vegetation. • Perched main channel. • Significant backwater in tributaries. • Uniform sediment deposition across the channel. • Hydrophobic vegetation located low on bank or dead in floodplain. • High channel sinuosity values. • Overwidened channel sections. • High width/depth ratios.
Evidence of Stability	• Access to active floodplain. • Vegetated bars and banks. • Limited bank erosion. • Older bridges, culverts, and outfalls with bottom elevations at or near grade. • Mouth of tributaries at or near existing main stem stream grade. • No exposed pipeline crossings.

(f) It is important to recognize the possible pitfalls of field assessments. These include observer bias, temporal limitations, and spatial limitations. Issues related to observer bias can be partially overcome with consistent use of experienced personnel to minimize relative differences between observations. Temporal bias can be minimized by examination of historical records, but these may be incomplete. Having the field team walk a continuous reach of stream can reduce spatial bias. Field investigation should extend both upstream and downstream of the project reach, and ideally, should be conducted at several different periods of the year.

(g) Landform Characteristics Associated with Bank Erosion. Lateral movement of the channel is a natural process and may occur even in a stable river. During the field investigation, careful inspections of bank erosion sites are required to determine if the bank erosion is part of natural lateral migration, or if it is an indicator of system-wide instability. Some of the features to look for at bank erosion sites are:

- Channel Bends. Inspect the point bar for sediment deposition that may be pushing the channel flow toward the outside of the bend. Normal channel meandering is expected to move the channel in the downstream direction. A resistant bank material or bank protection will interrupt that process.

- Gravel Bars – General Movement. In gravel-bed streams, it is common to view a train of gravel bars moving down the channel. The front of each bar is at an angle with the center line of flow, and that angle swings back and forth from one bar to the next. These bars are probably set into motion by the higher flows, but when the flow is relatively low, the front of the bar directs current into the bankline. Because the successive bars are angled toward alternate banks, the flow attacks first one bank then the other. The attack moves along the bank as the bars move down the channel.

- Gravel Bars – Transport. Gravel transport is influenced significantly by momentum forces so that gravel tends to maintain a straight travel path rather than following a meandering thalweg. This gravel transport characteristic influences the formation and growth of both middle bars and point bars.

- Gravel Bars – Stability. Recent flow conditions affect bar appearance during the field investigation. Inconsistent bar geometry, movement, and material size can be indicators of channel instability. However, gravel bar movement is often associated with bank erosion. Therefore, a field assessment of overall bank instability due to gravel bar movement is difficult.

- Increase in Channel Width. When both banks show erosion with no accompanying degradation so that there is a net increase in channel width, the channel is probably unstable. Possible causes are an increase in mean annual water discharge or an increase in slope. The channel is adjusting to a new flow regime. Such bank erosion is being produced by a completely different mechanism from point bar formation, gravel bar movement, or structural bank failure.

- Seepage and Tension Cracks. Inspect the bankline for seepage, clay lenses, slope failure lines, and tension cracks. Tension cracks suggest that the bank height is too great for the soil to be stable on the current bank slope.

- Dispersive Clays. Dispersive clay lacks the cohesive attraction common to most clays. Their allowable or permissible velocity (see paragraph 7-9c) is considerably below the range normally quoted for clay material. When making the field inspection, suspect such a clay where rills are cut deeply into a bank of clay material or into mounds of clay that have been excavated from a channel. Therefore, the presence of clay banks does not guarantee that bank material can resist high shear stresses or velocities.

- Farming or Maintenance Practices. Farming or maintenance practices that clear off native vegetation from the near channel bank vicinity will accelerate bank caving unless over-the-bank drainage is collected and controlled.

- Access/Egress Points. Cattle or vehicle access to the channel weakens the soil structure and removes native vegetation. Bank erosion often results. The problem typically migrates both upstream and downstream from the initial point of disturbance.

- Tributary Deposition. Delta deposits from tributary inflow can deflect flow to the opposite bank of the receiving stream. This deflection can be intermittent if high flows on the receiving stream wash away the delta deposit, and the deposit may not exist during the field reconnaissance.

(4) Identification of Geomorphologically Similar Reaches. The information gathered in the data assembly and field investigation should be used to divide the channel into geomorphologically similar reaches. When establishing reach limits, consideration should be given to: differences in channel slope, tributary locations, presence of geologic controls, planform changes, location of channel control structures (grade control structures, dams, culverts, etc.), changes in bed material size, major sediment sources (gravel mines, sediment laden tributaries, etc.), changes in channel evolution type, or other significant hydrologic or geomorphic changes. Initial reach limits may be made early during the field investigation that are refined following more detailed analyses.

(5) Assessment of Reach Condition. At the conclusion of a field investigation, a summary of channel stability in each reach is assessed. This summary may include the use of a general assessment and scoring techniques related to the existing condition of individual reaches. The many techniques available range in complexity and required effort. The choice of an assessment technique should be made with consideration of the study goals after the field investigations have been performed. Table 7-2 lists examples from a general reach condition assessment.

Table 7-2
Reach Condition Assessment

Condition	Bed	Bank
Stable	The channel bed is as close to a stable condition as can be expected in a natural stream. If the reach exhibits signs of local bed scour or deposition with a low rate of change, it falls into this category.	The channel banks are as close to a stable condition as can be expected in a natural stream and appear to have a low potential to erode. Banks are predominantly covered with extensive vegetation, boulders, or bedrock formations. If the reach exhibits signs of local bank erosion within an allowable rate of change, it falls in this category.
Moderately Stable	The channel bed in the reach is in a moderately stable condition. However, the reach may be in transition. Reaches where the bed is experiencing bed aggradation or degradation at a low rate of change fall into this category. In addition, moderate to high local bed scour or deposition fall into this category. For example, rapid aggradation immediately above and scour immediately below a minor debris blockage (such as a single tree blocking the channel).	The channel banks in the reach are in a moderately stable condition and exhibit medium erodibility. Banks are partially vegetated with moderately erodible soils. Typically, parallel flows would not result in bank erosion. The reach may be in transition. Reaches with low rates, but widespread bank erosion falls into this category. In addition, banks that exhibit moderate local bank erosion which does not appear to be spreading fall into this category. For example, in an otherwise stable reach, a single section of the bank could fall into the stream and result in local, moderate bank erosion.
Unstable	The channel bed in the reach is in an unstable condition. Reaches where the bed is undergoing widespread bed aggradation or degradation at a moderate rate fall into this category. Moderately scoured reaches or reaches where many of the pools are filled with loose sediment fall into this category.	The channel banks in the reach are predominantly unstable. Reaches where the banks are experiencing widespread erosion at a moderate rate fall into this category. Reaches where the channel banks are undergoing local bank erosion at a high rate of change and where the erosion is not likely to self-heal also fall into this category.

Condition	Bed	Bank
Very Unstable	The channel bed in the reach is in a very unstable condition. Typically, the channel shows no signs of approaching equilibrium with the current shape and planform. Reaches where the bed is undergoing widespread aggradation or degradation at a high rate fall into this category. Severely scoured reaches fall into this category. Reaches where all of the pools are filled with loose sediment also fall into this category.	The channel banks in the reach exhibit high erodibility and do not have any controls that restrict extensive changes in planform or shape. Typically, a significant riparian root mass that would slow rapid bank retreat is not present in the banks. Any parallel or impinging flows would cause extensive bank erosion. Reaches with near vertical to overhanging.

(6) Channel Typing, Classification, and Evolution. Channel typing or classification is a useful, though not essential, step in channel assessment. A channel can be described in detail without selecting a classification system and assigning the stream reach to a certain class. Typing or classification is useful if one is developing or using hydraulic geometry relations with separate regression equations for different types of streams. Such relationships should result in regression equations with better accuracy and less uncertainty.

(a) Determining a channel type relies on developing a channel description based primarily on observation. The channel description includes parameters such as channel and floodplain geometry, bed and bank material, planform, vegetation, bed forms, evidence of aggradation or degradation, grade control, alluvial or threshold conditions, etc. Channel typing is an elementary level of stream classification that uses generic terms. For instance, a stream may be typed as a meandering sand-bed channel.

(b) Channel classification involves the selection of a classification system, normally developed by a specific person (Brice 1984 or Schumm 1977), and the categorization of a channel into a specific class based on factors and measurements such as planform and planform features, dominant mode of sediment transport, entrenchment ratio, sinuosity, etc. There are numerous stream classification systems. Some of the most widely used are described in Montgomery and Buffington 1993; EM 1110-2-1418; Rosgen 1996; FISRWG 1998; and NRCS 2007. Some limitations of stream classification systems are:

- The classification is a snapshot of the existing condition of the stream and does not give any information about trends, such as whether a stream is stable, aggrading, degrading, or approaching a critical geomorphic threshold.

- Water quality or the biological health of a stream cannot be determined from a geomorphic classification system.

- The classification is a generalization of stream behavior, which the individual stream may conform to well or poorly.

(c) Channel evolution models differ from classification systems in that they are used to predict sequential stages in channel response. For example, the incised-channel evolution model developed by Schumm et al. (1984) predicts the sequence of changes that will occur in a channel as a headcut moves upstream. The model stages are shown in EM 1110-2-1418, Figure 2-23. Simon and Hupp (1986) and Simon (1989) have developed a similar model. Channel evolution models can be used to predict trends (aggradation, degradation, and channel widening) at a project site, and to prioritize restoration work along a stream channel.

(7) Methods for Assessing Historical Channel Stability. The analysis of historical data from stream gages, surveys, and mapping can give useful information about channel stability, any aggradation/degradation trends, rates of lateral movement, and planform changes.

(a) The review of aerial photographs taken at different time periods is a useful starting point. These are normally available for any site, even when gage data or historic surveys are absent.

(b) Using historic data requires a thorough understanding of system processes when comparing surveys performed several years apart or gage data with gaps in the record. For example, the existing thalweg elevation lower than the historic normally indicates that degradation is the dominant process, but it does not always indicate that the stream is currently degrading. The stream may have degraded to a point below the existing streambed, then reversed its trend of instability, and then aggraded so that the existing dominant process is aggradation (Schumm et al., 1984). Urban streams in the mid-Atlantic piedmont is incision that has been arrested by bedrock or other grade controls, resulting in a “degraded, now stable” stability classification.

(c) Results of the historical data analysis should be compared to both the results of the field investigation and the analytical stability assessment before reaching final conclusions. A specific gage analysis is an excellent method to examine historic channel stability. Additional information regarding specific gage analysis is provided in paragraph 7-7.

(8) Comparative Surveys and Mapping. One of the best methods for directly assessing channel changes is to compare both channel thalweg and cross sections. Thalweg surveys are taken along the channel at the lowest point in the cross section. Comparison of several thalweg surveys taken at different points in time allows the assessor to chart the bed elevation change through time and track the migration of headcuts or aggradation zones through the system. Cross-section surveys provide information about channel widening or narrowing. Haring et al. (2020) and Haring and Biedenharn (2021) provide a GIS-based toolbox (FluvialGeomorph) that analyzes LiDAR water surface profiles and cross sections from a temporal and spatial perspective.

(a) There are certain limitations that should be considered when comparing surveys on a river system. When comparing thalweg profiles, it is often difficult (especially on larger streams) to determine any distinct trends of aggradation or degradation if there are deep scour holes, particularly in bendways. The existence of very deep local scour holes may completely obscure temporal variations in the thalweg. This problem can sometimes be overcome by eliminating the pool sections and focusing only on the crossing locations, thereby allowing aggradation or degradation trends to be more easily observed. Reliable survey comparisons can be made only if the surveys are homologous in rivers and streams that have significant bed form movement and/or seasonal variations in sediment transport.

(b) While thalweg profiles are a useful tool, recognize that they reflect only the behavior of the channel bed and do not provide information about the channel as a whole. For this reason, it is usually advisable to also study changes in the cross-sectional geometry. Cross-sectional geometry refers to width, depth, area, wetted perimeter, hydraulic radius, and channel conveyance at a specific cross section.

(c) Cross-section surveys at permanent monumented range locations can be compared directly for different time periods. When available cross sections are not located by permanent monuments, it is often advisable to compare reach average values of the geometric parameters. This requires the study area to be divided into distinct reaches based on geomorphological characteristics. Individual cross-sectional parameters are calculated and averaged for the entire reach. The reach average values can be compared for each survey period. Cross-sectional variability between bends (pools) and crossing (riffles) can obscure temporal trends. Using channel crossing sections may be preferred to analyze long-term trends of channel change.

(d) Comparison of time sequential maps or aerial photographs can provide insight into planform evolution and the change or instability of the channel. Rates and magnitude of channel migration, locations of natural and manmade cutoffs, and spatial and temporal changes in channel width and planform geometry can be determined from analysis of historical information. With this type of data, channel response to imposed conditions can be documented and used to substantiate predictions of future channel response to a proposed alteration. Contemporary planform data can be obtained from aerial photos, maps, or from field investigations.

(9) Applicability of Geomorphic Assessments.

(a) Geomorphic studies provide insight into identifying dominant processes in a river system and can demonstrate trends. However, because geomorphic studies typically ignore water and sediment continuity with respect to both time and distance, predictions of future trends or direct identifications of cause and effect have significant uncertainty. Evidence obtained from gages and/or surveys that represent a single point in time and space is valuable, but conclusions must be tempered by what the evidence supports and not from a preconceived hypothesis. Geomorphic studies alone are typically insufficient for designing USACE FRM and environmental restoration projects.

(b) An example sediment impact assessment related to stable channel design is presented in Case Study 7A (Appendix N). Appendix C of this manual contains an example scope of work for a geomorphic assessment.

7-5. Channel Processes and Analysis Methods. As previously described in the fluvial geomorphology section, rivers continually mold and remold their streambeds by eroding and depositing sediments. Prediction of channel scour and aggradation are critical components of USACE project design, maintenance, and operations.

a. Channel Scour. Scour is perhaps the primary cause of riverine hydraulic structure failure. Analysis of potential scour is required for all types of USACE streambank protection and stabilization projects. For additional scour depth computational guidance, refer to the technical report Approaches for Assessing River Scour (Howard et al., 2021); National Engineering Handbook, Technical Supplement 14B (NRCS 2007); the Maricopa County Drainage Design Manual (Maricopa 2018); and Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges (Federal Highway Administration (FHWA 2012)).

(1) Long-Term Scour (Degradation). Long-term scour or degradation is the term describing a general lowering of the stream bed elevations due to erosion of the bed sediments. Long-term scour occurs over long reaches and may accumulate over time. Assessing future long-term scour is a critical component for USACE project design. Analytical tools such as HEC-RAS or HEC-6 are employed.

(a) Degradation factors include a reduction in sediment supply (upstream dams), an increase in water supply (watershed land use change), and a geometry change (channelization).

(b) The bed-material transport potential of a stream is determined by the magnitude of hydraulic shear and turbulence. When this potential is greater than the bed-material sediment supplied to a stream reach, hydraulic forces tend to meet the deficit by eroding sediment from the stream bed. Although the rate may be slow, degradation effects over the project life can be significant. For example, the sediment supply reduction caused by large storage reservoirs often result in downstream degradation. A tributary inflow sediment deficient can also induce degradation in the receiving river that can move both upstream and downstream. An example is the effluent from wastewater treatment plants in arid and semi-arid environments.

(c) Base-level lowering induced by channelization or cutoffs can cause degradation in both sand-bed and cohesive bed streams. Base-level lowering is an important consideration because the resultant degradation promotes bank caving, causes bridge failures as well as failure of other structures in its path, and increases the sediment discharge into the receiving stream.

- In sand-bed streams, the increased hydraulic shear stresses result in increased bed-material transport potential and bed scour to meet this deficit. The induced degradation moves upstream and frequently results in aggradation downstream.

- In cohesive-bed streams a similar process occurs, but the response is typically slower. Often a headcut or knickzone can be identified in a degrading cohesive-bed stream. A headcut is a rapid drop in the stream bed profile that moves in the upstream direction. A knickzone is a relatively steep section of the stream profile held in place by resistant bed deposits.

(2) General Scour. General scour represents the degradation of the channel bed during the occurrence of a design storm event. This scour generally represents an erosion impact to most of a channel cross section and may include impacts to both the channel bed and banks. The erosion is induced by applied shear stress and velocity above critical values for the surface material, primarily caused by the magnitude of the design storm event and the local area geometry (Maricopa 2018).

(3) Bend Scour. Channel bends generally have increased scour potential compared to straight reaches. Flow concentration on the outside of channel bends can cause bank erosion, and eventually, lateral migration of meandering streams. The complex flow patterns in bends have curvature-induced secondary flows as a balance is sought between the centrifugal force and a pressure gradient created by surface tilting (Bai et al., 2019). The secondary currents (velocity components not in the streamwise direction) scour material from the outside of the bend and deposit along the inside of the bend. The most pronounced scour location is usually on the outside bank of the downstream end of the bend (Howard et al., 2021).

(4) Contraction Scour. The scour that results from the acceleration of the flow due to a contraction, such as a bridge, is called contraction scour. This type of scour also occurs in areas where revetments are placed such that they reduce the overall width of the stream segment. Contraction scour is generally limited to the length of the contraction, and perhaps a short distance upstream and downstream, whereas general scour tends to occur over longer reaches. Contraction scour and local scour are combined in many references and empirical analysis methods. Contraction scour analysis at bridge structures may be evaluated with numerical models such as HEC-RAS or following analysis criteria presented by FHWA 2012 in Hydraulic Engineering Circular 18.

(5) Bed Form Scour. As discussed in paragraph 7-3e, bed forms are sedimentary features found in the stream bed that transition according to flow regime with lower regime bed forms such as ripples and dunes, followed by a transition to upper regime bed forms such as plane bed and antidunes. Dunes and antidunes in sand-bed rivers can result in additional scour as they migrate. The passage of a large dune may increase local scour depths as much as 30% (NRCS 2007). Bed form scour depth maybe computed with a bed form predictor equation or from field data and determined as one-half the amplitude (measured from dune top to trough) (NRCS 2007; Maricopa 2018). Bed form scour is usually much smaller in magnitude than other scour types in sand-bed rivers.

(6) Local Scour. Local scour is the term applied when erosion of the channel bed is limited to a particular location. It can occur in otherwise stable reaches of a stream as the direct result of a localized disturbance to the flow field. The maximum expected scour depth is difficult to measure since the most severe scour often occurs during the peak flow and deposition fills in the scour hole as the hydrograph recedes. Local scour should be regarded as a potentially severe problem in any mobile bed stream.

(a) Bridges. Because of their placement in the primary conveyance, bridge piers and abutments are the most frequent location of local scour problems. The process is usually very rapid. Scour gages are used to monitor scour at bridges, and consist of drilled holes in the stream bed that are filled with colored sand, brick chips, or chains.

(b) Drop Structures. Local scour shows up as a deep hole downstream from the structure, flanked by bank caving. Standard drop structure designs require bed and bank armoring to control this type of scour.

(c) Low Weirs. Local scour erodes the bank at the abutments of low weirs, causing structures to be flanked. Design recommendations include creating preferred flow paths to prevent short-circuiting flow, allowing for variable energy dissipation at initial overtopping, and assuring structure submergence for high flows.

(d) Miscellaneous. Local scour also occurs at the downstream junction between riprap or other revetment and the natural earth channel, especially when the bank protection is smoother than the adjoining bed and/or bank. Any protrusion into the channel will cause local acceleration of the flow and the potential for local scour. These include navigation and FRM structures such as training dikes; stream restoration structures such as boulders, root wads, and diversion weirs; and local infrastructures such as pipelines.

(7) Low-Flow Incisement. Rivers normally form a low-flow channel due to natural processes. The magnitude of the incisement is usually a best estimate derived by a field assessment and evaluation of detailed survey data. In cases where the low-flow channel is stable and the total scour is measured from the thalweg, this scour component may be ignored for design purposes. In most cases, the magnitude will be much smaller than other factors (Maricopa 2018).

(8) Scour Processes to Consider. Scour occurs due to several related processes, and estimated maximum scour is typically computed by summing the scour due to each individual process. The NRCS 2007 and FHWA 2012 relates the following qualitative principles that are useful in understanding scour processes and computations. Many of the scour computations are based on work originally provided by Laursen (1952), Vanoni (1975, 2006), and Lara and Pemberton (1984).

(a) The rate of scour is equal to the difference between the capacity for transport out of the scoured area and the rate of transport into the scoured area.

(b) Scour rates decline as scour progresses and enlarges the flow area.

(c) Scour asymptotically approaches a limiting extent (volume or depth) for a given set of initial conditions.

(d) Clear-Water and Live-Bed. FHWA 2012 and NRCS (2007) relates that there are two conditions for contraction and local scour: clear-water and live-bed scour.

- Clear-water scour occurs when there is no movement of the bed or the bed is transported in suspension through the scour zone. Clear-water scour is typically associated with coarse beds, flat gradient streams at low flow, local bed material deposits of bed material are larger than the greatest size in transport, armored beds, and channels or floodplains where transport is prevented by vegetation.

- Live-bed scour occurs when there is transport of bed material. Live-bed scour is regarded as cyclic with an expanding scour zone that develops during the rising stage of an event and refills during the falling stage.

- Scour in alluvial channel is usually live-bed. Most empirical equations developed for alluvial channels are applicable for live-bed conditions.

(e) Planform Effects. Channel planform has an influence on scour processes. Bend scour depths are directly impacted by planform. Lateral movement of the stream, thalweg movement, shifting of the flow direction in the area of concern, and other factors can affect the total scour depth. Planform changes are difficult to anticipate and may occur gradually or during a single major flood event. On the rising side of the event, the bed scours in meander bends and builds up in the shallower crossing between the bends. On the receding side of the event, the process is reversed. Braided channels often experience the greatest scour during intermediate flows, when flows attack banklines at impinging angles that may be nearly direct (NRCS 2007).

(f) Bank Structure Influence. In meandering channels, the thalweg in bends often moves toward the outer (concave) bank following placement of revetment or other types of direct erosion protection. The amount of additional bend scour is related to the relative erodibility of the bed and banks. Channels with highly erodible bed and banks can experience additional scour at the toe of a newly placed revetment.

(g) Armoring. Streambeds often feature a layer of coarse particles at the surface that overlies a heterogeneous mixture containing a wide range of sediment sizes. This surface armor layer is usually only one or two particle diameters thick. When a streambed contains at least some sediment that is too large to be transported by the imposed hydraulic conditions, finer particles are selectively removed. The layer of coarser materials left behind forms an armor layer that limits further scour unless higher levels of shear stress destroy the armor layer. Pemberton and Lara (1984) and NRCS (2007) present methods for computing scour depth until a limiting armor layer is formed. Paragraph 8-6c also describes armoring, and Appendix D provides guidelines for collection of bed material samples.

b. Scour Analysis Components. Scour analysis used for design of project features generally considers individual factors to develop an estimate of the total scour depth. NRCS (2007), Maricopa County (2018), and Howard et al. (2021) present a method to determine the total scour depth as the sum of all components of vertical bed change. The equation below computes total scour depth and is also summarized in Table 7-3.

$$z_t = \mathrm{FS}\left[z_{ad} + z_g + z_c + z_b + z_{bf} + z_{lf}\right] + FS_{local} \times z_s \qquad \text{Equation 7-2}$$

where:

z_t = total scour depth (ft)
FS = factor of safety
z_{ad} = long-term bed elevation changes (aggradation or degradation) (ft)
z_g = general scour (ft)
z_c = contraction scour (ft)
z_b = bend scour (ft)
z_{bf} = bed form trough depth (ft)
z_{lf} = low-flow incisement (ft)
FS_{local} = factory of safety for local scour component
z_s = local scour depth associated with a structure (ft)

Table 7-3
Scour Analysis Type by Process

Type of scour or process	Symbol	Type of Analysis
Total scour	z_t	Sum of all separate scour components. Note that many empirical equations may provide more than one component.
Factor of safety	FS	Factor of safety adopted for specific project based on analysis of risk, consequences, and mitigation feature performance risk (typical range 1.0 to 1.5); not appropriate to apply if previously included in the individual scour component estimates.
Long-term bed change	z_{ad}	Sediment continuity analysis using HEC-RAS or similar, may use simplified method such as equilibrium slope or armoring analysis if appropriate for project objectives.
General scour	z_g	Empirical or regime equations (add appropriate additional factors such as long-term degradation and structure scour when not included in basic equation) (Pemberton and Lara 1984; NRCS 2007; FHWA 2012; Baird 2019; Howard et al., 2021).
Contraction scour	z_c	Live-bed or clear-water contraction scour equations (NRCS 2007; FHWA 2012); HEC-RAS has incorporated standard methods for numerical analysis.
Bend scour	z_b	Empirical equations for bend scour, most include all scour except long-term bed change (Pemberton and Lara 1984; NRCS 2007; FHWA 2012; Baird 2019; Howard et al., 2021).
Bed form scour	z_{bf}	Occurs with dunes or antidunes. Estimate with field data or using bed form predictor equation (NRCS 2007; FHWA 2012; Maricopa 2018).
Low-flow incisement	z_{lf}	Low-flow incisement is estimated with field observations and survey data. For planning and design purposes, this may be estimated as no less than 1 foot and possibly in excess of 2 feet (Maricopa 2018).

Type of scour or process	Symbol	Type of Analysis
Factor of Safety, local	FS_{local}	Factor of safety for local scour from structures such as bridge piers, grade control, and similar. Typical range of 1 to 1.5, adjust to reflect uncertainty in the components of the local scour computation.
Local scour	z_s	Empirical equations for type of structure (NRCS 2007; FHWA 2012).

(1) Methods. Reasonably accurate prediction of flood scour potential is often necessary for USACE project design. However, calibration data that includes extreme events or that would account for USACE project impacts is seldom available. FHWA 2012 relates:

(a) Experience is usually the most reliable means of estimating scour depth. Lacking experience on a particular stream, scour depths may be estimated using physically based analytical models or empirical methods.

(b) Although scour depth can be estimated analytically or empirically, empirical methods were generally found to provide better agreement with observed data.

(c) In general, the recommended scour protection depth is the total scour depth below the channel thalweg for protection areas inside the main channel banks.

(2) Simple empirical equations can bound the potential scour risk in these situations. These equations are not universally applicable and often generate a wide variety of scour depths. Applying applicable equations in an ensemble approach can provide a possible range of scour depths. Many of these methods were collected in the U.S. Department of the Interior, Bureau of Reclamation (USBR) design manual (Pemberton and Lara 1984). Despite their age and limitations, they remain widely used. Recent guidance documents include NRCS (2007), FHWA (2012), and Baird et al. (2019). Howard et al. (2021) presents a review of equation applicability and development that is recommended to assist with method selection.

(3) Selecting Hydraulic Parameters from a 1D Model. The scour equations are all empirical regressions based on cross-section averaged hydraulic parameters from flume and prototype data. Therefore, while multidimensional hydraulic and morphodynamic models can be very useful to identify scour hot spots and evaluate shear stress distributions in critical scour reaches, multidimensional hydraulics (Q, W, D, V) should not be used in these equations. However, site-specific decisions are necessary to select results from 1D cross-sectional hydraulic computations that has two different sets of average hydraulic parameters. Considerations are as follows:

(a) Cross section: averages both the channel and the floodplain.

(b) Channel: average includes only the channel hydraulics.

(c) Using a section with a high-flow percentage within the floodplain can artificially reduce the average velocity and unit flow width in the likely scour zone.

(d) Channel average hydraulics (flow, velocity, top width, depth, hydraulic depth) is almost always more appropriate.

(4) Using an Ensemble of Scour Equation Results for Design. The scour equations usually compute a wide range of scour depths, making it difficult to choose one value. The range of scour depth results demonstrates the uncertainty in these equations. While the maximum value might be conservative, it may not be realistic. Simply averaging the values ignores the uncertainty in the result. These equations are best used in the following decision approach:

(a) Rule out equations that are not applicable to the setting or that generate unrealistic results (such as negative scour).

(b) Report the median value and the range. Look for clusters of results (several equations with similar scour depths) but recognize that some equations artificially generate similar results because they have similar forms or structure.

(c) In addition to using an equation ensemble, sensitivity analysis is also recommended to evaluate the impact of user-selected variables in the hydraulic model.

(d) Use these results with other qualitative assessments, local system expertise, and subject matter experts to agree on a design depth that reflects the likely scour depth, the risk tolerance, and the desired factor of safety. USACE (1994b, Plate B-41) presents results from multiple studies with a design curve for maximum scour depth location in both sand-bed and gravel-bed channels. NRCS (2007) presents the mean and maximum scour depth observations for typical alluvial rivers that was originally developed by Blodgett (1986).

c. Computing General Scour. The general scour equations were compiled by Neill (1973) to compute scour at constricted waterways. Pemberton and Lara (1984) present them as a method to "design buried pipe, buried canal siphon, or a bankline structure." These equations estimate scour in different river settings, including bends or straighter reaches, and are generally based on the hydraulic parameters at the design location (they do not refer to an upstream reference cross-section). General scour equations are presented in Table 7-4 (compiled from Howard et al., 2021).

(1) Most of the general scour equations (Neill Incised, Blench, Lacey, and USBR average velocity) have a similar form and use an empirical factor (Z) to account for sinuosity. Z is a categorical reduction factor with three options for straight, moderate, and severe bends. The value of Z varies for the different equations based on the bend category collected. But the Z coefficient ranges from 0.25 to 0.75 for most natural river settings, reducing scour more (lower Z) for straighter reaches.

(2) Assigning a quantitative reduction factor to qualitative bend categories introduces uncertainty (and nonlinearity/step functions) into the analysis. USACE provides rule-of-thumb approaches to calculating these categories based on the ratio of the radius of curvature (*Rc*) and the bankfull channel width (W) summarized in Table 7-5.

Table 7-4
General Scour Equations

Method	Equation	Parameters	Assumptions and Conditions
Neil Incised◊	$\Delta y = Z \cdot \overline{D}_{bf}\left(\frac{Q_d/W}{Q_{bf}/W_{bf}}\right)^m$	$Z = \begin{cases} 0.5\ if\ Straight \\ 0.6\ if\ Moderate \\ 0.7\ if\ Severe\ ■ \end{cases}$ $m = \begin{cases} 0.67\ for\ sand\ * \\ 0.85\ for\ gravel \end{cases}$	See table below for quantitative metrics for Z categories. Valid for reaches with channel constrictions.
Lacey◊	$\Delta y = Z \cdot 0.47\left(\frac{Q_d}{1.76\sqrt{d_m}}\right)^{\frac{1}{3}}$	$Z = \begin{cases} 0.25\ if\ Straight \\ 0.5\ if\ Moderate \\ 0.75\ if\ Severe\ ■ \end{cases}$	Silt bed rivers. Zero bedload conditions.
USBR Mean Velocity (Pemberton and Lara 1984)	$\Delta y = Z\overline{D}$	$Z = \begin{cases} 0.25\ if\ Straight \\ 0.5\ if\ Moderate \\ 0.75\ if\ Severe \end{cases}$	–
Blench◊	$\Delta y = Z \cdot \frac{\left(Q_d/W\right)^{\frac{2}{3}}}{(F_{B0})^{\frac{1}{3}}}$	$Z = 0.6$ $F_{B0} = f(d_m)$ $F_{B0}\ from\ chart$	Clear water flow.
Zeller†	$\Delta y = \frac{0.0685 \cdot D_{Max}V^{0.8}}{D_{h\_}^{0.4}S^{0.3}} - 1$	–	This is Zeller's bend scour equation without the bend term. Sand-bed channels.

Method	Equation	Parameters	Assumptions and Conditions
USBR Envelope Curve℄	$\Delta y = \begin{cases} 2.47 + \dfrac{0.937\left({Q_d}/{W}\right)}{3.45} \\ K\left({Q_d}/{W}\right)^{0.24} \end{cases}$	$If\ \left({Q_d}/{W}\right) < 3.45$ $If\ \left({Q_d}/{W}\right) \geq 3.45$	$K = \begin{cases} 2.45\ US\ Cust \\ 1.32\ SI \end{cases}$ Valid for: Relatively steep slopes. $0.004 < S < 0.008$ MS-CS bedload. $0.5 < d_m < 0.7$ $\left({Q_d}/{W}\right) < 3.45$

■Both Neil and Lacey have higher values of Z (1 to 1.25) for right-angle bends or vertical rock banks.

◊Neil, Lacey, and Blench in this document are from Pemberton and Lara (1984), which have been modified from the original documents.

*This equation applies a smaller power for sand than gravel, which tends to predict more scour for gravel than sand. This emerged from the data in this study but may not be broadly applicable.

†Variables with a "US" subscript indicates hydraulic parameters at the upstream reference cross section. This is more common in the bend scour equations, so it is described in the next section. But the General Zeller equations is derived from his bend scour equation, so it uses this concept.

℄ USBR Envelope Curve from Pemberton and Lara (1984) and Baird (2019).

Where:

Δy = the scour depth below the initial channel invert
Q_d = the design flow
d_m = median grain size
D = depth. Depth takes several forms in these equations including:
D_h, the hydraulic depth, which is the area divided by the top width
$\overline{D}_{bf}$, the average bankfull cross-section depth, but is often approximated with D_h and
$D_{\_Max}$, the maximum cross-section depth
Z = an empirical parameter accounting for channel sinuosity based on a categorical classification (See chart)
W = the flow width of the design event
W_{bf} = the bankfull flow width
Q_{bf} = the bankfull flow
F_{B0} = Blench's Zero Bed Factor (Figure 7-13)

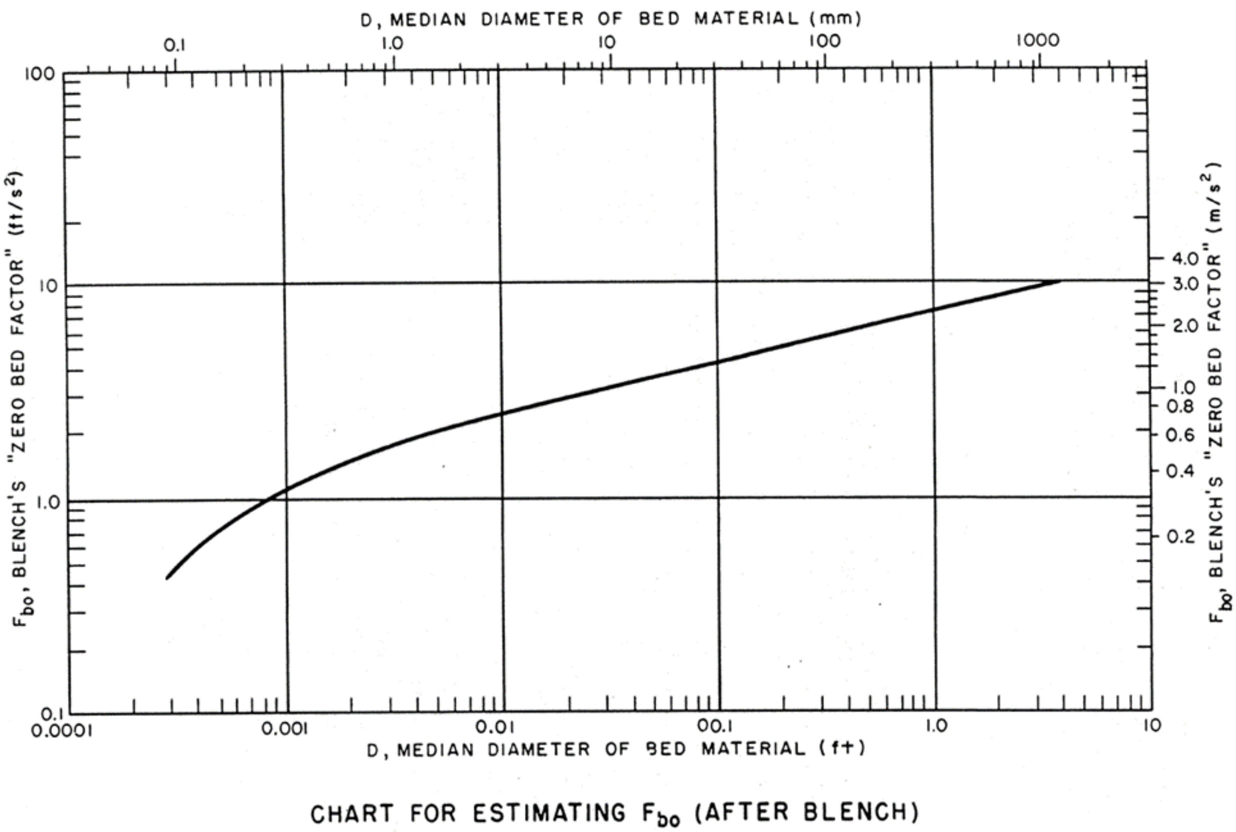


Figure 7-13. Chart to determine Blench's Zero Bed Factor as a function of d_m (from Pemberton and Lara (1984) after Blench (1969))

(3) Radius of curvature to width ratios for curvature categories used to determine the Z factor is presented in Table 7-5.

Table 7-5
Values to Determine the Z Factor

Channel Description (for Z parameter in Table 7-4)	**Rc/W Range** (ratio of radius of curvature to width)
Severe	$\frac{Rc}{W} < 3\ or\ 4$
Moderate	$3\ or\ 4 < \frac{Rc}{W} < 10$
Straight	$\frac{Rc}{W} > 10$

Note: Radius of curvature to width ratios for curvature categories used to determine the Z factor.

d. Computing Bend Scour. Empirical equations are available from multiple references to compute bend scour. Most of these equations were based on flows at or below bankfull. Maynord (1996), from Thorne and Abt (1993), notes empirical methods are valid up until significant interaction occurs between the main channel and overbank flow. Overbank depth should not exceed 20% of channel depth. Bend scour equations are presented in Table 7-6 (compiled from Howard et al., 2021).

Table 7-6
Bend Scour Equations

Method	Equation	Assumptions and Conditions
Maynord*	$\Delta y = \bar{D}_{US}\left(1.8 - 0.051\left(\frac{Rc}{W_{US}}\right) + 0.0084\left(\frac{W_{US}}{\bar{D}_{US}}\right)\right) - \bar{D}_{US}$	Sand-bed. $S \leq 2\%$, $1.5 < \frac{Rc}{W_{US}} < 10$** Recommends a safety factor of 1.0 to 1.19.
USACE EM 1110-2-1601	$\Delta y = \begin{cases} \bar{D}_{US}\left(-1.51\log_{10}\left(\frac{Rc}{W}\right) + 3.37\right) - D_{Max}\ for\ sand \\ \bar{D}_{US}\left(-1.62\log_{10}\left(\frac{Rc}{W}\right) + 3.375\right) - D_{Max}\ for\ gravel \end{cases}$	This method uses much of the same data as Thorne.
Zeller†	$\Delta y = \frac{0.0685 \cdot D_{US\_Max}V_{US}^{0.8}}{D_{h\_US}^{0.4}S_{US}^{0.3}} + \left(2.1\left(\frac{W}{4Rc}\right)^{0.2} - 1\right)$	Sand-bed channels.
Thorne	$\Delta y = \bar{D}_{US}\left(2.07 - 0.19\ln\left(\frac{Rc}{W_{US}} - 2\right)\right) - \bar{D}_{US}$	Includes data from large sand-bed rivers and smaller gravel-cobble systems. $\frac{Rc}{W_{US}} < 2$

Notes:

*For a factor of safety of 1. The method recommends a factor of safety between 1.0 and 1.19 based on the percentage of "significantly unconservative data."

**The algorithm has alternate forms for $\frac{Rc}{W_{US}} < 10$ and $\frac{Rc}{W_{US}} < 1.5$.

†This form replaces $\frac{\sin^2\frac{\alpha}{2}}{\cos\alpha}$ with the equivalent $\frac{W}{4Rc}$.

where:

Δy = the scour depth below the initial channel invert
W = the flow width (within the banks)
W_{US} = the flow width (within the banks) at the upstream, reference cross section
Rc = the radius of curvature
D = depth. Depth takes several forms in these equations including:
D_h, the hydraulic depth, which is the area divided by the top width
$\bar{D}_{US}$, the average cross section depth at the upstream, reference cross section, but is often approximated with D_h and
D_{Max}, the maximum cross-section depth before scour at the evaluation cross section

(1) Channel Curvature Defined for Bend Scour. The bend scour equations account for multidimensional forces on the outside of a bend by quantifying the bend severity based on the bend curvature and computing constriction. These equations avoid the complexities of multidimensional hydraulics at the bend by tying the equations to an upstream crossing or run, reference cross section as shown in Figure 7-14.

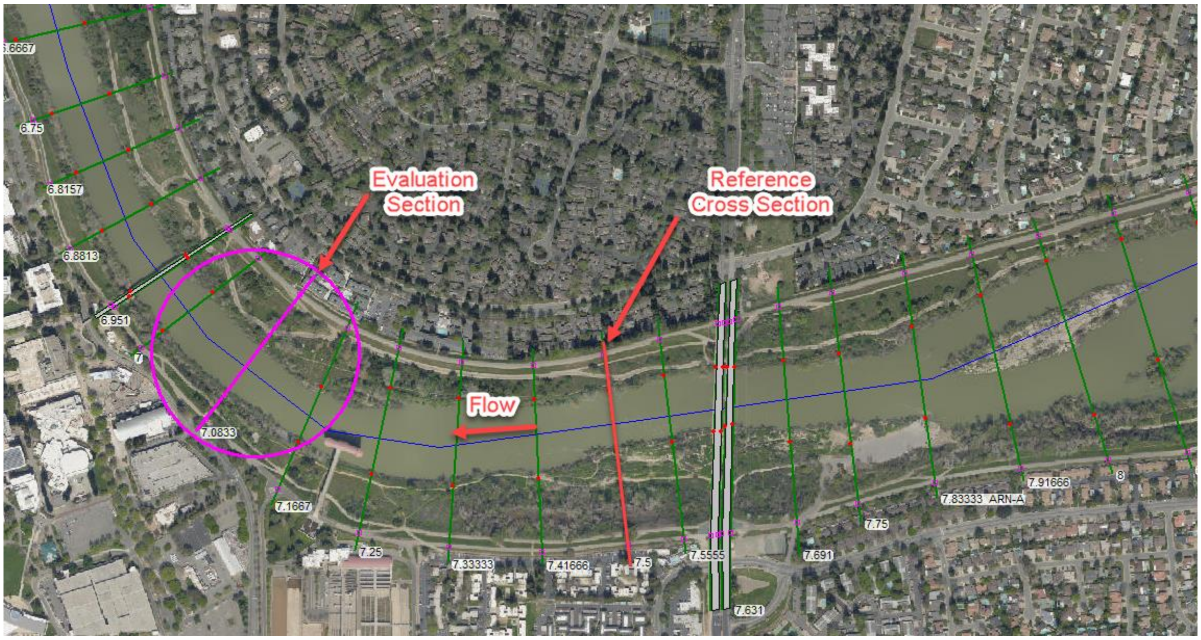


Figure 7-14. Bend scour equations evaluation (circled purple section) and reference section (red)

(2) Computing the Radius of Curvature. All of these equations require some measure of the bend curvature. In most cases, this is the radius of curvature (*Rc*). Most bend scour equations use this variable directly, while the general scour equations use *Rc* to determine the categorical curvature coefficient (Z from Table 7-5). The bend radius is typically calculated to the channel centerline of the bend. However, some equations require the use of centerline radius. Bend radius is computed using high-resolution aerial imagery in GIS software or Google Earth. To calculate the radius of curvature:

(a) Delineate the critical bankline, which is usually on the outside of the downstream third of the bend.

(b) Fit a circle to the critical section of the bend where the bend scour equation will be applied.

(c) Compute the radius of that circle.

(d) Figure 7-15 illustrates application. An evaluation cross section on the critical bend and the upstream reference cross section used for the bend scour analysis are also included for illustration.

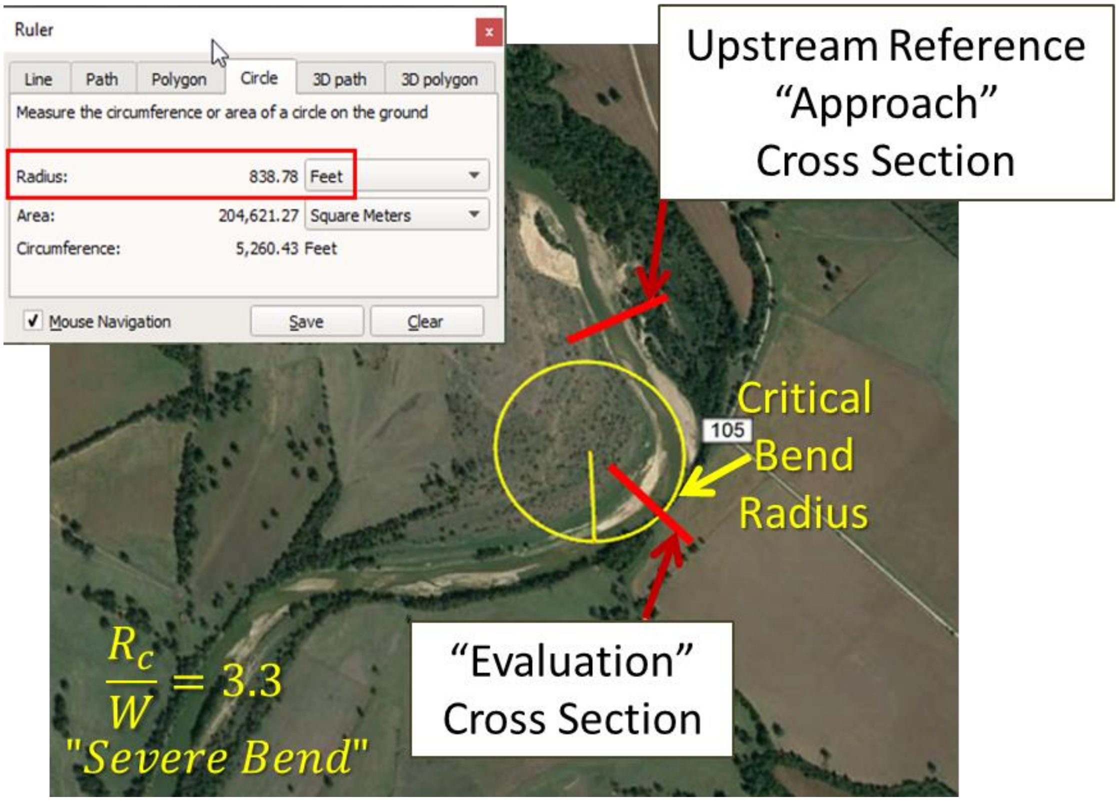


Figure 7-15. Radius of curvature calculation for a severe curvature bend on the Brazos River in South Texas

(3) Compound Bend Radius of Curvature. Compound bends do not have simple geometries but instead have curves on multiple scales. Reducing a bend to a single curvature estimate may not be sufficient at these locations. Scour computations are usually based on selecting the bend radius at the critical location on a compound bend to provide the critical (smallest) radius of curvature. Common practice computes scour at multiple locations in a compound bend. Additional detail on bankline and bend radius delineation is included by the National Cooperative Highway Research Program (2004) in Appendix B. Figure 7-16 illustrates a compound bend. The section most susceptible to scour is often the downstream, outside bank of a tight sub-meander (the critical radius in the image).

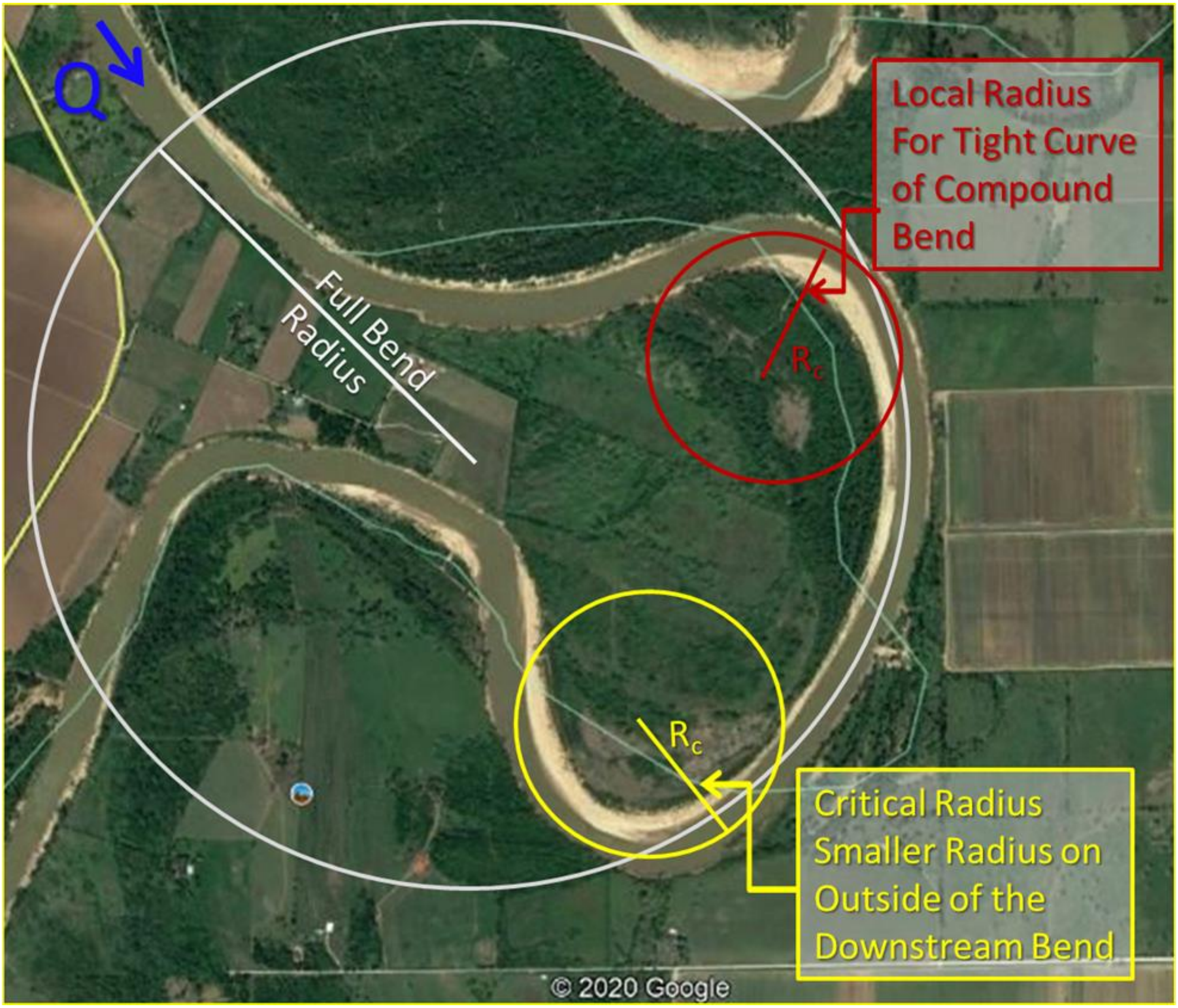


Figure 7-16. Compound bend radius of curvature

(4) Selecting an Upstream Reference Cross Section. Bend scour equations correlate complex multidimensional processes with one-dimensional hydraulic properties by tying the equation to an upstream reference or approach cross section (Figure 7-14). The reference or approach cross section should be a relatively symmetrical upstream section where one-dimensional assumptions are appropriate. This is usually the closest cross section in the straight crossing or run, upstream of the bend that also has similar hydraulic properties as the bend (bankfull depths/widths, channel slope, overbank depths/widths, roughness, etc.).

(5) Location of Maximum Scour in a Bend. The maximum scour depth in a bend varies considerably with bend geometry, hydraulics, material, and many other factors. Secondary currents (velocity components not in the streamwise direction) scour material from the outside of the bend and deposit along the inside of the bend. The most pronounced scour location is usually on the outside bank of the downstream end of the bend.

(a) A simple method is to apply the bend scour results from the point of curvature to downstream of the point of tangency (Simons, Li & Associates, Inc. 1982).

(b) NRCS (2007) states that the length of the scoured zone may be approximated using a relationship by Chen and Cotton (1988):

$$\frac{L_p}{R} = 0.0604\left(\frac{R^{1/6}}{n}\right) \quad \text{Equation 7-3}$$

where:

L_p = recommended length of protection, ft; measured downstream from bend apex
R = hydraulic radius (flow area/wetted perimeter), ft
n = Manning n value for the bed

(6) Revetment Toe Scour. Scour depths at bank toes on the outside of bends usually increase after construction of armored bank revetments.

(a) The Maynord (1996) equation presented in Table 7-6 is applicable for estimating revetment toe scour.

(b) FHWA 2012 presents guidance for estimating local scour at hydraulic structures. Field data for scour at abutments and similar structures such as a floodwall for various size streams are scarce, but data collected at rock spur dikes on the Mississippi River indicate the equilibrium scour depth can be estimated when the ratio of projected abutment length to the flow depth is greater than 25. FHWA (2012) presents the Froehlich, HIRE that was initially presented in Highways in the River Environment (Richardson 1990), and National Cooperative Highway Research Program (NCHRP 2010) equations along with extensive guidance for use to compute abutment scour. Both the Froehlich and HIRE equations are included within the HEC-RAS scour computations.

(c) Combining bend scour equation results with revetment toe scour is generally not applicable. However, parameters such as the factor of safety and sensitivity analysis should be considered when evaluating revetment toe scour within a bend.

e. Deposition.

(1) General deposition, like general scour, spans long reaches of a stream. When the concentration of inflowing bed-material sediment exceeds the transport capacity of the stream in that reach, deposition will occur. Initially, the deposition process starts at the upstream end of the reach and moves toward the downstream end. However, as the extent of downstream deposition increases over time and produces a backwater effect, deposition may begin to occur upstream. General deposition factors include a sediment supply increase, a decrease in water supply, and geometry change (such as channel slope reduction).

(a) When the bed material transport potential is less than the bed-material sediment supplied to a stream reach, sediment will deposit until the hydraulic forces driving sediment transport are in balance with the sediment supply. The aggradation rate depends on the magnitude of the difference in the sediment supply and sediment transport rates. This process can be induced by changes in watershed land use that increase the sediment supply, such as initial construction during urbanization or deforestation, or by channel improvements that decrease transport potential such as widening, dredging, or drop structures.

(b) Aggradation can be induced by decreasing the channel slope. The most dramatic example is construction of a dam. Slope reduction can also be induced by constructing drop structures or by increasing the channel length with the construction of new or restored meanders. Sea level rise will influence aggradation at the mouths of rivers flowing into the ocean.

(2) Local deposition refers to a deposition zone with limited aerial extent. It can occur at tributary confluence, downstream from local scour areas, and in areas of local instability. Channel width expansion typically decreases transport capacity. Sand and gravel will usually deposit as a center bar in a channel expansion because deposition occurs in the channel center where the material is being transported. The center bar will often deflect water toward the banks. When unprotected, bank erosion is expected, and a new plan-form alignment is initiated. On the other hand, silt and clay sediments, which are typically transported more evenly in a channel cross section, are expected to deposit along the sides of a channel expansion where the energy is lower. Deposition occurs until a narrower stream width is produced.

(3) A complex response to various engineering works or watershed changes is common. Both scour and deposition processes are possible. Complex responses may occur as the result of the imbalance of sediment load, sediment size, slope, and discharge as previously presented in paragraph 7-3g. For example, reservoir releases typically have a low sediment concentration and therefore an increased potential for degradation exists. However, reservoirs also typically reduce the magnitude of flood discharges downstream. Therefore, when there is a major tributary downstream from a dam (or many smaller tributaries) with significant quantities of sediment, the sediment supply may be greater than the sediment transport potential, and aggradation will occur downstream from a dam.

f. Numerical Modeling. Numerical modeling methods are preferred to evaluate complex responses to typical USACE projects. Chapter 9 of this manual discusses the application of numerical models. A brief discussion of numerical modeling applications follows.

(1) General scour and deposition can be determined with numerical models. The models, such as HEC-6T and HEC-RAS, can determine the locations, volumes, and bed-change elevations. Numerical modeling methods couple sediment transport equations with the continuity of sediment equation.

(2) Headcuts may be evaluated with sediment routing models like HEC-6T and HEC-RAS. The numerical models can be used to identify conditions conducive to a headcut by identifying zones of intense erosion. Models will transport sediment across a headcut. However, accurate model prediction of upstream headcut movement rate in cohesive materials is limited by algorithm accuracy and should be reviewed carefully.

(3) Scour depth at a bridge is the total combination of long-term bed elevation changes, contraction scour, and local scour at each individual pier and abutment. Numerical sedimentation models can be used for calculating long-term elevation changes and contraction scour.

(a) Local scour, which is caused by local turbulence and redirection of flow at piers and abutments, is estimated in mathematical models such as HEC-6T, HEC-RAS or 2D models such as AdH or SED-2D, using algorithms for calculating local scour external from the sediment transport algorithms. These algorithms are based on methods outlined by the FHWA 2012 in Hydraulic Engineering Circular No. 18, and are also discussed in USACE (HEC 2016a). Additional equations to predict local scour at bridge piers may be found in Richardson and Richardson (2008) and Simons and Sentürk (1992).

(b) While the equations vary somewhat, the basic variables are width of a bridge pier, shape of a bridge pier, skew angle of the bridge, depth of flow, velocity of flow, and in some cases, grain size distribution of the bed material.

(c) When non-typical complex pier shapes are used, the recommended practice is to conduct a physical model study (FHWA 2012). The study "Scale Factor Study for 1:30 Local Scour Model" (Sharp et al., 2016) provides test results for pier scour behavior for non-standard pier shapes and methodology for testing complex pier geometry to formulate site-specific factors.

(4) Analysis of channel and bank processes involves several challenging components of hydraulics and sediment transport analysis. Therefore, creating a low- and high-loss condition model is recommended. Refer to Chapter 9 for detailed modeling guidance.

(a) High-loss condition models using maximum roughness and energy loss coefficients are useful to evaluate river flood maximum levels and aggradation issues.

(b) Low-loss condition models using minimum roughness and energy loss coefficients are useful to evaluate maximum velocity conditions and design of stabilization features.

(c) Unsteady flow modeling and the illustration of the looped rating curve should also be considered when evaluating conditions for design of project features for river maximum levels and stability. Looped rating curves can also be derived from gage records and can be informative of geomorphic processes during extreme events.

7-6. Bank Erosion. Bank erosion is a major consideration from two perspectives: in natural rivers there is the loss of adjacent land with the associated introduction of sediment and debris into the stream, and in project reaches there is the possibility of project failure and land erosion outside the project real estate boundaries with potential to impact adjacent infrastructure and private lands.

a. Erosion Mechanisms. Stream banks are eroded by the channel flow, by waves, by ice, by local surface runoff cascading down the bank, and by geotechnical processes. Frequently, bank erosion occurs due to a combination of these processes at work. Erosion from surface runoff is generally a local scour problem and will not be discussed here.

(1) Channel Flow. When bank erosion occurs because water exerts stresses that exceed the critical shear stress for the bank soils, the erosion mechanism is attributed to hydraulic forces. These forces include:

(a) Tangential shear stress caused by drag of the water against the bank.

(b) Direct impingement of the water against the bank.

(2) Erosion from Waves. Boat waves can create bank erosion in confined reaches. Wind waves deserve attention in areas having long fetches. On most rivers, where fetch is generally limited, maximum short-period wave heights are caused by boat waves rather than wind waves. On larger rivers with longer wind fetches, the combination of high water and wind-driven wave action can be severe. Few river revetments, however, have been constructed solely as protection against boat waves.

(3) Erosion from Ice. Sediment processes can be significantly influenced by the annual cycle of ice formation and breakup (further described in this section).

(4) Geotechnical Failures. Often, eroding banks are due to bank slope instability and not to hydraulic forces.

(a) A common cause of geotechnical failure is excessive hydrostatic pressure in the soil column. As the flood stage increases, the hydrostatic pressure in the bank increases, especially in porous soils. If the river stage falls more rapidly than the pressure can equalize, a geotechnical bank failure may occur. This process occurs frequently in ephemeral streams with sandy banks. It is common for both hydraulic forces and hydrostatic bank pressure to contribute to bank erosion during flood recession. In wide ephemeral rivers, the receding flowline is more sinuous than during the flood and may impinge against a bank weakened by excessive hydrostatic pressures.

(b) Another cause of geotechnical failure is rainfall or snowmelt water that percolates into the soil column only to reach an impervious clay lens and be diverted to the stream bank. Proper control of bank drainage will correct this problem.

(c) A third cause of geotechnical failure results from degradation of the stream bed to the point where the bank can no longer support its height and slumps into the stream.

b. Erosion Rates and Quantities.

(1) Predicting bank erosion rates and quantities is a challenging task. Existing theories are often based on simplifying assumptions such as the banks contain uniform material or on specific field data when conditions vary significantly along the bankline. Empirical methods for determining bank erosion rates and quantities are typically employed. A discussion of bank erosion computation methods is available from Pizzuto (2008).

(2) The Bank Stability and Toe Erosion Model (BSTEM) developed by the National Sediment Laboratory, USDA-ARS, is a physically based model that accounts for the dominant stream bank processes but has an intermediate level of complexity and parameterization (HEC CPD-68B; Simon 2000; Langendoen 2008; Simon 2010). Refer to paragraph 8-6 for further discussion of bank erosion processes.

(a) Rates of bankline erosion are normally quantified using a series of historical aerial photographs. Bankline locations are traced onto a common base and the bank movement is measured and converted to units of surface acres lost per mile per year. This process is complicated by the need to identify common map points and accounting for photo distortion. Other uncertainties include water level variation, vegetation conditions (leaf-off/on), and image resolution. Details of a digital methodology and a list of potential sources for digital maps are provided in HEC No. 18 (FHWA 2012). When available, more precise lateral movement rates can be determined from historical survey or LiDAR data.

(b) Volumes of sediment eroded from the bank may be determined using bank height once the surface area is known. The average bank height may be estimated from the field reconnaissance or from channel cross sections.

(c) Both the gradation and specific weight of sediment eroded are needed from field measurements to calculate yield by grain size class.

(3) Destination of Bank Sediment. Whether or not the sediment eroded from the bank is being transported away by the flow can be determined by the appearance of the toe. If previously failed bank material is present and covered by tree growth, the bank is not active. Sediment that fell into the stream is being left there. If the bank is steep to the toe, the sediment falling from the bank is being transported away. Bank erosion in this case is active.

7-7. Specific Gage Analysis. If gage data are available, one useful tool available for assessing river stability is the specific gage record. A specific gage record is a graph of stage for a specific discharge at a particular gaging location plotted against time.

a. Application Overview. A specific gage record that shows an increasing or decreasing trend is indicative of aggradation or degradation, respectively. A record without a consistent increasing or decreasing trend over time can indicate equilibrium when supported by other sedimentation studies. A limitation of specific gage analysis is that trends are determined from historic data. When used with USACE project design, a specific gage record should not be interpreted as indicative of future trends. A specific gage record can be combined with other studies to investigate historic trends and incorporate projections into USACE project design. It may also be combined with sediment modeling (see paragraphs 7-9 and 9-4c) to evaluate model performance over time.

b. Methods. Fundamentally, a specific gage record can be developed by two methods (Biedenharn et al., 2017): (1) the rating curve method, and (2) the direct step method. Both methods have advantages and disadvantages but can produce reliable outcomes when properly applied.

(1) In the rating curve method, first establish the gage stage-discharge relationship for each year using measured discharge data. Using computed discharge data are not recommended because these data already involve a degree of interpretation and may mask or skew results. A best-fit curve is plotted through the measured discharge data using eye-fit or through an appropriate curve-fitting method such as regression. Since the specific gage record reflects only available measured data, it is important the best fit line is not extrapolated beyond the data limits for each individual year. There are often years without measurements for very high or very low discharges. Additionally, many gages have insufficient data to develop a reliable rating curve for every year. In these cases, the data from several years may be combined into a single rating curve.

(2) In the direct step method, specific gage data are not obtained from an annual rating curve, but instead come directly from the discharge measurements. The first step in the direct step method is to select an allowable band, or bin size, for the gaged discharges. There is no correct allowable band and is generally based on gage data variability and experience. Sizes typically range from approximately ±2.5% to ±5% of the discharge. For example, if a band size of ±2.5% is selected, a flow of 100,000 cfs would represent all the gaged stage data for discharges between 97,500 and 102,500 cfs. After developing the data bands, stage values observed within the selected discharge ranges are plotted against the date of measurement to produce a specific gage record (Biedenharn et al., 2017).

(3) Both the rating curve and direct step method have advantages and disadvantages. When sufficient data is available, it is generally recommended to perform both methods to compare results. Some advantages and disadvantages of each method are provided in Table 7-7

Table 7-7
Specific Gage Methods Advantage/Disadvantage

Method	Advantage[1]	Disadvantage[1]
Rating Curve	(1) Allows development of specific gage records for any flow within the range of flows measured for a particular year. (2) Produces relatively smooth stage-discharge curves that are easy to interpret visually.	(1) Requires sufficient measured discharge data to develop a reliable rating curve for each year. (2) Generates a single data point for each year, which masks variability of the actual measurements. (3) Bases the specific gage record on a regression curve fitted to the measured data, making the statistical analysis of trends less robust than is the case for the direct step method.
Direct Step	(1) Allows for more robust statistical analysis. (2) Shows the individual measurements, which means that the variability in the data is apparent to both those performing the analysis and end users of that analysis. (3) The direct step method is generally simpler to employ than the rating curve method (unless a program is available to automate the rating curve generation).	(1) Records sometimes include a lot of scatter, which makes for highly irregular stage-discharge curves. (2) It is only possible to derive specific gage records for those discharges that were actually measured in a particular year, which may lead to gaps in the record for some flows. (3) For many rivers where measured discharge data are limited, obtaining sufficient data points to produce a reliable specific gage record may require selecting a bin range that is unacceptably large.

[1] Summarized from Biedenharn et al., 2017, Samaranayake (2009).

c. Time Scale. Selecting the appropriate time scale for identifying trends is important because specific gage records typically are variable. Characteristically, the record variability is not random with cyclic trends commonly encountered. Biedenharn et al. (2017) states:

(1) Consequently, a short-term trend in the specific gage record may not reflect progressive change in the morphology of the river, being instead part of the between peaks and troughs that constitute the upper and lower boundaries of a band of variability about a steady, time-averaged mean value. In such cases, the period of which trends are identified may need to be lengthened to avoid focusing on a misleading short-term localized response.

(2) Conversely, trending over too long a time scale may also lead to misinterpretation of stage changes associated with a given discharge. Trends over multiple decades may mask changes due to one or more shorter periods of relatively rapid morphological response to individual disturbances of the fluvial system such as a channel cutoff, flood risk management project construction, or a dredging project. In this case, the record should be divided into two or more shorter periods to reveal the short-term trends.

d. Selecting Representative Flows. The discharges to be used in the specific gage record must be selected. This selection depends largely on the objectives of the study but should usually encompass the entire range of observed flows. Gaps in the data series for the highest and/or lowest discharges will usually occur. Stage values for the missing flows should not be generated by extrapolation of other annual data values because additional points created by estimating missing flows do not increase accuracy to the specific gage analyses for that discharge and may mask actual trends (Watson et al., 2013a).

e. Specific Gage Analysis Results and Interpretation. Specific gage records are an excellent tool for assessing the historical stability at a specific location. However, interpretation of a specific gage record can be challenging. The following examples provide specific gage analysis results and interpretation details.

(1) Brazos River at Richmond, Texas. Widespread bank instability has been a serious concern for many years along the Brazos River in the eastern outskirts of Houston, Texas. As part of a Planning Assistance to States (PAS) study in 2017, a specific gage record was developed for the Brazos River at Richmond. This USGS gage at Richmond (USGS 0811400) has measured discharge and stage data that date back to 1929. The Brazos River example is presented herein to illustrate two aspects of specific gage records: (1) to illustrate the importance of checking the raw data for gage changes throughout the period of record; and (2) to provide a comparison of the rating curve and direct step methods.

(a) Gage Datum Change. Specific gage records were developed for a range of flows between 2,000 cfs and 80,000 cfs. Discussion for 20,000 cfs, which has a return period of slightly less than one year, is as follows:

- Figure 7-17 shows the specific gage record which was based on the published USGS data at Richmond. Examination revealed that there had been an abrupt increase in stages of about 10 feet in 1989. As there was no physical change in the system that could account for this dramatic change, a closer examination of the data was necessary.

- After discussions with the USGS, it was discovered that there had been changes to the gage zero that were not listed in the published data. The original gage zero of 40.94 feet which had existed since 1929, was adjusted to 37.94 feet in 1976, and then again in 1989 to 27.94 feet.

- Diagnosing the gage datum shift was readily apparent because of the abrupt 10-foot change in 1989. However, if it had not been for this dramatic change, the more subtle 3-foot change that occurred in 1976 resulting from the smaller gage zero change may have gone unnoticed.

• As shown in Figure 7-17, the unadjusted stage in 1976 was only about a foot higher than 1975 and was well within the range of other data in this time period. Therefore, there would not appear to have been any issue with this data. However, the corrected stage in 1976 was actually almost 2 feet lower than in 1975. This clearly illustrates the importance of conducting a careful data review to ensure that all gage datum shifts and location changes have been included.

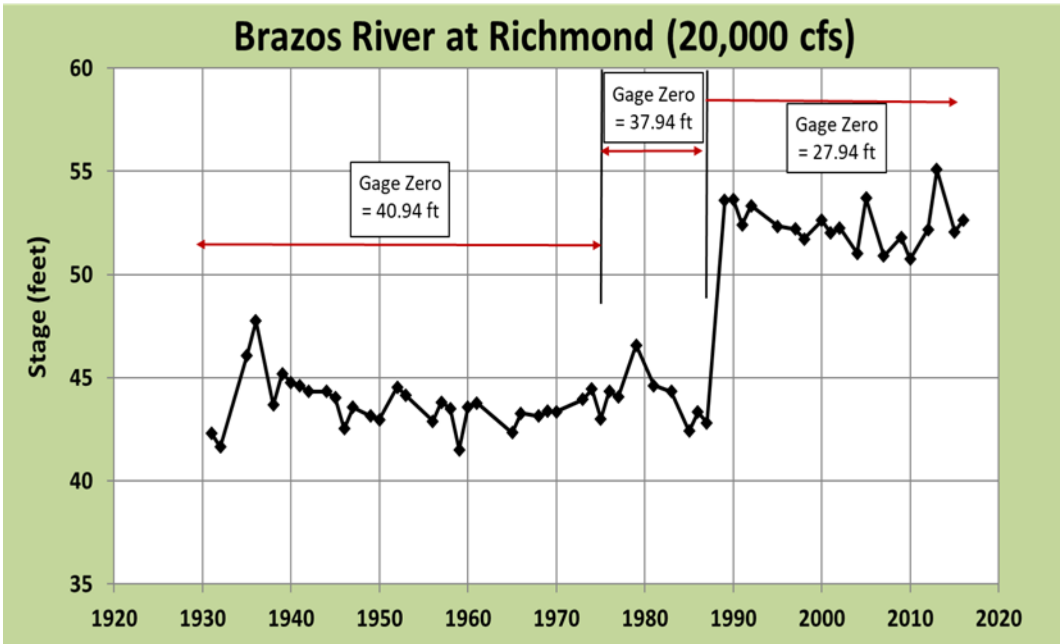


Figure 7-17. Specific gage record for the Brazos River at Richmond, Texas (based on original published data with gage zero set at 27.94 for the entire period of record; gage zero was changed in 1976 and again in 1989)

(b) Comparison of Rating Curve and Direct Step Methods. Figure 7-18 compares the specific gage results for the rating curve and direct step methods at the Richmond gage. Because of scarcity of data, a fairly large bin range of ±10% had to be used for the direct step method to produce enough data points for a reasonable specific gage record. Smaller bin ranges (2.5% and 5%) were attempted, but the results were not considered acceptable because there were too few data points.

• As shown in Figure 7-18, both the rating curve and direct step methods show similar overall trends. Prior to the mid to early 1970s, stages were fairly stable to possibly slightly degradational. After the mid-1970s, there is a degradational trend that appears to extend into the early 2000s. Stages appear to again stabilize somewhat after that period.

• There are also differences between the two methods that warrant some discussion. Although the direct step data does show some of the inter-annual stage variability (almost 3 feet in some years), there are many years where no data are plotted. This is particularly a problem for the period between the mid-1970s to the mid-1980s, and the post-2003 period. By contrast, the rating curve method produced a much more complete record with only a few years not having data. Thus, for this situation, the direct step method does not provide as comprehensive a view of the stage trends throughout the period of record.

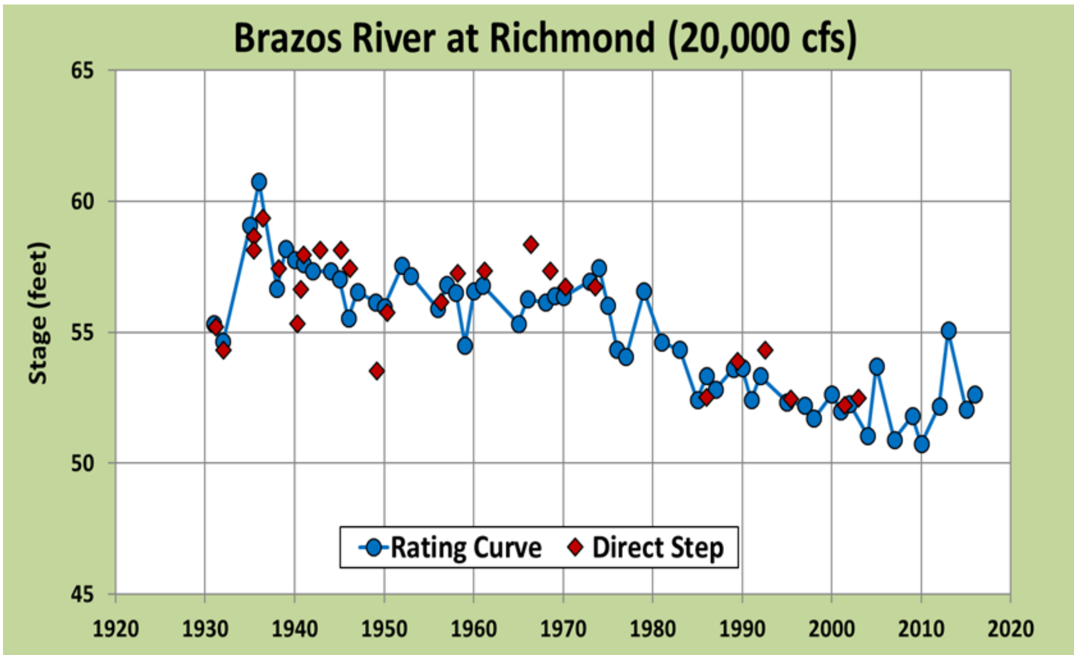


Figure 7-18. Specific gage record for the Brazos River at Richmond, Texas, comparison of methods (adjusted for gage zero changes)

(2) Missouri River at Sioux City, Iowa.

(a) A specific gage analysis was derived using the rating curve method on the Missouri River at Sioux City, Iowa, (RM 732) as show in Figure 7-19. The period of record extends from 1929 to current. Flood stage is above 1,085 feet National Geodetic Vertical Datum (NGVD) 29 at an extreme flow. The specific gage analysis illustrates what appears to be a continual degradational trend. Closer examination shows a complex response with many contributing factors:

• Sioux City trends are heavily influenced by: (1) the construction of the upstream Missouri River reservoir system with regulated flows and sediment trapping (construction initiated in 1933 at Ft Peck Dam (RM 1771), nearest to Sioux City is Gavins Point Dam (RM 811) that started construction in 1952 and closed in 1955); (2) construction of levees; and (3) private bank stabilization efforts and the Missouri River navigation channel that resulted in channel realignment, shortening, and numerous dike and revetment structures over a lengthy construction period from the 1930s through the 1970s.

• Due to the brief period of available data, it is unknown if the aggradation shown in the early 1930s is representative of early 1900 trends.

• Rapid degradation is apparent in high-flow years (1952, 1997, and 2011), followed by partial rebound during subsequent years with more normal flows. Long-term trends starting in the 1940s result from the combined influence of stated factors.

• Future trends may not resemble historic as the response to stated factors will continue to transition. Gage response is likely to continue to be heavily event-driven.

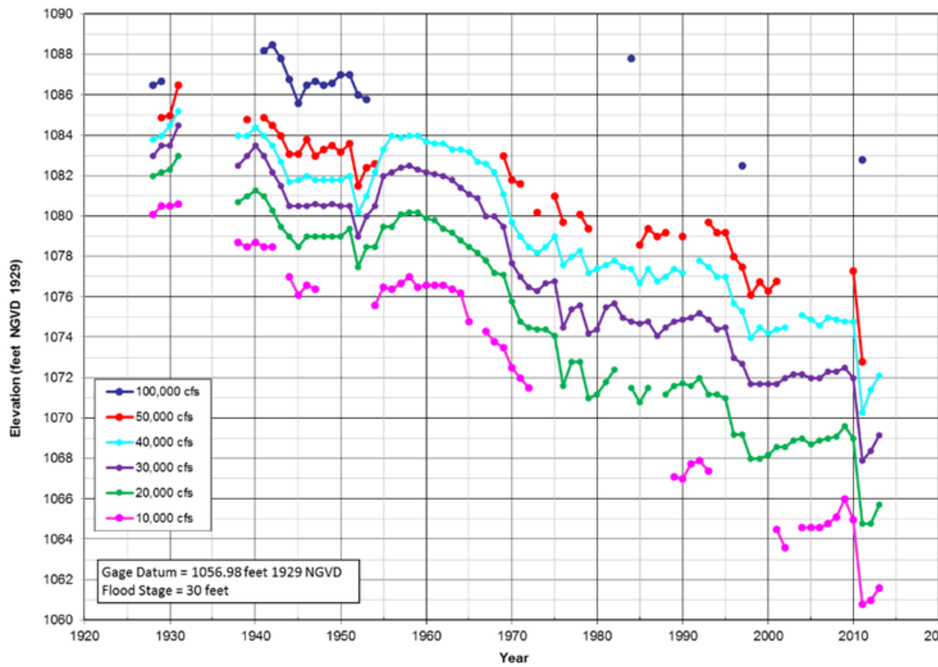


Figure 7-19. Specific gage analysis, Missouri River at Sioux City, Iowa (after USACE 2017d)

(b) Madeira River, Brazil. Specific gage analyses were performed by Gibson et al. (2019) on the Madeira River in Brazil at three stations located at Porto Velho (upstream), Humaitá (mid-reach), and Manicoré (downstream). A large planform change occurred in the 1980s with a major meander cutoff in the late 1980s. The cutoff was located about 40 km upstream of Manicoré and shortened the river by 37 km.

• The two gages upstream of the cutoff (Porto Velho and Humaitá, the top two plots in Figure 7-20) show no trend. However, the Madeira deposited sediment downstream of the cutoff with discernible impacts at the Manicoré gage (bottom panel of Figure 7-20). Deposited sediment increased water surface gage elevations between 0.2 to 0.4 m at the range of flows evaluated.

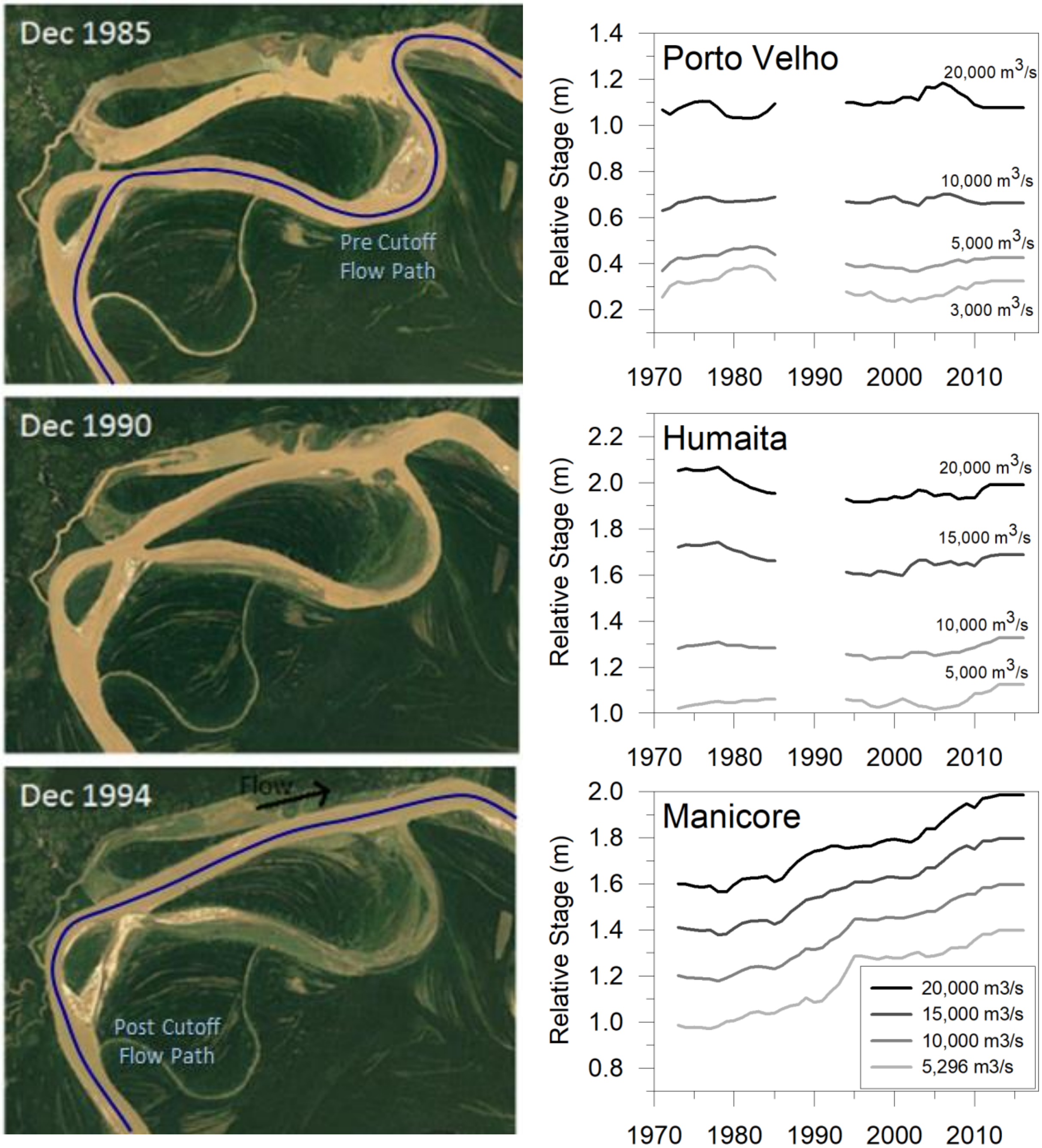


Figure 7-20. Bend cut-off upstream of Manicoré (left), specific gage curves (right) (from Gibson et al., 2019)

- Gibson et al. (2019) note that, outside of the local effects of the cutoff, the river is vertically stable and that the specific gage trends suggest that the reach is in dynamic equilibrium on the decadal scale. They also note that, despite the decadal scale, vertical stability upstream of the cutoff and the modest meander migration rate, the anabranching pattern of the river is relatively dynamic. Fifty-six percent of the vegetated islands between Manicoré and Porto Velho were not present in the 1984 aerial photographs (compared to 4% in the downstream reach, between Manicoré and the Amazon).

(3) Use of Trend Lines from Specific Gage Results. Trend lines may be fit through the specific gage results with the desire to establish historic rate of change. Common practice is to use an eye-fit or an appropriate curve-fitting method such as regression to create a trend line. Either method should be used with caution. Considerations are as follows:

(a) USACE FRM projects require estimating the impact of stage trends on project performance throughout the project life. Assuming that all processes that caused historic trends will continue unchanged in the future is generally suitable only for initial studies when combined with a robust sensitivity analysis. Sedimentation analysis is necessary to determine processes at work and the impact on FRM project performance (see paragraph 7-9). Specific gage analysis is a useful tool to inform sedimentation analysis scoping and model development.

(b) Watson (2010) states that a specific gage record does not provide any indication about future degradation or aggradation trends. Extrapolation of specific gage records into the future is extremely risky and is generally not recommended. Watson further recommends that even though the specific gage record is a valuable tool, it should be coupled with other assessment techniques and models to assess cause-and-effect relationships that may be manifested as trends in the specific gage.

(c) Eye-fit or standard regression analysis may be misleading when used to depict long-term trends.

- Linear regression can mask rates that are changing with time. For example, a long-term degradational trend may be accelerated by upstream dam construction or channel cutoffs. In another example, a gage located in a delta headwater will often have a non-linear aggradational trend as the delta location changes with time or responds to specific events.

- Eye-fit trend lines are subject to user development error that appear reasonable but can easily skew results.

- Nonlinear functions are very difficult to apply with accuracy to specific gage trends and are not typically used.

(d) Watson et al. (2013a) provided guidance for the use of statistical analysis to determine trend significance.

- The two statistical parameters commonly employed for this purpose are the coefficient of determination (R^2) and the probability (p-value) that the slope of a regression line fitted to the data is significantly different from zero.

- R^2 is a measure of the proportion of the change in the Y variable (stage) that is explained by change in the X variable (time). For example, an R^2 of 0.2 implies that only 20% of the observed change in stage can be explained by time.

- The p-value is the probability that the slope of a least-squares regression line fitted to the data is the product of random chance rather than the outcome of the influence of the X (independent) variable on the Y (dependent) variable. Using the p-value may not be applicable for many applications. For instance, rivers with limited size data sets will not provide meaningful p-value results. Using p-value criteria to interpret meaning with respect to historical trends in stages requires adopting subjective thresholds.

- Establishing that a trend is significant does not prove that the X and Y variables are related. Samaranayake (2009) emphasized that once a statistically significant relationship between two variables has been identified, one must evaluate whether the physical basis and possibility for the existence of a cause-and-effect relationship can be established.

(4) Limitations of Specific Gage Analysis (expanded from Biedenharn et al., 2017):

(a) Specific gage records indicate only the conditions in the vicinity of the particular gaging station and do not necessarily reflect river response upstream or downstream of the gage. Therefore, the specific gage record should be coupled with other assessment techniques to assess reach conditions when developing predictions about the ultimate response of a river.

(b) Since a specific gage analysis is empirically based, a key issue is the availability of reliable, measured data that provides the relationship between measured stages and measured discharges throughout the analysis period. Gage mean daily discharges are estimated values that are generally developed from a rating curve derived from periodically measured flows. Only the stage is measured daily or more frequently. Consequently, important limitations to the utilization of stage and discharge records for specific gage analysis are that only measured data should be used, and extrapolation is unacceptable. Uncertainty increases and subjectivity may result if these limitations are not accepted and observed.

(c) Specific gage records do not automatically correlate with cause. Detailed assessments and statistical analyses are often necessary to attribute trends to specific natural or anthropogenic factors. Assigning causal relationships among potential factors including changes in the hydrologic regime, basin sediment supply, and construction of features (dikes, revetments, cutoffs, levees, dams, or diversions) is challenging and will require comprehensive methods.

(d) A common error is to interpret a stage trend as always indicative of a channel bed elevation change. It is important to recognize that specific gage records track changes in water surface elevation. While stage trends may reflect bed elevation change due to degradation or aggradation, it may also result from some other change in channel properties (for example, roughness). Seldom would bed change be directly correlated at a 1:1 ratio to water surface change. As stages increase, the correlation of specific gage trends with bed changes typically diminishes. This is particular the case when overbank flows are considered, as these trends are more likely related to floodplain processes rather than bed changes.

(e) Stage and discharge records over long time periods should be examined for consistency. Gage movement, changes in gage zero of the gage, bridge construction, and levee construction are examples of specific events that can cause an immediate change in the stage-flow relationship. Data should be screened prior to use for effects including measurement quality, backwater, and ice. Brauer (2009) describes how historic measurements using different velocity measuring devices may bias historic records. Biedenharn et al. (2017) report that the Thebes gage, located at RM 43.7 on the Middle Mississippi River (MMR), is periodically subject to backwater effects because of the close proximity to the Ohio River. They elected to remove all measurements where the stage difference between Thebes and Cairo was less than 15 feet.

(f) Interpretation of a specific gage analysis will often need to consider historical and present-day fluvial processes and morphological responses. As previously discussed, trends on the Missouri River at Sioux City are influenced by a number of factors that will likely cause future stage trend rates to vary from historic. Rivers that exhibit large fluctuations in stage-flow due to seasonal trends, backwater effects, and looped rating curves require extra caution to develop trends.

(g) Limitations may be defined by statements of principle (expanded from Watson et al., 2013a):

- A specific gage record represents only the condition of the channel in the vicinity of the particular gaging station.

- Specific gage analysis examines past trends and changes and does not a priori give any indication of future trends or changes. In this context, Johnson and Bhattacharyya (1985) stated that: “Extreme caution should be exercised in extending a fitted regression line to make long-range predictions. Not only does the confidence interval become so wide that predictions based on it can become extremely unreliable, but an even greater danger exists. If the pattern of the relationship changes dramatically at a distant value of x, the data provide no information with which to detect such a change and then our estimate would drastically miss the mark.”

- Reliable interpretation of a specific gage record should be based on both the investigator’s visual observations and sound statistical analyses.

(h) Natural variability and sources of data uncertainty (such as data collection methodology variability over the period, general record homogeneity considering hydrologic variability, external influences such as regulation or flood risk management project effects, measurement error, and temperature effects) must be recognized. Watson et al. (2013b) demonstrate how an increasing stage trend at the Mississippi River at St. Louis may be detected that is the result of mixing two discrete observation data sets of disparate quality instead of river changes. They conclude that the pre-1930s discrete streamflow measurement data at St. Louis do not have sufficient accuracy to be compared with modern streamflow values when establishing long-term trends.

7-8. Additional Processes and Analysis Considerations.

a. Urbanization Effects on Sediment Processes.

(1) Urbanization within a watershed results in the conversion of multiple land uses such as forests, farmland, wood lots, wetlands, and pasture to urban land uses such as residential, commercial, and industrial developments. The process of urbanization directly impacts stream and riparian corridors by:

(a) Substantial stream channel alteration through straightening, lining, or even elimination with placement in underground conduits.

(b) Floodplain Encroachment.

(c) Increased sediment yield due to development construction and possibly increasing pollutant loading following development. Long-term sediment yield may be reduced if stream banks are armored and impervious area is extensive.

(2) Urbanization alters watershed sedimentation processes. Increased impervious area generally increases runoff and alters sediment yield characteristics of a watershed. Corresponding hydrologic changes include greater, more frequent, and longer duration flows with the consequence of additional energy acting on channel beds and banks. Within the urban environment, streams must adjust to these changes within the physical constraints imposed by multiple factors such as bridges, pipeline crossings, and bank stabilization measures. Thus, many urban streams are evolving to establish a new equilibrium condition that is markedly different from both the current and the historic, more natural condition. In response to urbanization, many channels experience rapid incisement.

(3) With respect to USACE project design and sedimentation analysis, the factors encountered in urbanization are a standard component of the necessary analysis that is described in various chapters throughout this manual. Specific conditions to consider include:

(a) Changes in sediment load, flow regime, and boundary conditions associated with channelization and urbanization result disrupt stream equilibrium.

(b) Floodplain encroachment can result in a significant confinement of channel flows with much higher energy acting in a narrow flow corridor.

(c) In a typical incising channel, the streambed degrades until a critical bank height is exceeded and the adjacent bank fails. Increasing channel width increases sediment load and is also a threat to adjacent infrastructure. In addition, channel degradation generally migrates upstream on both the mainstem and tributaries. This can destabilize a large part of the watershed. A new equilibrium may take decades or even centuries to achieve.

b. Sedimentation in the Floodplain.

(1) Sedimentation processes include the interaction between the channel and floodplain. River floodplains are complex environments with evolution dependent on many factors, including valley slope, channel morphology, sediment properties, hydrology, and confinement.

(2) In high slope and flow energy river corridors, floodplain sediment deposition often occurs over extended periods of time which are mobilized during extreme flow events. Floodplain material shear stress and sediment transport dynamics are strongly influenced by floodplain topography, valley wall alignment, and encroachments such as levees. Many floodplains exhibit both wide and narrow sections of effective flow width with a corresponding response in floodplain scour and deposition.

(3) Conversely, the river floodplain may be so wide that floodplain deposition is focused into a meander belt often much narrower than the valley width. Velocity reduction is typical when flows transition from the main channel to the floodplain This process results in elevated rates of material deposition adjacent to the channel and the formation of near bank natural levees. Sediment sorting during overbank flow results in lateral fining of flood deposits with the natural levees near bank typically consisting of coarse sediments. When natural levees exist, lateral channel migration results in cutting of the elevated banks and the establishment of new laterally accreted deposits that are often lower in elevation than the eroding cut banks.

(4) The magnitude of floodplain deposition can be quite large, affecting downstream sediment loads, the frequency of floodplain inundation by subsequent floods, and ecological conditions in the floodplain. This is a significant component to consider in a sediment budget as Chapter 6 of this manual describes. Factors affecting channel and floodplain flow distribution include channel alignment, levees, roadways, topography, and other site-specific features. Figure 7-21 shows a segment of the Missouri River where levees affect channel and floodplain flow. Floodplain feature influence varies with flow level and often contributes to long-term floodplain changes and aggradational or degradational stage trends.

(5) Floodplain sedimentation processes are a phenomenon to evaluate during detailed project design. Figure 7-22 shows conditions after an extreme event on the Missouri River in 2011 that significantly altered floodplain topography, with substantial areas of both deposition and scour.

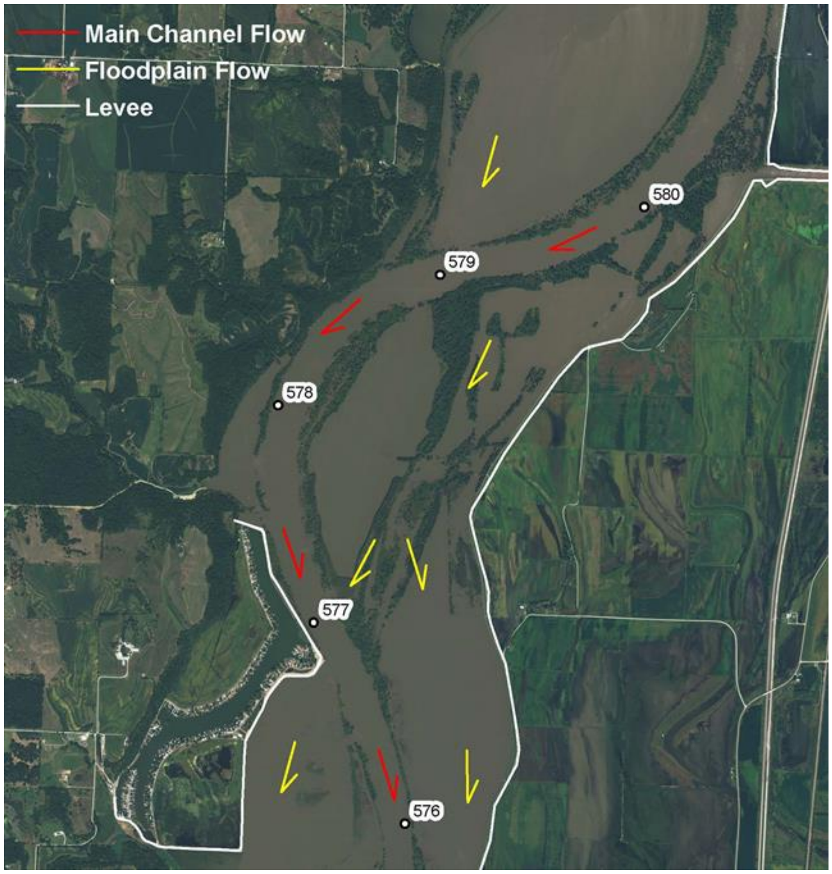


Figure 7-21. Channel and floodplain flow affected by levees, Missouri River

Figure 7-22. Floodplain scour and deposition, post-2011 event, Missouri River

c. Vegetation Impacts on Sediment Processes.

(1) Vegetation affects sediment processes both directly and indirectly. Sediment transport processes are affected by flow roughness. Direct impacts include increased flow roughness that varies according to vegetation height and density (Fischenich and Copeland 2001; Freeman et al., 2000). Numerous texts that evaluated vegetation effects on flow roughness are available for additional reference including Chow (1959) and USDA (1987) that present roughness as a function of vegetation height and density.

(2) An example of effective floodplain vegetation management incorporated with USACE project design is the Dallas Floodway Extension (DFE). The project consists of a chain of wetlands designed to provide unimpeded overflow for floodwaters along the Trinity River. Project capacity was designed using lower roughness values associated with wetland vegetation communities comprised predominantly of herbaceous species rather than woody species. The DFE channel and floodplain improvements result in complex hydraulics and sediment transport characteristics that influence the operation and maintenance of the project area. Monitoring has been conducted within the wetland areas to evaluate sediment deposition due to overtopping events and the long-term potential for impact (USACE 2014a). Figure 7-23 shows lower chain of wetlands features.

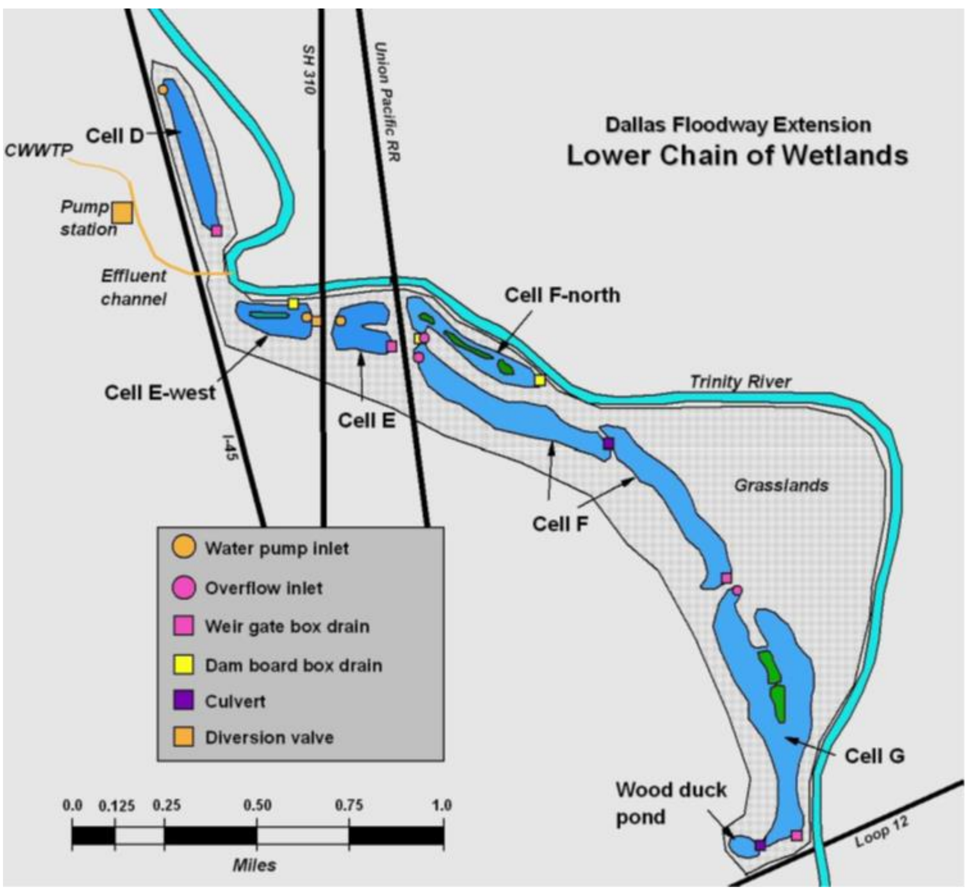


Figure 7-23. Trinity River, lower chain of wetlands primary features (USACE 2014a)

d. Hyperconcentrated Flows.

(1) Hyperconcentrated flows consisting of mud floods, mud flows, and debris flows (as discussed in paragraph 3-4) are an important consideration for USACE project design. Hyperconcentrated flows contain a high-density slurry mixture of water, sediments (boulders, cobbles, and soil), woody debris, and mud that can have enormous destructive power.

(2) Hyperconcentrated flow can be initiated by numerous causes including intense rainfall, rapid snowmelt, and volcanic actions. Steep slopes, ample erodible materials, and minimal infiltration are contributing factors. A wildfire can drastically increase the probability of debris flows in landscapes that otherwise have been historically stable.

(3) USACE project design in regions susceptible to hyperconcentrated flow that rely on traditional clear-water hydraulic design procedures for FRM works, can lead to undersizing of debris retention facilities by 10 to 100 times and flood conveyance channels by 3 to 10 times depending on event sequencing, the severity of the storm event and geomorphic characteristics of the basin (MacArthur et al., 1992).

(4) Watershed inspection, and the proper identification of risk for hyperconcentrated flows, is a critical aspect of USACE project analysis. Design of mitigation structures requires knowledge of the hyperconcentrated flow properties. Different mitigation strategies should be considered for mud floods, mud flows, and debris flows.

(5) USACE FRM project designs in areas at risk for hyperconcentrated flows that include conveyance improvement features should consider effects of sediment bulking, debris deposition, channel migration or avulsion, and similar. A channel lining using large rock riprap may be undesirable in steep slope channels as the riprap may launch and contribute to debris flow loading. Information regarding debris flow analysis methods and project design considerations are available from multiple sources (Hutter et al., 1994; Pierson 2005; Schumm and Harvey 2008; FEMA 2016, 2002). Additional information is also presented in Case Studies 8E and 10A (Appendix N).

e. Alluvial Fan.

(1) The general characteristics of alluvial fans were previously presented in paragraph 7-8e. Alluvial fans are a hazardous area where the degree of flood hazard is hard to predict. Individual flood events depend on geological, topographic, and hydrologic characteristics of the drainage basins and fan area.

(2) Non-uniform distribution of flow and sediment load (both size and concentration) over the fan may result in scour, deposition, blockage, avulsion, and flow redistribution over the fan. The size and location of fan channels can change rapidly. As a result, future floods may exhibit very different characteristics from those of historic events with variation in flow paths, depth, velocity, flow distribution of flow, and sediment load. These inherent characteristics of alluvial fans make quantitative analysis of fan processes, and correspondingly, of the design of USACE

project features, extremely difficult (HEC 1988). Figure 7-24 shows a typical alluvial fan, and Figure 7-25 shows an idealized plan and profile features of an alluvial fan.

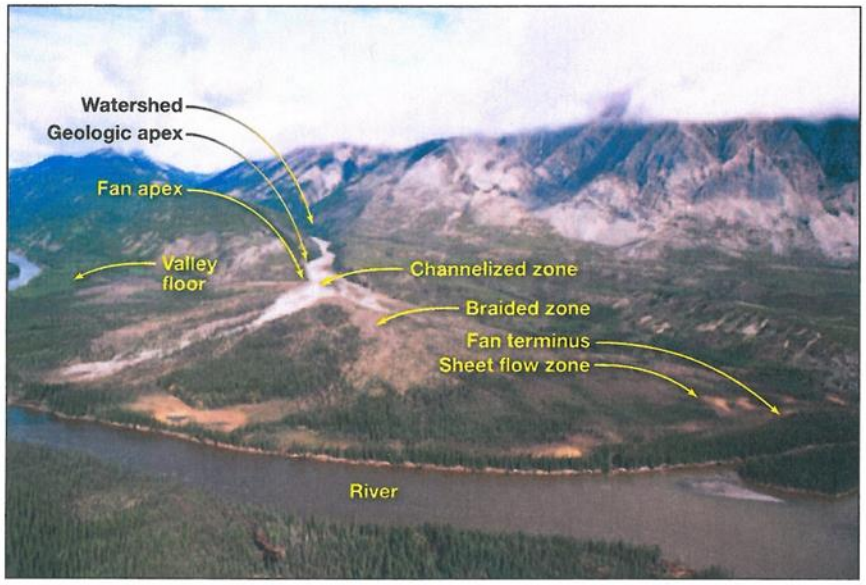


Figure 7-24. Alluvial fan flooding zones and features (FEMA 2012)

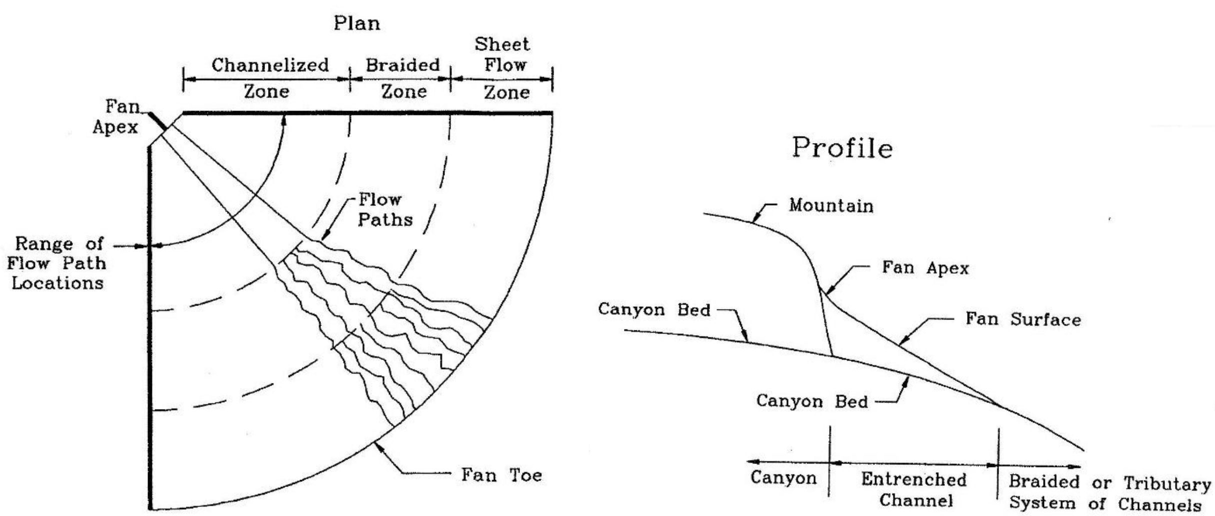


Figure 7-25. Alluvial fan, idealized plan and profile (adapted from HEC 1993a)

(3) Design analysis methods for alluvial fans are available from USACE in several documents (HEC 1988 and 1993a). Other relevant design guidance from numerous sources should also be consulted including National Research Council (NRC 1996), FEMA (1989, 2000, and 2012), and ASCE Manual No. 110, Chapter 19 (Garcia et al., 2008).

(4) Design analysis on alluvial fans requires careful consideration of sedimentation processes. Modeling mudflows and debris flows has long been an interest of hydraulic and sedimentation engineers. Because of the complicated fluvial physical processes of these flows, solving the fully dynamic equations for unsteady, non-uniform, non-Newtonian flows is still a challenging endeavor. Consequently, basic concepts of open-channel hydraulics are often applied to the simulation of mudflows. Currently design capability includes 2D models that are available for modeling flows on alluvial fans that can model both clear-water and sediment-laden flood flows. Garcia et al. (2004) presented an application for alluvial fan hazard mapping.

(5) The prediction of drainage basin sediment yield contributing to the alluvial fan and sediment transport along critical reaches of the fan is extremely difficult. In addition, alluvial fan flood hydrology is often faced with a lack of recorded data in many areas. The analytical methods used are highly dependent on regional data and experience. Successful evaluation of USACE projects on alluvial fans consideration of flow and debris movement through the project area. The natural channels are often unstable, and there is considerable uncertainty in the prediction of the size and location of the channel during and after a flood event. Channel avulsions are common during large events.

(6) Every factor affecting the nature of flood and debris problems, plus the development and its susceptibility to flooding, affect the feasibility of FRM options. There is no “cookbook” approach to developing effective project features. Planning and design of USACE FRM projects on alluvial fans must always consider project performance, both locally and downstream, for the entire range of floods, including floods larger than the design flood.

(7) Common Stability Problems and Project Performance.

(a) Application of standard channel and levee design criteria on alluvial fans will not correctly address the complex hydraulics and ensure project reliability. FEMA (1989) identified the following flood hazards for alluvial fans: (1) high-velocity flow (15 to 30 fps) that can produce significant hydrodynamic forces on structures, (2) significant erosion/scour depths, (3) deposition of sediment and debris to depths of 15 to 20 feet during a single event, (4) debris flow and their associated impact forces and large sediment loads, (5) flash-flooding with little warning time, (6) unpredictable flow paths, (7) hydrostatic and buoyant forces, and (8) inundation.

(b) Potential stability problems on alluvial fans are numerous and include avulsion of the stream at a point upstream of training works or channelization, thereby bypassing the constructed works, and infilling of the designed conveyance channel by coarse sediment deposits.

(c) FRM project channels constructed on alluvial fans generally have two types of problems (USACE 1993): (1) destruction of riprap protection with subsequent project feature damage, and (2) sediment deposits decreasing flow capacity. Riprap and other forms of bank protection tend to fail at the protection edges (inlet, outlet, or toe protection). Failure of toe protection due to excessive scour was noted in several projects. Riprap failures also have occurred due to impinging flow from tributaries or from flow meander in the channel. Refer to paragraph 7-7e for a discussion of general USACE FRM project design considerations.

f. Ice Effects on Sediment Transport. Sediment processes can be significantly influenced by the annual cycle of ice formation and breakup. The magnitude of the influences depends on a combination of factors with the prime considerations consisting of the amount of ice formed and the quantities of water and sediment to be conveyed. Consult available references such as Sediment Transport Under Ice (Ettema and Daly 2004) and Ettema (2008).

(1) Ice Processes.

(a) A brief background of ice processes is provided for context to the discussion of impacts on sediment transport. Refer to available references such as EM 1110-2-1612 for a thorough discussion on ice formation, thickness, breakup, and transport in rivers.

(b) Total ice volume and thickness are the result of the cumulative period of temperature degrees below the freezing temperature of water. Within natural rivers, the annual cycle of winter ice generally coincides with a decline in runoff and river flow. However, regulated rivers may experience winter flows due to reservoir releases that are well above the historic pre-dam natural condition and therefore increase ice formation. Runoff and channel flow typically increase significantly during spring thaws.

(c) Ice cover formation typically increases energy losses along a river reach. Ice cover usually increases and redistributes a channel's resistance to flow which reduces the overall capacity to move water and sediment. Ice may also locally influence erosion processes. The cementing of bank material by frozen water and ice armoring of bars and shorelines can reduce erosion and sediment transport rates. However, in the case of large ice jam breakup and ice gouging, ice may amplify erosion and sediment transport rates. The variability of river response to ice-cover presence makes it difficult to draw simple overall conclusions regarding the effects on the river bed and banks. The net effects vary from site to site (Ettema and Daly 2004). Ice processes may affect structures and bank stability in USACE river projects (Figure 7-26).

Figure 7-26. Ice at intake dam headworks and river bank, Yellowstone River

(2) Ice effects on sediment transport include:

- Ice affected flow distribution.
- Sediment transport by ice (sediment included in drifting ice).
- Sediment transport under ice.
- Ice influences on channel morphology.

(a) Ice cover adds an additional flow resistance boundary that can alter the channel velocity distribution and affect flow distribution. Ice cover may laterally redistribute flow and concentrate it, often along the thalweg. In addition to altering the sediment transport processes due to revised geometry, localized effects on bank erosion rates may occur (both increase and decrease) as the thalweg shifts toward or away from the near bank vicinity.

(b) Sediment may be transported by ice movement processes. Sediment-laden ice slush and clumps of ice-bonded sediment may appear during the early stages of ice formation. The ice slush and clumps usually include a mix of frazil and anchor ice that were once briefly bonded to the beds of such rivers and streams. The amounts of sediment entrained and rafted with the ice slush and clumps can produce a substantial momentary surge in the overall quantity of sediment moved by some rivers and streams. Much of the entrained sediment becomes included in an ice cover, where it remains stored until the cover breaks up. Ettema and Daly (2004) further discuss the mechanisms whereby ice entrains and transports sediment and the distances over which ice-rafted sediment typically may be transported.

(c) Sediment transport may occur under and ice cover. Typically, river morphology and flow resistance vary with flow and sediment conditions. Ice covers modify the interaction, doing so over a range of scales in space and time (Ettema and Daly 2004).

- Ice conditions affect water temperature and transport potential. Most empirical relationships for alluvial-channel hydraulics are based on data obtained with water in the range of 10 to 20 °C. Reduced water temperature increases kinematic viscosity, and slightly changes density. In so doing, it increases flow drag on the bed, decreases particle fall velocity, and thereby overall increases flow capacity to convey suspended sediment.

- The overall magnitude of tractive force that flow exerts on bed particles prescribe bed sediment motion and bed forms. Ice-cover presence influences bed tractive force and turbulence generation by redistributing flow and reducing the rate of flow energy expended along the bed.

- Local scour beneath ice jams can be significant. Ice cover may locally increase flow velocity, and thereby increase alluvial bed or bank erosion.

(d) River ice potentially may exert a combined impact of hydraulic and geomechanical effects that may continually destabilize channel geometry in rivers subjected to substantial, reservoir regulated flow during frigid winters. Ettema and Daly (2004) report that ice may influence channel cross-section shape, alignment, and bed elevation through the following:

- Reducing riverbank strength by increasing pore-water pressure or by producing rapid drawdown of bank water table during dynamic ice-cover or ice-jam breakup. This impact is part of the overall consequence of freeze-thaw behavior or riverbanks in frigid conditions.

- Tearing, battering, and dislodging riverbank material and vegetation during collapse of bank-fast ice (ice that is fastened or anchored to the bank, fast ice does not move with wind or current).

- Gouging and abrading riverbank material and vegetation during an ice run.

- These impacts reduce riverbank resistance to erosion and increase the local supply of sediment to the channel.

(3) Combined Impacts.

(a) While a single factor may result in minor channel change, a combination of hydraulic and geomechanical impacts may significantly alter channel stability. Channels that are usually considered less stable in open water conditions are more likely to be adversely affected by river ice (Ettema and Daly 2004).

(b) Freeze-thaw expansion of ice sheets during the winter and wind-driven ice movement during spring breakup may damage shoreline vegetation and cause ice shoves that can impact placed riprap protection in reservoirs and navigation pools. Figure 7-27 (left) illustrates riprap displaced, with a resulting damage to erosion protection, when ice shove occurred. The ridge of dark earth material was formed by windblown ice forces (a compressive up and out of shoreline material). Figure 7-27 (right) shows a rock groin after ice damage. The rock groin should be 2 feet above the water surface and extend 30 feet away from the shoreline. Ice expansion during freeze-thaw cycles significantly displaced structure rock.

Figure 7-27. Ice damage on shoreline, Lake Onalaska, Minnesota

(4) Ice Impacts on USACE Projects. During USACE project design, situations where performance can be negatively affected by river ice dynamics should be evaluated. For instance, ice jam magnitude, frequency, and location are often impacted at reservoir deltas. Consult available references (Ettema and Daly 2004) and Ettema (2008) for additional details. A thorough evaluation of potential for ice processes to alter ongoing sedimentation processes at USACE projects is recommended in ice affected regions.

7-9. Stability Analysis Concepts and Methods. Approaches and techniques that have been used for quantitative evaluation of channel stability include allowable velocity, allowable shear stress, stream power, hydraulic geometry relationships, sediment transport analysis, and bank slope stability analysis. Most of these techniques do not provide a complete solution and are best regarded as judgment aids rather than complete analysis tools. For example, available analytical techniques cannot determine reliably whether a given channel modification will cause meander development, which is sensitive to difficult-to-quantify factors like bank vegetation and cohesion.

a. Additional Guidance. The information provided below is in summary form and is focused on allowable velocity and shear stress. More extensive information is available in numerous textbooks and manuals on mobile boundary hydraulics and sediment transport including EM 1110-2-1418 (USACE 1994a); Stability Thresholds for Stream Restoration Material (Fischenich 2001a); ASCE Manual No. 110, Chapter 9 (Shields et al., 2008); Design of Open Channels, Technical Release No. 25 (USDA 1977); and Chow (1959). Qualitative analysis of channel stability applicable for typical USACE projects is demonstrated in Appendix F. Locally or regionally developed approaches and data that have been found to give satisfactory results are often preferred over the more general approaches.

b. Sediment Budget. Channel stability is ultimately determined by the ability of the channel to pass the incoming sediment load while not scouring its bed. If sediment transport capacity is less than sediment supply, then the channel will aggrade. On the other hand, if the capacity is greater than the supply and the bed is alluvial then the channel will degrade. Aggradation or degradation potential in a channel reach requires an assessment of the reach-scale sediment budget. The sediment budget compares the quantity of sediment transported into the reach with the sediment transport capacity of the reach.

(1) The Sediment Impact Assessment Model (SIAM), a sediment budget tool also incorporated into the hydraulic design package in HEC-RAS, provides a fixed bed sediment analysis tool. Further information on SIAM is available in Chapter 9.

(2) In addition to channel processes, long-term sediment budget evaluation should also be performed at the watershed level as previously discussed in Chapter 6.

c. Allowable Velocity and Shear Stress Concepts. The concepts of allowable velocity and allowable shear stress are closely linked. They have been used mainly to design channels without boundary erosion. In channels transporting sediment, however, design should ensure that sediment outflow equals sediment inflow.

(1) Modifications of allowable velocity or shear stress to allow for sediment transport have been proposed in a few references but are of limited applicability. The allowable velocity approach is advantageous in that velocity is a parameter that can be measured within the flow. Shear stress cannot be directly measured; it must be computed from other flow parameters. However, shear stress is a better measure of the fluid force on the channel boundary than velocity. In addition, conventional stability analysis guidelines rely on the shear stress as means of assessing the stability of erosion control materials.

(2) Allowable Velocity.

(a) The concept of allowable or permissible velocity for various soils and materials dates from the early days of hydraulics. The allowable velocity is defined as the maximum velocity of the channel that will not cause erosion of the channel boundary. It is often called the critical velocity because it refers to the condition for the initiation of motion.

(b) Considerable empirical data exist relating maximum velocities to various soil and vegetation conditions. However, this simple method for design does not consider the channel shape or flow depth. At the same mean velocity, channels of different shapes or depths may have quite different forces acting on the boundaries. Critical velocity is depth-dependent, and a correction factor for depth must be applied in this application. Despite these limitations, maximum permissible velocity can be a useful tool in evaluating the stability of various waterways. It is most frequently applied as a cursory analysis when screening alternatives.

(c) The data in Table 7-8 give examples of simple allowable velocity data, which provide a guide to non-scouring velocity in FRM channels (EM 1110-2-1601). In the reference, the table is supplemented by graphical data for coarse gravel and boulder materials.

Table 7-8
Example of Simple Allowable Velocity Data

Suggested Maximum Permissible Mean Channel Velocities*	
Channel Material	**Mean Channel Velocity, fps**
Fine sand	2.0
Coarse sand	4.0
Fine gravel[1]	6.0
Earth	
Sandy silt	2.0
Silt clay	3.5
Clay	6.0
Grass-lined earth (slopes less than 5%)[2]	
Bermuda grass	
Sandy silt	6.0
Silt clay	8.0
Kentucky bluegrass	
Sandy silt	5.0
Silt clay	7.0
Poor rock (usually sedimentary)	10.0
Soft sandstone	8.0
Soft shale	3.5
Good rock (usually igneous or hard metamorphic)	20.0

Notes:
[1] For particles larger than fine gravel (about 20 mm = ¾ in.), see Plates 29 and 30 (EM 1110-2-1601).
[2] Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.
*From Table 2-5, EM 1110-2-1601.

(d) Figure 7-28 shows another example for allowable velocity, using data provided by the Soil Conservation Service (SCS), USDA (1977), presented by Shields et al. (2008). This discriminates between “sediment-free” and “sediment-laden” flow. Adjustment factors are suggested by the USDA for depth of flow, channel curvature, and bank slope. In this context, sediment laden refers to a specified concentration of suspended sediment.

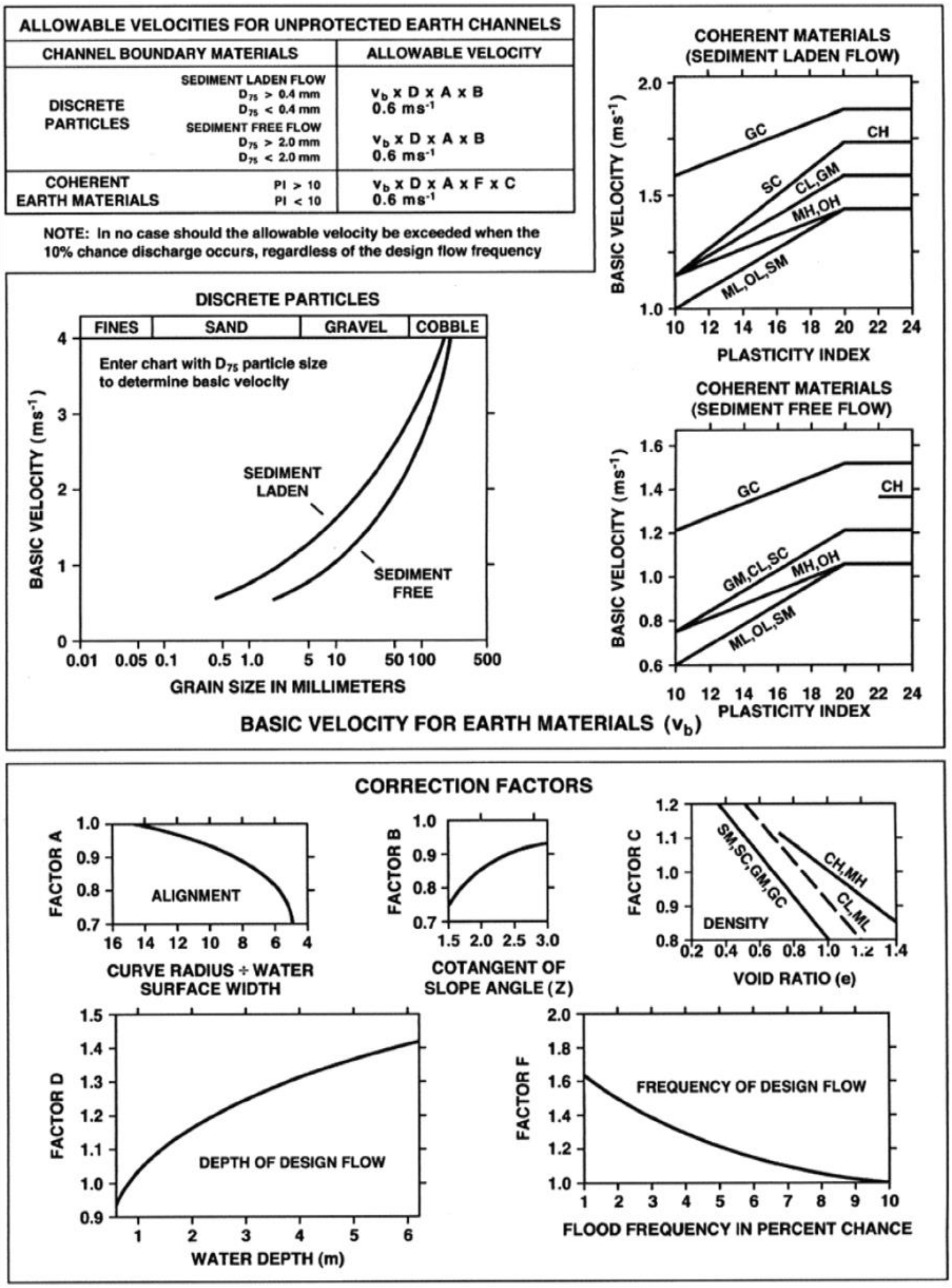


Figure 7-28. USDA (1977) allowable velocity charts for unprotected earth channels (from Figure 9-9, Shields et al., 2008)

Used with permission of ASCE, from Chapter 9, Sedimentation Engineering, Processes, Measurements, Modeling, and Practice, Manual No. 110, Shields et al., 2008; permission conveyed through Copyright Clearance Center, Inc.

(e) Allowable velocities are the maximum cross-sectional average flow velocities that do not cause serious boundary erosion. Allowable velocity for a given channel is determined using the formula in the box at the top left. The basic velocity, V_b, is given by one of the group of three plots at the top of the figure, whereas correction factors A, B, C, D, and F are obtained from the five plots in the bottom group. The letters PI = plasticity index, and the abbreviations GC, SC, CH, CL, GM, MH, OH, ML, OL, SM, etc. refer to the type of boundary as classified using the Unified Soil Classification System (Figure 9-9, Shields et al., 2008).

(3) Allowable Shear Stress.

(a) By the 1930s, shear stress (sometimes called tractive force) was generally accepted as a more appropriate erosion criterion. Unless otherwise noted, shear stress refers to the average shear stress along the bed (flow boundary) of a river. Refer to paragraph 5-2 and Figure 5-2 for information regarding the average boundary shear stress in uniform flow.

(b) The critical shear stress is defined as the shear stress where applied forces and resisting forces are balanced (paragraph 5-2). Shields (1936) determined the critical shear stress for noncohesive materials by measuring sediment transport for a range of shear stress values. Shields started with high shear stress conditions involving significant sediment transport and reduced the shear stress to the point at which sediment transport vanished. The shear stress at the point that sediment transport vanishes, and conversely the shear stress at the point of incipient motion, is the critical shear stress.

(c) Typically, the terms "allowable shear stress" and "critical shear stress" are used interchangeably since the allowable shear stress is usually viewed as not exceeding the critical shear stress. The term allowable shear stress will be used in the rest of this chapter unless the discussion is specifically about field or lab work conducted to determine the balance point of applied and resisting forces. Allowable shear stress values for noncohesive materials are usually presented in nondimensional form.

(d) The allowable shear stress concept has also been applied to semi-cohesive and cohesive soils, but values do not correlate well with standard geotechnical parameters because erosional resistance is affected by such factors as water chemistry, history of exposure to flows, and weathering (Raudkivi and Tan 1984). Analysis of experience with local channels and hydraulic testing of local materials are generally recommended. Empirical data for allowable shear stress versus grain size in canals are widely published, including EM 1110-2-1418. Figure 7-29 shows allowable shear stresses for a range of noncohesive and cohesive materials. Values should be compared against the results of field observation or laboratory testing.

(e) Figure 7-29 (left) shows recommended allowable unit shear stress values for canals in noncohesive material (USBR), and (right) allowable unit shear stress values for canals in cohesive material as converted from USBR data on allowable velocities (figures reported in Chow 1959).

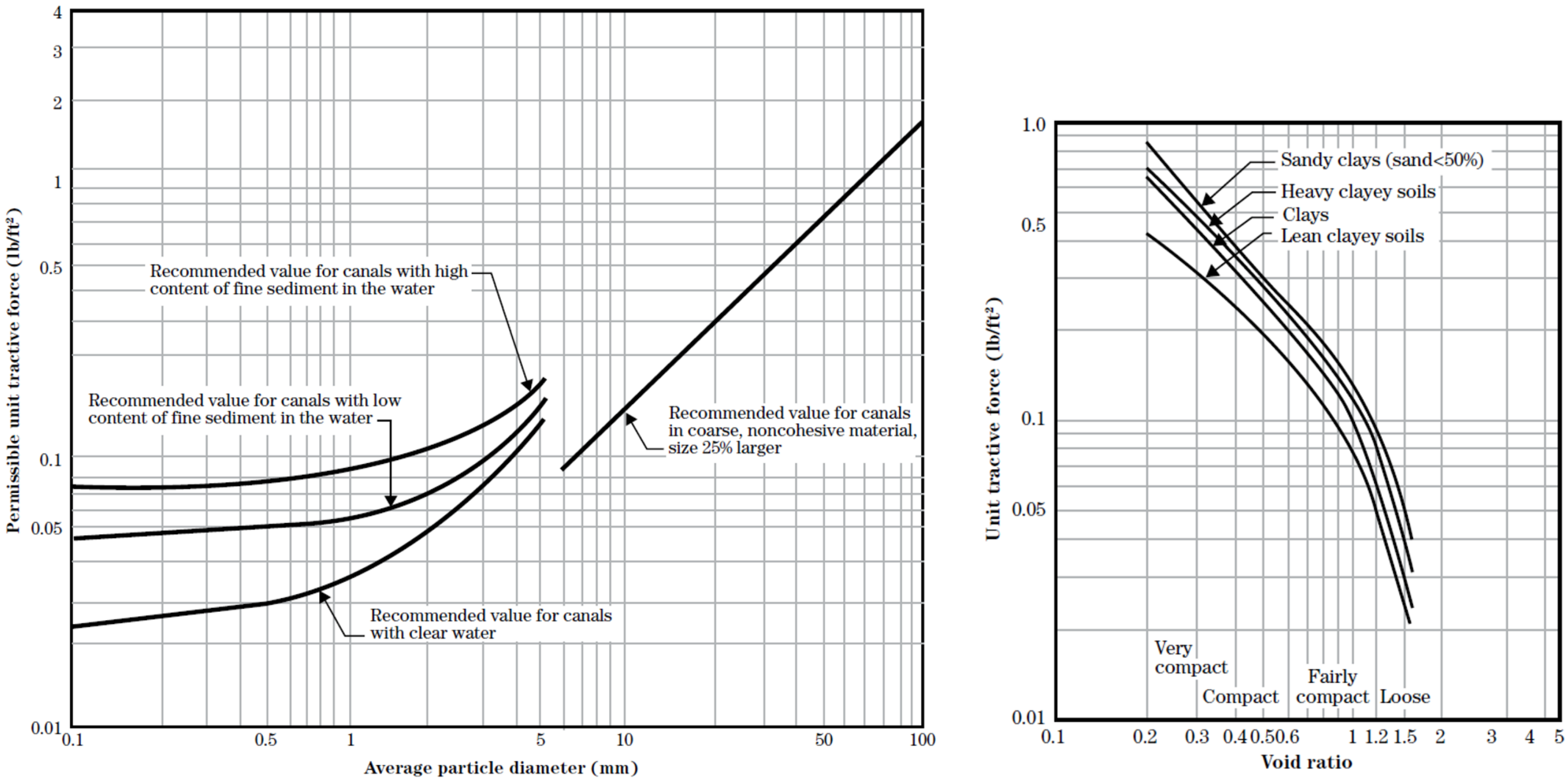


Figure 7-29. Allowable shear stresses (permissible tractive forces) (NRCS 2007)

Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

(f) Table 7-9 lists allowable shear stress and velocity values for uniform noncohesive sediments (also presented in Stability Thresholds for Stream Restoration Materials, TN EMRRP-SR-29 (Fischenich 2001a)).

Table 7-9
Allowable Shear Stress and Velocity for Uniform Noncohesive Sediments (after Fischenich, 2001a)

Class Name	D_s mm (inch)[1]	Φ(deg)[2]	τ_c	τ_{cr} (lb/sf)	V_{*c} (ft/s)
Boulder					
Very large	>2,048 (80)	42	0.054	37.4	4.36
Large	>1,024 (40)	42	0.054	18.7	3.08
Medium	>512 (20)	42	0.054	9.3	2.2
Small	>256 (10)	42	0.054	4.7	1.54
Cobble					
Large	>128 (5)	42	0.054	2.3	1.08
Small	>64 (2.5)	41	0.052	1.1	0.75
Gravel					
Very coarse	>32 (1.3)	40	0.05	0.54	0.52
Coarse	>16 (0.6)	38	0.047	0.25	0.36
Medium	>8 (0.3)	36	0.044	0.12	0.24
Fine	>4 (0.16)	35	0.042	0.06	0.17
Very fine	>2 (0.08)	33	0.039	0.03	0.12
Sands					
Very coarse	>1 (0.04)	32	0.029	0.01	0.07
Coarse	>0.5 (0.02)	31	0.033	0.006	0.055
Medium	>0.25 (0.01)	30	0.048	0.004	0.045
Fine	>0.125 (0.005)	30	0.072	0.003	0.04
Very fine	>0.0625 (0.003)	30	0.109	0.002	0.035
Silts					
Coarse	>0.031 (0.002)	30	0.165	0.001	0.03
Medium	>0.016 (0.001)	30	0.25	0.001	0.025

[1] Particle diameter D_s in mm (see table 3-1), tabulated inch diameter by Fischenich (2001a).

[2] Φ – Angle of repose for noncohesive sediment, τ_c Shields critical shear (see paragraph 5-2), τ_{cr} critical shear stress, V_{*c} critical shear velocity (Fischenich, 2001a)

(g) Table 7-9 lists limits that reflect idealized conditions for the stability of sediments in the bed. Sediment mixtures tend to behave differently than uniform sediments. Within a mixture, coarse sediments are generally entrained at lower shear stress values. Conversely, shear stresses larger than those listed in Table 7-9 are required to entrain finer sediments within a mixture (Fischenich 2001a).

(h) Cohesive soils and vegetation can be similarly evaluated to determine empirical shear stress thresholds. Cohesive soils are usually eroded by the detachment and entrainment of soil aggregates. Resisting forces are primarily the result of cohesive bonds between particles. The bonding strength, and hence the soil erosion resistance, depends on the physio-chemical properties of the soil and the chemistry of the fluids. Field and laboratory experiments show that intact, undisturbed cohesive soils are much less susceptible to flow erosion than are noncohesive soils.

(i) Vegetation has a profound effect on the stability of both cohesive and noncohesive soils. It increases the effective roughness height of the boundary, increasing flow resistance and moving higher velocities upward away from the soil, which has the effect of reducing the forces of drag and lift acting on the soil surface. Flippin-Dudley et al. (1998) presented an evaluation of flow resistance for vegetated channel and floodplains that examined different vegetation types.

- As the boundary shear stress is proportional to the square of the near-bank velocity, a reduction in this velocity produces a much greater reduction in the forces responsible for erosion.

- The roots and rhizomes of plants bind the soil and add erosion resistance. The presence of vegetation does not prevent soil erosion, but the critical condition for erosion of vegetated banks is usually the threshold of failure of the plant by snapping, stem scour, or uprooting, rather than from detachment and entrainment of the soils themselves.

- Soils protected by vegetation usually fail at much higher levels of flow intensity than for bare soil.

(j) Table 7-10 lists limiting values for shear stress and velocity for a number of different channel lining materials. Included are soils, various types of vegetation, and number of different commonly applied stabilization techniques. Information presented in the table was derived from a number of different sources. Ranges of values presented in the table reflect various measures presented within the literature (Fischenich 2001a).

Table 7-10
Allowable Shear Stress and Velocity for Selected Materials and Sediments (after Fischenich, 2001a)

Boundary Category	Boundary Type	Allowable Shear Stress (lb/sq ft)	Allowable Velocity (ft/sec)	Source[1]
Soils	Fine colloidal sand	0.02–0.03	1.5	A
	Sandy loam (noncolloidal	0.03–0.04	1.75	A
	Alluvial silt (noncolloidal)	0.045–0.05	2	A
	Silty loam (noncolloidal)	0.045–0.05	1.75–2.25	A
	Firm loam	0.075	2.5	A
	Fine gravels	0.075	2.5	A
	Stiff clay	0.26	3–4.5	A, B
	Alluvial silt (colloidal)	0.26	3.75	A
	Graded loam to cobbles	0.38	3.75	A
	Graded silts to cobbles	0.43	4	A
	Shales and hardpan	0.67	6	A
Gravel/Cobble	1 inch	0.33	2.5–5	A
	2 inch	0.67	3–6	A
	6 inch	2	4–7.5	A
	12-in.	4	5.5–12	A
Vegetation	Class A turf	3.7	6–8	C, D
	Class B turf	2.1	4–7	C, D
	Class C turf	1	3.5	C, D
	Long native grasses	1.2–1.7	4–6	E,F,G,D
	Short native and bunch grass	0.7–0.95	3–4	E,F,G,D
	Reed plantings	0.1–0.6	NA	C, D
	Hardwood tree plantings	0.41–2.5	NA	C, D

[1] Values consolidated by Fischenich (2001a), from A-Chang (1988), B-Julien (1995), C-Gray and Sotir (1996), D-Fischenich (2001a), E-Kouwen (1980), F-Norman (1975), G-Temple (1980).

(4) Cautions Regarding Allowable Velocity or Shear Stress.

(a) For channels with substantial inflows of bed material, a minimum velocity or shear stress to avoid sediment deposition may be as important as a maximum to avoid erosion. Such a value cannot be determined using allowable data for minimal erosion.

(b) Literature values previously presented generally relate to average values of shear stress or velocity. However, velocity and shear stress are neither uniform nor steady in natural channels. Flow pulses and 3D currents can give velocities or stresses of two to three times the average. While literature values were developed empirically and implicitly include some of this variability, natural channels typically exhibit much more variability than the flumes from which these data were developed.

(c) In bends and meandering channels, bank erosion and migration may occur even if average velocities and boundary shear stresses are well below allowable values. Conversely, deposition may occur in local slack-water zones even if average values are well above depositional minimums. EM 1110-2-1601 provides information on cross-sectional distributions of velocity and shear stress in bends.

(d) An allowable velocity or shear stress will not, in itself, define a complete channel design, because it can be satisfied by a wide range of width, depth, and slope combinations. It therefore has to be supplemented by additional guidelines for slope, width, or cross-sectional shape. In many channel modification cases, the slope will be predetermined within narrow limits, and practicable limits of width/depth ratio will be indicated by the existing channel.

(e) The Shields relationship (Figure 5-3) applies basically to uniform flow over a flat bed. However, sediment load influences the ability of flow to erode channel materials. In sand-bed channels especially, the bed is normally covered with bed forms such as ripples or dunes, and the shear stresses required for significant erosion may be much greater than indicated by the Shields diagram. Bed forms and irregularities also occur in many channels with coarser beds. Sediments in suspension have the effect of damping turbulence in the flow. Turbulence is an important factor in entraining materials from the channel boundaries. Thus, velocity and shear stress thresholds are often 1.5 to 3 times those presented in the previous tables for flows carrying high-sediment loads.

(5) Guidelines for Application (from EM 1110-2-1418, Chapter 5). The following guidelines are suggested for computations and procedures using allowable velocity and shear stress concepts:

(a) Determine cross-section average velocities and/or shear stresses over an appropriate range of discharges. Under overbank flow conditions, determine in-channel values, not averages, over a compound section. For existing channels, where possible, use stage discharge relations established from gaging stations or known watermarks, otherwise use hydraulic computations with estimated roughness. Stage-discharge relations in compound channels are reviewed by Williams and Julien (1989).

(b) A practical design approach for modification of existing channels is to match the velocity-discharge curve of the existing channel as far as possible by controlling cross section, slope, and roughness. Experience with response to local constrictions and widenings in alluvial channels generally supports this approach; these tend to scour or fill to restore more or less the natural velocity.

(c) In active alluvial streams, roughness may change appreciably between low and high stages (Figure 5-8). Bed roughness predictors (EM 1110-2-1601) can be used as a guide. For erosion checks it is conservative to estimate roughness on the low side, whereas for levee design it is conservative to estimate on the high side.

(d) If cross sections and slope are reasonably uniform, computations can be based on an average section. Otherwise, divide the project length into reaches and consider values for small, medium, and large sections.

(e) Determine the discharge for incipient erosion from the stage-velocity or discharge-velocity curve and determine its frequency from a flood-frequency or flow duration curve. This may give some indication of the potential for instability. For example, if bed movement has a return period measured in years, which is the case with some cobble or boulder channels, the potential for extensive profile instability is likely to be negligible. On the other hand, if the bed is evidently active at relatively frequent flows, response to channel modifications may be rapid and extensive.

d. Hydraulic Geometry. Hydraulic geometry theory assumes that a river system tends to develop in a predictable way, producing an approximate equilibrium between the channel and the inflowing water and sediment (Leopold and Maddock 1953).

(1) Hydraulic geometry theory typically relates a dependent variable, such as width, to an independent or driving variable, such as discharge or drainage area. Therefore, a primary weakness of hydraulic geometry theory is that dependent hydraulic design variables are assumed to be related only to a single independent design variable and not to any other design variables. In addition, USACE projects are often located in areas where the river system has been affected to a large degree and is not functioning in a sediment/water equilibrium.

(2) Hydraulic geometry analysis may be used in a geomorphic assessment of the study reaches to provide semiquantitative information on channel stability and sensitivity to change. Further discussion on hydraulic geometry relationships is available in Copeland et al. (2001), National Engineering Handbook Part 654 (NRCS 2007), and Soar and Thorne (2001).

e. Integration and Concept Application.

(1) A stability assessment of a channel system includes integrating the information from all the available analyses with respect to the channel bed and banks.

(2) Often the individual assessments produce contradictory results. For example, field investigations might indicate vertical channel stability, while computational analysis determined that the channel should be degrading. Assigning a level of confidence to the various components based on data reliability and analysis tool experience and performance may be used to reconcile contradictory results.

(3) The information gained from the channel stability assessment can be applied to determine potential evolutionary trends in the stream system. This depends on having a clear

understanding of the dominant geomorphic processes at work in the watershed, and a conceptual model of how the stream system reacts to imposed changes. For example, the incised channel evolution model (Schumm et al., 1984) is a conceptual model of the reaction of a stream system to a base-level lowering without changes in the upstream land use or sediment supply.

(4) Complex processes occur in many watersheds. For example, the effects of base-level lowering may be combined with other perturbations, such as increased runoff or decreased sediment supply. Multiple changes at various stages will cause a complex response in the stream system that is not described by a channel evolution model.

(5) Regardless, the stability evaluation should attempt, as much as possible, to develop a conceptual understanding which explains the historic and future evolution of the streams within the study watershed. There is no cookbook answer. Sound judgement based on insight and experience must always be incorporated when making a stability assessment.

7-10. General Design Guidance Related to Bank Protection, Aggradation, and Degradation. The following sections briefly discuss USACE project design features typically related to bank protection, aggradation, and degradation, and provide additional reference materials.

a. Design Features to Arrest Bank Erosion.

(1) Surface Armor.

(a) Surface armor is applied directly to the bank and includes stone armor such as riprap, other self-adjusting armor such as concrete or soil-cement blocks, rigid armor such as concrete and soil cement, and flexible mattresses such as concrete blocks and gabions.

(b) Toe protection is the key to successful surface armor performance. Studies should confirm that the bed profile is relatively stable before attempting bank protection with surface armor.

(c) Flexible or self-adjusting techniques are preferred over rigid techniques due to uncertainties associated with the future magnitude of additional local scour induced by the protection works.

(d) Surface armor may increase local turbulence because the streamlined bank may result in higher velocities. Care must be taken to ensure that local erosion is controlled at the upstream and downstream ends end of protection.

(e) Riprap design criteria in can be found in EM 1110-2-1601. Additional guidance for surface armor can be found in Chapter 7 of Biedenharn et al. (1997); and NRCS (2007), Chapter 14.

(2) Indirect Protection.

(a) Indirect protection techniques include permeable and impermeable dikes and retards, bank barbs, and bendway weirs. Indirect protection is used to alter thalweg alignment and is constructed away from the bank in such a manner to deflect or dissipate the erosive forces of the stream.

(b) Care must be taken to ensure that deflected currents do not induce erosion at some other location. Consequently, it is much more difficult to design indirect bank protection structures than surface armor because the 3D flow and sediment distribution has to be very carefully defined.

(c) Indirect protection is subject to increased maintenance due to drift accumulation.

(d) An extensive treatment of indirect protection methods used in smaller streams can be found in Chapter 8 of Biedenharn et al., 1997. Additional guidance can be found in Hydraulic Engineering Circular No. 23, FHWA 2009. Design guidance for navigation in large streams and rivers is contained in EM 1110-2-1611, Chapter 7.

(3) Grade Control.

(a) When banks are failing on both sides of a channel, the cause is typically due to system instability. The bed of the channel is degrading, which results in excessive bank heights—excessive in the sense that the soil properties are unable to sustain the banks over time. When this occurs, bank instability may be widespread throughout the system. Traditional bank stabilization measures may not be feasible where systemwide bank instabilities exist, and grade control may be an appropriate solution.

(b) Grade control structures can enhance the bank stability of a stream in several ways. Bed control structures indirectly affect the bank stability by stabilizing the bed, thereby reducing the length of bankline that achieves an unstable height. Two additional bank stability advantages are that bank heights can be reduced due to sediment deposition upstream of the structure, increasing bank stability, and by reducing upstream velocity and scouring potential (Thorne 1990). Therefore, if systemwide bank instability is a significant concern, consideration might be given to raising and/or constricting the grade control weir to promote bank stability.

(c) Grade control is an effective method to arrest channel degradation, and it is often appropriate to use in conjunction with surface armor and/or indirect protection to protect channel banks. A series of well-designed grade control structures can adjust sediment transport capacity to sediment supply and can improve bank stability by reducing bank height and reducing shear at the bank toe. Additional guidance is provided in the following section on design features to control degradation.

(4) Bioengineering.

(a) In situations where erosive forces are small or can be controlled to some extent, live vegetation can be effectively used to protect banks. Rock or wood structures and bioengineering products that encourage vegetation growth are often used in conjunction with vegetation.

(b) Streams with well-stabilized vegetation on their banks enhance water quality and habitat for both fish and wildlife.

(c) Successful application of vegetative bank protection requires stabilization of the toe. Bioengineering projects typically consist of a combination of rock toe protection, some type of temporary soil stabilization measure, such as fabric or wattles, and native vegetation planting.

(d) Guidance for bioengineered bank protection can be found in Allen and Leech (1997); Fischenich (2001); Federal Highway Administration HEC-23 (2009); NRCS (2007), Appendix 14 I; and USBR (2015). Guidance for incorporation of vegetation into the design and maintenance of FRM channels and stream restoration projects can be found in Fischenich and Copeland (2001).

b. Design Features to Control Aggradation. USACE engages in preventing aggradation and removing aggraded material when it impacts navigation or FRM projects, or when special authorities have been assigned by Congress. Ecosystem restoration projects must also consider the possibility of project-induced aggradation. The approaches are stable channel design, debris basins, maintenance dredging, and reduction of incoming sediment load by stabilizing upstream channels or watersheds.

(1) Stable Channel Design. Aggradation will not occur if the sediment transport capacity of the project channel is the same as the upstream supply reach. Although this goal cannot always be achieved, there should be an attempt to minimize changes in sediment transport capacity throughout project reaches. Maintenance requirements due to induced aggradation can be estimated using approaches outlined in TR-01-28 (Copeland et al., 2001).

(2) Debris Basins. Debris basins are common features at the upstream end of fixed bed channels, especially supercritical-flow concrete channels. Chapter 8 of this manual discusses the design of debris basins.

(3) Maintenance Dredging. Periodic dredging may be feasible to address project impacts due to aggradation. Long-term maintenance requirements in dredged channels can be determined using numerical models such as HEC-6T and HEC-RAS.

(4) Watershed Sediment Management.

(a) USACE has implemented the Regional Sediment Management (RSM) program, an approach for managing sediment projects that incorporates many of the principles of integrated watershed resources management (USACE 2002b). USACE has received multiple congressional authorities that support watershed approaches in various Water Resources Development Acts and that provide authority to assess the water resource needs of river basins and watersheds. Authorities include (1) section 2037 of WRDA 2007 pertaining to regional sediment management as well as watershed and river basin assessments; and (2) section 202 of WRDA 2000, Watershed and River Basin Assessments, which amended section 729 of WRDA 1986.

(b) Although not typically an authorized function in USACE projects, sediment management practices in the watershed upstream from the project can be effective in reducing sediment yield. In-channel techniques, such as grade control and streambank protection, and land management practice adjustments can be an effective part of a watershed sediment management plan. Techniques such as these have been applied successfully in the Delta Headwaters Project in Mississippi (paragraph 7-10d).

c. Design Features to Control Degradation. USACE engages in preventing degradation when it impacts navigation, flood risk management, environmental enhancement projects, or when special authorities have been assigned by Congress. Frequently, projects are designed to convey high discharges at lower elevations, which can increase channel velocities and possibly increase degradation potential. Project-induced degradation can damage or destroy other project features such as bank protection, in-stream structures, and infrastructure. Design approaches that address degradation include stable channel design and grade control. There are a variety of grade control options, including high-drop structures, low-drop structures, low-water weirs, and hard points.

(1) Stable Channel Design Applied to Degradation. Degradation will not occur if the sediment transport capacity of the project channel is the same as the upstream supply reach. Although this goal cannot always be achieved, there should be an attempt to minimize changes in sediment transport capacity though project reaches. Stable-channel geometry can by analytically determined using approaches outlined in TR-01-28 (Copeland et al., 2001). Modules in SAMwin and HEC-RAS can be used to make these computations for a representative cross section. A more rigorous computational approach using a 1D model, such as HEC-RAS or HEC-6T, should be conducted in cases where degradation potential is significant.

(2) Grade Control Structures. Grade control is often an essential component to stabilize a degrading stream or one that is subject to conditions that may cause degradation.

(a) Basic Types. There are two basic types of grade control structures: (1) a bed control structure, and (2) a hydraulic control structure. A bed control structure is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the degradational zone. A hydraulic control structure is designed to function by reducing the energy slope along the degradational zone to the point that the stream is no longer capable of scouring the bed. Due to the complex hydraulic behavior of grade control structures, it may be difficult to distinguish between the two types because a specific structure may have characteristics of both. The characteristics of a grade control structure can change with discharge and over time.

(b) Methods. The most grade control methods are the construction of in-channel structures or full channel lining. Grade control may be provided using large or small structures with a crest elevation raised or at the existing bed elevation. Large structures may be channel and flood plain spanning structures that incorporate concrete and large rock riprap. Smaller scale grade control structures include rock riffles and stone weirs. These structures are often shaped to focus energy toward the channel center to reduce bank erosion, and can also be configured to provide environmental benefits. Based on the structure height, the use of a sheet pile cut-off may be required to seal the structure preventing flow-through failure. Performance during high-flow events can be an issue if hydraulic jumps occur and the downstream scour at each structure is not addressed.

(c) Design Requirements. Primary design requirements for grade stabilization methods are determining the slope required for stability, designing the grade control structure system to not cause an increase in flood levels, and preventing the migration of bed or bank stability issues upstream or downstream of the structure. Continuity of water and sediment through the project reach must be provided. A series of well-designed grade control structures can adjust sediment transport capacity to sediment supply and can improve bank stability by reducing bank shear stress.

(d) Impacts. Streambank stabilization affects many of the structural characteristics and functions of a stream. The basic purpose of any stabilization project is to interrupt erosion processes where they are deemed to conflict with social needs or ecological requirements. These efforts also interrupt or affect other processes and alter the physical environment. Grade control structures modify water surface elevation, velocity, sedimentation processes, and sediment transport over a significant distance. Fischenich (2001b) presented a review of impacts of stabilization measures.

(e) Hydraulic Design of Grade Control Structures. Grade control design includes determining the height, spacing, and dimensions and composition of the control section, accounting for energy dissipation and providing for protection at the abutment, approach, and exit. Determining the height and spacing requires an evaluation of stable channel geometry and may require a sediment transport analysis. Design details of several different types of grade control structures is available in EM 1110-2-1601 and in Biedenharn et al. (1997). Biedenharn and Hubbard (2001) present criteria for structure siting. Stream Restoration Manual, National Engineering Handbook Part 654 (NRCS 2007) presents environmental feature guidance such as fish passage and habitat, along with recreational and aesthetic considerations.

(f) Design Slope, Height, and Spacing of Grade Control Structures. The height and spacing of grade control structures depends on the selected design slope of the project reach. The design slope should be set equal to the equilibrium slope, the slope that will provide the sediment transport potential to carry the inflowing sediment load through the reach without degradation or aggradation.

- The required boundary condition variables to determine height and spacing are the full range of inflowing water discharges, the concentrations, by size class, of the inflowing bed-material sediment and the resistance to erosion of particles on the channel bed. It is not satisfactory to assume historical concentrations and particle sizes when designing drop structures, because the structures, if they are successful, will reduce the sediment concentration and may even alter particle size distributions. Numerical models such as HEC-6T and HEC-RAS provide the computational framework for setting height and spacing.

- One method for designing the spacing of grade control structures can be determined by extending a line from the top of the first structure, at a slope equal to the design slope, upstream until it intersects the original streambed (Figure 7-30). Theoretically, the siting of grade control structures is straightforward and can be determined by the following equation:

$$H = \frac{S_o - S_f}{L} \qquad \text{Equation 7-4}$$

where:

H = the amount of drop to be removed from the reach
S_o = the original bed slope
S_f = the design slope (set equal to the equilibrium slope)
L = the length of the reach

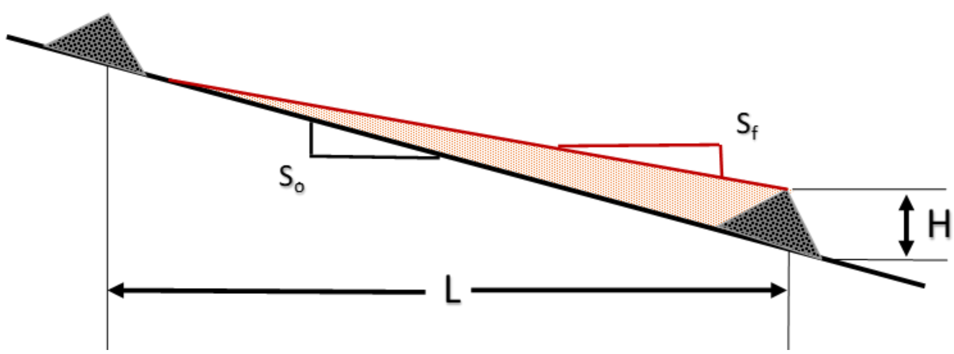


Figure 7-30. Spacing of grade control structure (Biedenharn et al., 1997)

- The method shown in Figure 7-30 was used to design grade control structure spacing in the Delta Headwaters Project (DHP) in Mississippi. The DHP is described in numerous design and monitoring reports that includes sedimentation process and design overview (USACE 1981c; Biedenharn et al., 2000; Martin et al., 2010).

- For riffle spacing, Newbury suggests spacing that varies with location as shown Figure 7-31.

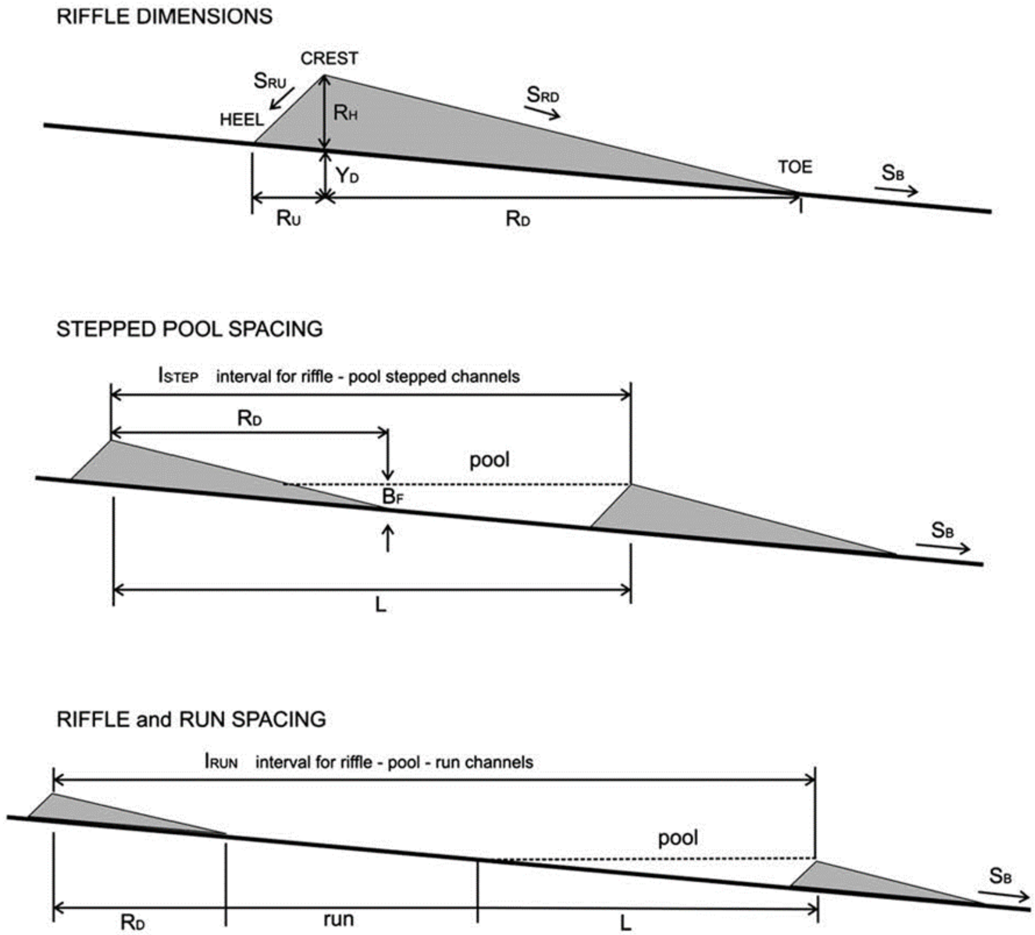


Figure 7-31. Riffle, run, and pool dimensions (Newbury 2013)

Used with permission of Taylor and Francis, from Canadian Water Resources Journal, Newbury, R.W., Vol. 38, ed. 3, 2013; permission conveyed through Copyright Clearance Center, Inc.

Note: Newbury notes that riffle spacing may be adjusted to create stepped pools or run and pool profiles typically found in meandering channels. The downstream slope S_B is shallow (< 0.01) to allow the flow to decelerate in an undulating surface hydraulic jump in the run or pool below.

Where riffle dimensions:

R_H = riffle height above channel bed
S_B = channel bed slope
S_{RU} = slope of the upstream riffle face
S_{RD} = slope of the downstream riffle face

R_U = distance of heel to crest $R_U = \frac{R_H}{(S_{RU} + S_B)}$ Equation 7-5

R_D = distance of crest to toe $R_D = \frac{R_H}{(S_{RD} - S_B)}$ Equation 7-6

Y_D = height of bed at the crest above the toe $Y_D = R_D \, x \, S_B$ Equation 7-7

where Total elevation drop is sum of Y_D and R_H

Riffle spacing:

L = pool length with no back − flooding $L = \frac{R_H}{S_B}$ Equation 7-8

B_F = height of back-flooding on upstream riffle

I_{step} = interval between crests with back − flooding $I_{step} = L - \frac{B_F}{S_B} + R_D$

Equation 7-9

I_{run} = interval between crests with run and pool $I_{run} = L + R_D + run$

Equation 7-10

- In addition to the above, there are many other methods for determining the design slope for a channel. The decision to use one method or another depends on several factors, such as the level of study (sediment impact assessment or detailed design), availability and reliability of data, project objectives, and time and cost constraints. In some cases, the design slope may be based solely on field experience with similar channels in the area. Initial planning level procedures may involve using empirical regime relationships in alluvial streams or tractive force or permissible velocity methods in threshold streams.

- The design analysis should include confirmation that sufficient sediment will be supplied from upstream to establish the design slope over time. Upstream channel modifications can reduce sediment supply, which could lead to the equilibrium slope being attained by degradation (Figure 7-32). The potential for this condition to occur increases with decreased height of the grade control structure. This condition would create a decrease in bed elevation at the downstream end of the upstream structure, which could lead to undercutting.

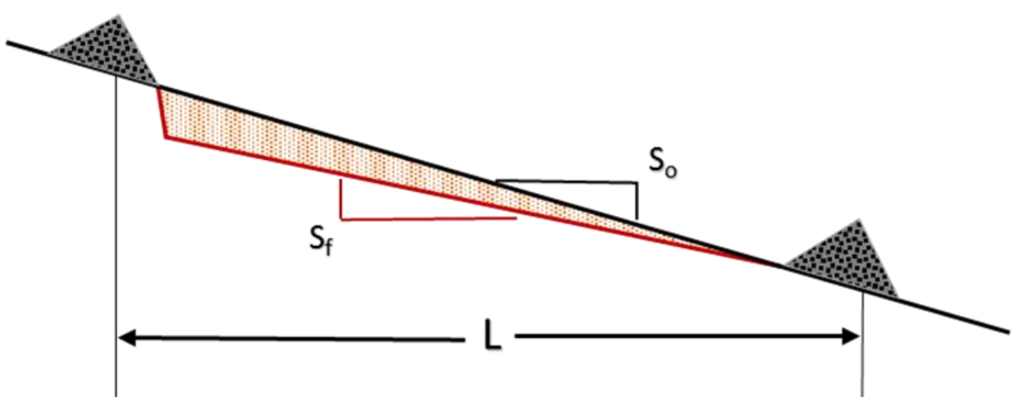


Figure 7-32. Possible effect of reduced sediment supply on grade control slope

- Since determination of the design equilibrium slope is often subjective to numerous parameters that are difficult to assess (like sediment inflow and initiation of transport), include provisions to increase structure reliability such as increasing the length, thickness, and toe protection provided by energy dissipation features.

- Some designs include provisions to focus energy along the channel centerline via a lowered weir section or similar. This may further reduce bank erosion and align approaching flow direction. However, additional centerline scour may affect the structure and downstream channel. This should be addressed with relevant scour protection features. Inclusion of a lowered center section should be examined with site-specific analysis to evaluate performance. Regardless of the procedure, the engineer must recognize the uses and limitations of that procedure before applying it to a specific situation.

(g) Examples of grade control structures are shown in Figure 7-33 and Figure 7-34.

Figure 7-33. Soil cement grade control structure, Rio Salado, Phoenix, Arizona

Figure 7-34. High drop, Hotophia Creek, north Mississippi

(3) Low Weirs. Low weirs are minimal-height channel structures that can provide environmental enhancement by creating aquatic habitat during low-flow periods when the channel would normally be dry, and can maintain conveyance by keeping sufficient channel water depth to prevent the growth of obstructive vegetation. Weir height is normally less than one-third of the tailwater depth at the project design flow line to provide minimal head loss at the design flow. However, at low flows, the low weir acts like a drop structure and must be designed accordingly (Ables and Boyd 1969). Protection must be provided to the stream bank tie-in to prevent local erosion and flanking. Figure 7-35 shows a low weir example at which sediment supply is insignificant and sediment does not deposit upstream from the weir crest.

Figure 7-35. Low weir, Big Creek, Louisiana

(4) Hard Points. Channel grade control can be constructed at the existing bed elevation to arrest degradation. Hard points can be constructed of rock, concrete, sheet piling, or other available materials. Typically, scour occurs downstream from these structures at low flow. Design should include sufficient toe-down depths, both upstream and downstream, to preclude scour from undermining the structure. The structure should be appropriately keyed into the banks to prevent flanking. The overall configuration of the structure should be modified to accommodate fishery and/or recreational issues. Multiple shorter height stabilizers can be designed in lieu of using one large structure. Shorter height structures may increase total project construction cost but reduce the drop at each individual structure which can improve structure reliability and reduce impacts.

(a) Application at Mill Creek. An example of using hard points in a FRM project channel is Mill Creek in Walla Walla, Washington. The bed of Mill Creek is composed of sand and gravel. The project design included concrete-capped wire-bound gravel stabilizers constructed at intervals varying between one-half to one times the channel width. The stabilizers were initially constructed at the design channel grade.

- At the upstream end of the project is a diversion dam designed to divert both peak flood flows into an offstream storage reservoir and irrigation water onto adjacent farmland. The reservoir reduces sediment supply to the FRM project channel, inducing degradation during high flow. Consequently, an adverse slope developed upstream from most of the stabilizers, with a significant local scour hole at the downstream face of each stabilizer. Fortunately, a sheet pile cutoff wall at each stabilizer prevented failure of most of the structures.

- A physical section model study of the Mill Creek FRM project (Robinson and Copeland 1986) revealed several significant conclusions related to hard point stabilizers:

– Since the final bed profile is significantly dependent on the sediment inflow, one cannot assume a positive (or even a horizontal) slope upstream from the hard point. Armoring of the stream bed significantly reduces bed scour. Therefore, the practice of bulldozing bed material into local scour holes after floods, which destroys the armor layer (see paragraph 8-6c), results in greater long-term degradation.

– Numerical sedimentation modeling using HEC-6 predicted similar volumes of degradation. While the HEC-6 results are informative, the physical model results were used to improve accuracy of predicted local scour depths critical to design of apron protection at the stabilizer (Figure 7-36). Design of USACE hydraulics structures should consider accuracy of methodology and include project features to mitigate risk, as necessary.

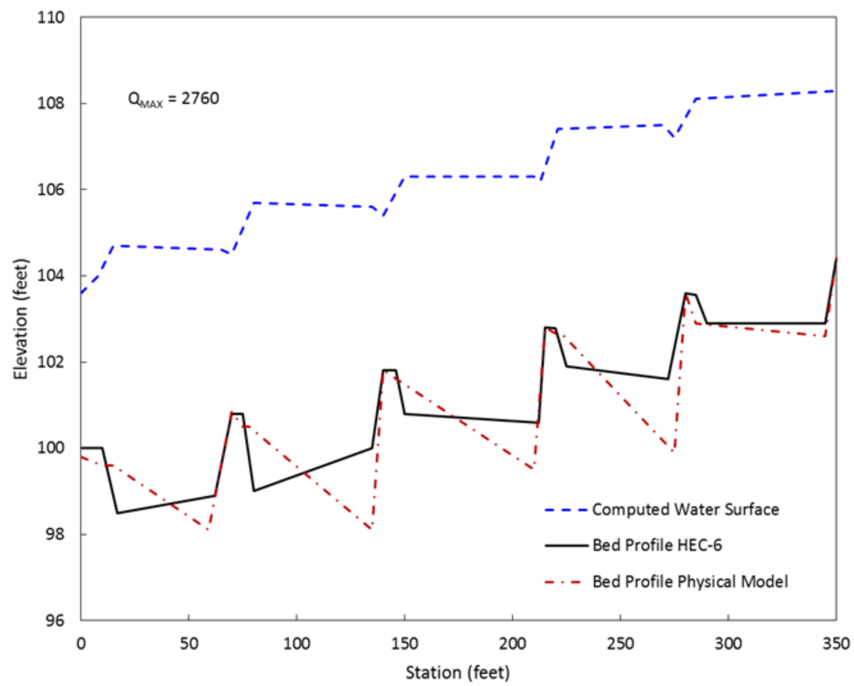


Figure 7-36. Mill Creek, comparison of HEC-6 and physical model bed profiles after 1945 flood (redrawn from Robinson and Copeland 1986)

(b) Additional Applications. Figure 7-37 shows an example of using multiple stabilizers that use a series of masonry stabilizers. In this application, the crest centers are a bit lower than the edges, which helps to concentrate low flows to the center of the channel. Figure 7-38 points to the importance of providing adequate toe-down depths and keying the abutments into the banks. Figure 7-39 shows how rock stabilizers are frequently used in gravel-bed channels to achieve grade control and to improve aquatic habitat.

Figure 7-37. Masonry stabilizers, Wilson Creek, Yucapia, California

Figure 7-38. Stabilizers are ineffective if flanked or undercut

Figure 7-39. Engineered rock riffle, Sinsinawa River, Menominee, Illinois

d. Example Program Applications. USACE has conducted multiple programs under specific congressional authorizations, as well as research activities directed at sedimentation processes. Examples of two large-scale authorities that led to project construction and monitoring are provided with a brief discussion of design features follows. Both of these programs provide a wide range of stabilization structures and information on performance.

(1) The Section 32 program was authorized by the Stream Bank Erosion Control Evaluation and Demonstration Act of 1974 (Section 32, Public Law 93-251) and reported by USACE (1981b). The legislation authorized a 5-year program, which, among other things, consisted of an evaluation of existing bank protection techniques, construction of demonstration projects, and monitoring the projects to determine the most promising methods. The final report is quite extensive and comprehensive. Copies of the report and its various appendixes are available from the National Technical Information Service in Springfield, Virginia (USACE 1981b).

(2) Delta Headwaters Project (DHP), formerly known as the Demonstration Erosion Control (DEC) Project), was authorized by several congressional actions in the 1980’s. A joint project with USACE and the USDA, the program provided for the development of a system for control of sediment, erosion, and flooding in the foothills area of the Yazoo Basin, Mississippi. Activities were targeted at 16 watersheds comprising 2,600 square miles. Rehabilitation included

grade control structures, pipe drop structures, bank stabilization, bio-engineering techniques, and a combination of detention and retention reservoirs. The DHP is described in numerous design and monitoring reports that includes sedimentation process and design overview (USACE 1981c; Biedenharn et al., 2000; Martin et al., 2010).

7-11. Flood Risk Management Projects.

a. FRM Sedimentation Response. Design features associated with FRM project formulation include channel deepening/widening, levees, flood walls, reduced hydraulic roughness, channelization, cutoffs, and diversions. These project features often cause a significant impact on sediment processes. Impacts can extend throughout the project reach, upstream, downstream, and within tributaries. Analysis should be performed to evaluate the impact of FRM project features on sediment processes.

(1) Typical FRM project objectives are to reduce damages by confining flows with levees/flood walls or lowering flood stages with methods including storing flood flows, diverting peak flood flows around the protected area, channelization, and reducing hydraulic roughness.

(2) FRM projects often impact, to some extent, one or more of the six degrees of freedom of the natural river (channel width, depth, slope, hydraulic roughness, planform, and lateral movement). FRM project sedimentation impacts should be addressed in project design, maintenance, and adaptive management. Project features to address sedimentation (such as debris basins, stability features, sediment removal to maintain capacity) and maintenance costs must be captured in the planning phase benefit-cost ratio and addressed in the risk register.

(3) Project alternative formulation should include sufficient detail to identify sedimentation-related issues and the design of necessary mitigation features.

(4) Minimizing conveyance changes to leave the natural river corridor as untouched as possible generally reduces the risk of unexpected or dramatic geomorphic response. Constructed projects can result in large sedimentation imbalances and have the potential for significant stage-discharge shifts during the project life. Projects that include features such as levees constructed near the channel bank, channel widening or deepening, and flow diversions can significantly alter existing conveyance. Projects that increase channel velocities may exacerbate bank erosion and long-term degradation problems. Lower river levels may also affect floodplain wetlands and groundwater levels with associated environmental impacts.

(5) In an overview of stability problems associated with FRM channel projects (USACE 1990), major potential stability problems were summarized as:

(a) When depths are increased but the original slope is maintained, velocities at the higher discharges will be increased and the bed and banks may erode. The erosion response will be increased if bank stability depended on features removed for the project construction, such as cohesive sediment deposits, armoring (paragraph 8-6c), or vegetation.

(b) When the cross-section enlargement is too large and there is a substantial sediment transport load, the cross section may partly infill with sediment deposits and the calculated flood capacity may be compromised in a single event.

b. Key Locations. Not all locations in a project are equally likely to experience sedimentation problems. Problems are likely to start at the following locations:

(1) Braided channels.

(2) Changes in channel width.

(3) Bridges or other structures built across the stream.

(4) Channel bends.

(5) Abrupt changes in channel bottom slope.

(6) Long, straight reaches.

(7) Tributary and local inflow points.

(8) Diversion points.

(9) Upstream from reservoirs or grade control structures.

(10) Downstream from dams or grade control structures.

(11) The downstream end of tributaries.

(12) The approach channel to a project reach.

(13) The exit channel from a project reach.

c. General Design Considerations. General guidance for design considerations are as follows:

(1) FRM project works should be carried sufficiently far upstream, and consideration should be given to trapping or removal of coarse sediment upstream of the FRM zone.

(2) Debris basins, debris barriers, constructed levees, erosion protection features, and constructed channels may require considerable maintenance to achieve performance objectives.

(3) Sediment deposits in channels, without debris basins, were found at tributaries and other locations where there was a decrease in energy slope. USACE or natural features incorporated in the project that cause the inflowing water to slow will probably produce sediment deposition.

(4) Avoid project components that require flows to change direction or change velocity quickly in a short distance. Sediment erosion, deposition, channel avulsion, and possible structural failure may result from attempting to force flows to change their course abruptly.

(5) Levees may also cause channel sedimentation in streams with high-sediment loads by restricting transport and deposition of sand on the overbank areas. More sand is then retained in the channel to deposit further downstream in reaches of flatter slope. Additional items that should be accounted for in the total levee height include: (1) sediment deposition, (2) superelevation/surface wave formation, (3) bed form (anti-dune height), and (4) residual freeboard (Philips and Williams 2008).

(6) At flow diversions, the division of sediment between channels is not necessarily proportional to the division of flow. Further sedimentation problems may arise if there are substantial downstream inflows of sediment that the reduced flows (after diversion) are unable to transport. When flood flows are diverted into a channel, but the channel is not deliberately modified to accommodate the increased discharges, serious erosional problems may ensue. The channel tends to respond by widening and deepening, and by flattening its slope through upstream degradation and downstream gradation.

(7) Critical detention storage factors are size, outlet works, and downstream protection. Storage capacity should be sufficient good performance during large flood events. When storing sediment, the downstream channel often must be protected from the relatively clearer outflow. Channel protection is usually required in the vicinity of the overflow spillway.

(8) Debris barriers have also been effective in keeping some of the debris from moving downstream, as shown by their extensive use in southern California. While they are not considered effective for FRM due to limited storage capacity during extreme events, they may be considered as a component of a total plan. Even when full, they have been credited for reducing the stream slope and thus reducing the rate of debris movement downstream.

d. Extreme Events. Design should examine how extreme events may alter sedimentation processes and consequences to USACE flood damage reduction project performance. A significant shift in sedimentation processes or crossing a stability threshold during an extreme event may shift the project stage-damage relationship. USACE FRM project designs are based on detailed hydrologic and hydraulic modeling. Robust design methods, as explained in this manual, are employed to reduce the risk of project nonperformance due to unanticipated hydrologic and sedimentation processes. Chapters 9 and 10 present modeling and risk evaluation guidance. Case Study 10A (Appendix N) presents additional information regarding extreme event consequences.

e. Determining the Boundary of the Study Area.

(1) The study area for a FRM project is always larger than the project area itself. The study area needs to include the watershed area that will be affected by the project. The limits of the study area are often difficult to determine because project effects can extend for a

considerable distance upstream and downstream. The effects may also extend up tributaries. Consequently, a large area can be affected by changes along one reach of a stream.

(2) In some instances, the boundary may be well defined by control points such as dams or geologic controls. In most instances, the study boundary will not be well defined, and the engineer must estimate the extents. In these cases, the final boundary must be selected after consideration is given to the historical behavior of the river, current behavior, the relative size of the project and the type, amount and location of available data, and the results of modeling and analysis.

(3) Data collection is a critical component of a successful study. When insufficient data is available for areas outside of the project boundary, these areas cannot be included in the analysis. However, necessary analysis in the upstream and downstream reaches cannot be neglected. Data collection or other suitable methods to address missing parameters is recommended to reduce project performance risk. Schedule and cost for data collection efforts should be identified in the sediment studies work plan (Chapter 2).

(4) The decision to include or exclude specific reaches for detailed analysis will likely depend on how much the proposed project features deviate from the characteristics of the natural river.

(5) If the project reach is on a small tributary to a larger stream, it may have no effect on the larger stream even though the project causes drastic changes to the tributary. For example, if a tributary contributes 2% to the total sediment discharge of its receiving stream, it would be unlikely that a project that doubled this contribution to 4% would have any significant effect on the receiving stream.

(6) Project design features should be included to ensure that hydraulic conditions in the reaches upstream and downstream from a project retain pre-project hydraulic and sedimentation conditions. Figure 7-40 illustrates degradation downstream of the lined channel which likely occurred due to high exit velocities and upstream sediment trapping in a debris basin. Providing project features to accommodate changes in sediment continuity is recommended to reduce potential project impact on the river system (Lane's balance, paragraph 7-3g(5)).

Figure 7-40. Degradation downstream of channel exit,
Iao Flood Risk Management Project, Maui, Hawaii

f. General Sedimentation Concerns of Typical Project Features. USACE FRM projects usually alter the stage-flow-damage relationship. As a result, significant impacts can occur to ongoing sediment processes that can have severe consequences to project performance. Although design is site specific, general guidance for sediment concerns related to typical USACE project features to reduce flood risk is provided below. Each project will have unique considerations. Items are not listed in order of USACE implementation preference or potential sedimentation issue magnitude.

(1) Influence on Hydrology. The design flow for a typical FRM project differs greatly from the more frequent flows that shape the channel. However, extreme events can significantly alter the river corridor with the potential to seriously damage USACE FRM project features. Therefore, the full range and sequence of flow conditions should be evaluated in a sediment study. Possible hydrologic responses to project features that also will affect sedimentation include:

(a) Projects can have significant impacts on storage and channel conveyance. Flow changes can be magnified by changes in timing. Evaluation of storage impacts and timing is determined with routing computations in an unsteady flow model such as HEC-RAS.

(b) Hydrologic impacts often extend a significant distance both upstream and downstream of the constructed project. The magnitude of hydrologic change should be considered when determining sedimentation study boundaries.

(c) Timing of peak flows may also be affected by project features. Large scale projects may alter timing such that peaks tend to combine more often downstream of the project which can raise water levels. A coincident flood analysis may be required.

(d) Storage may be reduced when features such as levees or channel improvement constrict the available floodplain area. Storage may be added if project features raise water levels and induce upstream storage. However, in this case, areas upstream of the project may be negatively impacted with higher water levels and longer duration flooding.

(e) Projects that decrease river levels may decrease floodplain storage and consequently increase downstream flow peaks and aggravate ongoing stability issues. Unless the leveed reach eliminates a significant portion of the floodplain storage and/or conveyance, the effects on peak discharge will likely be negligible (for example, a small community ring levee vs. many miles of a large river levee system).

(f) Channelization may increase channel velocities during floods, which decreases the time of concentration of flood peaks. Consequently, downstream peak flows may be increased.

(g) The influence on hydrology can be significant when the project lowers water levels over a large area in the watershed. Decreased river levels will decrease floodplain storage and percolation that will alter flood runoff.

(2) Potential Sedimentation Issues. Many USACE projects alter sediment processes. Areas with ongoing sediment issues can be magnified by the FRM project. Typical project sedimentation issues include:

(a) Projects often alter several components of Lane's balance (paragraph 7-3g(5)) with significant effect on both flow and sediment supply. Both aggradation and degradation are potential responses.

(b) Examples of post-project aggradational trends are numerous on FRM levee projects. Pre-project warning signs include aggradational stage trends, the presence of near bank natural levees, and rivers with higher sediment concentrations.

(c) The post-project channel flow percentage may increase during floods to the point of causing bed degradation. If the channel is a threshold channel, allowable velocity or allowable shear stress methods may be suitable to evaluate project condition stability. Alluvial channels should be evaluated using a numerical sediment model such as HEC-RAS or HEC-6T.

(d) Projects that raise river levels, such as a levee, will reduce flow velocity and induce deposition upstream of the project.

(e) Ongoing floodplain aggradation can be exacerbated by project construction. For instance, a levee project that causes a significant floodplain conveyance reduction will concentrate sediment deposition within a smaller floodplain area.

(f) At the downstream transition from the project reach to the natural channel, an increase in energy slope during floods often occurs due to the project hydraulic impacts. This often results in an erosion zone followed by deposition. If the erosion is significant, bed and bank protection in the exit channel should be included in the project plan.

(g) Projects that lower the stage-discharge curve below the existing condition create the potential for upstream degradation and headcuts. Tributary streams also have the potential for headcuts because of the lower base level in the receiving stream.

(h) Evaluate existing planform stability and bank migration. The historic bank erosion and migration rates will probably continue and may be increased by project impacts on sedimentation. Protection features may be required when the migrating bank is close to critical infrastructure and project features such as a levee.

(i) Features can be damaged or destroyed by degradation induced by the project itself or by downstream degradation that migrates into the project reach.

(j) Lateral migration of the river channel and bank erosion may damage project features and allow flood waters to inundate adjacent areas.

(3) Long-Term Effects. Aggradation can affect both project channel maintenance requirements and the long-term level of protection. These factors must be considered in the long-term assessment of project benefits.

(a) Sediment deposition and erosion may be different from historic rates because of project impacts on sediment transport. Long-term aggradation or degradation in the project reach should be calculated using a numerical sedimentation model such as HEC-RAS or HEC-6T.

(b) Numerical simulations for a period of record (POR) that is 50 years or greater is recommended. The POR is generally based on historic flow data but should also assess future changes in both flow and sediment. Including synthetic events may be necessary if the POR is not adequate to examine the effects of extended low-flow periods and extreme events. Future conditions assessment should follow current USACE climate change guidance.

(c) Long-term simulations should consider floodplain vegetation change due to both project impacts and natural processes.

(d) Aggradation can affect both project channel maintenance requirements and the long-term level of protection. These factors must be considered in the long-term assessment of project benefits.

(e) Long-term simulations should consider floodplain vegetation change due to both project impacts and natural processes. Encroachment of vegetation into the pre-project channel should be expected if the project alters the occurrence of flushing flows such as with flow diversion.

(4) Evaluation. Evaluation may include a combination of physical and numerical modeling. Appropriate methodology will vary with the design phase as identified in the sediment studies work plan (Chapter 2). Design requires the capability to evaluate the complex flow patterns associated with the project features such as inlets/outlets and the flow/sediment response from existing to project conditions. Two-dimensional models with sediment have been shown to produce satisfactory results. Physical models are also capable and in some complex cases may be the preferred design method. Chapter 9 of this manual provides additional information on numerical and physical models. Guidance on determining regime or stable hydraulic geometry is contained in Copeland et al. (2001).

(a) One-dimensional numerical sediment modeling with HEC-RAS or HEC-6T can provide useful design information relative to sedimentation trends in the pre-project and project condition channels.

(b) Two-dimensional models are often preferred to provide a method to evaluate various alternative designs. Caution is required to define the percentages of water and sediment inputs. While model capability is increasing, accurate model analysis on the flow/sediment distribution at the boundaries is difficult. A robust sensitivity analysis is one method to address model limitations.

(c) Physical modeling of a large scale, such that scale effects do not adversely influence water/sediment distribution, may be necessary for projects such as flow diversions when it is necessary to accurately predict sediment transport between the main channel and diversion.

(d) Measured sediment and flow data from the existing project area is highly recommended for model input, calibration, and when evaluating results.

(e) The project sediment studies work plan should discuss evaluation methods, data requirements, schedule, and budget as discussed in Chapter 2.

g. Sedimentation Concerns for Specific Project Types. In addition to the previously presented general sedimentation concerns, additional information for specific types of USACE FRM projects is described in the following sections.

(1) Levee and Flood Wall Projects. Levees and flood walls are design features intended to reduce the flood damage area. Levee impacts on sediment processes are reduced when constructed on the floodplain and set back from the channel, with minimal change in flow area vegetation, and when changes to cross section or bottom slope are minimal. Typical impacts include raised water surface, possible degradation and deposition areas, and potential impact to areas upstream of the levee extent. Figure 7-41 illustrates the potential sedimentation impacts typical of levee and floodwall projects. Specific concerns are further described as:

(a) Lateral migration of the river channel and bank erosion must be addressed to ensure levee and floodway integrity.

(b) Levee construction that reduces floodplain width also concentrates the sediment deposition in a smaller area. Conveyance loss may occur that can affect project performance.

(c) Levees and floodwalls can be damaged or destroyed by degradation induced by the project or by downstream degradation that migrates into the project reach.

(d) The water surface profile at the upstream end of the leveed reach is likely to be higher than it was under natural conditions. This will induce deposition upstream of the project.

(e) A transition to the natural channel occurs at the downstream end of the leveed reach with the potential to increase the energy slope due to the additional flow expansion loss and the raised river level through the leveed reach. An erosion zone may occur in the transition followed by deposition.

(f) Levees block inflow from local drainage areas so that outlet structures and pumping stations are typically required. On the landward side, these features can be impacted by sediment deposition that can block intakes. Flap gates and elevated river levels can create prolonged pool conditions with sediment deposition. On the river side, drainage outlets can be buried by river deposits.

(g) Tie-back levees are required on larger tributaries. Sediment deposition and a loss of tributary conveyance often occurs due to raised main channel river levels and constriction of the floodplain depositional area.

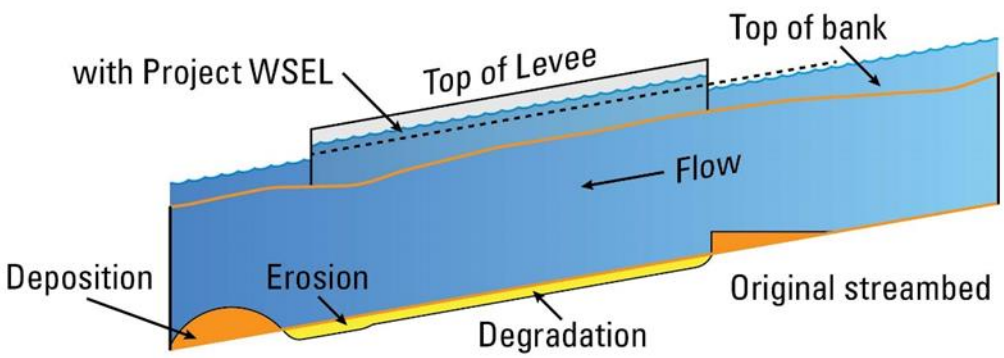


Figure 7-41. Typical sedimentation impacts of levee/flood wall projects

(2) Increased Conveyance. Projects can increase conveyance using multiple methods such as removing channel obstructions, altering vegetation (Figure 7-42), or streamlining the channel shape by methods such as removing abrupt expansions/contractions or smoothing irregular channel banks (Figure 7-43). Figure 7-44 illustrates sedimentation impacts of reduced roughness projects. Specific concerns are further described as:

(a) Increased conveyance results in lower water surface elevations. As a result, floodplain storage may decrease and sediment transport potential may increase.

(b) Channel degradation may be a potential response. The project stage-discharge curve will likely be lower than under existing conditions. Tributary streams also have the potential for headcuts because of the lower base level in the receiving stream.

(c) The downstream end of the project may experience deposition due to the decrease in slope where the channel transitions back to natural conditions and due to increased sediment supply from degradation in the project reach.

(d) Removed vegetation from natural channels will reemerge unless specifically addressed with maintenance (Figure 7-45).

Figure 7-42. Reduced roughness by removing vegetation, Mojave River, Victorville, California

Figure 7-43. Reduced roughness by streamlining banks, Bouquet Canyon Wash, Santa Clarita, California

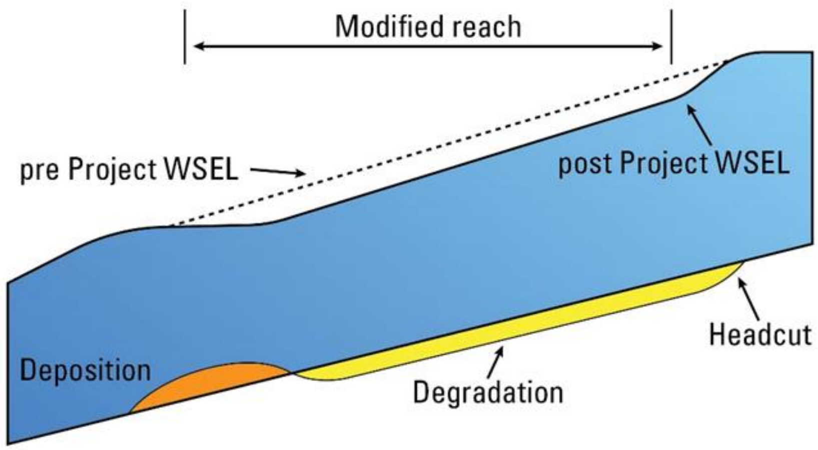


Figure 7-44. Typical sedimentation impacts of increased conveyance

Figure 7-45. Controlling vegetation is critical to maintaining conveyance, Santa Inez River, California

(3) Channelization with Natural Boundaries. Channelization projects lower the flood stage of the stream by widening, deepening, smoothing, straightening, or streamlining the existing channel. Since the natural channel dimensions are changed with this approach, a detailed sediment study should be conducted. Typically, it is not possible to achieve both natural channel conditions and flood-stage reduction at the same time. The design should attempt to achieve a balance considering both objectives.

(a) Dimensions. Channel dimensions that are not in equilibrium can lead to channel instability; channel velocities that increase can lead to bank erosion during flood events; and, if the channel is enlarged, channel velocities that decrease may lead to excessive aggradation and loss of conveyance. Sediment problems are typically less severe when the design cross section modifies only one bank and is elevated (Figure 7-46).

(b) Width. The most common problems arise when the design bottom width is not in regime with the natural system. Perennial streams typically have a low-flow channel. If a wide, flat-bottom channel is constructed, a low-flow channel will often develop inside the wide channel with a meander pattern that attacks alternating banks. Therefore, channel designs for perennial streams should follow the natural channel cross-section shape where possible. Ephemeral streams in the southwestern United States, on the other hand, often exhibit a wide, flat sand-bed and no low-flow channel. Designs for those streams should follow that cross-section shape.

(c) Depth. Sediment problems can result in deeply incised channels with excessive depths. Slope stability problems are common. Stable bank design depends on the geotechnical properties of the soil and the method of bank protection.

(d) Alignment. Following the existing channel alignment is usually preferred if the channel is stable. To achieve reduction in flood stages, the alignment may be straightened to increase slope. This alternative may require additional bank protection and grade control to limit degradation. To increase aquatic habitat, more sinuosity may be added to the alignment. This alternative may induce significant reduction in sediment transport capacity and aggradation due to reduced slope. A detailed sediment study should be conducted to ensure sediment continuity. Selection of channel alignment should consider that not all natural stable streams are meandering streams and that no natural stable streams are straight.

(e) Stratigraphy. It is important to evaluate the variability in bed and bank materials within the project reach, upstream, and downstream. Sediment transport potential changes significantly with changes in bed gradation. The bed and bank sediment layers should be checked for variability with depth and spatially. Project channels constructed along new alignments can have significantly different bed and bank materials. Severe problems can occur when the design cuts through a clay or erosion resistant layer into a less resistant material that can be eroded by the flow (see Figure 7-47 showing boring logs and material layers). This figure also illustrates how limited data of only three borings for a channel project length of nearly three miles can be misleading.

(f) Specific concerns are further described as:

- Channel width and depth affect sediment transport continuity. A deepened channel tends to confine more flood flows in the channel, which increases channel velocities during floods and may induce degradation. On the other hand, a deepened channel that does not lower water levels will have greater flow area which reduces channel velocity and may induce aggradation. Increased channel width can also lower velocity and result in aggradation. A detailed sediment study should be conducted to determine the most effective channel geometry to achieve sediment continuity and identify any maintenance requirements.

- A channelization project is often more likely to experience sediment problems than either a levee project or a reduced hydraulic roughness project. The type of problems and their severity depend on how stable the pre-project channel was in the project reach and how much the design channel dimensions depart from regime values.

- Channelization may reduce storage, increase velocity, and alter flood peak timing with a potential to increase downstream flood peak flows. Increased flows may affect stability in the downstream reach.

- Loss of conveyance due to aggradation after construction, combined with underestimation of maintenance requirements, can lead to a project’s failure to perform at design levels.

- Channelization impacts on the stream system exhibit the same trends as increased conveyance, but the impacts are often more extreme. The upstream end of the project reach has a potential for a headcut because the stage-discharge curve is lower than pre-project conditions. Tributary streams also have the potential for headcuts because of the lower base level in the receiving stream. The downstream end of the project may experience deposition due to the decrease in slope where the channel transitions back to natural conditions.

- Channelization that results in lower river levels may also affect floodplain wetlands, floodplain connectivity, and groundwater levels. Stabilization may be required for substantial portions of the bank. These types of impacts may have significant environmental impacts.

- Channelization has the potential to significantly alter entrance and exit flow hydraulics. Sedimentation problems in the approach and exit reaches are illustrated in Figure 7-48 and Figure 7-49.

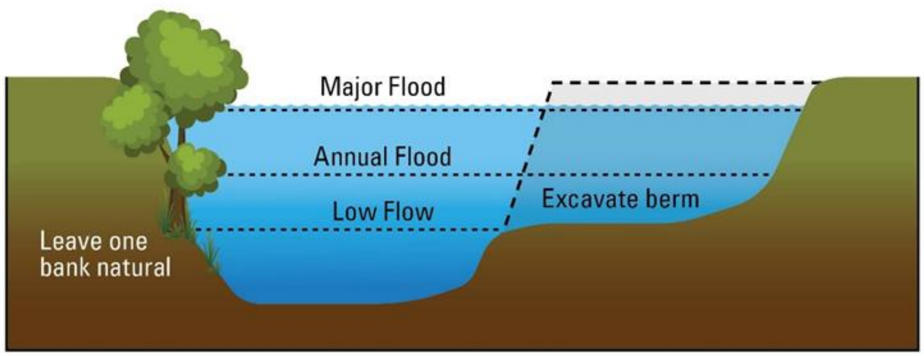


Figure 7-46. Composite cross-section shape (modification of one bank)

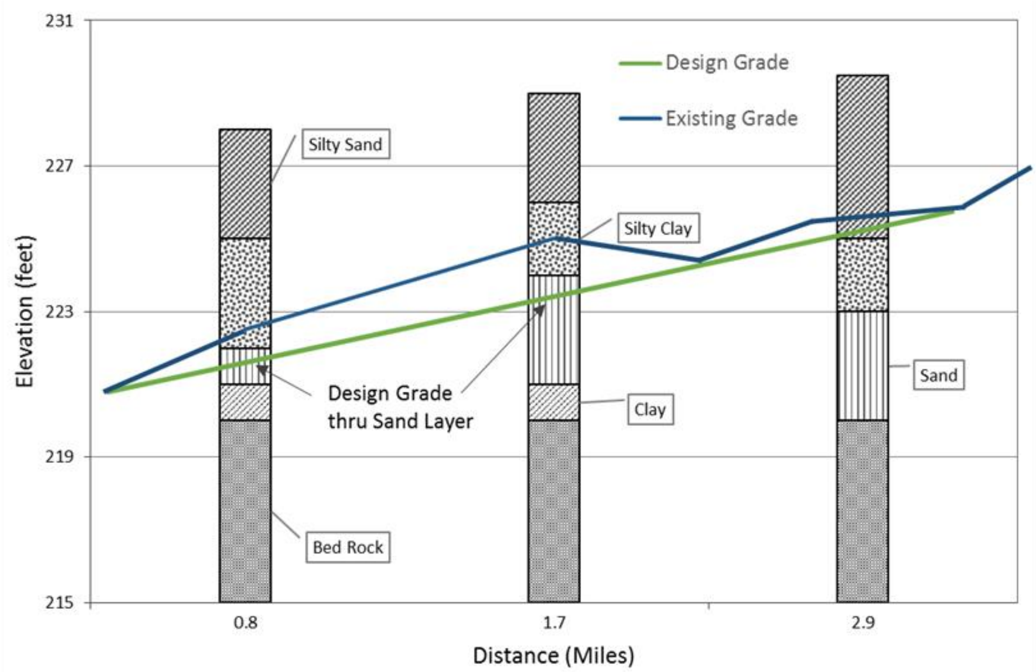


Figure 7-47. Use of boring logs in selecting design grade

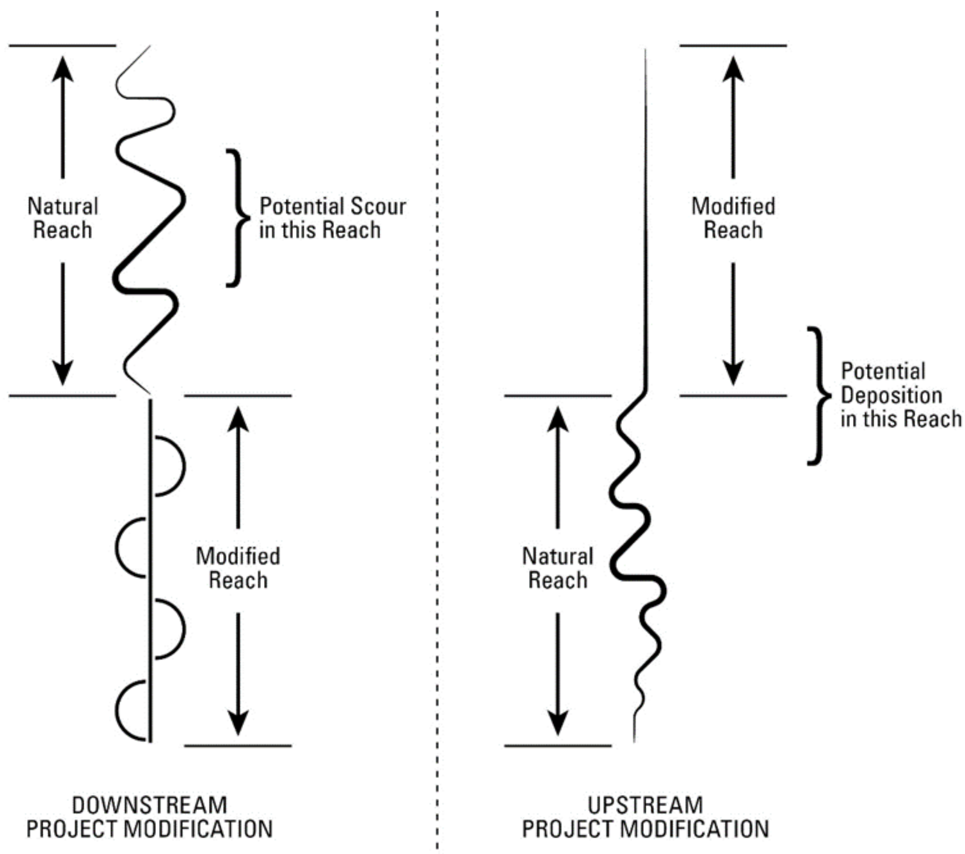


Figure 7-48. Effects of abrupt channel modification

Figure 7-49. Gully development, Yucaipa Creek, Yucaipa, California (developed as a consequence of upstream channelization, which reduced overbank percolation and concentrated flood discharges)

(4) Channelization with Rigid Boundaries. Lined channels are often used to minimize real estate needs and protect against high velocities. Lined channels may be necessary in some areas due to extreme project reach slopes. Channel lining can be used to reduce flow roughness and increase channel capacity. Project reach erosion is prevented by the protected bed and banks. Specific concerns are further described as:

(a) Sediments can roughen the channel lining by abrasion (Figure 7-50) with potential for significant channel lining damage (Figure 7-51).

(b) Deposition problems may occur if the channel flow area is smaller or if the channel slope is milder than the upstream reach (Figure 7-52). Sediment deposition can lead to a significant increase in channel roughness, which in turn exacerbates the deposition process (Figure 7-53).

(c) Debris basins that eliminate or reduce coarse sediment inflow to the design channel are commonly included with this design approach.

Figure 7-50. Increased roughness due to abrasion, Corte Madera Creek (Copeland 2000)

Figure 7-51. Destruction of channel lining by heavy bed material load, Santa Paula Creek, Santa Paula, California

Figure 7-52. Remnant of sediment deposition in concrete channel after removal of deposit, Corte Madera Creek, California (Copeland 2000); sediment falls out when channel enters the tidal zone, even during floods

Figure 7-53. Increased roughness due to deposition at downstream end of Iao Stream Boulder-Concrete Channel in Maui, Hawaii, caused by decrease in channel slope

(5) High-Level Cutoffs. Cutoffs that are active only during flood flows are called high-level cutoffs (Figure 7-54 and Figure 7-55). They provide immediate and significant reductions in flow levels within and upstream of the cutoff reach. Analysis of the potential scour and deposition extent is necessary to ensure that the cutoff will function as designed. Figure 7-56 illustrates sedimentation impacts of high-level cutoffs. Specific concerns are further described as:

(a) Advantages include the retention of natural channel vegetation and dimensions at low flow, a reduction in potentially destructive velocities in the natural channel, and periodic connectivity with a terrestrial floodplain.

(b) Disadvantages include a decrease in flushing in the old channel bend during floods and a new requirement for channel maintenance in the design floodway.

(c) Sediment transport capacity in the natural channel is reduced by the flow reduction while the channel size is unchanged. These conditions often lead to deposition in the natural channel reach affected by the cutoff.

(d) Varying with cutoff flow magnitude and sediment size, the likely response within the high-level cutoff is degradation.

(e) The high-level cutoff lowers the project reach river level, which increases energy slope upstream of the cutoff during high flows. The increased energy could lead to bed and bank stability problems in the upstream reach and increased sediment load to the project reach.

(f) The local energy slope is decreased downstream from the cutoff. This decrease in slope, and the extra sediment supplied by degradation in the cutoff channel itself, introduces a potential for deposition in the downstream channel.

(g) Turbulence created at the downstream junction of the natural channel and the cutoff channel has the potential to create local scour. Local bank and bed protection may be necessary.

(h) Projects that include significant gravel transport may not follow the same pattern. Gravel transport is heavily affected by momentum and may "ramp up" at the cutoff channel to block the entrance.

(i) The high-level cutoff will require vegetation maintenance and land use restriction. Due to the loss of flushing flow in the natural channel, it is likely that periodic excavation of sediment and clearing of vegetation will be required within the natural channel.

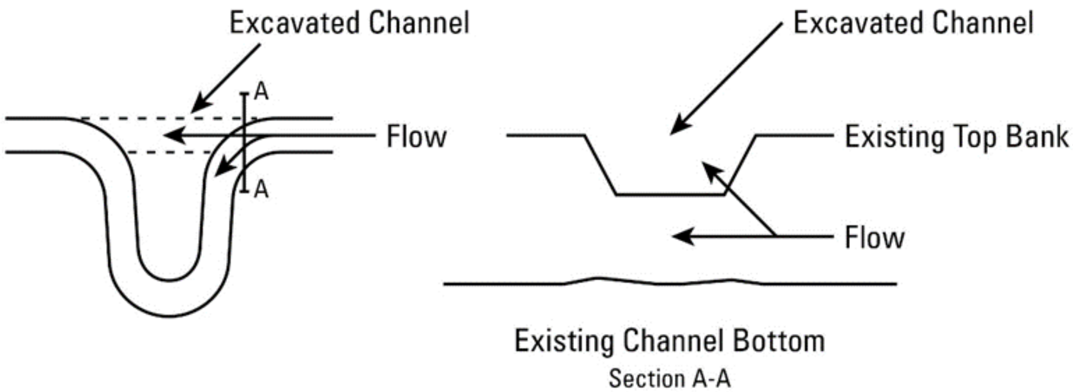


Figure 7-54. Illustration of high-level cutoff

Figure 7-55. High-level cutoffs, Little Blue River, Missouri

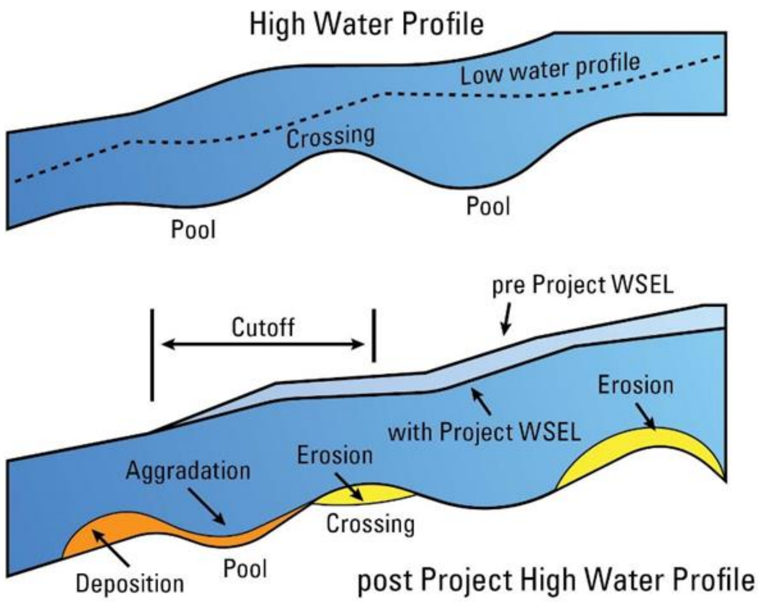


Figure 7-56. Typical sedimentation impacts of high-level cutoffs

(6) Diversions or Bypass Channels. Diversion or bypass channels are used to convey flow around a high-damage reach rather than improving reach capacity. Diversions/bypass channels are similar to a cutoff in concept. The difference is that diversions and bypass channels are usually longer than a cutoff and may require a meandering channel planform. The entrance to the diversion/bypass channel is often controlled with a hardened structure such as a weir to regulate the flow split between the main channel and the bypass/diversion channel.

(a) Inlet Geometry. The location and elevation of the diversion/bypass inlet, relative to the channel planform, is a significant design feature.

- As Figure 7-57 shows, inlets located at a crossing will have lower sediment concentrations than will inlets located at a point bar. Inlets located on the outside of a bend will have even lower sediment concentrations; hence, the concave bank of a bend is the favored location for irrigation diversions.

- The elevation of the diversion/bypass inlet sill is also a significant design parameter. This elevation not only controls the flow diversion percentages, but also the sediment diversion percentages. The coarser sediment is concentrated at the bottom of the water column, while the finer sediments (very fine sand to clay) are typically uniformly distributed in the water column. Consequently, a higher sill elevation will decrease inflow of coarser sediments and reduce deposition within the diversion/bypass channel.

- Sediment deposition issues can occur in the main channel if excess flow is diverted with very little sediment. In this case, the remaining channel flow does not have sufficient transport for the increased sediment concentration.

- ERDC Coastal and Hydraulics Engineering Technical Note CHETN-VII-9 (Letter et al., 2008) provides a review of numerical models and capabilities as well as recommendations. ERDC Technical Note CHETN-VII-13 (Brown et al., 2013) provides multiple conclusions on the effects of sediment diversion on the sediment transport processes. A significant conclusion from these analytical studies demonstrates that maintaining equilibrium in the source river is dependent on the diversion ratio of bed material and water.

(b) Figure 7-58 illustrates sedimentation impacts of diversion/bypass channel projects. Specific concerns are further described as:

- Advantages include lowering flood stages and reducing destructive high velocities while retaining the natural pre-project channel vegetation and channel dimensions. Substantial ecological benefits are associated with this alternative because terrestrial connectivity and nutrient subsidies are promoted.

- Deposition is common because sediment transport potential is reduced when the discharge is divided into two flow channels. Deposition can occur in both the pre-project channel and the diversion/bypass channel. Both local deposition and general aggradation are likely. Local coarse sediment deposition occurs just downstream from the inlet, and general aggradation occurs as finer sediments drop out more slowly in the downstream reach. In cases where the diverted flows enter a wide floodway, deposition of even the finest sediments can occur.

- Upstream of the diversion inlet, the water surface elevation is reduced at the diversion/bypass inlet, which increases the local energy slope. This condition introduces erosion potential upstream from the bypass, which could precipitate a headcut in the upstream direction.

- Downstream at the bypass outlet, where the diverted flow is returned to the channel, the local energy slope is often decreased. This creates a potential for local deposition. The immediate area at the exit often has a turbulence zone that can create local scour that can aggravate downstream deposition. Local bank and bed protection should be considered in the vicinity of the outlet structure.

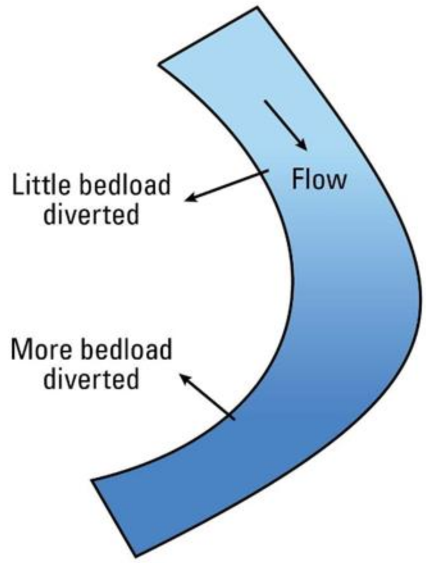


Figure 7-57. Impact of diversion location

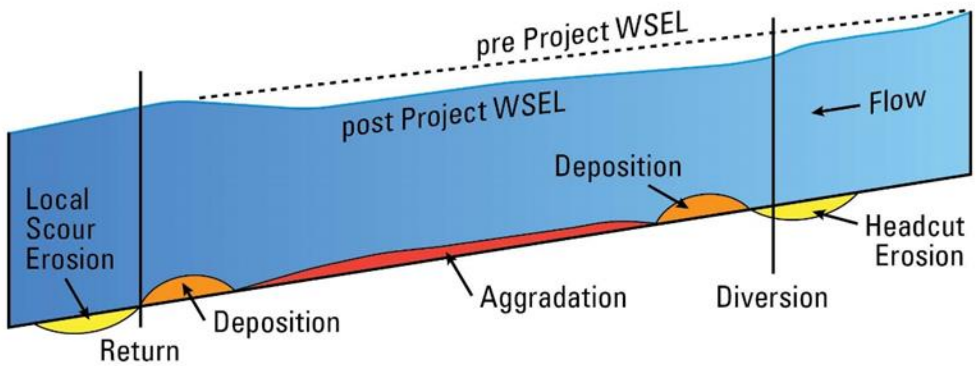


Figure 7-58. Typical sedimentation impacts of diversion/bypass channels

(7) Reservoirs and Detention Basins. Reservoirs/detention basins have numerous impacts on sedimentation processes and are further discussed in detail within Chapter 8. Impacts with respect to channel processes are summarized as:

(a) Advantages of upstream reservoirs and detention basins with respect to sedimentation issues include: (1) reduction of channel instability induced by destructive high velocities and stages during flood events, (2) decreased aggradation, which causes a reduction in channel conveyance in the downstream channel, (3) FRM can be achieved without destroying the natural channel vegetation and natural channel dimensions downstream from the dam, and (4) low-flow augmentation may prevent encroachment by vegetation.

(b) Disadvantages with respect to sedimentation include: (1) aggradation at the upstream end of the reservoir where deltas are created by the main channel and tributaries, (2) introduction of degradation problems downstream from the dam, and (3) decrease in channel flushing during floods, which may leave sediment deposits at tributary confluences and may allow more vegetation to encroach into the channel downstream from the dam.

(c) Typical impacts are illustrated in Figure 7-59. Within the reservoir delta headwaters, channel aggradation progresses upstream from the reservoir. Over time, river levels rise to an elevation above the pre-dam construction level. This can cause significant channel conveyance and flooding issues such as on the Missouri and Niobrara Rivers above Gavins Point Dam (Figure 7-60). At this location, sediment deposition caused a rise in groundwater levels that forced relocation of the town of Niobrara, Nebraska. Chapter 8 of this manual discusses reservoir sedimentation in more detail.

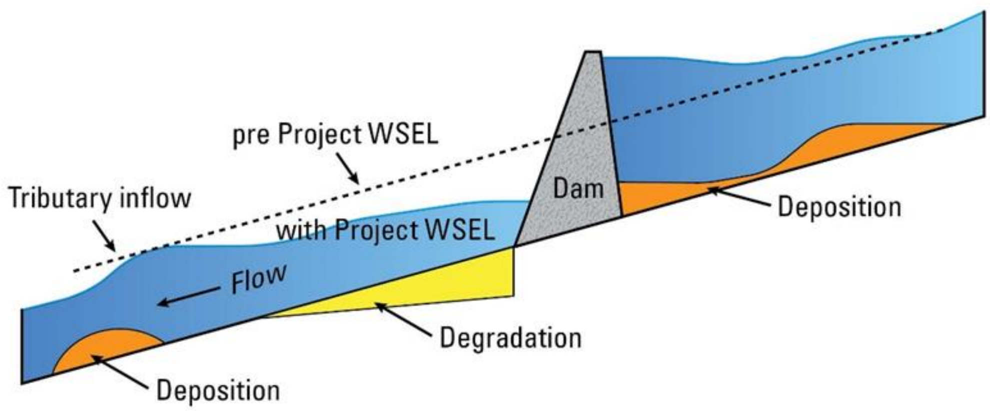


Figure 7-59. Typical sedimentation impact of reservoirs

Figure 7-60. Aggradation at the Niobrara and Missouri River confluence located upstream from Gavins Point Dam and Lewis and Clark Lake

(8) Debris Basins. Debris basins and check dams are used in the headwaters of FRM project channels to trap large bedload debris before it enters the modified project channel. Unlike sediment detention basins, which can be designed for multiple objectives, a debris basin is designed to trap only the coarse material or hyperconcentrated load associated with debris flows. Chapter 8 of this manual discusses design considerations in more detail. Impacts with respect to channel processes are summarized as:

(a) Debris basins are employed in areas with highly unbalanced sediment transport. The basin is used to prevent aggradation in the downstream channel.

(b) Debris basin location is typically at the slope break from the upper mountainous reach to the lower valley floor. Incorrect location can lead to degradation downstream of the basin followed by a deposition zone within the FRM project reach.

(c) Debris basins can reduce the damage from the passage of large debris loads through reinforced concrete channels. Damaged concrete lining can increase hydraulic roughness and reduce channel capacity.

h. General Maintenance Requirements.

(1) The role of sedimentation on measures necessary to maintain project function is critical to a comprehensive operations and maintenance (O&M) plan. This maintenance must be defined in the project O&M Manual and typically includes an inspection program, monitoring,

stability feature repair plan, description of trigger levels when maintenance actions are necessary, discussion of sediment excavation or filling areas, and vegetation maintenance requirements.

(a) Defined maintenance should be adequate to preserve project function and design capacities. Maintenance is a critical component to ensure FRM project performance and avoid catastrophic project failures. Maintenance will usually require annual O&M inspections.

(b) Effective maintenance will require a detailed monitoring program to detect change at critical project features.

(c) When significant sedimentation issues are involved, an adaptive management approach is appropriate.

(d) Environmental compliance should be performed for anticipated O&M during the initial project design and construction. Items such as vegetation management, rock structure maintenance, and sediment removal with disposal should be addressed. Resolve all issues that have the potential to prohibit or reduce the project maintenance extent.

(e) Maintenance parameters should be defined for each project component. Parameters should specify reasonable tolerances and recognize annual fluctuations. For example, channel modifications may begin to deteriorate as soon as completed, vegetation may begin to grow on the banks, and the channel may begin to change its alignment to a less efficient configuration. Define the allowable magnitude for these changes before maintenance actions are necessary.

(f) Annual maintenance is required for some features, while others may be impact-specific. Annual maintenance may also be preferred to maintain costs at a reasonable level and avoid wide cost fluctuations.

(g) Inspection intervals should be defined for all items necessary to project function.

(h) Project response to extreme events and associated repairs should be identified.

(i) Additional funding may be necessary for items such as vegetation management and sediment removal. Real estate ownership or easements are required for maintenance items and should be included in the initial project.

(2) Maintenance of Vegetation and Organic Debris. FRM projects often are based on assumed vegetation levels and associated roughness. Mowing and live vegetation control to maintain hydraulic roughness may be necessary to maintain project conveyance. Dense floodplain vegetation often traps sediment which raises floodplain elevations and reduces conveyance. Construction of levees may influence levee river-side land use change with conversion from farming to higher roughness species. Organic debris (uprooted trees) is both transported and deposited. Removing organic debris can reduce hydraulic roughness, lower water surface elevations, and reduce the risk of bank scour from diverted flow.

(3) Vegetation O&M and Removal Concerns. Vegetation removal is often an O&M issue. The growth of vegetation in the river channel reduces conveyance and may induce sediment stability problems. However, healthy riparian vegetation may stabilize streambanks, provide shade to limit water temperature fluctuations, provide a water quality benefit, improve aesthetic and recreational aspects of the site, and provide hydraulic and physical complexity. Woody vegetation on levees is a controversial issue in which levee structural integrity may conflict with habitat benefits. Vegetation removal may be especially sensitive in semi-arid and arid climates where river and stream channels provide a significant portion of terrestrial habitat. Further guidance is provided in Fischenich and Copeland (2001).

(4) Maintenance to Remove Deposits from Aggrading Channels. Aggradation problems are prevalent in channel modification projects because project designs frequently include increased channel dimensions to improve flow conveyance and lower flood water surface elevations. This often has the unintended consequence of decreasing sediment transport potential. Maintenance to remove sediment deposits may occur as long term and after large flood events.

(a) Long-Term Maintenance. The average annual volume of dredging can be estimated by calculating the difference between the average annual sediment yield entering the project reach and the project's average annual sediment transport capacity. If the result shows deposition, that value is the average annual dredging that will be required to maintain hydraulic capacity. This approach provides only an approximation of dredging requirements because deposition will vary with the wet and dry cycles associated with annual hydrographs. During dry years, no dredging may be needed, but during wet years dredging may be required more than once. Note that the dredging process itself increases aggradation rates. Long-term numerical period of record simulations and designated dredging schedules can capture these feedbacks and should be conducted where dredging maintenance is significant.

(b) Large Flood Event Maintenance. Maintenance and bank protection repair will occur after large flood events. Maintenance records of nearby projects or the average value for streams in the area provide the best information for estimating this maintenance requirement. Catastrophic project impacts such as channel avulsion or sediment blockage are nearly impossible to accurately predict. However, project hydraulics can be used to identify areas that will be susceptible to local scour or deposition.

(5) Maintenance of Channel Degradation Features. Many projects include design features to prevent channel degradation. Although degradation may improve conveyance, large scale degradation can threaten adjacent levees and critical infrastructure. Necessary O&M of bed and bank protection features should be identified. Some projects may elect to defer initial construction of mitigation measures until degradation has reached a certain level. Initial project design should identify future stabilization needs and ensure that single-event catastrophic failure is not possible, that future construction costs are not significantly increased, that environment compliance has been performed, that funding is available, and that real estate requirements are met.

(6) Maintenance of Overbank Areas. Overbank sand deposits can make agricultural land unusable, although deep tilling and other methods may be effective for deposition depths less than 2 or 3 feet. Unless gradual, minimal deposition in the range of 1 foot can kill many hardwoods (some hardwoods such as cottonwood, willow, and bald cypress are more tolerant). These problems are usually too great to be resolved by maintenance.

(7) Maintenance of Tributaries. Tributaries may need to be included in O&M requirements. If the main channel aggrades, water surface elevations increase. This, in turn, raises the water surface on tributary streams. In relatively flat terrain, the increased water surface elevation at the mouth of the tributary will create backwater effects and induce deposition up the tributary. On the other hand, if the mainstem channel degrades, then degradation can propagate up the tributary and threaten infrastructure.

(8) Estimating O&M Costs. Sedimentation impacts can be a significant component of O&M costs throughout the project life. Monitoring and reporting requirements for sedimentation should be included in the O&M manual. Monitoring costs include equipment installation and maintenance, data collection for annual and large flood events, data analysis, and reporting requirements. Maintenance action costs include design, documentation, and the actual construction. Cost estimates are based on analysis and records from similar projects. Accurate identification of costs during the project life are critical to prepare local sponsors for funding needs. Without maintenance, sediment processes can reduce the project level of protection, jeopardize stability during extreme events, and result in catastrophic consequences.

7-12. Navigation Channel Projects.

a. Extensive literature and guidance are available for sedimentation studies in river channels. With respect to navigation channels, EM 1601-2-1611 and the ASCE Manual No. 124 for Inland Navigation Channel Training Works (ASCE 2013) are excellent sources of sedimentation guidance. It is not the intent of this EM to duplicate this guidance, but rather to provide supplemental information concerning sedimentation processes specific to shallow-draft navigation channels.

b. Navigation channel projects typically have significantly different objectives from those of the typical FRM project. USACE navigation channels are required to provide a channel of minimum depth and width at a normal or minimum flow level, whereas FRM projects are focused on providing adequate channel conveyance at high-flow levels. Navigation channel alignment must also provide a reliable sailing line that does not migrate from side to side across the channel. Navigation channels must be reliable since a single shallow crossing can obstruct navigation and shut down an entire waterway.

c. Sedimentation Problems in Shallow-Draft Navigation Channels. Previous chapters in this EM have addressed typical sedimentation problems, most of which are applicable to navigation channels. However, there are some sedimentation processes that are specific to navigation channels that warrant further discussion. For this discussion, the sedimentation processes are addressed in two broad categories: (1) open river reaches, and (2) pool reaches.

(1) Sedimentation in Open River Reaches. Sedimentation problems (deposition) typically occur in open river reaches due to local variations in cross-section geometry and planform such that the local sediment transport capacity is reduced to the point that sediment deposition occurs. Typical sedimentation locations include channel crossings, long-radius meander bends, straight reaches, divided-flow reaches, tributary confluences, lock approaches, and entrances to slack water harbors. Deposition typically occurs at these points due to local geometry processes that are not necessarily associated with long-term aggradational or degradational trends occurring in the system, and consequently can often be effectively addressed with local management alternatives, such as river training structures and maintenance dredging.

(2) Sedimentation in Pool River Reaches. Many navigation systems are viable only through the use of a locks and dams, which create a series of backwater areas called pools. These pools have the same localized problems discussed for the open river reaches, with the added problem of sedimentation resulting from the reduced sediment transport capacity throughout the pools. Sedimentation may occur throughout the pool reach, and therefore may be more associated with system-wide aggradational processes than local geometry issues at specific locations. Figure 7-61 shows an example in which the Whitewater River enters Pool 5 on the Upper Mississippi River in Minnesota.

Figure 7-61. Sediment plume from the Whitewater River, entering the Mississippi River, Pool 5

d. Sediment Management Alternatives. According to ASCE (2013), engineering solutions to managing sediment deposition in navigable waterways may be placed in three categories: prevention, treatment, and accommodation (Table 7-11). While each of the management alternatives listed in Table 7-11 are theoretically feasible, the most commonly used approaches to managing sediment in navigation channels involve training structures (keep sediment moving, KSM) and maintenance dredging (dredge and remove sediment, DRS, or dredge and place sediment, DPS).

(1) KSM involves constricting channel width to the point that the sediment transport capacity remains high enough to prevent sediment deposition (self-maintaining channel). KSM can be thought of as manipulating natural processes to manage deposition; however, it may increase deposition problems downstream.

(2) DRS includes removing and disposing of the dredged material out of the channel, such as in upland containment areas.

(3) DPS primarily consists of in-channel disposal.

Table 7-11
Sediment Management Methods for Navigation Channel (ASCE 2013)

Category	Strategy	Examples
Prevention	KSP – keep sediment in place	Erosion control on land and/or bed and banks
	KSO – keep sediment out	Sediment traps, gates and dikes, channel separations
	KSM – keep sediment moving	Training structures, agitation, flocculation reduction, flushing flows
Treatment	KSN – keep sediment navigable	Nautical depth definition, aerobic agitation
	Maintenance dredging DRS – dredge and remove sediment	Placement in confined disposal facilities or offshore, permanent beneficial uses
	Maintenance dredging DPS – dredge and place sediment	Bypass sediment (KSM)
Accommodation	Adapt to changing sediment regime	Flexible infrastructure, light loading, seasonal navigation

(4) Maintenance Dredging.

(a) Maintenance dredging of troublesome navigation reaches is a common method of addressing immediate sedimentation issues. However, the results are often temporary, with dredging required during subsequent periods of low water. The morphologic impacts of maintenance dredging depend on the magnitude of the operation and the method of disposal (DRS or DPS).

(b) Historically, on many navigable rivers, dredge material was typically disposed in upland containment areas. However, as these areas began to fill up, it became more difficult to find suitable disposal sites. Currently, on many navigable rivers, dredge material is disposed in the channel, typically in dike fields or in the channel thalweg. However, on some navigable rivers, environmental law may prevent in-water disposal due to contaminated sediments or turbidity concerns and thus requiring the use of upland containment sites. On rivers where in-water disposal is not allowed, disposal sites management is likely required. Identifying and promoting the beneficial use of dredge material can be an effective way to manage containment sites. Sediment removal expense can be significant.

(c) Additional guidance on dredging and disposal is available in USACE (1987, 2015b) and USEPA/USACE (2007).

(5) Training Structures in Navigation Channels.

(a) River training structures are used in local reaches where the geometry is such that excessive sediment deposition is adversely affecting navigation channel dimensions (width and depth). The training structures typically function by constricting the channel, thereby increasing the velocities and corresponding sediment transport capacity through the reach.

(b) The most used name for training structures employed in USACE projects are “dikes,” which are most often constructed of stone. Figure 7-62 shows a photograph of typical stone dikes on the Lower Mississippi River. These features are also referred to by many other names including groins, jetties, vane dikes, kicker dikes, spur dikes, wing dams, spur dams, transverse dikes, bendway weirs, contraction structures, and spurs.

(c) On rivers with high sediment loads, structure design should consider the channel response and impacts of deposition in reduced velocity areas. Structure design that results in channel constriction should consider the potential for channel degradation.

(d) The intended function and design specifications for dike structures vary considerably. A description of each of these structure types is provided by USACE (1997b) and ASCE (2013). Figure 7-63 shows an illustration of a typical navigation channel section on the Missouri River. Figure 7-64 shows an illustration of the typical navigation channel configuration plan consisting of dikes on the inside bend and revetments on the outside bend, employed on a self-scouring reach of the Missouri River navigation channel.

Figure 7-62. Typical stone dikes on the Lower Mississippi River

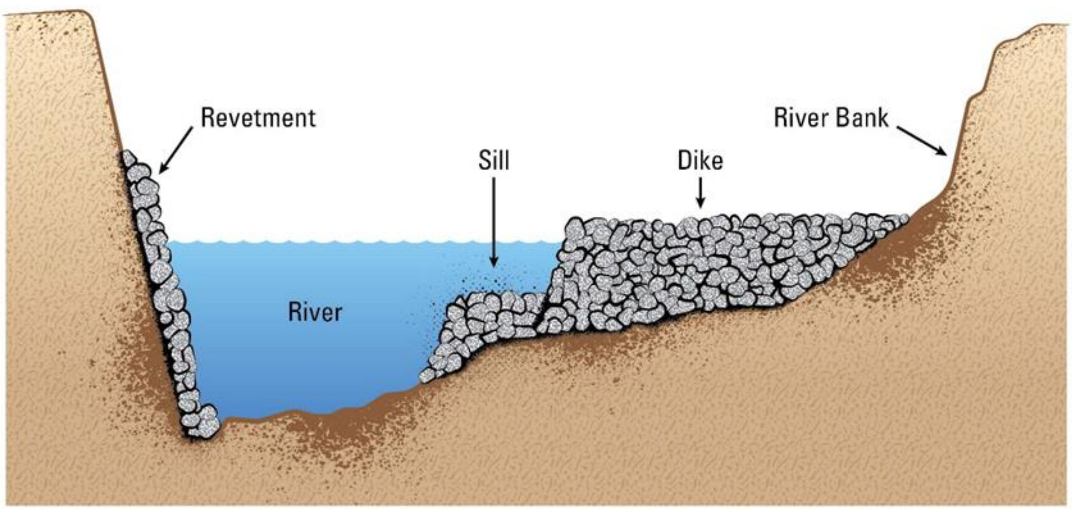


Figure 7-63. Missouri River navigation channel typical section

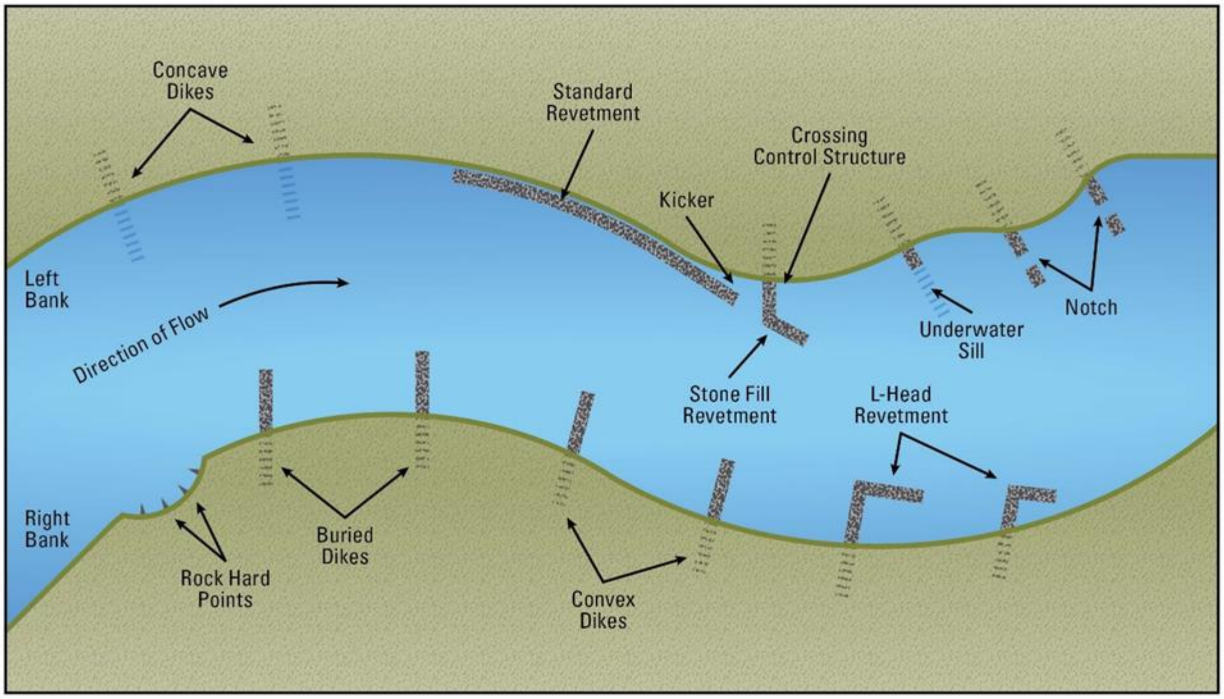


Figure 7-64. Missouri River navigation channel training structure typical plan layout

(e) Whereas maintenance dredging provides temporary relief from sedimentation, training structures essentially function continuously and usually reduce the need for maintenance dredging. Figure 7-65 illustrates that maintenance dredging decreased as the cumulative length of dikes increased on the 290-mile reach of the Lower Mississippi River in the Vicksburg District. Significant maintenance dredging was required during the 1960s through the mid-1970s while the dike construction program was in its infancy. As dike construction progressed, maintenance dredging decreased. During 1988, and again in 2012, extreme low water was experienced on the Lower Mississippi River. During those 2 years, more dredging was required than during any of the previous 30 years. However, this dredging was a manageable volume.

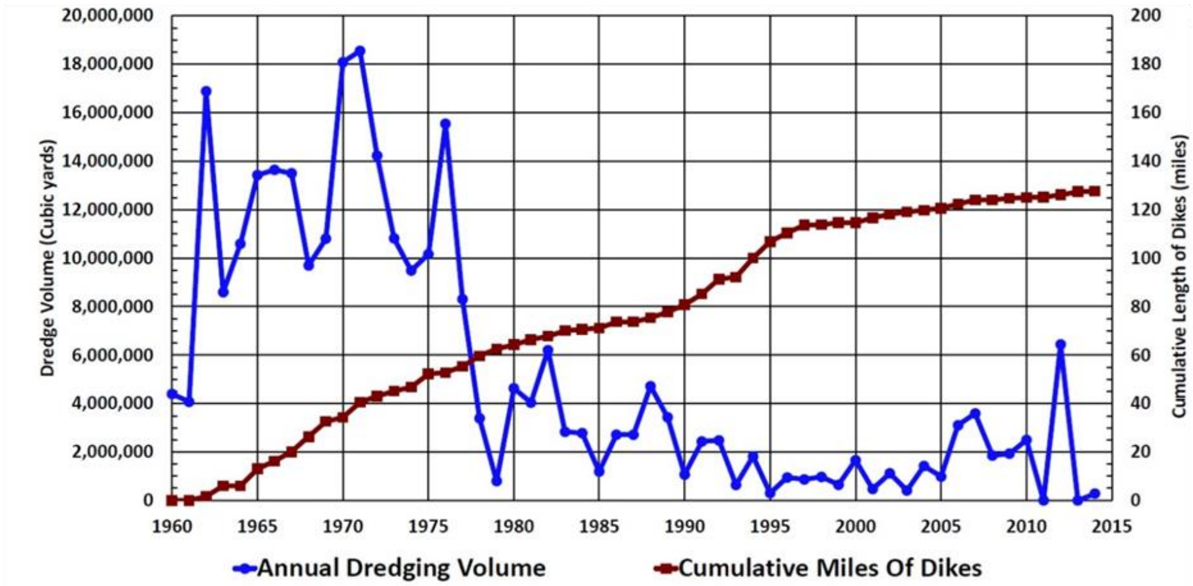


Figure 7-65. Cumulative dike length vs. channel maintenance dredging on the Mississippi River in the Vicksburg District (redrawn from Mayne et al., 2021)

(6) Operational Procedures at Locks and Dams. Sediment deposition can occur throughout the pool in a lock and dam system. While localized maintenance dredging and training structures may be effective for local deposition reaches, they may be less effective for aggradation along long reaches in a pool section. In these instances, it may be possible to increase the delivery of sediment through the pool by operational modifications at the dam. Hinged pool operation is one example whereby the pool is lowered below normal pool during critical sediment transport flows to increase the sediment movement (rather than deposition) through the pool. USACE (1997b) provides a description of hinge pool and other operational procedures.

(a) On the J. Bennett Johnston Waterway (Red River Waterway), Pool 3 uses a hinge pool operation. As flows increase and the water surface levels begin to rise in the upper end of the pool, the water surface at the structure is drawn down below the normal pool level. The maximum design drawdown for Lock and Dam No. 3 is 7 feet. The primary purpose of this operation at this structure is to reduce real estate requirements.

(b) Hinge pool operation also provides the added benefit of increasing the sediment transport capacity due to increased velocities associated with increased slope in the pool. During the design, a balance was developed that allowed an increase in velocities to accommodate sediment transport but limit the increase in velocities so as to not become a hazard to tows navigating the pool.

e. Analysis Methods.

(1) Sedimentation issues in navigation channels are typically addressed using empirical analyses and physical/numerical models. There are numerous studies documenting the use of physical and numerical models for navigation projects, but for a general overview, the reader is referred to Ettema et al. (2000a) for physical model studies and Ettema (2008) for numerical model studies. Regardless of the methods used, the key to a successful plan is to have an experienced river engineer involved in the process.

(2) While sand load is the most significant material size when evaluating material transport and function of the navigation channel, silt and clay are also commonly dredged from navigation channels. Sedimentation issue analysis methods that are capable of meeting site-specific project needs will generally require tracking material size classes. Low current velocities typically desired by the navigation industry may conflict with sediment transport requirements. An example evaluation of sedimentation processes with respect to a navigation channel is presented in Case Study 7B (Appendix N).

f. Monitoring.

(1) A key aspect of maintaining required channel dimensions (width and depth) is monitoring, especially during periods of low water. Monitoring is often a function of available funding, so on rivers with limited funding, open lines of communication with the United States Coast Guard and the navigation industry are critical in identifying problem areas.

(2) On many navigable rivers, frequent surveys are conducted using sounding equipment to identify and subsequently monitor problem reaches. Also, some Districts conduct annual hydrographic surveys to determine channel changes that could impact navigation. These surveys typically include single-beam cross sections. Repeating surveys at the same cross-section locations allows tracking change over time. Figure 7-66 shows the results of a typical single-beam cross-section survey on the Lower Mississippi River at Natchez, Mississippi.

(3) In historic problem reaches, densely spaced single-beam or more detailed multi-beam surveys may be required.

(4) Divided-flow reaches with side channels and chutes are often problematic. As flow increases in these side channels, sediment transport capacity is reduced in the navigation channel, potentially resulting in sediment deposition. In these instances, closure structures may be required to limit the amount of flow in these side channels and chutes. Due to the environmental benefits of side channels and chutes, close coordination is required with environmental stakeholders in the design of these closures.

(5) The following example from a divided-flow reach on the Lower Mississippi River illustrates the importance for monitoring to ensure the long-term viability of the navigation channel. Figure 7-67 shows an aerial photograph of the divided-flow reach at Cottonwood Bar (RM 470) on the Lower Mississippi River. Figure 7-68 (zoomed view) shows the results of a

typical multi-beam survey for the Cottonwood Bar reach. The areas on this survey that are colored yellow and green are potential channel constrictions during periods of low flow. This reach has experienced accidents and groundings in recent years. Also, the narrow channel creates high velocities during periods of low water that result in the reach being difficult to navigate.

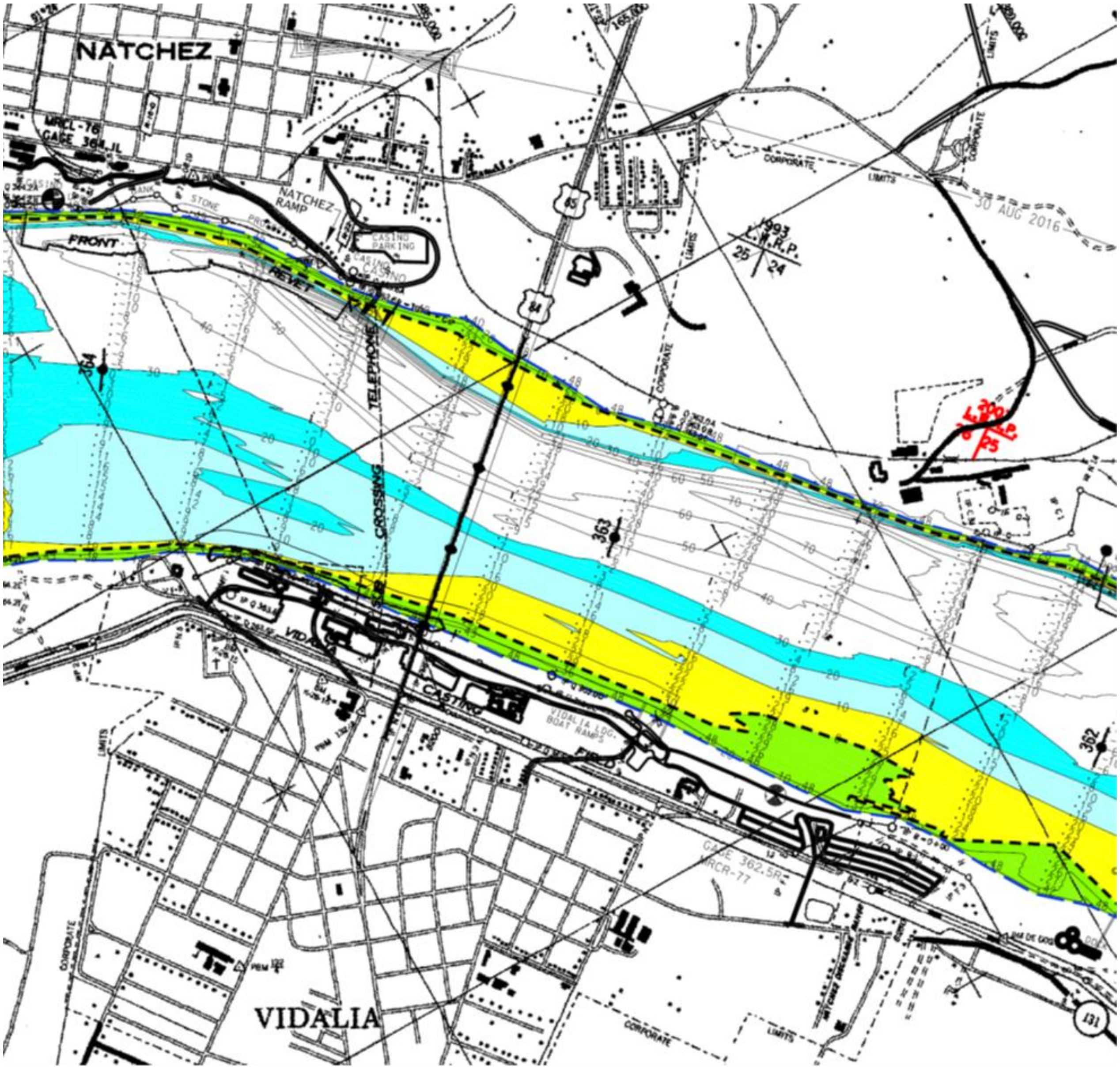


Figure 7-66. Single-beam cross-section survey on the Mississippi River

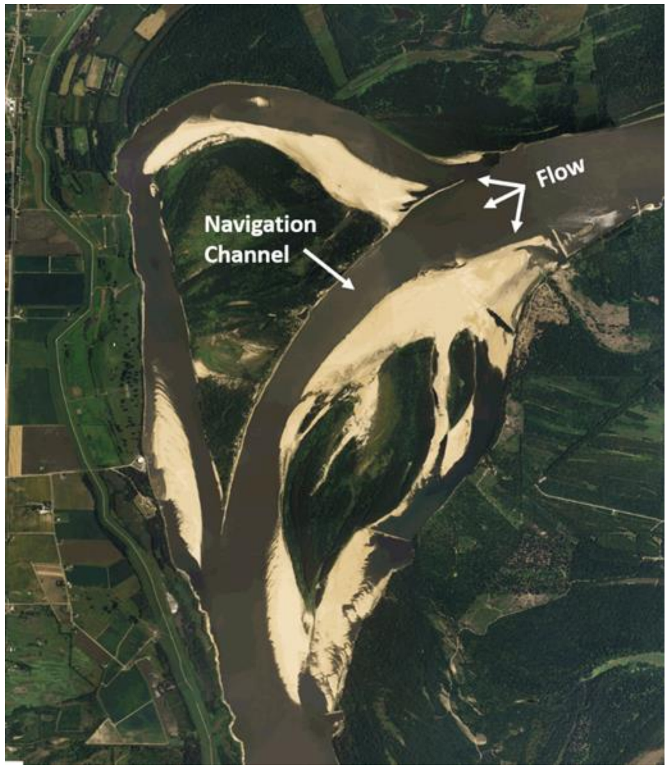


Figure 7-67. Divided-flow reach at Cottonwood Bar on the Lower Mississippi River

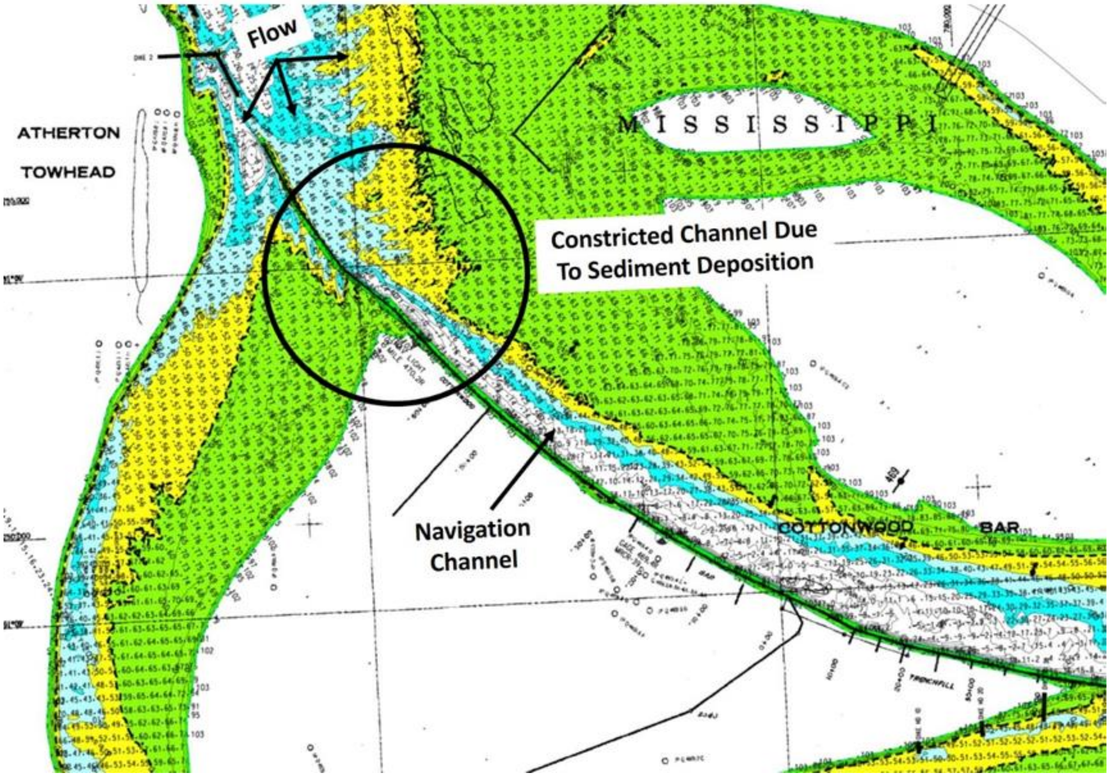


Figure 7-68. Multi-beam survey of the divided-flow reach at Cottonwood Bar on the Lower Mississippi River

7-13. Channel Mining.

a. Mining sand and gravel from the beds of natural rivers and streams can significantly affect sedimentation processes and upset the stability of the fluvial system. Significant sediment removal can eliminate the sediment supply to downstream reaches, which, in turn, results in channel degradation. Sediment removal can also induce headcuts and degradation in an upstream direction. Degradation can result in significant infrastructure failures, including bridges, roadways, bank protection, and pipelines on the main channel or tributaries.

b. To avoid the problems of in-channel sediment extraction pits, gravel mining is sometimes conducted adjacent to the stream channel. In such cases, it is important to determine if the designed protection works are adequate to prevent capture of the main channel during floods. This is especially critical when the main channel is constricted by the protection works and when the main channel flow impinges on the protection works.

c. Mining may be carried out by dredging or mechanical excavation. Mining often consists of trenches and pits in the channel or by gravel bar skimming that removes the top coarser material. Pits in river floodplains or terraces are also common methods. Mining removes the channel aggregate material for use as construction aggregate for multiple purposes such as highway base, pipeline bedding, and concrete aggregate. Sand and gravel subjected to prolonged transport (such as active channel deposits) are particularly desirable sources of aggregate because weak materials are eliminated by abrasion and attrition, leaving durable, rounded, well-sorted gravels (Barksdale 1991). An example evaluation of gravel mining impacts is provided in Case Study 7C (Appendix N).

d. Allowable Quantities and Rates of Removal. No general guidelines have been established to govern removal quantities and rates. If the stream does not have an excess of inflowing bed material (if it is not aggrading), then sand and gravel mining of significant quantity will likely cause a river response with significant economic and environmental impacts. When excess material is available, the removal rate could conceivably be equal to the aggradation rate and alleviate downstream deposition. Aggradation rates can be determined using historical surveys, evaluating measured sediment data, and by long-term numerical modeling. A sediment budget approach to determining allowable removal rates for gravel mining is provided in Case Study 7D (Appendix N).

e. Impact of Mining on the Stream System.

(1) Upstream. The most common effect upstream from an in-channel pit is headcutting with resultant bank failure and channel widening. Degradation that results in base-level lowering on the main stem can also induce degradation up tributary streams. Channel degradation should be calculated for a distance sufficiently far upstream to ascertain if bridges and other structures are adequately founded (Figure 7-69). SLA (1982) illustrates a case history in which the San Juan Creek in Orange County, California was adversely affected by a gravel mining operation when the upstream headcut eroded the channel bottom to a depth of 30 feet. Steepened banks failed and the channel widened. Grade control could be considered in many applications to limit infrastructure impacts.

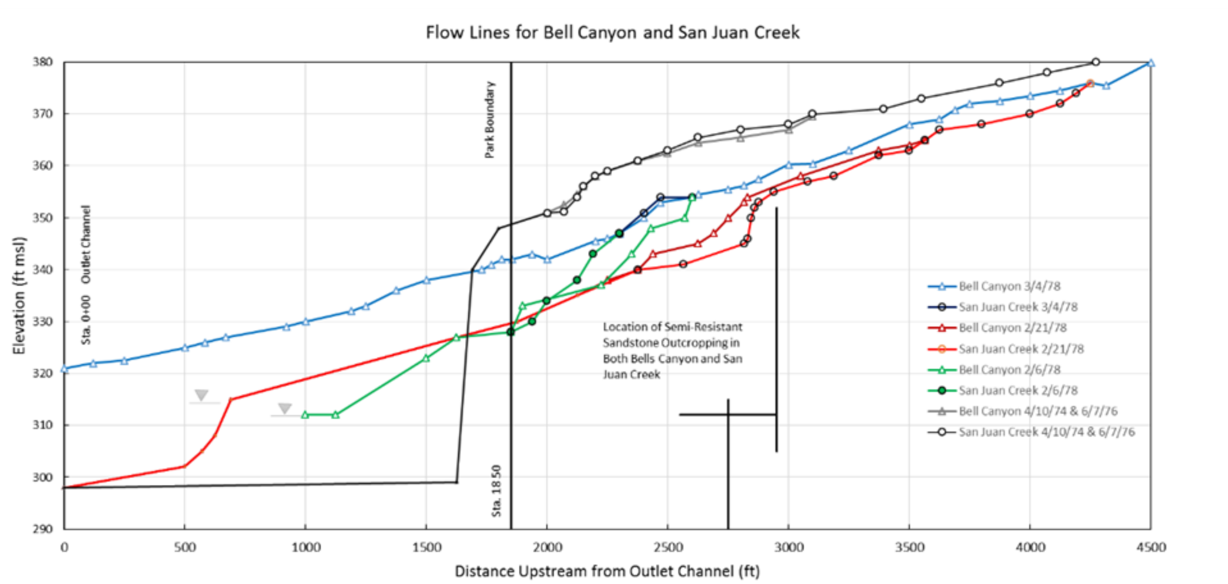


Figure 7-69. Measured bed profile upstream of excavation pit (redrawn from SLA 1982)

(2) Downstream. The downstream channel may be affected by a significant distance depending on the removal rate, normal sediment load, and channel stability prior to initiating mining operations.

(a) Scour will usually occur downstream from in-channel mining operations. This occurs because the pit traps the inflowing bed material sediment load and stream flow exiting the pit is sediment-depleted, similar to the sediment-deficient release from a dam. This deficit from sediment transport capacity results in removal of bed material from the channel downstream. The bed may eventually become armored if sufficient coarse material is present. The degree of channel scour in response to in-channel mining will be roughly proportional to the inflowing sediment load, removal rate, and downstream transport capacity.

(b) In aggradational systems, channel mining may offset or reduce the aggradation rate.

(c) Once the mining extraction of sediment is terminated, channel segments downstream of the mining reach tend to aggrade in response to the ongoing upstream processes, which includes headcut advancement and an elevated sediment load.

(3) Environmental. Evaluation of mining impacts and channel response should also consider environmental impacts. The extraction of gravel materials, including bar skimming at low extraction rates, often has environmental consequences. Gravel bar removal can result in a wider, shallower streambed. Enlargement of the active channel will often occur through bed lowering and bank erosion. The revised geometry leads to increased water temperatures, modification of pool-riffle distribution, alteration of inter-gravel flow paths, and thus degradation of salmonid habitat (Kondolf 1994). Other effects include reduction of fine particulate organic matter, density and biomass of invertebrates, and total densities of fish in pools (Brown et al., 1998).

f. Empirical Analysis. Empirical analysis of gravel mining impacts is usually not sufficiently accurate for practical application. Many factors beyond channel mining, such as mainstem and tributary impoundments, bank protection, channel stabilization, and land use changes, can also contribute to channel degradation, which hinders isolating the effect of a single activity such as channel mining. In addition, the effects of channel mining may lag the mining event by several years (such as in the case of a slowly moving headcut).

g. Numerical Analysis. Gravel mining operations should be evaluated using a numerical sedimentation model such as HEC-RAS or HEC-6T.

(1) The bed material composition throughout the expected excavation and degradation depths is essential and will require borings.

(2) Careful attention to the numerical model armoring calculations is essential.

(3) The numerical simulation should be based on a long-term historical hydrograph that includes both drought and flood years.

(4) It is critical that the upstream and downstream model boundaries be outside the influence of the proposed mining operation.

(5) Simple sediment budget analyses using models such as SIAM and SAM are not recommended due to the complex interaction between the processes of sediment inflow, erosion, entrainment, and deposition; and also due to the complex nature of the armoring process and the interaction of the various bed sediment particle sizes.

h. Adaptive Management. Even with robust sedimentation modeling, adaptive management of rivers with channel mining operations should be considered.

(1) An adaptive management approach allows for realistic uncertainties in determining an allowable extraction rate and for the myriad of other factors that can also cause bed degradation. If river degradation has occurred for any reason, the adaptive management process can be employed to limit channel mining to an acceptable level.

(2) When using an adaptive management approach, the surveys should be sufficiently detailed and comprehensive to allow an accurate assessment of the distributed effects of the current mining operation and to provide a sufficient baseline for future mining operations that might open in other locations on the river.

7-14. Stream Restoration Studies.

a. Stream restoration is the focus of many USACE projects through a variety of authorities. Typical restoration projects will influence the entire fluvial system. Design methodology should consider all aspects of the system, including the channel, banks, riparian zone, and floodplain. Design guidance for sedimentation process analysis is available from many suitable references, including Sediment Sampling and Analysis for Stream Restoration Projects (Fischenich and Little 2007), Hydraulic Design of Stream Restoration Projects, ERDC/CHL TR-01-28 (Copeland et al., 2001), and Stream Restoration Design, National Engineering Handbook Part 654 (NRCS 2007).

b. Many stream restoration efforts are related to sediment transport processes that affect geomorphic change or water quality in some way.

(1) An imbalance between the sediment load and the hydraulic energy available to transport it can cause channel aggradation or degradation, a change in substrate size, and bank erosion. This could reduce connectivity from one habitat to another, reduce water depths and width-to-depth ratios, alter benthic conditions, or degrade floodplain forest communities.

(2) Excess sediment in the water column can reduce light penetration and aquatic plant growth.

(3) Many of the aquatic habitat evaluation procedures (USFWS 1980) used by USACE to determine habitat benefits require input on sediment parameters, including things such as substrate size, suspended sediment, or riffle-pool patterns.

(4) Maintenance due to sediment deposition within the project area or erosion of project features can significantly affect geomorphic processes. Care must be taken to ensure that maintenance is adequately defined to prevent unintended river responses.

c. Sedimentation Problems Affecting River and Stream Ecology. Stream restoration projects can restore any number of physical or chemical processes in an attempt to create a more desirable ecological condition. Sediment mobilization, transport, and deposition in the aquatic ecosystem are commonly linked to changes in geomorphology, water quality, and biology. Sedimentation can result in a variety of stressors on aquatic ecosystems including:

(1) Aggradation or degradation resulting in altered cross-section depth, size, and shape.

(2) Bank erosion.

(3) Altered substrate size.

(4) Reduced light penetration.

(5) Contaminant and nutrient transport.

(6) Reduced connectivity between habitats.

(7) Sediment oxygen demand.

(8) Burial of aquatic organisms.

(9) Changes in the frequency and duration of floodplain inundation.

d. Typical Restoration Project Goals.

(1) USACE project goals and objectives can be formulated to address physical, chemical, or biological processes and functions. For instance, reducing sediment deposition, increasing light penetration, and increasing submerged aquatic vegetation (SAV) growth may be objectives that, when combined, result in increased fish populations. The first two objectives are directly linked to sediment transport, and the third objective can be only achieved if the first two objectives are met. In these cases, the hydraulic design engineer is challenged with modifying a fluvial system to provide specific habitat requirements as well as sustainable physical conditions.

(2) If criteria is available describing the linkage between physical/chemical parameters and biologic response, this should be included as part of the objective. Often, however, this criteria is lacking, and a significant amount of judgment is needed in identifying project features that will result in the achievement of objectives. Many historic USACE projects were designed for the primary purpose of flood risk management or navigation without consideration for environmental performance. Figure 7-70 shows an example of a typical USACE project focused on FRM.

Figure 7-70. Goose Creek, Sheridan, Wyoming, USACE flood risk management project, completed in 1963

(3) The Goose Creek project has performed well as a FRM project since construction, with minimal maintenance needs. The concrete U-channel provides channel bed and bank stability with high reliability. However, environmental impact is severe, with a current local desire to modify the project and provide a more natural setting. In this case, stream restoration design requires that hydrology, hydraulics, sediment transport, and channel morphology are all evaluated for the restoration site and for other adjacent impacted areas.

(4) Stream restoration projects often change channel characteristics that impact sedimentation processes within the project reach as well as upstream and downstream reaches. Restoration projects require analysis of channel stability due to modification of stream width, depth, slope, planform, bank erosion potential, hydraulic roughness, and bed material gradation. Successful and sustainable stream restoration requires a thorough, contextual understanding of the dynamic physical, chemical, and biological processes occurring in a river, along with the life cycle and seasonal needs of organisms.

(5) This document contains a thorough discussion of sedimentation processes that can be employed in stream restoration design. In addition to evaluating sedimentation processes for design issues such as with project condition stability, a successful stream restoration project will also require adding knowledge of channel form and function to attain biologic goals. Numerous texts contain excellent references on desirable features such as step-pool design, incorporation of riffles, and habitat structures. A few of the notable stream restoration design guidance references include Copeland et al., 2001; Shields et al., 2008; and USDA 2007b.

e. Sediment Management Alternatives.

(1) The alternatives used to deal with sediment in stream restoration projects fall into several categories including: (1) eliminate or reduce sources of sediment, (2) maintain sediment transport potential through the project, and (3) create sinks for sediment with periodic maintenance to preserve project capacities and depths.

(2) Eliminating or reducing sources of sediment could include upstream bank stabilization and grade control, improved land use, barriers such as dikes or islands to prevent sediment from entering a project area, or reduction of erosive forces such as river currents or wind-driven wave action within the project reach. Eliminating sources of sediment at a watershed scale requires that the major sources of sediment be identified, which could be prohibitively costly.

(3) Maintaining sediment transport potential by altering discharge, adjusting water levels, or using in-stream structures is often desirable since it enhances the sustainability of restoration projects. However, this can be difficult to achieve if the project reach is already depositional, since it requires increasing the total discharge, hydraulic slope, or both.

(4) Creating sinks for sediment can be done through flow diversions or by creating in stream sediment traps by dredging. Periodic maintenance dredging, although not a sustainable solution, is sometimes the only option for maintaining adequate water depths and connectivity in some projects or for creating sediment traps. If a beneficial use of dredge material can be found, maintenance dredging could be a feasible alternative. A reasonable estimate of future annual maintenance should be provided to the local sponsor.

f. Analysis Methods. Sedimentation issues associated with stream and river restoration projects are addressed using different methods that depend on the amount of and timing of funding, the amount of risk that is acceptable, and study schedules.

(1) Analysis methods range from relatively simple analytical methods to more complicated 1D or 2D sediment transport models. State floodplain permits and other constraints often require at least some type of numerical hydraulic modeling that might help with the interpretation of project effects on sediment transport.

(2) Two-dimensional models provide more detailed spatial information, which can be useful in defining sediment transport throughout a project area. However, the cost and risk associated with the project may not justify a 2D model. The pertinent questions are "How much better is the 2D answer than the 1D answer?" and "Does the risk management achieved by selecting 2D justify upgrading from the 1D option?" (Gibson and Pasternack 2015).

(3) Regardless of the methods used, the key to a successful plan is to have an engineer experienced with hydraulic and sediment transport analysis involved in the process. Analysis methods must be capable of meeting site-specific project needs that could include determining the geomorphic (sediment loads, particle size) and biogeochemical (sediment concentrations) processes associated with the project.

g. Restoration Design Considerations. Restoration design has considerable potential for negative impacts to USACE project function. Following guidance for evaluation of sedimentation processes will reduce the risk of poorly performing restoration projects. Several of the more common considerations in restoration design are as follows:

(1) In addition to establishing biological restoration goals and objectives, the engineer should also ensure that designs have acceptable outcomes with respect to the physical processes of erosion, transport, and deposition of sediment. For example, achieving objectives over the project life, should not result in sedimentation problems within the project area, adjacent locations, or problems that would occur beyond the project life.

(2) The term "stream restoration" may suggest returning to the former natural channel. ER 1105-2-100 states "restored ecosystems should mimic as closely as possible, conditions that would occur in the area in the absence of human changes to the landscape and hydrology." However, few USACE project watersheds have not encountered significant anthropogenic disturbances to sedimentation processes. In these altered conditions, it is nearly impossible to restore the former natural channel from the pre-disturbance historic period.

(3) During analysis, it is critical to assess existing, future without, and future with project conditions, including the effects of altered stream flows and sediment inputs. Alternatives should be selected that meet project objectives, partially restore natural and sustainable conditions, and minimize O&M requirements due to sedimentation processes.

(4) When performing stream restoration of USACE FRM projects, it can be particularly challenging to avoid compromising the existing project function. For instance, replacing a straightened concrete lined channel with a meandering stream will reduce channel conveyance and increase water surface profiles. Therefore, a typical restoration project, even if adequately designed to be sustainable and in balance with sedimentation processes, will also require additional conveyance to avoid impacting water surface elevations.

(5) Stream flow diversions to create floodplain wetland habitat can alter sediment transport throughout the system. In some cases, a clear-water diversion to avoid wetland area deposition is the goal. In other cases, the diversion may be designed to convey sediment to off-channel areas. In either case, if diversion flow volumes are sufficiently large, the loss of stream power downstream of the diversion may result in sediment deposition in the channel.

(6) Conventional USACE projects evaluate project stability with respect to maintaining project performance for the project life. For instance, a FRM project minimizes the risk of future channel meandering affecting an adjacent levee by including stability measures. However, stream restoration goals often include a more natural condition of channel evolution that includes the processes of channel migration and bar building. Therefore, combining project goals may often require incorporating project features, such as offset buried rock riprap, that do not directly impede the restoration goals, but provide long-term protection to adjacent critical infrastructure.

(7) Many of the stream channel environments targeted for restoration consist of confined and realigned historic channels. Subjected to the urban environment, these former natural channel processes are now forever altered. Conducting restoration in these conditions often results in regular, long-term management and maintenance requirements (including annual monitoring, permitting, and funding to support these activities) that may have been unanticipated by project proponents and local sponsors.

h. Monitoring and Adaptive Management.

(1) Section 2039 of WRDA 2007 directs the Secretary to ensure that, when conducting a feasibility study for a project (or component of a project) for ecosystem restoration, the recommended project includes a plan for monitoring the success of the ecosystem restoration. This guidance applies to specifically authorized projects or components of projects as well as to those ecosystem restoration projects initiated under the Continuing Authority Program (CAP).

(2) Necessary monitoring for a period not to exceed 10 years will be considered a project cost and will be cost-shared as a project construction cost and funded under construction.

(3) An adaptive management plan will be developed for all USACE ecosystem restoration projects. While the goals and objectives of most ecosystem restoration projects focus on restoring ecological processes (connectivity of habitats) and/or producing a biological response (sustained species population range), to achieve these objectives, quite often the physical/chemical effects of sediment have to be altered.

(4) The team that develops the monitoring plan should consider whether obtaining data on physical parameters such as sediment loads, concentrations, size, and water quality parameters such as light penetration, contaminants, or nutrients is needed to determine project success. If the objectives focus on biologic parameters such as the number of fish or ducks, extensive sediment data may not be needed. However, if the objectives are focused on physical parameters such as reduced bank erosion or channel depths, then pre-project data must be collected to form a baseline, followed by post-project data to determine success.

(5) Adaptive management is a structured decision process that promotes flexible decision-making that can be adjusted as outcomes for management actions and other events become better understood. There is not a standard prototype for implementation that will immediately constitute an adaptive management program. It is context-specific and involves feedback and learning between designers, managers, and stakeholders (NRC 2004).

i. Operation and Maintenance. Stream restoration projects may have different maintenance requirements than a typical USACE FRM project. Expectations during the operation and management phase following construction should be clearly defined in the O&M manual.

(1) Stream restoration project environmental goals may include a desirable level of dynamic processes including bend migration, bar building, and woody debris. These processes should be defined with tolerable limits for measurable metrics such as aggradation, degradation, and lateral bank movement rates.

(2) Unacceptable impacts to other concerns such as surrounding infrastructure, flood risk, navigation channel risk, and similar should be clearly stated along with any constraints on maintenance methods.

(3) Extreme event performance and expectations should also be considered with reasonable limitations defined. For instance, several Missouri River habitat chutes were completely filled with sediment, while several others experienced severe degradation during the 2011 extreme event. Restoration costs for several projects following this extreme event were in excess of original construction costs. Large magnitude impacts should be addressed in O&M manuals. Reasonable local sponsor maintenance responsibilities and costs should be considered when developing O&M requirements. Case Study 10A (Appendix N) presents additional information regarding extreme event consequences.

7-15. Staged Sedimentation Studies for River Projects. Chapter 2 of this manual discussed staged sediment studies and the sediment study work plan (SSWP) in detail. Once study objectives have been identified, it is up to the engineer to select an appropriate evaluation procedure. ER 1110-2-8153 requires that a sediment impact assessment be prepared for all projects. A staged sediment studies approach should be followed in which contingency factors are assigned and revised as more data and analysis are available to decision-makers. Typical components of the staged study, as applied to river projects, are as follows:

a. Stage 1 – Sediment Impact Assessment (Reconnaissance). An example of a limited-detail sediment impact assessment for a project with negligible sedimentation issues is provided in Case Study 7A (Appendix N).

(1) Sediment impact assessment of with and without project channel stability.

(2) If sediment problems are determined to be negligible, the sediment impact assessment can be the final stage.

(3) Examples include:

(a) Geomorphic assessment.

(b) Specific gage analysis.

(c) Qualitative assessment (Appendix F) or limited-detail modeling such as HEC-RAS stable channel design.

(d) Hydraulic geometry relationships.

b. Stage 2 – Detailed Sedimentation Study (Feasibility).

(1) Sediment impact assessment predicts a sedimentation problem.

(2) Similar, existing project is experiencing sedimentation problems.

(3) River analysis using a numerical sedimentation model of existing and future conditions.

(4) Examples include:

(a) Numerical evaluation of aggradation/degradation trends.

(b) Sedimentation impacts from river diversion/bypass channel.

(c) Low head dam installation/removal.

(d) Bank stabilization that alters sediment processes.

c. Stage 3 – Feature Design Sedimentation Study (Preconstruction Engineering and Design).

(1) Extension of the detailed sedimentation study to test the final project design.

(2) Conducted at a specific location on a stream where extensive data are available.

(3) Complex models that will require intensive data sets to conduct analysis.

(4) Examples include:

(a) Depth of both local and general scour at bridges.

(b) Head loss and potential local scour at weirs and drop structures.

(c) Potential deposition in expansions and at inflow points.

(d) Performance of control structure/debris basins in the design.

(e) Stability of the channel invert against erosion.

(f) Ability of the approach structure to eliminate headcuts upstream from the project.

(g) Local erosion at the approach structure and the changes in tailwater as the result of changes in the exit channel.

7-16. Report Requirements. River sedimentation study reporting requirements will vary widely with study methods and details. Refer to the sediment budget and modeling report chapters for respective reporting content. As a minimum, the following topics should be considered for inclusion in a typical study report.

a. River Reach and Study Area Map.

(1) Aerials (current and/or historical).

(2) Meanders.

(3) Bank erosion areas.

(4) Critical infrastructure/land use classifications.

(5) Soil type map.

b. Stream Profile.

(1) Water surface profile(s) and bed elevation vs. river stationing.

(2) Hydraulic controls.

(3) Structures.

(4) Distributaries/tributaries.

c. Description of Sediment-Related Issues.

(1) Current.

(2) Future (with and without project).

d. Sediment Sampling and Data Collection.

(1) Sampling location map.

(2) Gradation plots.

e. Modeling Summary.

(1) Assembly.

(2) Calibration.

(3) Summary (cumulative mass change, bed change, etc.).

(4) With/without project results.

Chapter 8
Reservoir Sedimentation

8-1. Introduction. Reservoir sedimentation is a progressive, systemic, infrastructure failure mode that affects USACE dams and impacts the present and future USACE CW Program. USACE has not constructed many new reservoirs since the 1970s. Agency focus has shifted to operation and maintenance of existing reservoirs instead of new reservoir design. As USACE operates existing facilities, stakeholders often modify or add operational objectives, while sediment deposits reduce flexibility. These sediment deposits require periodic evaluation of reservoir sedimentation and the impacts on operations. Additionally, while new dam analysis is rare, some FRM studies include new reservoir alternatives, which require careful sediment analysis.

a. Chapter Preview. This chapter reviews the physical processes associated with reservoir sedimentation (upstream and downstream of the dam), methods for monitoring and analyzing reservoir sedimentation, climate change factors, and implications for reservoir management and sustainability strategies. Figure 8-1 illustrates chapter content.

(1) The chapter focuses on information applicable to USACE projects and studies, which emphasizes large reservoirs with FRM missions. The chapter supplements, and sometimes summarizes, excellent literature on reservoir sedimentation that provides broader understanding of general sediment processes, reservoir sedimentation processes, and operational experiences, both domestic and internationally.

(2) Evaluation of reservoir sedimentation processes will play a key role for USACE in the future as projects continue to age and sediment-related issues become more frequent and severe. Reservoir sedimentation studies are performed to address a wide variety of USACE project issues at varying levels of complexity. USACE investigations should be designed to invest resources in an efficient and cost-effective way to obtain a risk-informed solution. Refer to paragraphs 8-16 and 2-2 for guidance and tools to perform staged sediment studies.

(3) USACE reservoir sediment scientists, managers, and modelers should consult classic references such as Morris and Fan (2010) and the ASCE Manual No. 110, Chapter 12 (Morris et al., 2008), as well as other chapters in this EM for additional information related to reservoir sedimentation.

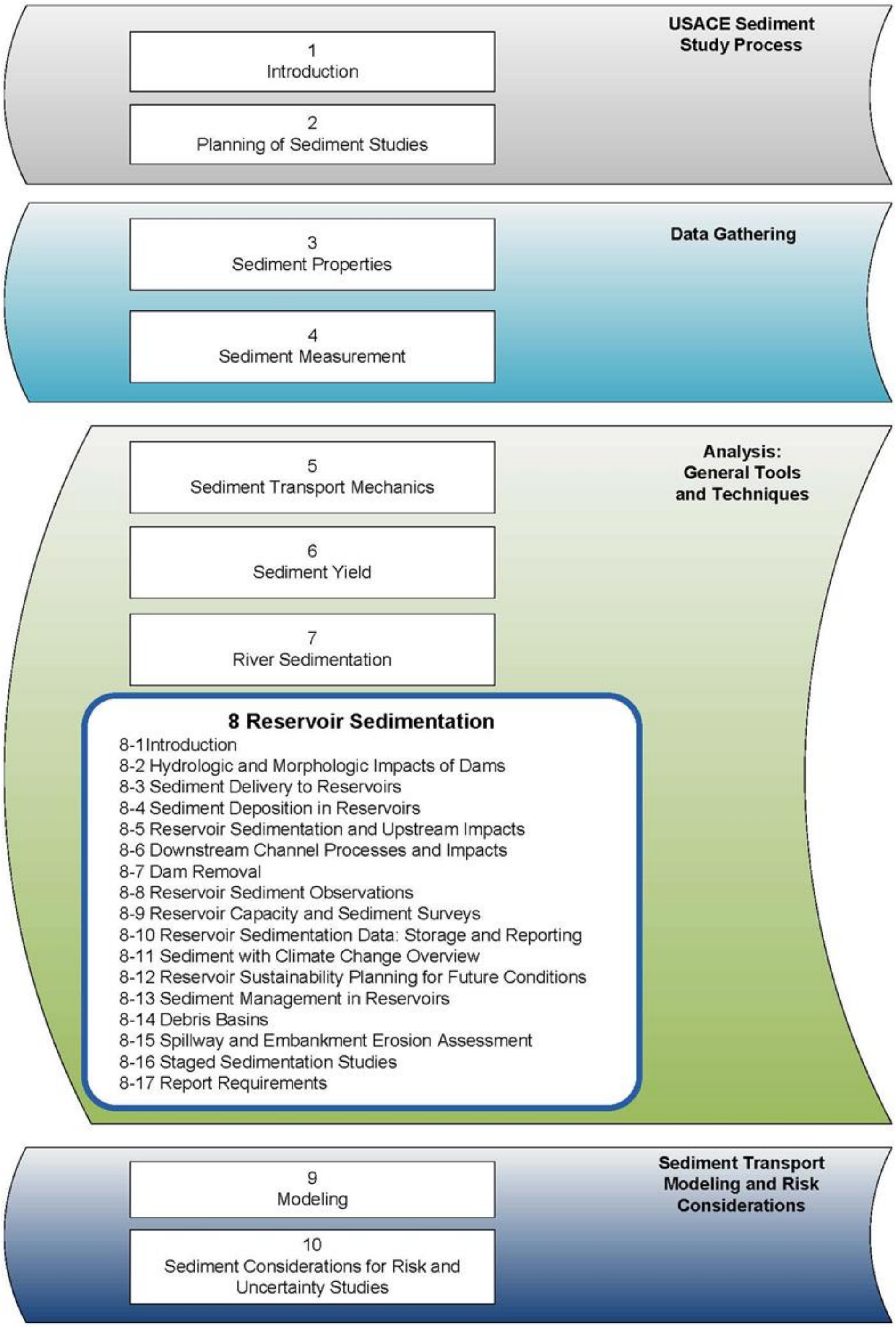


Figure 8-1. Chapter 8 content and general document structure

b. Reservoir Sedimentation Significance.

(1) Global Reservoir Sediment Context.

(a) From 1950 through about 1980, reservoir storage supply grew faster than population, as shown in Figure 8-2. Annandale (2013) noted that global dam construction has declined since the 1980s. The cost of additional storage space continues to climb due to the scarcity of prime dam sites, as well as more restrictive regulatory compliance hurdles as resource agencies recognize the environmental impacts of dams (Kondolf et al., 2014).

(b) Sediment deposition reduces global reservoir storage by approximately 0.8% per year. The International Commission on Large Dams (Kondolf et al., 2014) estimated that sediment impacts associated with reservoirs, cost $21 billion per year, about 37% of the $57 billion total annual reservoir investment, operation, and maintenance cost.

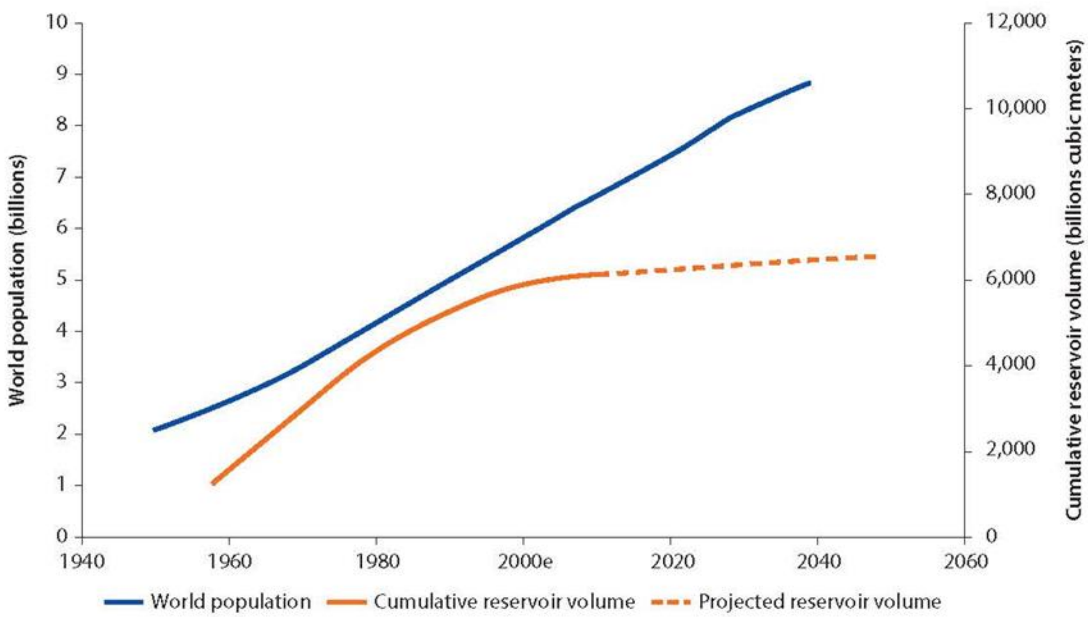


Figure 8-2. World population and reservoir volume trajectories (Annandale et al., 2016)

(2) National Significance. Federal agencies own only 5% of the dams in the United States, but these dams possess 61% of reservoir storage capacity. The nation's dependence on the storage capacity provided by these large dams for water supply, flood risk reduction, and hydropower generation continues to grow along with the population and water demand (Podolak and Doyle 2015). Reservoir sedimentation reduces the storage available to satisfy increasing demand, making reservoir sediment processes critical to national water security, flood risk reduction, and environmental resilience.

(3) USACE Dam Portfolio.

(a) The USACE National Inventory of Dams (NID) database (Figure 8-3) shows that dam construction in the United States peaked between 1950 and 1979, then dropped significantly after 1980. USACE's dam portfolio parallels this national trend (USACE 2015e), highlighting the current need to operate and maintain existing reservoirs in the most effective ways.

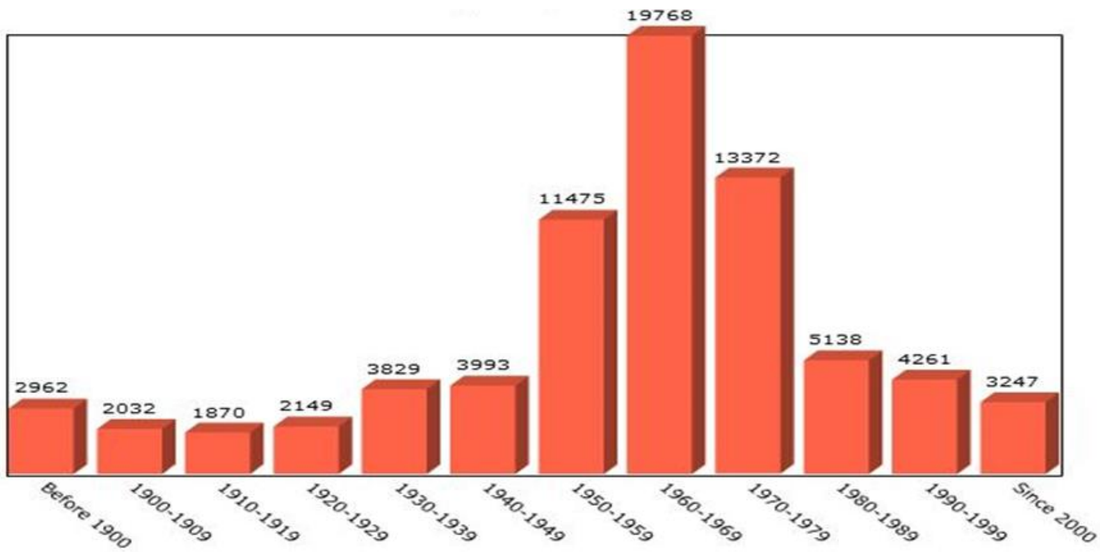


Figure 8-3. Number of U.S. dams, by completion dates

(b) Morris and Leach (2013) note that USACE operates and maintains approximately 383 reservoirs. The USACE Dam Safety Program portfolio states that USACE "operates and maintains approximately 700 structures nationwide and in Puerto Rico" (including locks and dams which do not have permanent reservoirs) (USACE 2015e). The USACE dam portfolio includes six of the 10 largest dams in the United States and 50% of the federal dam inventory.

(c) USACE's dam portfolio represents a large, but aging investment in nationally valuable infrastructure. USACE (2015) notes that 95% of USACE-managed dams are more than 30 years old and that 52% of USACE dams have reached or exceeded the 50-year design service life. Since USACE focus is now primarily on the operation and maintenance of existing agency reservoirs rather than new reservoir design, understanding and managing for sediment becomes more critical. Within the aging reservoir portfolio, sediment deposition impacts to reservoir operations and dam safety are growing issues.

c. Reservoir Sustainability.

(1) Historically, USACE designed reservoirs to store sediment over a finite project life. USACE analyses predicted sediment impacts over typical project lives of 50 to 100 years. Historic sediment yield estimates were generally reasonable. Relatively few projects have substantially exceeded predicted sedimentation rates. Therefore, most reservoir sedimentation

issues encountered thus far have been localized maintenance problems rather than problems with a loss of capacity.

(2) Many USACE reservoirs are near or beyond the 50-year time length since construction, and projects are now being operated outside of the design life assumptions. Reservoir deposits are exceeding the planned design life storage and are beginning to impact pool volumes that generate project benefits (Morris et al., 2023). Additional sedimentation will continue to shift priorities at existing projects, causing impacts from sediment accumulation and downstream sediment starvation long before the reservoir fills completely. Some of the sedimentation impacts include:

(a) Reduction in firm yield (maximum yield delivered during drought).

(b) Reduction in reservoir storage with impacts to project purposes (such as flood risk management, navigation, water supply, hydropower, etc.).

(c) Increased spillway use.

(d) Decreased recreation.

(e) Increased cost for hydropower generation.

(f) Loss of flexibility for environmental flows.

(g) Water quality impairment.

(h) Upstream flooding due to backwater effects from delta growth.

(i) Ice jam location/frequency changes.

(j) Blocked inlets.

(k) Downstream bed degradation and bank erosion.

(l) Downstream channel habitat impairment.

(m) Increased O&M costs.

(n) Dam safety issues.

(3) In the long term, without active intervention, reservoir sedimentation will replace most storage volume in all reservoirs. The current "trap and store" strategy for sediment management does not provide sustainable USACE project benefits.

(4) Sustainable reservoirs release the same sediment volume that enters the reservoir, resulting annually in a net zero loss in storage. In most cases, passing sediment through the reservoir to the sediment-starved reach downstream is the most sustainable alternative. In

general, the closer the timing, volume, and gradation of outflowing sediment match the inflowing sediment (taking into account the altered hydrology of the reservoir), the less negatively and more beneficially it will impact downstream ecosystems. Sustainable reservoirs will continue to provide project benefits well beyond the original design life. Reservoir sustainability is further discussed in paragraph 8-12.

(5) Planning for reservoir sediment sustainability will ensure that USACE reservoirs continue to meet their authorized purposes effectively into the future. Portfolio-wide prioritization of USACE reservoirs with respect to benefits and costs can inform data collection efforts, analyses, reservoir sustainability plans, and sustainable actions. Similarly, new reservoir projects should include sustainable operational infrastructures and strategies.

(6) Additional information regarding reservoir sustainability planning is provided in paragraph 8-12 and reservoir sediment strategies is provided in paragraph 8-13.

8-2. Hydrologic and Morphologic Impacts of Dams.

a. Reservoir Impact Overview.

(1) Dams and reservoirs impact the flow of water and sediment that can induce both rapid and long-term morphological changes.

(2) Below a dam, the channel adjusts to both reduced sediment inputs and the altered flows produced by reservoir releases. Dams are highly efficient bedload traps. Even reservoirs operated for sediment release may trap most of the inflowing bed material. Reservoir trapping of bed material often, but not always, results in channel degradation below the dam and lowering of the river base level. Depending on the severity of the degradation, this can trigger many responses, including degradation of tributaries, destabilization and undercutting of streambanks, degradation that undermines bridge piers and river structures, and sediment depletion of river bars (Morris et al., 2008). In some cases, downstream channel aggradation may occur when factors such as flow regulation, sediment inputs, flow roughness change, and similar combine to reduce channel capacity.

(3) Reservoir storage and dam operations that reduce flow extremes can, to some degree, offset the downstream channel instability. However, the flow-sediment relationship following dam construction, with a significant reduction in sediment supply, is typically sufficiently different from the pre-dam condition to initiate channel adjustment processes.

(a) Dam flow releases that are sediment free, often referred to as hungry water, entrains sediment load until it is in balance with the water’s transport capacity. Stated differently, the released water has excess energy that is utilized in the entrainment and movement of sediment from the downstream channel’s bed and banks.

(b) Predicting the magnitude of the morphological changes often requires significant computational evaluation because it is not always clear to what extent reduction in peak flows will offset the reduced sediment supply downstream of the dam. Furthermore, degradation may induce other morphological processes that decrease the transport capacity such as a decreased slope, channel widening, and bed armoring.

(4) Long-term morphological changes are not limited to the downstream river channel. Dams can reduce slope and change flow depths considerable distances upstream of the reservoir pool. Large fluctuations in operating pool elevations can further extend these changes, as can adaptive operational responses such as pool elevation reductions. Figure 8-4 summarizes these typical responses to dam construction, both upstream and downstream of the dam.

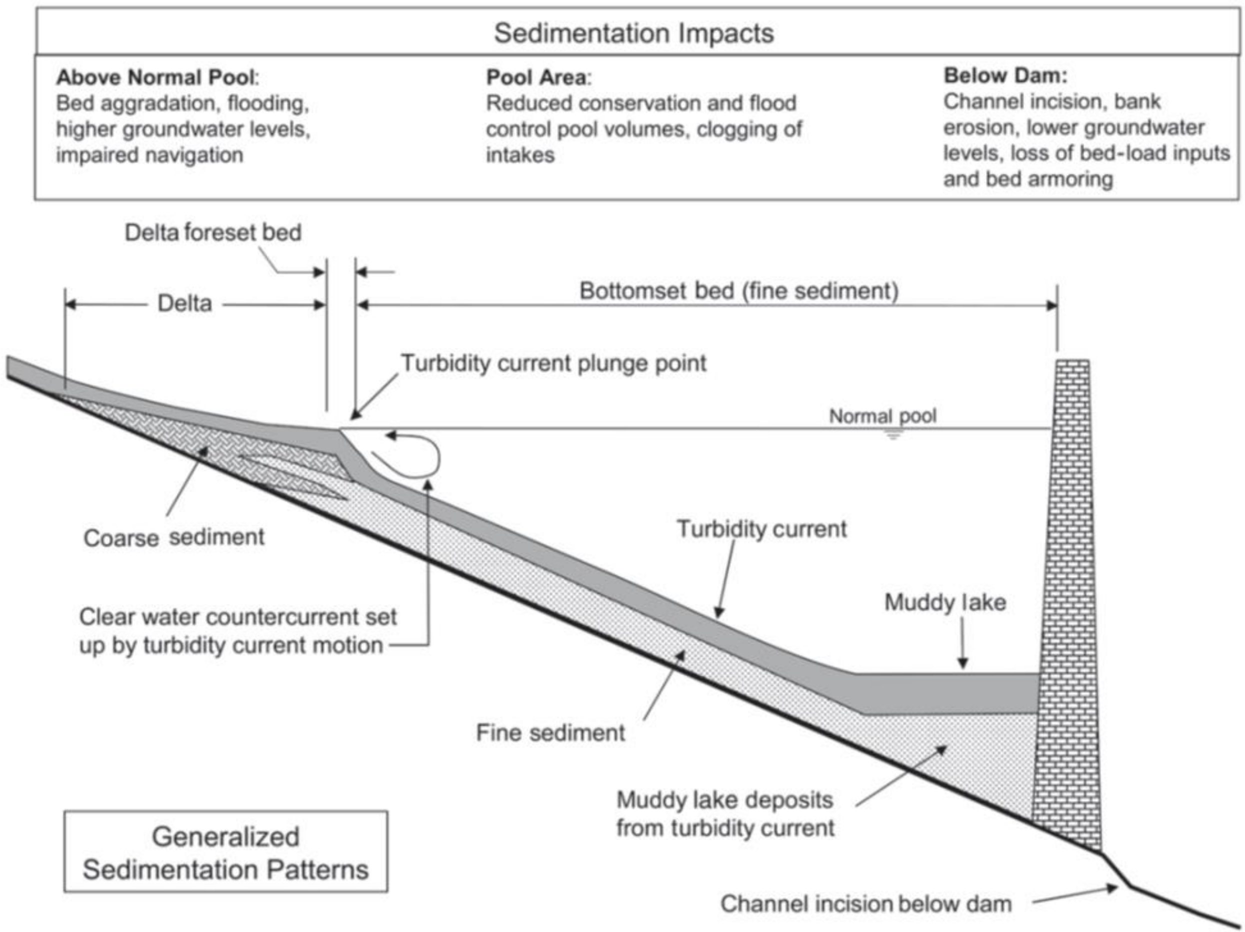


Figure 8-4. Typical sediment responses to dam construction (Morris et al., 2008)

Used with permission of ASCE, from Chapter 12, Sedimentation Engineering, Processes, Measurements, Modeling, and Practice, Manual No. 110, Morris, G., Annandale, G., and Hotchkiss, R., 2008; permission conveyed through Copyright Clearance Center, Inc.

b. Post-Dam Hydrologic Changes. Dams impact the hydrologic state of the river through impoundment of water. The objectives for this impoundment can vary widely and may include providing storage for downstream FRM, storing water for downstream water supply, impounding hydrostatic head for irrigation distribution or hydropower, and/or increasing the surface area for recreation. As population and water demand increase, the benefits derived from an impoundment grow in importance (Podolak and Doyle 2015).

(1) Generally, the operation of reservoirs for downstream flood risk reduction reduces flow variability by capturing and storing higher peak flows and runoff volumes. Downstream water uses such as navigation and water supply often result in a much higher base flow than the pre-dam condition.

(2) Overall, reservoir operations tend to reduce the peaks and augment the troughs of downstream flows (USACE 1997). This transformation narrows the range of flow variability below the dam compared to pre-dam conditions. Figure 8-5 shows the hydrologic effect of upstream reservoir regulation on the Missouri River flow at Sioux City, Iowa, located about 70 river miles downstream from Gavins Point Dam.

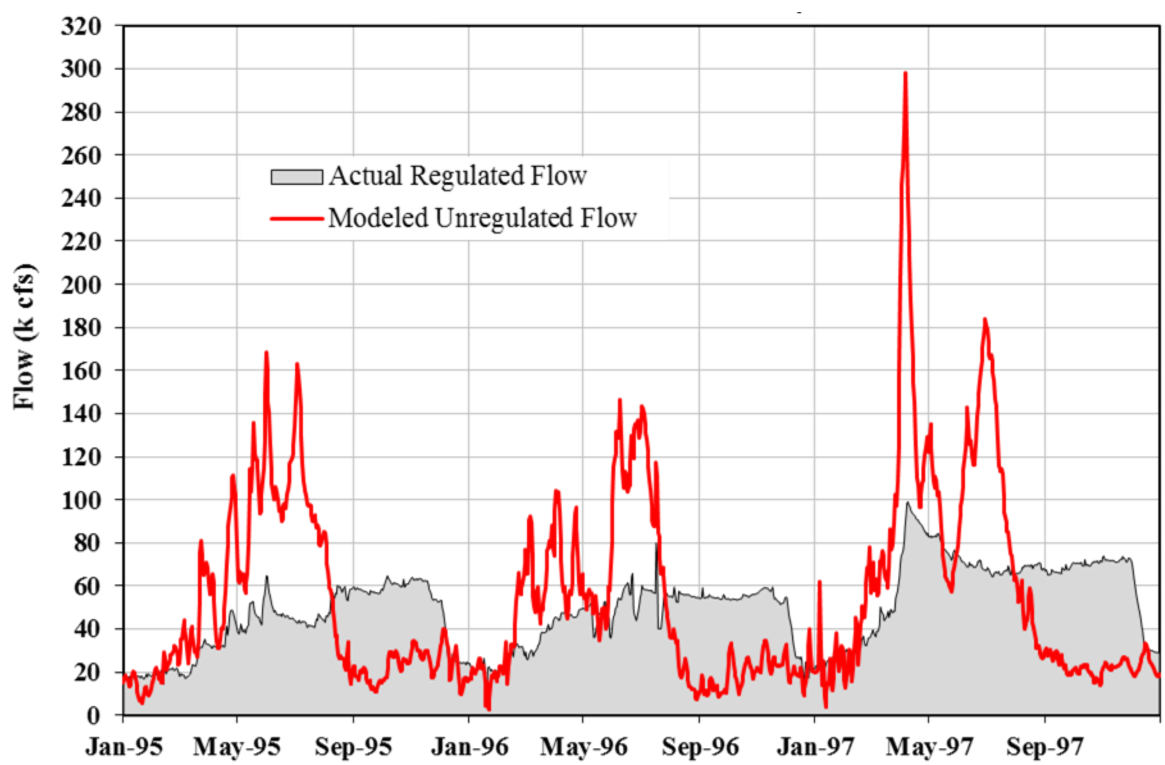


Figure 8-5. Reservoir regulation flow change, Missouri River at Sioux City

c. Upstream Reservoir Impacts.

(1) Impounding water behind a dam decreases the river's kinetic energy, reducing its ability to transport sediments. Impacts include:

(a) Lower energy reservoir flows allow coarser sediments to drop out of the water column. These particles deposit on the channel bed, forming a delta at the interface of the open channel flow and the reservoir pool. At reservoirs with limited variability of the pool elevation, this delta can grow rapidly, extending upstream into the river channel and downstream into the reservoir.

(b) When pool levels rise, the delta deposits may initiate further upstream. At lower pools flows may remobilize and transport sediments farther downstream. For example, an extended period with low reservoir pool levels may result in the river channel incising within previously deposited delta sediments.

(c) Tributary junctions within the reservoir may form additional deposition zones.

(d) Figure 8-6 shows post-dam sedimentation at Cochiti Lake that illustrates a typical progression of the delta over several time periods. Upstream reservoir impacts are further discussed in paragraph 8-5.

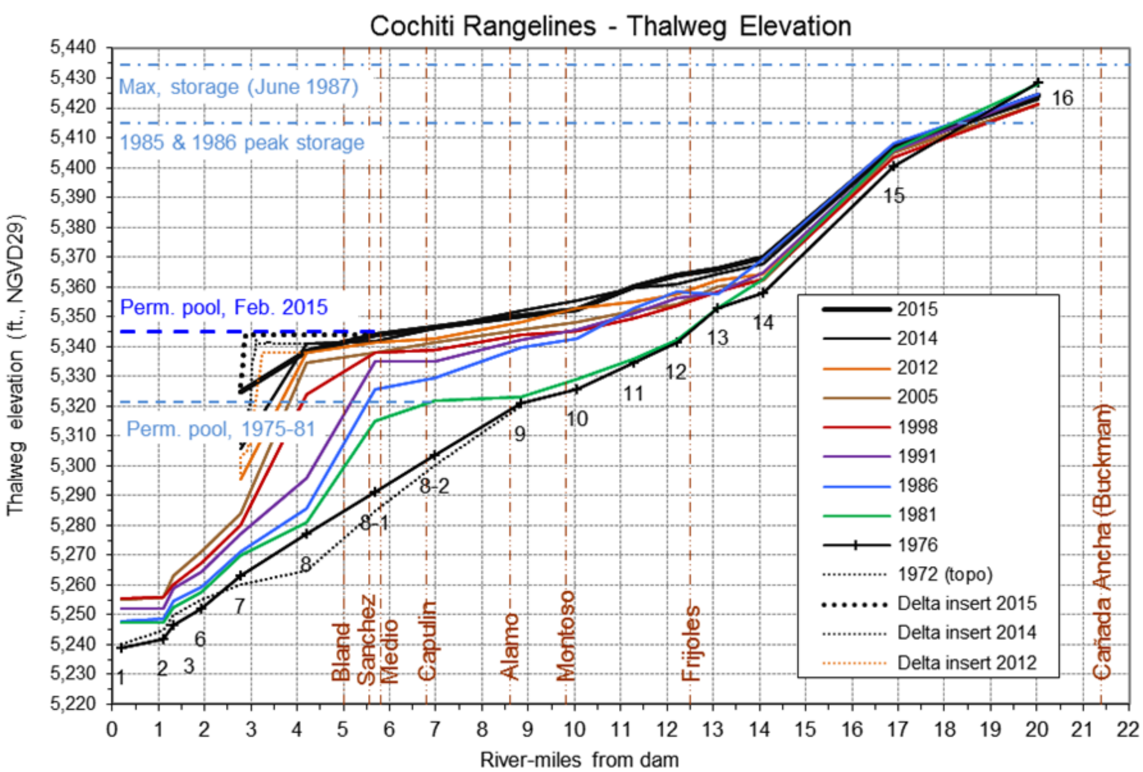


Figure 8-6. Delta progression, Cochiti Lake (USACE 2013b)

(2) The delta depositional features can increase riverine flood risks and have far-ranging impacts on aquatic habitats. Lateral variation in the delta is common. Figure 8-7 depicts the downstream progression of the delta at Lewis and Clark Lake. Paragraph 8-5 of this chapter provides methods for estimating the shape and progression of the delta.

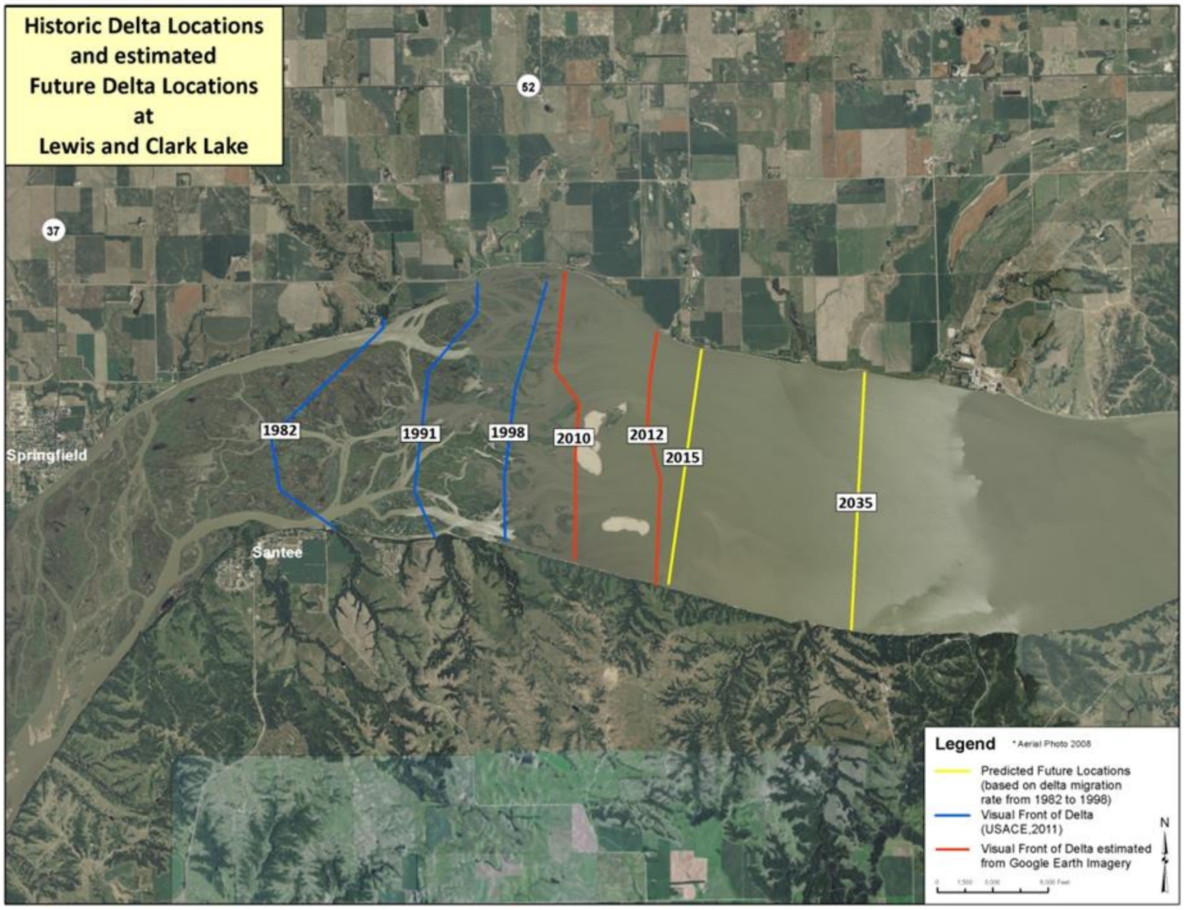


Figure 8-7. Planview delta progression, Lewis and Clark, South Dakota

(3) Backwater effects can create large, deep deltas of coarser materials as the river enters the reservoir. Finer sediments, which require less energy to remain in suspension, travel further into the reservoir before settling. Some portion of the finer sediments may pass through or over the dam. Paragraph 8-4b of this chapter provides methods to estimate reservoir trap efficiency. Secondary forces, associated with temperature, turbulence, or density differences between the water and sediment mixture, may keep some portion of these finer sediments in suspension as a turbid density current. These turbid density currents may sometimes travel through the length of the reservoir and on through the outlet works.

(4) Reservoir pools also impact tributaries. For example, the Black River deposits its bed material load in a large delta that dominates the upper portion of Pool 7 on the Upper Mississippi River (Figure 8-8).

Figure 8-8. Upper Mississippi River, Minnesota, Pool 7 (flow left to right), distributary channels of the Black River provide diverse habitat

d. Downstream Reservoir Impacts.

(1) Trapping of coarse sediments in the reservoir results in the discharge of sediment-poor water, also termed “hungry” water. This often causes downstream channel incision and the formation of a bed armor layer. Morphological responses can propagate downstream many kilometers until the fluvial system reestablishes equilibrium (Williams and Wolman 1984).

(2) While in some systems, the trapping of fine sediments may be seen as a positive environmental function of reservoirs, in other systems, the discharge of unnaturally clear water favors non-native fish over native, turbidity-dependent species. Potential infrastructure and environmental effects of these physical changes are discussed further in paragraph 8-6.

(3) In addition to the effects of sediment trapping, flow regulation may induce significant geomorphic and ecologic change. Flow regulation typically attenuates peak flows and augments base flows, which changes the downstream channel sediment transport capacity. This may increase or decrease the overall transport capacity, depending on the site, but it generally decreases transport capacity.

(a) Due to the decreased transport capacity, dams often cause deposits where tributaries join the river downstream of the dam. The regulated flow regime no longer has the capacity to transport sediments delivered by tributaries, causing deposits that can impact ecosystems (such as burry spawning gravel).

(b) Ayles and Church (2015) relate how dam construction on the Peace River in northern British Columbia and Alberta has significantly reduced peak flows while leaving sediment delivery to the river largely unchanged. In this case, aggradation has occurred near tributary confluences and other sediment sources.

(c) Altering the duration and frequency of floodplain inundation may lead to vegetation changes in the riparian corridor with a variety of both physical and environmental impacts.

e. Setting the Baseline to Evaluate Morphologic Changes. Defining the baseline condition is a prerequisite to understanding the morphologic changes that occurred following project construction or will occur following new projects. Baseline conditions can often influence project objectives and evaluation methods. Multiple options exist for formulating this baseline. The most appropriate formulation depends on project goals and data availability.

(1) The first option is to assume the river was in a state of dynamic equilibrium (see Chapter 7) before dam construction. This condition may correspond to the river condition before the agricultural or urban development of the watershed that coincided with European settlement.

(2) The second option for a baseline assessment is to start with the pre-dam condition. The pre-dam condition is usually described by the period of study before dam construction, including channel geometry, flow regime, sediment measurements, and other data collection that characterized pre-dam conditions. This may have been a period of river instability, as the river adjusted its dimensions and character in response to large scale watershed and stream network changes.

(3) The third option for a baseline assessment is to create a current watershed "no-dam" condition, in which the current system is analyzed with the reservoir effects removed. This analysis usually requires upstream gaging, numerical modeling, and application of geomorphic adjustment concepts (FISRWG 1998; Copeland et al., 2001; NRCS 2007).

f. Numerical Modeling. A range of numerical modeling methods are available as analysis tools.

(1) Simplified analysis methods including empirical relationships and simplified transport solutions may be appropriate for general trends and reconnaissance-level studies.

(2) Spreadsheet tools may be suitable for rapid qualitative assessments. Parker (2006) provides a suite of spreadsheet tools on his website "1D Sediment Transport Morphodynamics with applications to Rivers and Turbidity Currents."

(3) Numerical (and in some cases physical) modeling is recommended for feasibility studies and designs where sedimentation processes may alter plan selection or design choices.

(4) Both 1D and 2D models have been used with success to model reservoir and downstream channel responses.

(5) Refer to paragraph 2-2 for guidance on numerical modeling levels of investigation and approved model software and available guidance (USACE 2011a, internal HH&C CoP SharePoint). See Chapter 9 for guidance on model selection and parameterization.

8-3. Sediment Delivery to Reservoirs.

a. Sediment Delivery to Reservoirs. The sediments delivered to a reservoir are quantified by sediment yield—the amount of sediment transported beyond or delivered to a specified point in the drainage network over a specified time period. The sediment yield is typically much less than the combined inputs from watershed erosion and stream bank and bed erosion, as much of the sediment that erodes in a watershed redeposits elsewhere in the system.

(1) Sediment delivery is affected by stream dynamics and equilibrium processes as discussed in Chapter 7. Sources of sediment include fine sediments originating in upland sources and contributed via tributaries, fine and coarse sediments originating from river channel banks and bed, and fine and coarse sediments originating from reservoir margins.

(2) Methods for predicting and evaluating sediment yield from watersheds are discussed in detail in Chapter 6. Chapter 6 also provides guidance on the selection and application of procedures for calculating sediment yield, along with the underlying assumptions in these methods.

(3) The following sections discuss the temporal and spatial variability in sediment yield and reservoir banks as an important source of sediment input to the reservoir. Understanding these topics is important for selecting and designing effective sediment management measures.

b. Variability in Space.

(1) A small part of a watershed can contribute a disproportionately large amount of the total sediment yield depending on underlying geology, land use, and history of stream straightening. As a consequence, dividing total sediment discharge by total basin area to obtain an average yield can mask an underlying variability in sediment yield. Variations in specific sediment yield, the sediment yield per unit of land area, can be particularly significant in watersheds subject to disturbance from human activities or from catastrophic natural events.

(2) Watersheds can have distinctly different sediment delivery characteristics: (a) watersheds where the sediment delivery ratio is low due to considerable storage of sediments in the channels and valleys (source dominated systems); and (b) watersheds where the channels are efficient conveyors of sediment, and the sediment delivery ratios are high (pathway-dominated systems).

(3) In watersheds with considerable sediment storage and a low downstream sediment delivery ratio such as those discussed above, the upland sediment supply may correlate poorly with downstream sediment delivery. In systems such as these, upstream erosion control methods may not have a significant impact on downstream sediment delivery, particularly in the short term. However, in many watersheds the channels efficiently convey sediment and act as dominant sediment sources rather than sinks (Leech and Biedenharn 2012).

(4) In Midwest agricultural watersheds, sediment fingerprinting and modeling of three events demonstrated that the eroding banks contributed from 40% to 60% of the total sediment (Wilson et al., 2014). These percentages are watershed specific.

(5) Land use within the watershed affects sediment delivery. For example, the geomorphic effects of forest roads range from consistent long-term contributions of fine sediment into streams to mass failures of road fill material and slopes during large storms.

(a) Although mass erosion rates from roads are typically one to several orders of magnitude higher than from other land uses, roads usually occupy a relatively small fraction of the landscape, so their combined effect on erosion may be more comparable to other activities such as logging (Gucinski et al., 2001).

(b) Swanson and Dyrness (1975) reported that along road rights-of-way in an Oregon study area, slide erosion was 30 times greater than on forested sites; however, only about 8% of a typical area of deforested land in the study area consisted of road right-of-way. At comparable levels of development, road right-of-way and clear-cuts in the study area contributed about equally to the total impact on erosion by landslides. In the study, the combined impacts of roads and logging appear to have increased slide activity on roads and clear-cut sites fivefold relative to forested areas over a period of about 20 years. Many other studies, such as Megahan (1978) in Idaho, report comparable effects.

(6) High spatial variability in sediment yield also applies to basin scale analyses. Using a sample set of 1,358 gaging stations worldwide with watershed size between 350 and 100,000 km^2, Jansson (1988) showed that over 69% of the sediment yield was contributed by less than 9% of the total land area.

c. Variability in Time.

(1) The sediment yield from a watershed into a reservoir is subject to wide variability at different time scales ranging from single storm contributions to variations in yield over decades. Because reservoirs need to be managed for sustainability in the long term, predicting temporal variations in sediment yield is very important for understanding reservoir sedimentation and sediment management planning. Moreover, several sediment management methods depend on the temporal variability in sediment delivery to bypass or pass through sediment-laden water while retaining clearer water. The patterns and causes of some of the variability in time for sediment yield are discussed below.

(2) Single Storm Variation. Plots of suspended sediment concentration (SSC or C) and discharge (Q), called C-Q plots, exhibit several different behaviors. Concentration increases as flow increases that lead to an exponential increase in sediment mass at higher discharges. In general, higher flows deliver a disproportionate amount of sediment, a fact that forms the basis for sediment bypass strategies.

(3) Over the course of a single storm, however, sediment transport is not necessarily uniquely tied to flow in the river. A single discharge may carry different sediment loads at different times in the storm hydrograph. Figure 8-9 shows examples of different types of C-Q plots. When sediment concentration and water discharge are measured continuously for the duration of a flood event, the data pairs often produce a hysteresis effect in the concentration-discharge time-history (Williams 1989; Tananaev 2015; Smith et al., 2009).

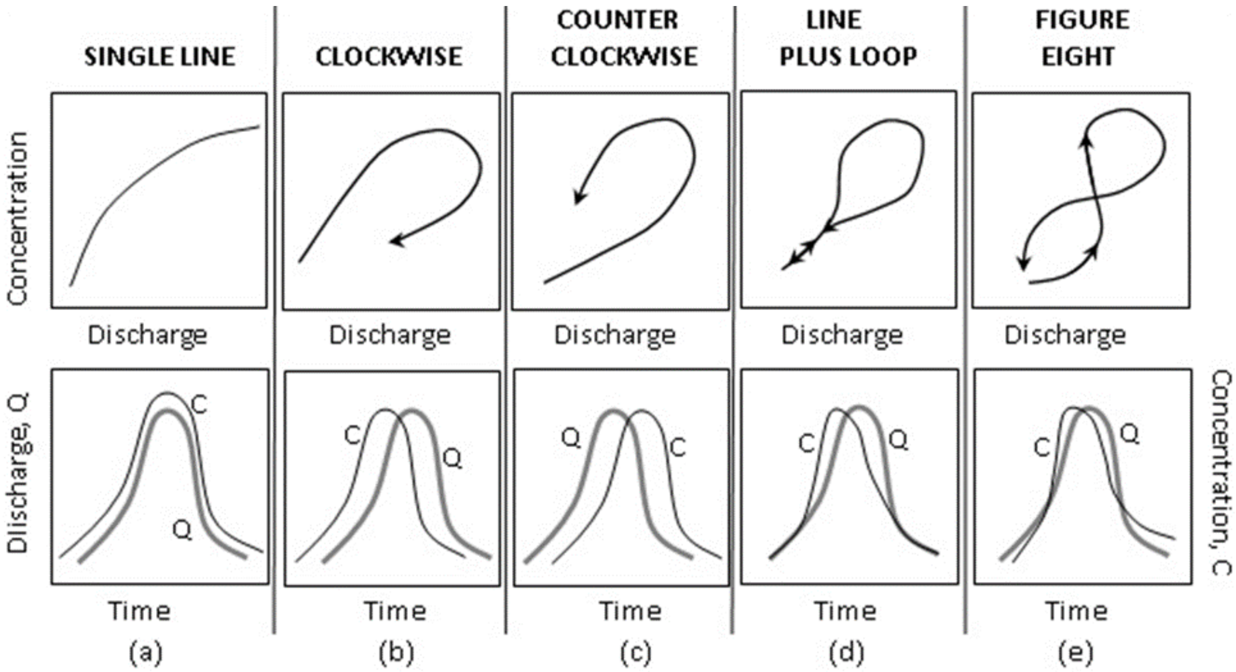


Figure 8-9. Hysteresis in sediment concentration vs. water discharge (redrawn from Williams 1989)

(4) Figure 8-9 provides theoretical relationships between discharge and concentration during a single hydrologic event on rivers. The lower graphs show the variation in concentration and discharge over time, and the upper graphs show the hysteresis effect when these data are plotted as C-Q graphs. In addition to the single-valued relation between flow (discharge) and sediment inflow (concentration) shown in Figure 8-9 panel (a), Figure 8-9 shows several other distinct behaviors that have been observed either on different rivers or on the same river for different storms.

(5) The hysteresis in sediment production is the direct result of the specific location and amount of sediment available for transport. As summarized in Morris and Leach (2013, pg. 53): “The more common pattern is for sediment concentration to peak before discharge peaks,

producing a clockwise concentration-discharge (C-Q) hysteresis loop (panel (b)). This can occur when the first part of the flood washes out readily mobilized sediment, leaving the latter portion of the hydrograph relatively deficient in sediment. Counter-clockwise loops (panel (c)), which are also common, can occur when more distant areas of the watershed have more erodible soils or when landslides develop as soils become oversaturated as the storm progresses. The hysteresis pattern is not necessarily a fixed watershed characteristic, and different storms can produce different patterns in the same watershed."

(6) Other C-Q behaviors such as the line-plus-loop and the figure eight relations have also been observed in rivers (panels (d) and (e)).

(7) Seasonal Variation. Most watersheds have one or two seasons with high flows and sediment yield from high rainfall or snowmelt.

(a) The delivery rate for the fine fraction of total sediment load, the wash load, depends in part on rainfall-runoff processes and erosion in the watershed.

(b) Transport of the coarser bed material that comprises the stream bed is driven primarily by stream hydraulics and correlates well with discharge so long as no sediment pulses are present in the system. For this reason, early in the flood season, a coarse-bedded river may carry a high total suspended load consisting mostly of fine sediments, while later in the flood season, at the same discharge, the flow may have a much lower SSC of fines and a changed bedload fraction. This constitutes a variation of both the amount of sediment transported as well as the size gradation of sediments, depending on the time in the flood season of a particular storm.

(c) Late-season suspended-sediment yield may also be reduced by factors such as seasonal freezing and thawing, vegetation growth during the wet season, and the availability of readily mobilized sediment.

(8) Multi-Year Variation Due to Flooding. Climate and the recent flood history can also affect sediment supply.

(a) For example, after several years with moderate rainfall and minimal flooding, sediment storage within the watershed and in river and stream channels can become significant, temporarily reducing the yield to a reservoir. A large flood after such a dry period can transport the stored sediment downstream and into the reservoir in a single event, as well as reduce the sediment available for transport in future floods.

(b) Meade and Parker (1984) found that 50% of the annual sediment load is discharged on 1% of the days.

(c) Extreme storms or cycles of wet and dry years can dramatically influence annual yield, and it is not unusual for a single large storm event to deliver more sediment than an entire year of average flows (Morris et al., 2008).

(9) Figure 8-10 shows the influence of single high-precipitation storm events on suspended-sediment outflow for a watershed in Puerto Rico on the Valenciano River near the site of a proposed new dam and water treatment facility. The increases in sediment yield from Hurricane Hortense in September 1996, and Hurricane Georges in September/October 1998, are clearly visible. During Hortense, the 24-hour rainfall at San Lorenzo near the upper reaches of the Valenciano River recorded the highest rainfall of the event in Puerto Rico: 24.6 in. of rain.

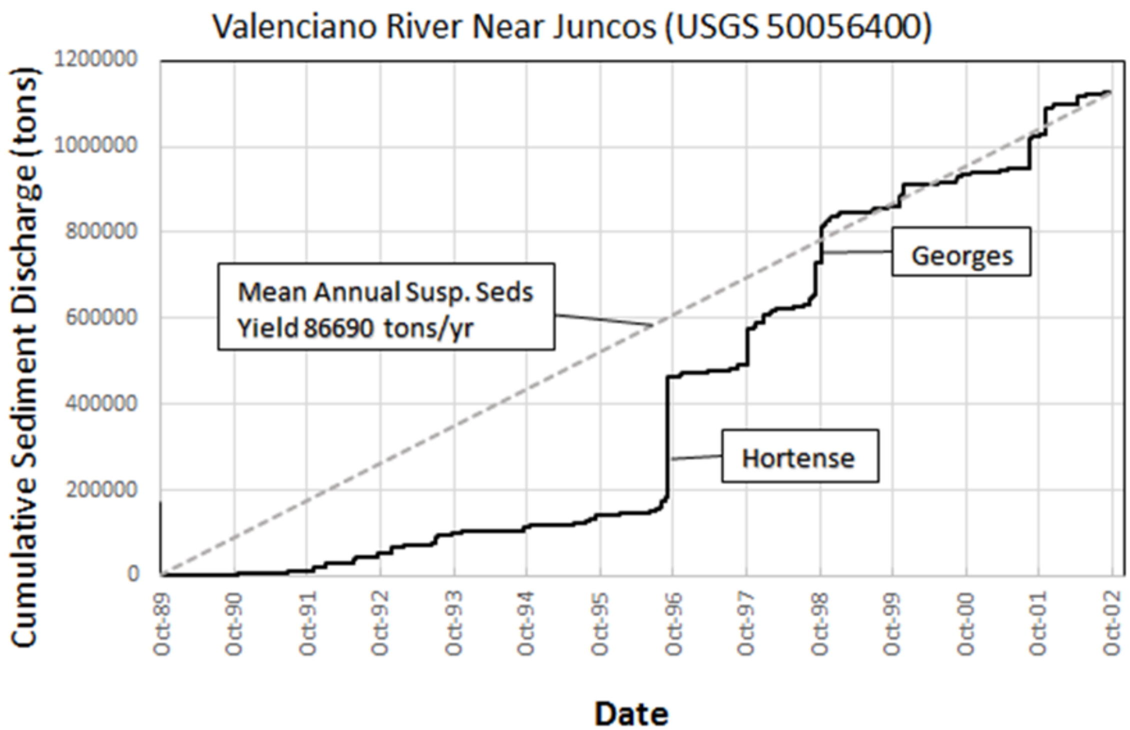


Figure 8-10. Cumulative sediment yield, Valenciano River near Juncos, Puerto Rico

(10) As seen in Figure 8-10, the amount of sediment transported in one event was well in excess of the accumulated transport in the previous 7 years. This highlights not only the importance of high-load events for predicting the long-term sedimentation in a reservoir, but also the need for long records for sediment yield to capture trends and transport extremes.

(11) Long-Term Variation. Estimates of long-term sediment yield have been used for many decades to size the sediment storage pool and estimate reservoir life. Many USACE projects report sedimentation rates with reasonable agreement to design rates. However, because these estimates were often based on limited data taken at low inflow rates, some USACE reservoirs have accumulated sediment more rapidly than originally planned.

(a) The delivery of sediment into a reservoir is highly variable over time, with most sediment being delivered by large floods. Furthermore, since this transport occurs during large events, it is often difficult to measure accurately and may not be represented properly even in watersheds with a long history of gage data.

(b) Using sediment yield estimates for reservoir studies should recognize accuracy limitations, especially when short-duration or poor-quality data are used in the analysis.

(12) Kirchner et al. (2001) used cosmogenic radionuclides to determine the erosion rates of rock outcrops in Idaho and found that long-term sediment yields over a period of 5,000 to 27,000 years were, on average, 17 times higher than those measured using 10 to 84 years of gage data.

(a) Caution is advised when interpreting differences between short- and long-term trends as multiple other factors such as the effects of redistribution of sediments in the watershed, anthropogenic factors such as land management and water resources development, climatic affects, and similar phenomena may significantly affect transport over the years of gage data.

(b) Events such as landslides, debris flows, fires, and drought also play a large role in the production of sediment for transport. Hillsides and channels may accumulate these sediments over a period of decades and have this load washed out and delivered downstream by an extreme flood.

(13) Human activities and land use changes can also affect long-term sediment yield rates. Reservoirs designed and built in relatively undisturbed areas can be impacted by roads and development in the watershed as well as changes in shoreline influx of sediment from recreational use and pool operations of the reservoir itself.

(a) Garbrecht (2011) used a sedimentation survey of the Fort Cobb Reservoir in west central Oklahoma and sediment load measurements on contributing tributaries to evaluate the effectiveness of soil conservation practices at reducing watershed sediment yield and reservoir sediment accumulation rates. He found that sediment yield reductions achieved by conservation efforts of approximately 60% to 65% were offset by a shift in the mid-1980s toward wetter climatic conditions that increased soil erosion and sediment delivery to the reservoir. In this case, Garbrecht recommends that the interaction between wetter climate and effectiveness of conservation practices be considered when projecting future sedimentation rates and reservoir life span.

(b) Miller et al. (2005) used radionuclide, stratigraphic, and cartographic data covering a span of 111 years to deduce sedimentation rates in Fairfield Lake, North Carolina. The 0.3 km^2 lake lies in a drainage area of 7.3 km^2. They found that sedimentation rates in the lake increased by 1.5 to 10 times during the period from the 1970s to 1980s to 2001, likely in response to land use changes. While rates of sedimentation were expected to increase with development, the degree to which depositional rates were affected by land use change was surprising given the limited amount of development within the watershed (0.12 buildings per hectare in sub-basins that deliver sediments to the lake).

(14) While development may increase reservoir sedimentation, long-term trends can also have reduced sediment yields. Factors include increased water use, soil and water conservation measures implemented, upstream dams constructed, and reduced stream flow to transport sediment. Figure 8-11 shows the trend over 50 years in the Middle Reaches of the Yellow River

(MRYR). Increased demand for water resources in the Yellow River due to the rapid development of China's national economy has dramatically increased water extraction and diversion for agricultural, irrigation, urban, and industrial use, especially after the 1980s. Erosion control measures implemented over about 49% of the land area has also reduced sediment yield (Gao et al., 2011).

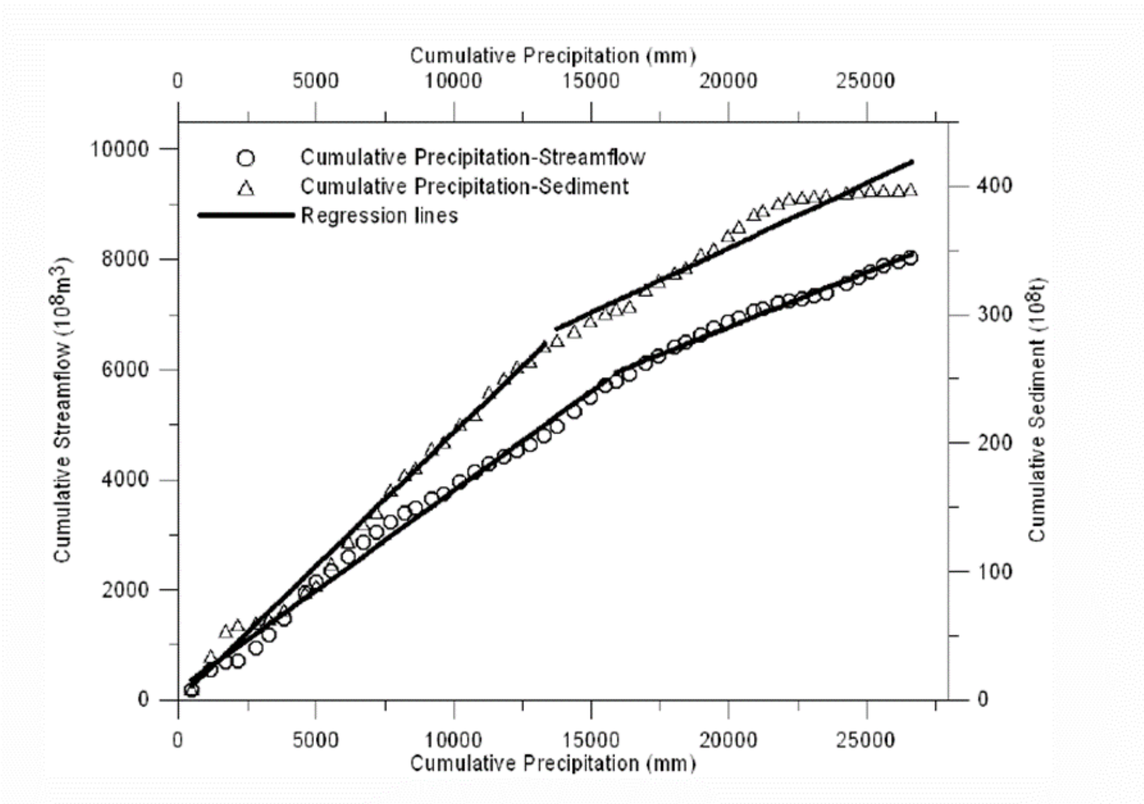


Figure 8-11. Curves for cumulative precipitation-streamflow and precipitation sediment from 1957 to 2008 in the Middle Yellow River (Gao 2011)

(15) The slope of the flow and sediment response curve changes in the 1980s (around 15,000 cumulative mm since 1957).

d. Wildfires. Extensive and frequent wildfires expose the reservoir watershed to erosion. Widespread wildfire followed by regional storms of even moderate intensity can drastically increase short-term sediment yield and disturb the drainage system. The magnitude of watershed hydrologic and sediment delivery changes following wildfire depend on many factors including burn severity, landscape susceptibility to erosion, and the timing and magnitude of storms that follow the fire. Refer to paragraph 6-7 for further information on the evaluation of wildfire impacts on runoff and sediment yield. Wildfire severity can be increased by reservoir operations as discussed in paragraph 8-5.

e. Erosion of Reservoir Banks.

(1) In addition to sedimentation delivered from upstream sources, reservoirs also recruit sediments from their shorelines, either by gradual soil loss and scour or through slumps and landslides. Erosion can be gradual or catastrophic. Depending on variable conditions such as reservoir pool shape, prevailing winds, recreational use, and soil types, shoreline erosion rates can be substantial. Refer to paragraph 8-5c(3) for additional discussion of sediment loading due to landsides.

(2) Lyons (1996) surveyed shoreline problems at USBR reservoirs. Erosion was reported at 114 of 154 reservoirs surveyed. Erosion problems affected 27% of the 9,700 miles of shoreline where data was available. Much of the shoreline damage was reported as minor. The most common potential impacts of shoreline erosion were to archaeological sites, government property, water quality, and fish and wildlife. However, at 14 sites shoreline erosion was considered of sufficient severity to affect project life.

(3) Shoreline erosion can occur for a wide range of reasons. Shoreline recession rates are location and event specific, and are also highly affected by the underlying geology of the shoreline. The new bankline established by a reservoir may be at an elevation in the adjacent landforms that is not resistant to erosion. In reservoir reaches that are broad or are aligned along the prevailing winds, large erosive waves often undercut and erode banks. The wind-wave action can cause a littoral type of drift along the reservoir bankline and transport eroded material further into the reservoir.

(4) Reservoir shorelines is usually erode faster in alluvial systems than mountainous regions. Gatto and Doe (1987) provide an extensive list of the most frequently observed processes responsible for reservoir bank erosion, some of which are summarized below:

(a) Waves and Currents. Wind-driven waves and boat wakes cause periodic hydraulic loading to shorelines, while currents from circulation, tributary inflows and wind also contribute to soil removal and bank failure. A frequent mechanism of the wave and current erosion is the undercutting of banks leading to subsequent slumping of bank sediments.

(b) Groundwater. In addition to preexisting groundwater seepage and piping on slopes, groundwater flow can occur after rainfall or in response to fluctuations in reservoir levels. Rapidly dropping water levels can cause excess pore pressures and slope failure.

(c) Ice. Ice forces include frost wedging and heaving of soil, scour and impingement by ice on the shoreline, and ground ice slumping.

(d) Rainfall-Driven Runoff. The same erosive processes that contribute to erosion upstream are also present at the reservoir shoreline. Weathering, raindrop impact, rill and gully erosion, and sheet flow can transport sediments from the shoreline areas into the reservoir.

(e) Bioturbation. Activity of ground-dwelling and burrowing animals and insects near the reservoir can loosen soil and destabilize banks. Additionally, plant roots and tree fall also affect soil stability.

(f) Soil Heterogeneity and Preexisting Failure Planes. The presence of weak or erodible soils and previous landslides can contribute to accelerated erosion or failure of shoreline slopes. This can be of particular importance during the filling of a reservoir or during water level raises that saturate areas not usually exposed to water.

(g) Human Activity. Besides the impacts of boat wakes mentioned above, roads and development of reservoir shorelines can increase both soil erosion and the potential for landslides.

(5) Gatto and Doe (1987) investigated the amount and causes of shoreline recession in 10 reservoirs. Table 8-1 provides a summary of their results. The study shows that more than one erosion mechanism can be present at the same location, leading to complex interactions and difficult-to-predict erosion patterns.

Table 8-1
Summary of Historical Recession (revised from Gatto and Doe 1987)

Reservoir	Average Recession (m/yr)	Monitoring Period	Dominant Processes
Allegheny	1.2	1968–1977	Wind wave erosion, groundwater-induced sliding.
Berlin	1.5	1951–1979	Wind wave erosion, groundwater-induced sliding.
Big Sandy	0.6	1939–1969	Wind wave erosion, landslides.
Orwell	1.8	1953–1970	Wind wave erosion, initial reservoir filling, freeze-thaw effects, raindrop and sheetflow erosion, groundwater-induced sliding.
Oahe	11.9	1968–1976	Wind wave erosion, initial reservoir filling.
Sakakawea	5.2	1958–1986	Wind wave erosion, initial reservoir filling.
Fort Peck	0.9	1950–1978	Wind wave erosion.
Dworshak	–	–	Landslides, positive pore pressures.
Pend Oreille	1.5	1958–1970	Wind wave erosion, freeze-thaw effects, groundwater-induced sliding, boat waves.
Rufus Woods	2.4	1955–1977	Wind wave erosion, groundwater-induced sliding, initial reservoir filling.

(6) Several studies have further examined wave effects on reservoir sedimentation from shoreline sources. Elci et al. (2007) developed a shoreline erosion prediction methodology that describes shoreline erosion as a function of lake levels, wind direction and magnitude, fetch, and beach profile shape using data from Hartwell Lake upstream of the USACE-operated Hartwell Dam. In their study of the Baskatong Reservoir in Quebec, Saint-Laurent et al. (2001) concluded that wave action was the primary erosion mechanism, especially in highly exposed areas with large fetches. They also suggested a correlation between the sediment characteristics, fetch distribution, and the wave erosion energy.

(7) Reservoirs may have narrow or dendritic planforms, which can give them very long shorelines relative to their surface area. For example, Lake Nasser upstream of the High Aswan Dam is 550 km (334 mi) long, but the shoreline is more than 10 times longer (8,900 km, 5,600 mi). The six Missouri River lakes have 745 miles of river impoundment with 5,940 miles of shoreline (Dorough and Reid 1991).

(8) Average shoreline recession rates in the six Missouri River lakes were determined to range from 2.3 to 8.2 ft/yr with some areas of 15 to 40 ft/yr (Dorough and Reid 1991). Gatto and Doe (1987) reported a recession rate of 11.9 ft/yr in the Oahe reservoir, one of the Missouri River reservoirs in South Dakota. Shoreline erosion may mobilize into the lake or remain near the source. Figure 8-12 shows a typical slide with eroded material at Lake Sakakawea.

Figure 8-12. Shoreline erosion with deposited material, Lake Sakakawea (2010)

(9) Shoreline erosion rates at Lake Sharpe are quite high due to multiple factors including bank height, soil types, and prevailing winds (Figure 8-13).

Figure 8-13. Vertical cut retreating bank, Lake Sharpe (2012)

(10) While shoreline erosion is typically not a threat to the maintenance of overall reservoir capacity, it can cause impacts to authorized purposes by causing siltation near water intakes, boat ramps, or other infrastructure. Depending on the extent of government-owned land, shoreline erosion may have the potential to impact private lands. Furthermore, even minor erosion can threaten the stability of costly shoreline properties and infrastructure. Figure 8-14 shows the undermining of adjacent infrastructure at Bowman-Haley in 2010.

(11) Project purposes and operations can be impacted by small amounts of shoreline erosion in cases where facilities are directly adjacent (water intakes, boat ramps, recreations trails, roadways, etc.).

(a) In some cases, it may be feasible to modify the operating plan to reduce water level fluctuations and the resultant erosion. Otherwise, because of the long length of shoreline, it is not generally feasible to protect against shoreline erosion except in localized areas where high-value property or structures are threatened.

(b) Practices to stabilize shorelines in specific localities may include the use of riprap, sheet piling, or other conventional engineering measures, and reduction of boat speeds to minimize wake (Morris and Fan 1998). Bank and shoreline stabilization design methods are presented in multiple USACE references, including EM 1110-2-1100, EM 1110-2-1601, and EM 1110-2-1614.

Figure 8-14. Infrastructure failure, Bowman-Haley (2010)

f. Jennings Randolph Example.

(1) Jennings Randolph reservoir provides an example of how a better understanding of the temporal and spatial variability of sediment yield leads to better information for reservoir planning and sediment management purposes.

(2) Based on several sediment studies conducted before impoundment in the 1960s, the annual sediment yield to the Jennings Randolph reservoir was estimated to be approximately 20 acre-feet per year. Thus, 2,065 acre-feet of reservoir storage was allocated to sediment for the 100-year life of the reservoir. The dam was completed in May 1981, and the conservation pool was filled in May 1982.

(a) A large amount of deposited sediment was noticed in the headwaters of the reservoir in November 1984 and January 1986. Reservoir surveys computed total deposition of 270 and 900 acre-feet respectively, a five- to ten-fold increase from expectations. The study concluded that extreme events tended to dominate sediment transport in this mountainous watershed.

(b) Although the original estimate of 20 acre-feet per year was based on valid data, and was computed using accepted methods; a sediment discharge rating curve based on data up to the 2-year event, as used in the pre-impoundment studies, could not be accurately extrapolated to reflect the sediment discharge for major flood events. A single extreme event may produce one to two orders of magnitude more sediment yield than a typical 2-year event.

(c) Burns and MacArthur (1996) re-evaluated the reservoir using 10 different methods, including the original studies, and found the annual sediment yield to Jennings Randolph Reservoir ranges from a low of 0.06 to a high of 1.75 acre-feet per square mile per year. They concluded that an annual sediment yield to the reservoir of 92 acre-feet per year would be a reasonable value of storage loss for planning purposes. This is much higher than the pre-impoundment estimate of 20 acre-feet per year.

(3) In summary, sediment yield from the watershed is a critical input to understanding the sedimentation behavior in a reservoir. Because of uncertainties in collected data, natural variability and both spatial and temporal variations, sediment yields need to be assessed with caution. White (2005) summarizes some recommendations to use when evaluating sediment yield.

(a) Understand what is controlling sediment yield in the basin of interest.

(b) Be realistic about how things (land use, land management, climate) have changed and how they are likely to change.

(c) Never make a single number prediction of sediment yield; at least include some error bars or a distribution of likely yields.

(d) Recognize that all estimates require some sediment yield data (for model development, calibration, and validation).

(e) Put available data into a longer term context.

(f) Include extreme events in the assessment, as they often control sediment yield over the long term.

8-4. Sediment Deposition in Reservoirs. Sediments that accumulate in reservoirs are transported primarily by fluvial processes. Mass wasting, ice forces, and aeolian transport represent much smaller contributions. To properly predict sediment transport and deposition by hydraulic forces, it is important to characterize the bed material, the sediment that comprises the bed of the river or reservoir. These sediments may be transported as either bedload, particles that move along in frequent contact with the bottom flow boundary, or as suspended load, particles that are maintained within the water column.

a. Stages of Reservoir Life.

(1) Sedimentation conditions in a reservoir reach can be broken down into four main phases:

(a) Pre-Dam Sediment Balance or Quasi-Balance.

(b) Stage 1: Initial Rapid Changes Accompanying Reservoir Construction.

(c) Stage 2: Transition from Horizontal Sedimentation to Channel and Floodplain Morphology.

(d) Stage 3: Long-Term Sediment Balance Including Reservoir.

(2) These four phases are summarized below using descriptions from Morris and Fan (1998) and Morris et al. (2008).

(a) Pre-Impoundment Sediment Balance. Before dam construction, most river reaches are perceived as being approximately balanced with respect to sediment inflows and outflows. Sediments may temporarily accumulate in some channel reaches, but are mobilized and transported downstream by larger floods. The total amount of sediment transported through a reach is much larger than the rate of aggradation or degradation within the reach. Note that in some watersheds, this assumption may not be valid due to anthropogenic and other influences on watershed processes and sediment balance.

(b) Stage 1: Continuous Sediment Trapping. Dam construction drastically alters hydraulic conditions in the river, converting the flowing stream into a pool characterized by low velocity and efficient sediment trapping. During the first stage of reservoir life, continuous sediment trapping occurs during all inflowing flood events.

- Coarse bed material load is deposited as soon as stream velocity diminishes as a result of backwater from the dam, creating delta deposits at points of tributary inflow.

- Finer sediments are carried further into the reservoir by either stratified or non-stratified flow and accumulate downstream of the delta deposits. These finer sediments fill in the deepest part of the cross section first, eventually producing sediment deposits that are essentially flat, after which continued deposition produces horizontal sediment beds extending across the width of the pool.

(c) Stage 2: Main Channel and Growing Floodplain. At some point, the reservoir transitions from continuous deposition to a mixed regime of deposition and scour, and the rate of sediment deposition is reduced compared to deposition in the earlier stage.

- In wide reservoirs, this stage is also characterized by the transition of sediment deposits from horizontal beds to a channel-floodplain configuration, while in narrow reaches, the channel may occupy the entire reservoir width.

- This transition will occur naturally when sedimentation reaches the spillway crest; a main channel will be maintained by scour, and its base level will be established by the spillway. The inflow and discharge of fine sediment may be nearly balanced, but coarse bed material continues to accumulate.

- Sediment management techniques, such as drawdown to pass sediment-laden flood flows through the impounded reach or periodic flushing, can produce a partial sediment balance to help preserve useful reservoir capacity.

(d) Stage 3: Full Sediment Balance. A long-term balance between sediment inflow and outflow is achieved when both the fine and the coarse portions of the inflowing load can be transported beyond the dam or artificially removed on a sustainable basis. Considerable upstream aggradation may occur above the spillway crest, and delta deposits must reach the dam before this balance is reached.

(3) Most of the world's reservoirs are relatively young and are not managed for sediment, placing them in Stage 1. The structural and operational requirements for operation in Stage 2 or 3 have been investigated or implemented at relatively few sites worldwide. A notable example is the Three Gorges reservoir on China's Yangtze River, designed to reach full sediment balance after about 100 years.

(4) A concept commonly applied to reservoir sedimentation is the "reservoir half-life," the time required for sediment to fill half the original reservoir capacity. This is often substantially less than one-half of the total design life since sedimentation declines as the reservoir fills.

(a) Half-life can be a good approximation of when sedimentation problems will become serious. At many sites, however, sediments can seriously interfere with reservoir function even when much less than half the original capacity has been lost.

(b) Loehlein (1999) describes sedimentation issues at the Conemaugh River dam in Pennsylvania and at two other USACE flood control dams where sediment-related problems occurred with less than 6% storage loss. Problems included hindered floodgate operations and clogging of hydropower and water supply intakes.

b. Trapping and Releasing Efficiency.

(1) General.

(a) Trap efficiency (TE) is the percent by weight of the sediment inflow to a reservoir that the reservoir retains over a specific period of time. The complement to TE is release efficiency, which is the percent by weight of inflowing sediment that exits a reservoir. While release efficiency is perhaps a more useful concept for sediment management, it is often easier to measure the amount of sediment accumulated in a reservoir, so TE is more commonly used. The concept of TE can be applied to a single flood event or to a long-term trend in reservoir deposition.

(b) The TE of reservoirs depends on a wide variety of parameters. It can be quantified for the different sediment sizes present in inflows. Generally, the TE for incoming bed material load is 100% while finer material can travel much further into a reservoir with some portion reaching the dam outlets. The input variables that need to be considered to estimate sediment trapping include:

- Retention time of runoff and transported sediment effects.

– Reservoir characteristics such as shape, storage volume, stratification, and bottom roughness.

– Inflow characteristics such as runoff volume, inflow geometry sediment concentration, and base flow.

- Settling velocity of sediment particles, which depends on sediment characteristics such as grain size, sediment density and potential for flocculation.

- Dam characteristics such as the position, shape and number of outlets, the type of dam and its operation.

(c) Several empirical methods have been developed to predict TE based on a few key parameters where data availability, time, or budget constraints do not warrant a data collection campaign or modeling analysis. These methods are useful for reconnaissance-level or preliminary analyses. When more than a preliminary analysis is required, physical measurement, coupled with mathematical modeling of sediment transport will generally be required to assess project performance risk.

(d) An early method, proposed by Brown (1950), characterized TE in terms of the ratio of reservoir capacity to watershed area. Brune (1953) and later Dendy (1974) related TE to the ratio of reservoir capacity to mean annual inflow. Churchill (1948) presented a relationship between TE and a Sedimentation Index (SI), where SI is the ratio of retention period to the mean reservoir velocity. Churchill's curve was developed using data from five reservoirs operated by the Tennessee Valley Authority where sediment inflows were primarily silts and clays. Figure 8-15 shows trapping efficiency estimates of Brune and Churchill, in which Churchill's SI has been nondimensionalized using the gravitational constant.

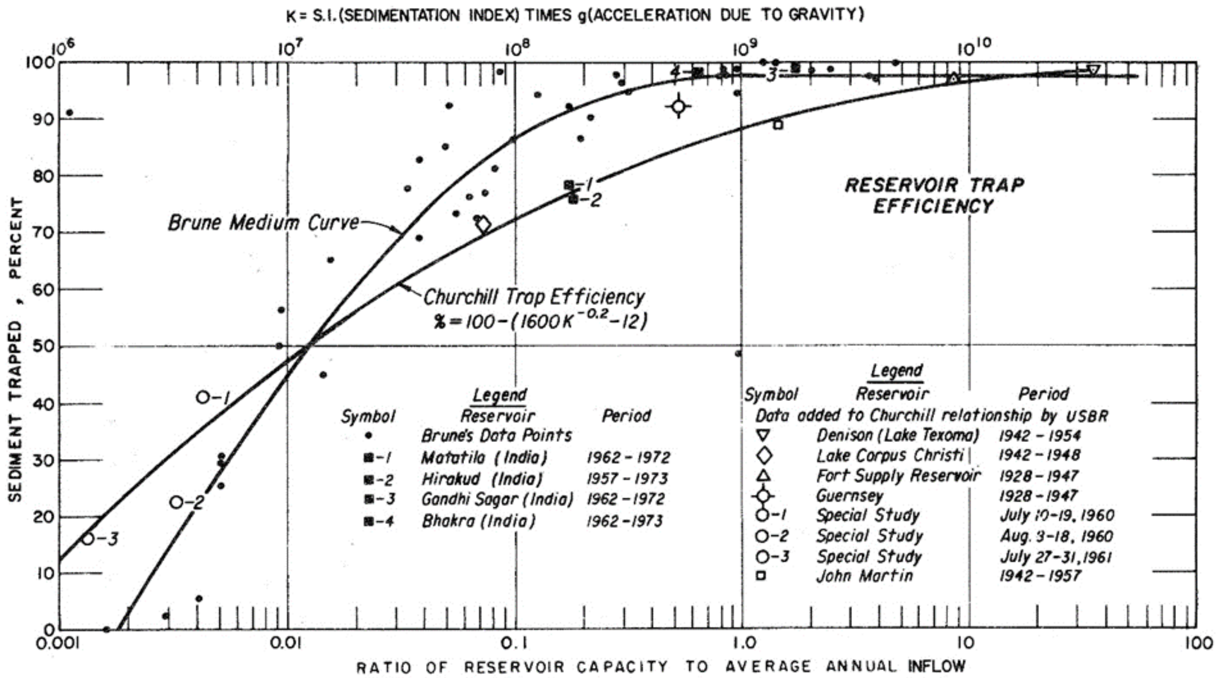


Figure 8-15. Relation of sediment trapping efficiency to capacity to inflow ratio of Brune and to SI of Churchill (Strand and Pemberton 1987)

(e) Strand and Pemberton (1987) recommend Brune's method for large storage reservoirs or reservoirs with a significant operating pool, while they found Churchill's method more applicable to small reservoirs, settling basins, and flood control facilities. Appendix K contains a detailed summary and example calculations using the Brune and Churchill methods.

(f) Espinosa-Villegas and Schnoor (2009) conducted a comparison of several TE prediction methods on data from the Coralville reservoir. They found that annual TE varied greatly from year to year, with the observed TE varying from 5.6% to 95.8% with a mean of 74.7%, a median of 79.9%, and a standard deviation of 17.3% over the period of record (1973–2005). For this study, the Churchill relation was the best predictor of the mean long-term TE.

(g) Trimble and Carey (1992) compared Brune and Churchill predictions for TE to measured sediment accumulation on 27 reservoirs in the Tennessee River Basin. Estimated trap efficiencies by the Brune method were equal to or higher than Churchill (1948), but, in general, the two values were similar. Trimble and Carey concluded that, for a system of reservoirs, the Churchill method, which accounts for sediment received from an upstream reservoir, provides a more realistic estimate of sediment yields than the Brune method.

(h) Many recent studies of TE focus on small capacity reservoirs. Heinemann (1981) provided a revision of the Brune curves using data for reservoirs with drainage area of 15 square miles or less. Kantoush and Schleiss (2014) investigated the effect of a reservoir shape factor on sediment deposition and removal with physical experiments for shallow reservoirs. They found that the evolution of TE has an increasing or decreasing effect according to the geometry shape factor.

(i) Verstraeten and Poesen (2000) investigated several empirical methods and four numerical models in their ability to predict TE for small dams and retention ponds. They found that TE changes based on the magnitude of each event. The large runoff events, which deposit thick sediment units, have lower TE than the smaller events. Additionally, they concluded that given the high range in TE using empirical models, the use of theoretical models to predict TE for small ponds was recommended.

(2) Dry Detention Structures. Dry detention structures have a minimal permanent pool and may be sized to meet a variety of USACE project objectives. When calculating sediment yield for an existing dry detention structure, allow for some scouring and removal of previously deposited material during times of low to moderate in-channel flow through the reservoir area. Selection of appropriate TE adjustment factors should be conducted on a project specific basis that includes an assessment of project goals and performance risk affected by TE estimates.

(a) Dendy (1974) found little difference in deposition between the dry detention storage and permanent pool reservoirs. TE relationships applied equally well for permanent pool or dry reservoirs, although the dry reservoirs in Dendy's study had only a 5-year maximum length of record.

(b) The USDA-SCS (1983) suggests that, for dry reservoirs, TE should be lowered by 5% for streams with dominant sand loads, and by 10% for streams that carry predominantly fine material (silts and clays).

(c) Verstraeten and Poesen (2000) reported that these values may not be based on available field data and could lead to an overestimation of dead storage in project planning.

(3) Run-of-River Structures. Unlike dry detention or permanent pool reservoirs, run-of-river structures are not designed for flood storage, but to maintain a minimum depth for navigation. Consequently, deposition in the navigation pool is much less than in a flood control reservoir, primarily occurring during normal flow periods. During flood periods, when the gates of the navigation dams are open and the river profile is about the same as the pre-project profile, some erosion of the previously deposited material may occur.

(a) A numerical sediment transport model, such as HEC-RAS, with long-term simulation capability can determine TE by calculating depositional changes in a navigation pool. Results from a period-of-record simulation can determine average annual yield at the structure and investigate removal rates and areas to maximize benefit. Assessment of deposition and erosion patterns would also generally be performed using a numerical sediment transport model as discussed in Chapter 9.

(b) Although primarily empirical, two simplified techniques for estimating TE in a run-of-river pool are briefly described in Appendix K.

(4) Debris Basins. Debris basins are a special type of dry reservoir designed to retain the coarsest sediments. The volume and cleaning rate are monitored, but it is extremely difficult to estimate total sediment inflow because TE typically changes drastically as the basin fills. Short-circuiting during single-event deposition, high concentrations of fines, and the wide variation in grain size are common issues that limit TE accuracy estimates and complicate the use of debris basins to define watershed sediment yield. A reliable method is to evaluate the system with a numerical model with calibration of the inflowing sediment discharge rating curve to historic records of debris basin sediment removal volume and material size. Refer to paragraph 8-14 for further information on debris basins.

(5) Additional Considerations for Evaluating Trap Efficiency.

(a) The amount of sediment trapped by a reservoir or a debris basin depends on the flow velocity, flow depth, and sediment particle sizes. With the possible exception of dry detention areas or small pond structures, it is reasonable to assume the TE of fine sands and larger (particle sizes greater than 0.125 mm, Table 3-1) is 100%. Silts and clays settle more slowly, but pools with a ratio of reservoir capacity to average annual inflow as small as 0.1 may settle 80% to 95% of all sediments.

(b) Specific methods of computing settling velocities for sediment materials of various sizes were previously discussed. The time required for sediment particles to settle out of the water column relative to the time required for flow to pass through the reservoir is a check against empirical TE estimates.

c. Delta Formation. Where a river or tributaries enter the reservoir, flow velocities drop significantly and sediments, predominantly those carried as bedload, create a deposit or delta (Figure 8-16). Delta deposits deplete reservoir storage and can also cause channel aggradation and other hydraulic changes extending a great distance upstream from the reservoir pool because of backwater effects.

(1) Delta deposits usually represent the coarse fraction of the sediment load and can range from silt to cobbles. At the downstream end of the delta, a slope break signals the terminus of the deposit. Downstream of the slope break, the steeper portion of the deposit is called the foreset slope, while the milder slope upstream of the break is called the topset slope. The foreset material is usually finer than the topset material and downstream of the foreset slope, the deposits are formed chiefly from suspended sediments settling, with the grain size smaller than the foreset sediment. This change in grain size may occur even in reservoirs lacking an obvious delta (Fan and Morris 1992).

(2) Figure 8-17 illustrates the development of the depositional delta at the Aswan High Dam in Egypt from start of filling in 1964 to 2003 (Moustafa 2013). The figure shows about 50 meters of deposition and the consequent loss of storage at the downstream end of the delta in a period of 40 years. The transition from topset to foreset slope occurs approximately 350 to 370 kilometers upstream of the dam. During the delta formation period illustrated in Figure 8-17,

initial reservoir filling occurred for about the first 10 years after 1964. During the operational period after filling, reservoir levels were generally between 160 and 180 meters.

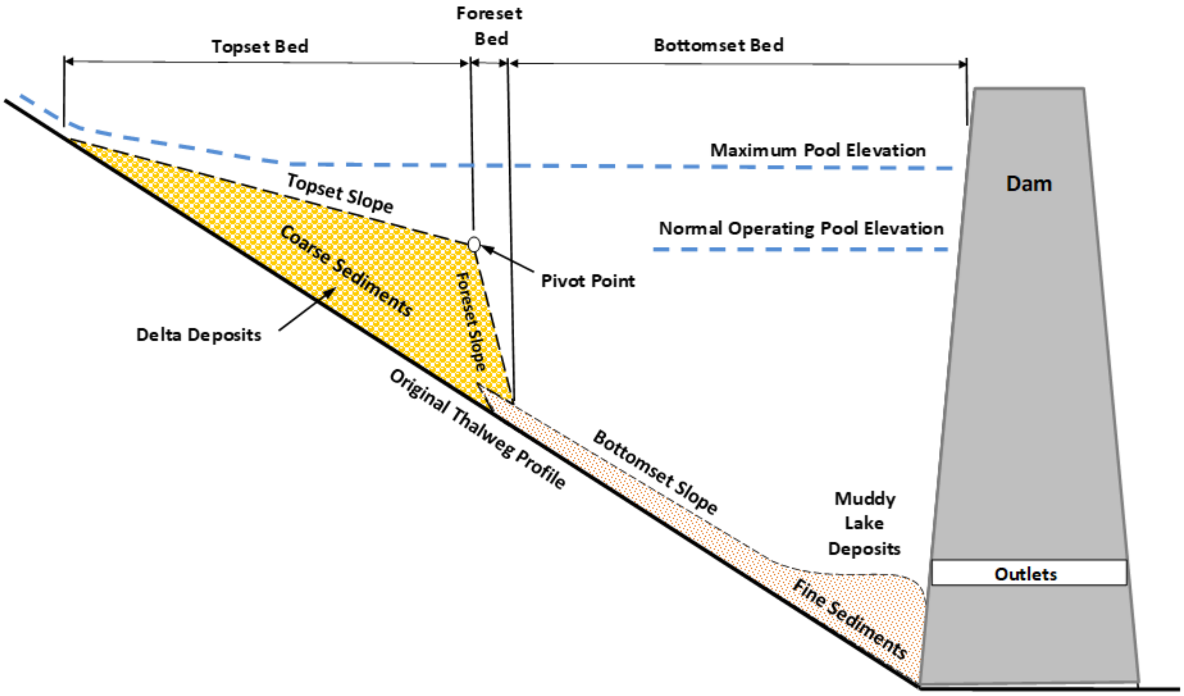


Figure 8-16. Depositional zones in a reservoir

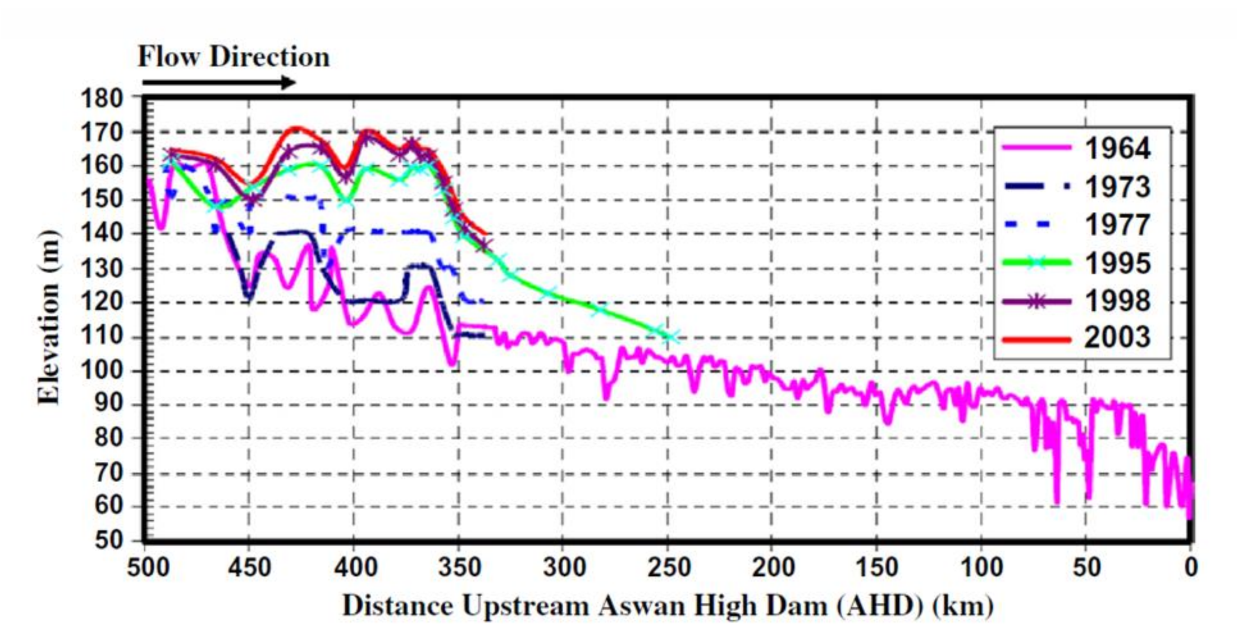


Figure 8-17. Longitudinal section of the lowest bed elevation of Aswan High Dam Reservoir (AHDR) from year 1964 to 2003 (Moustafa 2013)

(3) Figure 8-18 shows the development of the delta in Lewis and Clark Lake on the Missouri River. The delta growth and movement of the distinct edge of the topset slope is visible when comparing photos from 1973 and 2016.

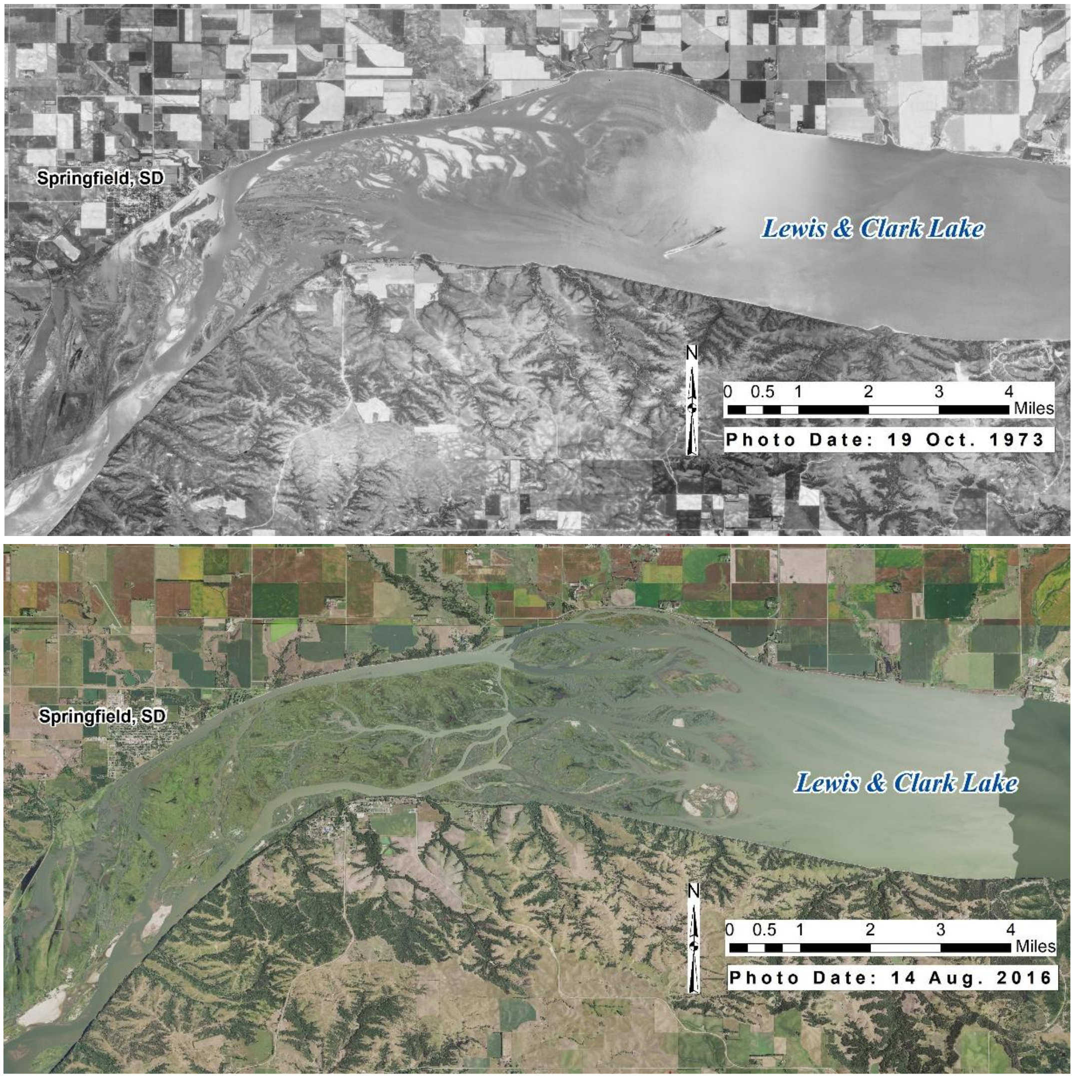


Figure 8-18. Development of delta in Lewis and Clark Lake, upstream of Gavins Point Dam on the Missouri River between 1973 and 2016

(4) Numerical modeling can be used to accurately predict deltaic deposition patterns including location, rate of growth, and material size.

(a) Kostic and Parker (2003a) provide model results that illustrate interacting topset, foreset, and bottomsets that show a structure quite similar to that observed in the delta of the Colorado River on Lake Mead. The model consists of two linked sub-models: (1) a turbidity current model that describes the evolution of the bottomset deposits, and (2) a fluvial delta model that describes the evolution of the prograding topset and foreset deposits.

(b) Moustafa (2013) used a 2D depth-averaged unsteady flow and sediment transport model (CCHE2D) to predict sedimentation, both erosion and deposition, at the Aswan High Dam delta. Figure 8-19 shows the predicted sedimentation for the Aswan delta for the 5 years following a 2009 survey using the calibrated 2D model. Note that for reservoirs where the cross section varies significantly in the downstream direction, it is difficult to identify a single topset slope.

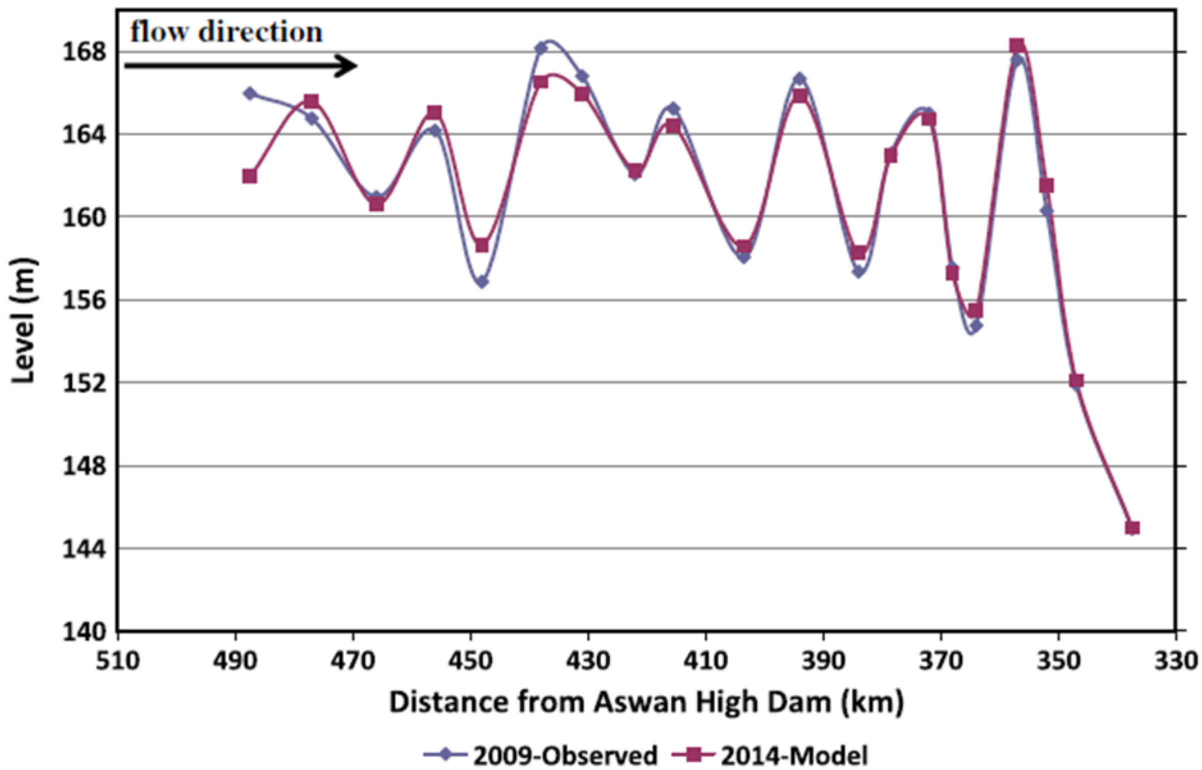


Figure 8-19. Prediction of longitudinal bed profile for AHDR for the years 2009 to 2014 (Moustafa 2013)

(5) Appendix L presents historic methods for predicting sediment deposition in a reservoir. Empirically based and simplified analysis methods for predicting preliminary topset and foreset slopes before detailed modeling are:

(a) Topset Slope.

- The topset slope of the delta can be estimated by one or more of the following methods:

− Statistical analyses of existing delta slopes support a topset slope value equal to one-half of the existing channel slope (Strand and Pemberton 1987; Morris and Fan 1998).

− Alternately Strand and Pemberton (1987) developed a relationship (Figure 8-20) to refine topset slope estimate from an existing reservoir that is geomorphically and operationally similar.

– Parker (2006) presented a spreadsheet calculator for evolution of a longitudinal river profile and migrating delta using a backwater formulation.

– If it is assumed that the delta has developed to the point where all bedload transported into the delta is deposited, then the slope yielding zero bedload transport using an appropriate bedload transport equation can be determined.

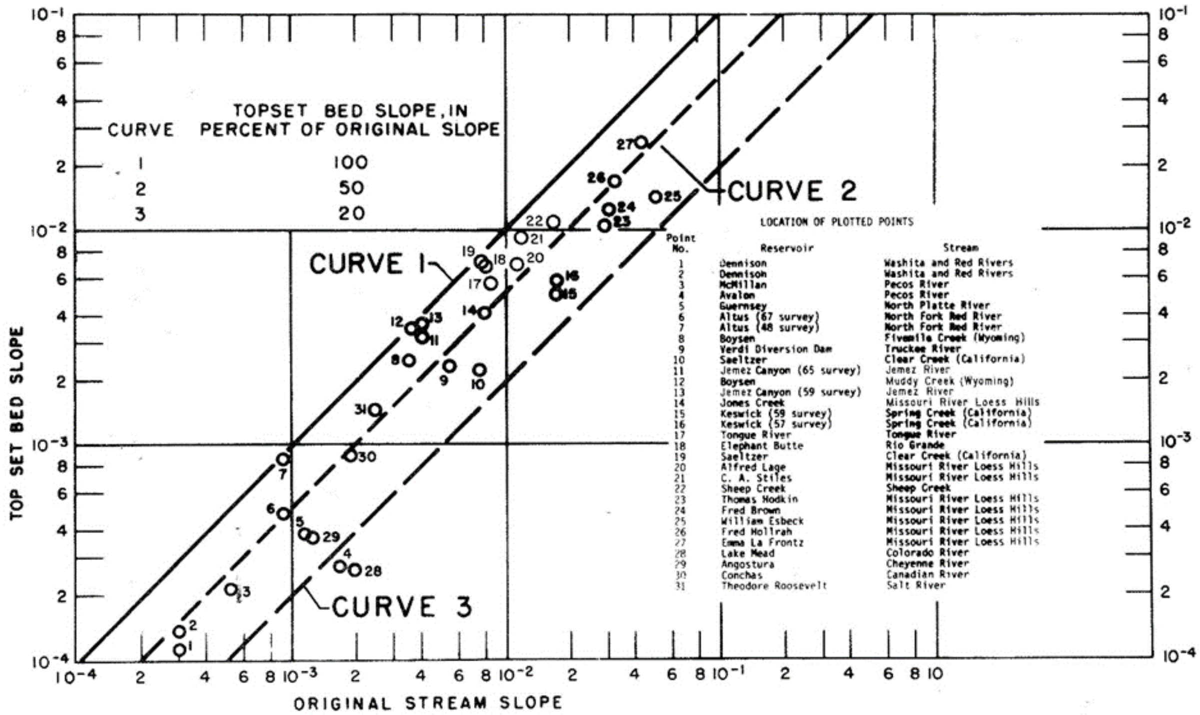


Figure 8-20. Topset slope vs. original streambed slope from existing reservoirs (Strand and Pemberton 1987)

- For example, solving the Meyer-Peter and Muller equation (1948) for slope at zero bedload transport yields:

$$S_T = K \frac{\frac{Q}{Q_B}\left[\frac{n}{(d_{90})^{1/6}}\right]^{3/2}}{D} d \qquad \text{Equation 8-1}$$

where:

S_T = topset slope
K = coefficient equal to 0.19
Q/Q_B = ratio of total flow in ft^3/s to flow over bed of stream in ft^3/s (Q/Q_B is normally equal to 1). Discharge is referred to as dominant discharge and is usually determined by either channel bankfull flow or as the 1.5-year flood peak

d = diameter of bed material on topset slope, usually determined as weighted mean diameter in millimeters
d_{90} = diameter of bed material for which 90% of material is finer than d_{90} (in millimeters)
D = maximum channel depth at dominant discharge in feet
n = Manning's roughness coefficient for the bed of the channel

- The Meyer-Peter and Muller equation, or any other equation selected for zero transport, will yield a slope at which the bed material will no longer be transported, which is a necessary condition for the formation of a delta.

- Subsequent research by Wong and Parker (2006) have shown that for plane-bed conditions (without bed forms) that the form drag correction in the original Meyer-Peter and Muller equation overpredicts transport. In those cases, the reference should be consulted, or another transport function used.

(b) Foreset Slope. Generally, deltas composed of coarse sediments such as sands and gravels will have a steep foreset slope, whereas deltas containing fine-grained sediment will have much more shallow slopes. The average of foreset slopes observed in USBR reservoir resurveys is 6.5 times the topset slope. However, some reservoirs develop much steeper foreset slopes. Lake Mead's foreset slope is 100 times the topset (Morris and Fan 1998). Reservoirs on rivers with heavy silt loads in China, however, have foreset slopes about 1.6 the original river bed slope (Wuhan College of Hydropower 1982).

(c) Pivot Point. The location of the pivot point between the topset and foreset slopes depends primarily on the operation of the reservoir and the existing channel slope in the delta area.

- If the reservoir is operated near the top of the conservation pool a large portion of the time, the elevation of the top of the conservation pool will be the pivot point elevation. Conversely, if the reservoir water surface has frequent fluctuations and a deeply entrenched inflow channel, a mean operating pool elevation should be used to establish the pivot point.

- In the extreme situation when a reservoir is emptied every year during the flood peak flows for sluicing sediment, the pivot point will be at the sluiceway (USBR 2006a). The location of the pivot point between the topset and foreset slopes progresses downstream toward the dam over time.

- Comparison ratios of the foreset slope to bed slope may be misleading due to differences in selecting the pivot point between the topset and foreset as shown in Figure 8-16.

(6) Simplified methods for predicting delta formation (topset, foreset, pivot point) can also be performed using a spreadsheet application such as those by Parker (2006). However, in most USACE project applications, accurate predictions of delta formations will require detailed numerical modeling (See Chapter 9)

d. Turbid Density Currents.

(1) Reservoirs have a strong tendency to stratify because of vertical differences in fluid density. These differences are caused by variations in the concentration of suspended sediments, by the temperature gradation between colder lower water and warm top layers, and occasionally from changes in salinity. Density differences due to suspended sediments are the primary contributors to stratification in reservoirs although under conditions of low suspended solids, inflow temperature can also have a large role in stratification. A more complete discussion on turbidity density currents can be found in Morris and Leech (2013).

(a) For example, in reservoirs when inflowing turbidity is low, at 25 °C, the density difference caused by a 1 °C difference in temperature is equivalent to the effect of approximately 420 mg/L of suspended solids with a specific gravity of 2.65 (Morris and Fan 1998).

(b) Kostic and Parker (2003b) state that when a sand-bed river carrying mud as wash load enters a reservoir, in general, the sand tends to deposit out to form a fluvial topset and an avalanching foreset, and the mud tends to deposit out as a bottomset. During floods, many sand-bed rivers carry sufficiently high concentrations of wash load to render the river water heavier than that of the body of standing water. In such a case, the mud-laden river flow plunges to form a bottom turbidity current.

(2) Reservoir stratification can cause several different behaviors for inflowing sediment depending on the degree of stratification and the density, both from temperature and from suspended load, of the incoming flows. The incoming sediment may be transported as overflow, interflow, or underflow, as illustrated in Figure 8-21.

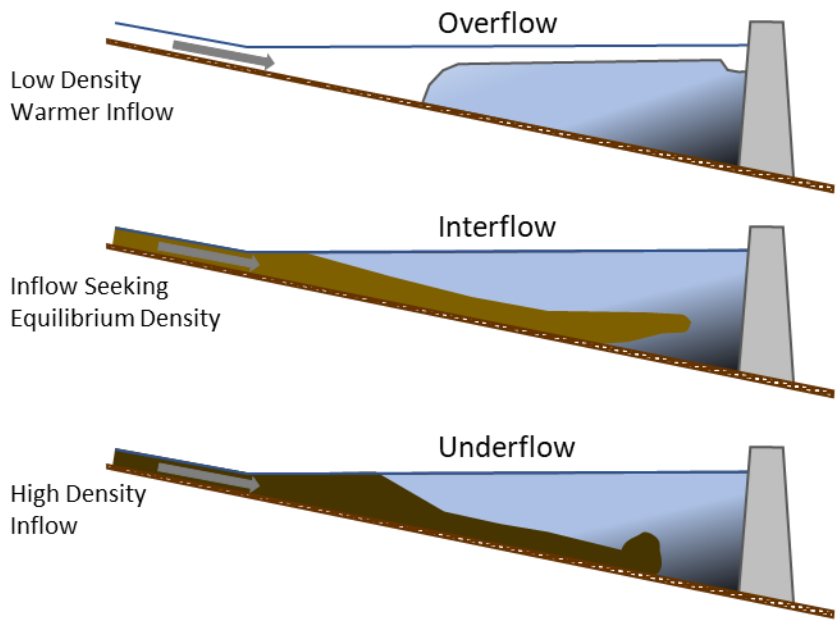


Figure 8-21. Conceptual view of reservoir vertical stratification patterns

(3) Warm or lower density water will flow across the top of cooler, higher density water as overflow, water of intermediate temperature or density will seek an equilibrium at a matching density or thermocline in interflow, and cool or sediment-laden inflow will sink beneath warmer water as a bottom current or underflow. It is these low-density bottom currents that are commonly referred to as turbidity currents, density currents, or gravity-driven currents.

(4) As the sediment-transporting inflow waters enter the impounded waters, they plunge beneath the clear waters and travel downstream along the submerged thalweg. The point where the turbid inflowing water plunges beneath the ambient water is called the plunge point, or plunge line. The maximum thickness of the turbid current generally occurs at the plunge point. Plunging flow establishes a weak countercurrent in the overlying clear water just downstream of the plunge line, causing clear surface water to travel upstream.

(a) Since both downstream-moving and upstream-moving currents converge along the plunge line, floating debris carried by the flow typically accumulates immediately downstream of the plunging flow. Debris accumulation and marked changes in the water color are strong indicators of the plunge point (Figure 8-22).

Figure 8-22. (a) Turbid water plunging in Nurek reservoir, Tajikistan, (b) accumulation of floating debris at the density current plunge point in Miel-1 reservoir, Colombia (Photos G. Morris, from USACE 2021)

(b) The location of the plunge point is determined by the balance between stream momentum, the pressure gradient across the turbid-clear water interface, and the resisting shear forces. It is also influenced by morphologic factors, such as bed slope, bed roughness, and cross-sectional shape and area. In practice, the plunge point can be estimated from the densimetric Froude number (USBR 2006a) defined as

$$F_p = \frac{V}{\sqrt{\varepsilon_i g D_p}} \qquad \text{Equation 8-2}$$

where:

F_p = densimetric Froude number
V = inflow velocity
ε_i = relative density difference = $(\rho_i - \rho_r)/\rho_i$
ρ_i = density of the inflowing water
ρ_r = density of the receiving (ambient) water
g = acceleration due to gravity
D_p = depth at the plunge point

(c) When a sediment-laden flow discharges into a wide reach, the turbid surface of the water may extend into the reservoir as an irregular tongue-like current that shifts location depending on inflow concentration, temperature, and cross currents.

(5) For a turbidity flow to be self-sustaining, the mean rate of energy input to the flow by the sediment must exceed the mean rate of energy loss from the turbulent mixing required to keep the sediment in suspension, often called the Bagnold criterion. A second requirement for a self-sustaining turbidity flow (Parker et al., 1986) is that sediment entrainment from the bed equal or exceed the depositional rate. Entrainment can be expected to be a function of boundary shear stress and sediment-related parameters.

(a) For example, a slow-moving turbidity current may exert a shear stress slightly larger than that required to entrain sediment into suspension, producing entrainment. This entrainment rate is less than the deposition rate, so the turbidity current will experience a net loss of granular material, decreasing the sediment concentration in the turbidity current and causing the current to decelerate and eventually vanish.

(b) By contrast, a higher flow velocity may produce a rate of sediment entrainment from the bed that is greater than the depositional rate. As the current entrains more sediment, the gravity forcing on the flow increases and the current will accelerate, further augmenting entrainment. This self-reinforcing cycle allows the development of a self-sustaining turbidity current that can gradually reach high speeds and flow all the way to the dam. Bed sediment availability for entrainment, reservoir geometry, and turbulence dampening at high concentrations limits the growth of such a flow (De Cesare et al., 2001).

(6) Sustained turbidity currents can travel long distances downstream. Before the construction of Glen Canyon Dam further upstream, turbid density currents were documented to travel 129 km along Lake Mead to Hoover Dam (Grover and Howard 1938), the longest documented travel distance of turbidity currents in any reservoir. At Luzzone reservoir in Switzerland, turbidity currents have been documented to travel with velocities as high as 2.5 m/s (8 ft/sec) in a narrow canyon, decreasing to about 1.5 m/s (5 ft/sec) in the larger part of the reservoir (De Cesare et al., 2001).

(7) Historic observations of turbidity currents have documented their ability to excavate and transport materials. In an early study of this phenomenon, Forel (1888) concluded that the Rhône River entering Lake Geneva had cut a canyon in its own alluvial cone, explaining the origins of the underwater canyons in the delta region of the lake. His studies showed the sediment-laden Rhône River created a plunging turbidity current flowing into Lake Geneva.

(8) In some reservoirs, turbidity currents can transport a significant amount of sediment toward a dam.

(a) Sundborg and Jansson (1992) investigated flushing operations at the Cachí hydropower reservoir in Costa Rica and found that 18% of the total inflowing sediment load was accounted for by turbidity currents that ran along the deepest part of the channel and passed through the turbines. Fifty-four percent (54%) of the inflowing load deposited along the length of the flushing channel and did not reach the hydropower plant. Flushing operations subsequently removed the deposited sediments.

(b) De Cesare et al. (2001) used field data and physical modeling of turbidity currents at the Luzzone reservoir to validate a numerical model of turbidity flows. The reservoir is about 7,800 feet long with an average bed slope of about 4%. Turbidity currents had caused sediment accumulation in front of the dam that required raising of the intakes even though the reservoir had lost only 1% of its capacity to sedimentation. Turbidity currents had focused sediment accumulation beneath only 8% of the reservoir surface area.

- Observation and numerical simulation show that turbidity currents were the main transport medium for the incoming fine granular material and also redistributed sediments inside the reservoir by entraining bed material and transporting it closer to the dam.

- Maximum turbidity current velocities along the lake bottom reached values of 2.6 fps for a small flood event with an annual return interval, while values measured near the dam were close to 1.6 fps.

- For the modeled event, although the water was 1 °C to 2 °C warmer than on the lake bottom, an increase of sediment concentration occurred, and the inflow density was much higher than that of the lake water.

(9) As demonstrated by Toniolo et al. (2007), the front of a turbidity current that reaches the dam face will be reflected upstream, creating a hydraulic jump at the interface between sediment-laden fluid near the bed and overlying clear water. If the elevation of this interface is located below the dam outflow elevation, the TE will equal 100%, as no fine material can be transported out of the reservoir. Through time, as the reservoir fills with sediment, the turbid-clear water interface elevation will rise above the outflow elevation and the TE is less than 100%. Turbidity currents that reach the dam and are not released will accumulate as a submerged lake of turbid water. Sedimentation from repeated events will create nearly horizontal sediment beds extending upstream from the dam.

e. Spatial Variation.

(1) General.

(a) Sedimentation rates between reservoirs can differ due to variations in watershed sediment yield (as discussed in paragraph 8-3). In addition, the spatial distribution of sediment can also vary dramatically between reservoirs. Longitudinal and lateral depositional patterns can vary depending on reservoir characteristics, sediment inflow attributes, and dam operations. Understanding the reservoir deposition helps plan dam operation and maintenance to prevent service interruptions and upstream and downstream impacts.

- A straight, narrow reservoir with few tributaries, fine load, and a relatively narrow range of operating pool elevations may deposit at a relatively consistent rate over most of the reservoir.

- A sinuous branching reservoir with large seasonal changes in pool level may deposit sediment with more pronounced spatial and temporal variations.

- In reservoirs with fluctuating water levels or substantial drawdowns, previously deposited sediments may be extensively eroded and reworked by downcutting, slope failure, and wave action.

- Significant sediment inputs from multiple tributaries add additional complexity.

(b) Sediment transport within a reservoir varies by process:

- Transport of coarse material as bedload along the topset delta deposits: Deltas usually form where the main river or tributaries discharge into a reservoir. Delta deposits contain practically all of the inflowing bedload, tend to fine downstream, and can vary greatly in grain size from cobbles to silt. The delta may be divided into the topset and foreset deposit areas, with the downstream limit of the delta characterized by an abrupt reduction in grain size. In high flows, deltas create backwater, which causes bed aggradation above the normal pool level. Delta topset slopes are often about one-half of the original river streambed, but can vary significantly.

- Transport of fines as non-stratified flow: As flow velocity decreases approaching the dam, suspended fines in the water column have time to deposit out from suspension and accumulate on the reservoir bed. In reservoirs held at high pool levels, gradual fine deposition can form a tapering deposit that gets progressively thinner as it approaches the dam. For large reservoirs that are operated at low levels for flood control, most of the fine sediment is carried to the dam and creates a wedge-shaped deposit that is deepest at the downstream end (Morris and Fan 1998). Narrow reservoirs with frequent water level variations can form nearly uniform deposits.

- Transport of fines in turbidity currents: As described in the previous section, turbid density-driven currents transport fine sediments long distances. Over time turbidity currents may create a nearly horizontal reservoir bed upstream of the dam with a wedge-shaped deposit beneath. Fine sediments, which are highly variable within each watershed, can comprise a high portion of the incoming sediment load. Turbid density currents are particularly important in explaining both the depositional patterns and transport processes for fine sediment in deeper reservoirs (Morris et al., 2020).

- Longshore drift can transport sediments along the reservoir shoreline. This process is also known as littoral drift or longshore transport. Longshore sediment drift in a common direction is caused by a combination of the currents, wind direction, and oblique wave action on the shore. Most design guidance is found in ocean applications. Refer to Part II, Chapter 4 on surf zone hydrodynamics of EM 1110-2-1100, for additional guidance.

(2) Depositional Distribution. The factors considered to be the most significant in the sediment distribution in the reservoir are:

(a) Reservoir size and shape.

(b) Sediment quantities and characteristics.

(c) Sediment sources (mainstem, tributaries, shoreline, etc.).

(d) Vegetative growth and density on frequently exposed deposits.

(e) Consolidation of deposits.

(f) Magnitude, duration, frequency, and sequence of hydrologic events.

(g) Reservoir regulation practices (pool levels and variability).

(h) Reservoir release methods (outlet works, spillway, bypass, etc.).

(3) Longitudinal Distribution.

(a) Sediment normally accumulates longitudinally along the whole reservoir, from the upstream topset delta to the dam face (Figure 8-16). While the accumulation pattern varies, Figure 8-23 provides data from several reservoirs used to develop sediment distribution design curves for planning studies (Strand and Pemberton 1987). The Figure 8-23 format allows comparison between deposition profiles by plotting the percent sediment deposited to the percent water depth (reservoir depth at top of operational pool). Within that format, the location of 100% sediment deposited and 100% reservoir depth would occur at the dam.

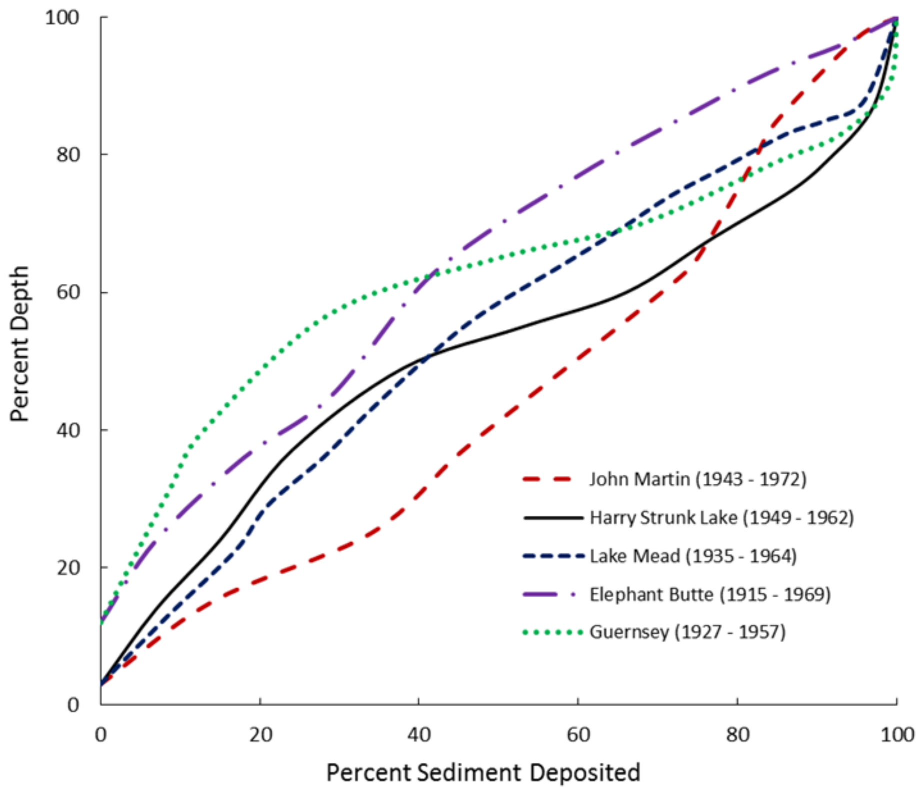


Figure 8-23. Sediment deposition profiles of several reservoirs (redrawn from Strand and Pemberton 1987)

(b) The longitudinal profile may vary within the same reservoir by season or flood severity. The sediment profile passes from the delta topset slope through the foreset slope and along the reservoir, eventually reaching a muddy pool below the lowest dam outlet, often called dead storage since it cannot be evacuated through the dam (muddy deposits below the outlet Figure 8-16, also shown on Figure 8-31).

(c) The term "dead storage" used in historic reservoir evaluations refers to a permanent sediment storage pool. However, most reservoir sediment does not deposit in the dead storage zone. In long reservoirs without turbidity currents or without a significant inflow of fines, most sediments are deposited in delta areas and the dead storage pool remains relatively empty until the delta reaches well into the reservoir. In other cases, where fines are present, turbid water entering a reservoir plunges to the bottom and flows along the submerged riverbed. A flat longitudinal bed profile that extends upstream from the dam indicates turbid density currents reach the dam that often creates a submerged muddy lake at the stratified interface.

(d) USACE studies generally consider three approaches to predict the longitudinal distribution of reservoir sediment:

- Empirical Extrapolations. Most USACE reservoirs have repeated bathymetric surveys. District engineers can use these surveys to calculate deposition rates and patterns and to empirically extrapolate the extent and pattern of longitudinal deposition. For some applications, predictions based on trend lines from historic rates and directions of geomorphic change may be sufficient.

- Analytical Curve Analysis. Historically, agencies used several analytical relationships to compute the longitudinal pattern of sediment deposition. These methods fit curves to reflect future depositional patterns based on sediment yield rates, sediment properties, reservoir geometry and, occasionally, operational parameters. The area reduction method was the most popular method along with several other methods (see Appendix L and USBR 2006a). Recently, Annandale (1987) applied Yang's stream power theory to predict sedimentation profiles using data from 11 reservoirs in South Africa. However, analytical methods are no longer widely applied. They mostly predated good, repeated bathymetry data and modeling capabilities that drive the other two methods. When study requirements call for more detail than an empirical extrapolation of cross-section data, numerical modeling is generally appropriate.

- Numerical Modeling. Numerical modeling is recommended where predicted flow and sediment yield differ from historic conditions, where the study questions are sensitive to the timing and spatial distribution of sediment accumulation, and where alternative management strategies (such as flushing) violate the assumptions of historical extrapolations.

– A primary consideration is whether 1D (cross-section averaged), 2D (depth-averaged), or 3D sedimentation modeling is more appropriate (see discussion on model dimensionality in paragraph 9-1g).

– One-dimensional models are appropriate for narrow reservoirs, where the flow is highly channelized, closely follows the thalweg, and transverse mixing is well accomplished. One-dimensional models are well suited to applications requiring long-term simulations or multiple alternative analyses. 1D models excel in computing longitudinal responses to deposition and drawdowns.

– Where the reservoir pool is wide, lacks a single clear flow direction, or when lateral sediment distributions are important (such as computing deposition at a shoreline marina), multidimensional models are more applicable. Model complexity and capability varies, and the right level of complexity should be selected to answer the study question (USSD 2015). See detailed discussion of model selection and available models in Chapter 9.

– Both 1D and 2D models have been used with success to model reservoir sedimentation. Figure 8-24 illustrates a 1D, unsteady HEC-RAS mobile bed model of the Arghandab reservoir in Afghanistan with station 0 at the dam (Gibson and Pridal 2015).

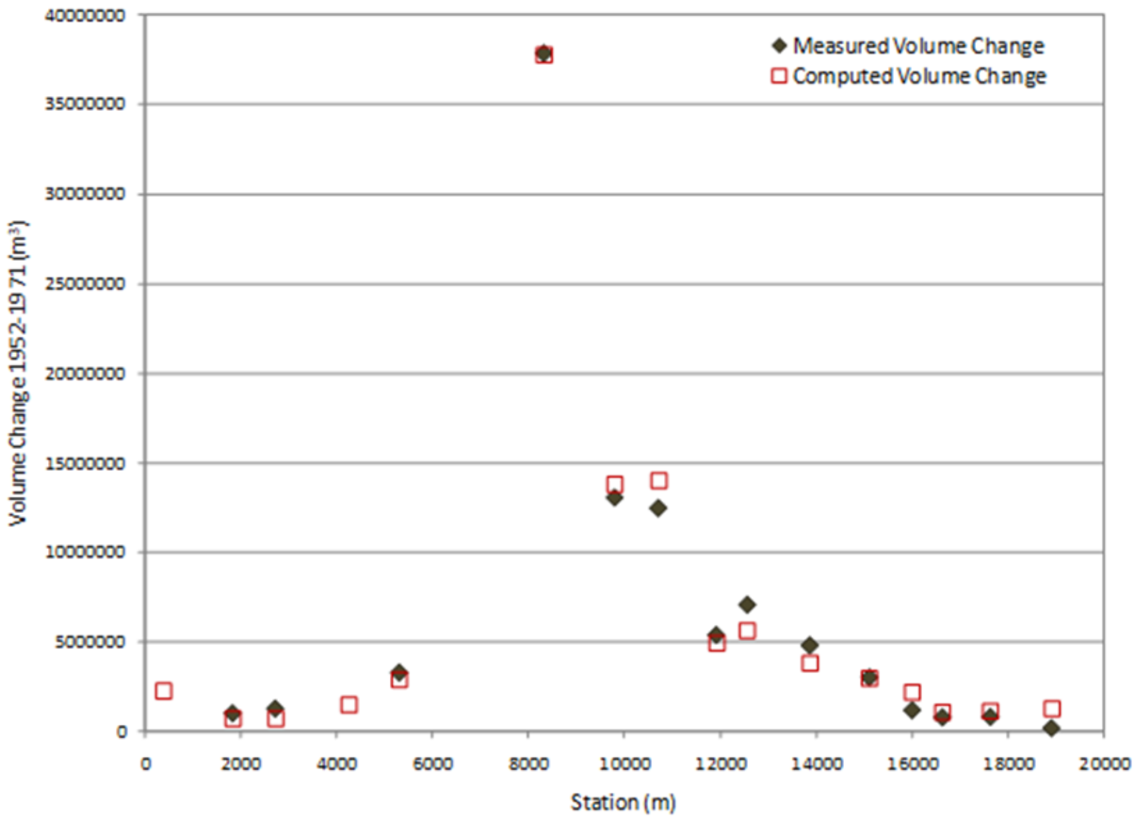


Figure 8-24. Performance of a 1D HEC-RAS model at Arghandab Reservoir, Afghanistan (Gibson and Pridal 2015)

− In a study of sedimentation at Conowingo Dam using a 2D model (AdH-2D), Scott (2012) concluded that since the bulk of the annual sediment load (95%) is passed into the reservoir at high flows (greater than 30,000 cfs), 1D and 2D models were adequate for predicting sediment transport. A river discharge of 30,000 cfs requires approximately 4 days to transit through the reservoir. Flows less than the median flow of 30,000 cfs may have a high degree of sediment stratification in the water column, but these flows deliver only about 5% of the sediment per year. The water column at high flows is relatively well mixed and stream-wise velocity is the dominant transport process.

(4) Lateral Distribution.

(a) The lateral depositional sequence in a single cross section varies between reservoirs, influenced by the spatial variation factors previously listed. It can be complex, as illustrated in Figure 8-25, where upstream sections within the delta area have proportionally more sediment deposited within the channel while downstream sections may have a more even sediment deposition veneer spread laterally across the entire section.

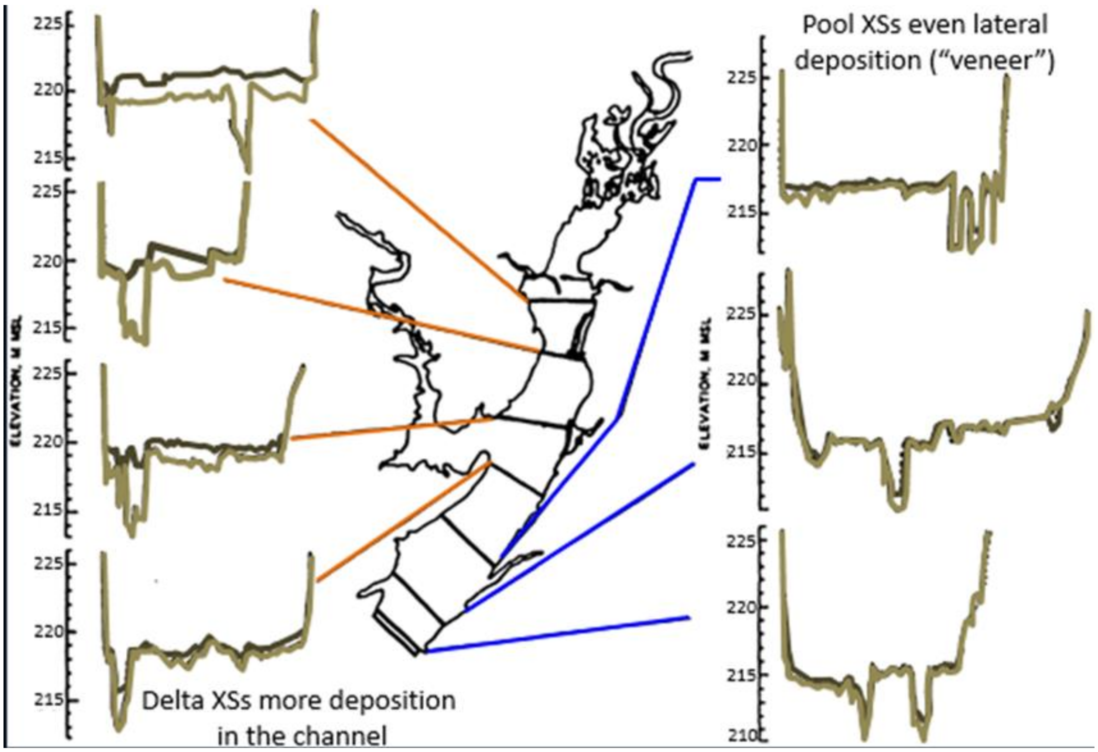


Figure 8-25. Longitudinal distribution of lateral variation in cross-section deposition

(b) In other reservoirs, as illustrated in Figure 8-26, deposition may be a simple process such that sediments first fill the deepest part of a cross section and subsequently spread out across the submerged floodplain to create broad flat sediment deposits. The horizontal focusing of fine sediments into the deepest part of the channel occurs because turbidity currents flow along the lowest part of the channel and the distribution of flow velocity (and sediment flux) throughout a reservoir cross section will preferentially aggregate sediments at the deepest point. Depending on the pool operating conditions, the deepest part of the cross section may contain an incised channel in the flat sediment bed.

(c) Another mechanism that affects lateral distribution is the contribution of sediment to the reservoir bed from erosion of the shoreline. As sediment flows through the reservoir, horizontal focusing is often responsible for the bypassing of side channels that have no significant incoming sediment supply of their own. Finally, the sediment load from tributaries, as further described in paragraph 8-5, will result in unequal cross-section deposition.

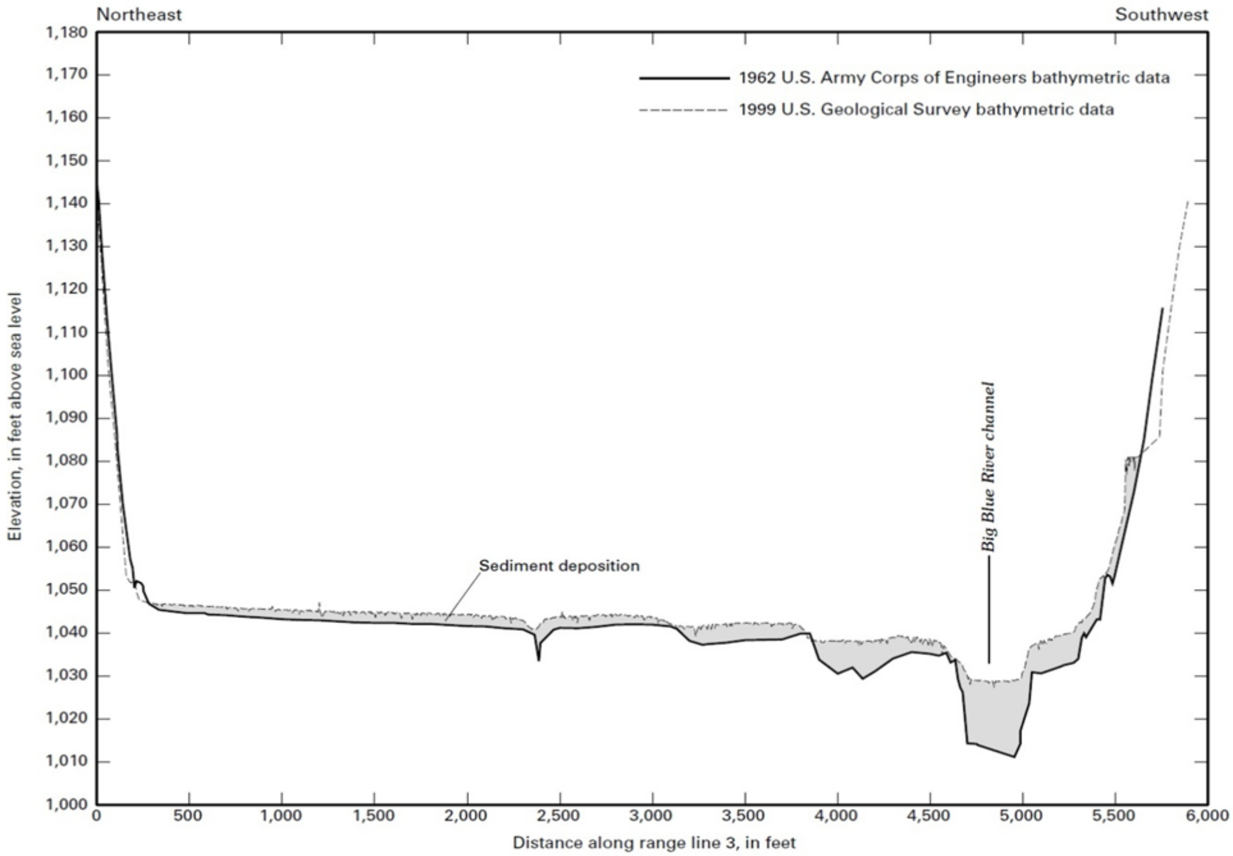


Figure 8-26. Sediment deposition between 1962 and 1999 in Tuttle Creek Lake Reservoir at range line 3, 4.6 miles upstream of dam face (Juracek and Mau 2002)

(5) Vertical Distribution.

(a) Sediments deposit in reservoirs in an unsteady sequence. Rare high-flow, high-concentration events punctuate long periods of relatively low flows and low transport.

(b) The vertical structure of sediment deposits (reservoir stratigraphy) often records this sequence with alternating layers of fine and coarse sediments. In some reservoirs, coarse or sandy deposits are delivered during high flows, and separate layers of fine material are laid down during lower flows (Evans et al., 2002). In other reservoirs, debris-laden fine lenses from flood events separate clean sand layers deposited during lower flows (Gibson and Boyd 2016).

(c) Sediment layers also preserve seasonal variations in concentration, grain size, and organic content of the inflowing sediment. Because of this process, sediment cores of undisturbed sediment layers can be used to infer the history of flows into a reservoir. Consolidation also generates vertical trends in sediment density, which are discussed later.

(d) In addition to layering, vertical focusing also plays a role in the fate and transport of sediments through a reservoir. As described in paragraph 8-4c and Figure 8-21, incoming flows of differing temperature and sediment concentrations can come to equilibrium at different vertical levels in a reservoir depending on whether they are overflows, interflows, or underflows. The elevation of outlets in the dam can have an effect on whether these sediments remain or are passed through the dam.

(6) Tributaries. In the same way that a delta forms on the topset slope at the upstream end of a reservoir, tributaries with a significant sediment load can also form deltas when they enter the reservoir. These deltas can create backwater and bed aggradation above the normal pool level. Tributaries can also discharge fine sediments into the reservoir that contribute to sedimentation well downstream of the tributary inputs.

(a) Wulandari et al. (2015) detail how the Wonogiri Reservoir in Java, Indonesia has lost 33% of its storage capacity between 1980 and 2011 due to tributary inflows that come from rivers located around the reservoir. Coarse sediment inputs deposit at the tributary deltas around the Wonogiri Reservoir, while fines have reached the deepest part of the reservoir at the center.

(b) Lewis and Clark Lake is formed by Gavins Point Dam on the Missouri River near Yankton, South Dakota. The Niobrara River has delivered almost all of the sediment that has formed an extensive delta of about 20 miles in length in the lake and filled about 25% of the reservoir storage volume.

(7) Need for Forecasting Distribution by Elevation and Purpose. At multi-purpose dams, it is important to evaluate current and forecast future depositional patterns so that operational decisions can be made regarding storage allocation, sediment management, and the impacts on reservoir operations. Reservoirs with flood storage are often multi-purpose with active storage for other uses such as for water supply, water quality, navigation, hydropower, and recreation. At these facilities, it is common to allocate storage by pool elevation. The variable sedimentation pattern will impact reservoir storage and uses differently depending on the longitudinal, lateral, and vertical distribution of the sediment.

f. Temporal Variation and Consolidation of Deposits.

(1) Temporal Variation.

(a) Depositional patterns can change over time, growing more complex and difficult to interpret. Sediment deposits can be reworked substantially by large floods. Coarse sediments can be carried deeper into the pool, overlaying finer sediments. Turbidity currents can scour out channels in the fine material that lies in horizontal layers at the bottom of the reservoir. Multiple deltas can be formed at the reservoir inlet and in tributaries, each corresponding to a different pool elevation. Sediments from both different sources and flood events can deposit and erode over time. Spatially distributed reservoir sediment cores can be extremely useful to reconstruct the rate and history of deposition.

(b) In addition to natural variability in sediment yield, reservoir operations, whether by intent or indirectly, affect the time variation of sedimentation in the reservoir and upstream.

Figure 8-27 shows sediments typically deposited in a single flood event near the intake of Mud Mountain Dam, a flood control dam near Auburn, Washington. The sediments range from large cobbles to fine silts and sands washed down from the slopes and glaciers of Mount Rainer. Normal post-flood dam operations erode most of these forebay deposits during the drawdown following the flood event.

Figure 8-27. Forebay of Mud Mountain Dam, on the White River in Washington

(c) Reservoir operations also affect sedimentation indirectly, by controlling flow and water surface for the live storage allocations in the reservoir pool. Petkovsek and Roca (2014) modeled Tarbela Dam in Pakistan. They showed that keeping minimum water levels at the beginning of the flood season to preserve live storage accelerated sediment progress toward the dam, possibly causing damage to machinery and blocking of low-lying outlets. Conversely, if minimum water levels were kept high, sediment advance was retarded, but live storage for water supply and hydropower fell faster.

(2) Role of Extreme Events.

(a) During extreme floods, sediment deposition in the reservoir can accelerate. Landslides and debris flows can compound the effects of high-rainfall events. In 1979, sediment and debris 17 m deep were deposited in front of the Valdesia dam in the Dominican Republic during the passage of Hurricane David, a Category 5 Cape Verde Hurricane. The power intakes were clogged for approximately six months (Morris and Fan 1998).

(b) In mountainous areas, landslides can contribute over half the sediment load during extreme events. A single event of this kind can produce flows of sediment and debris equivalent to many decades of normal events. (Morris et al., 2023). Caine (1980) proposed that widespread landslide activity could be associated with an intensity-duration threshold of the form $I=AD^{-B}$, where I is rainfall intensity in millimeters per hour, A and B are coefficients, and D is duration in hours.

(c) For temperate zones, Caine used the same relationship and established values of A = 14.82 and B = 0.39 for the threshold condition. Larsen and Simon (1993) extended the analysis to focus on humid climates, specifically Puerto Rico, with adjusted values of A = 91.46 and B = 0.82. Figure 8-28 shows the threshold relations for the temperate dataset (Worldwide threshold) and the humid-tropical conditions (Puerto Rico threshold).

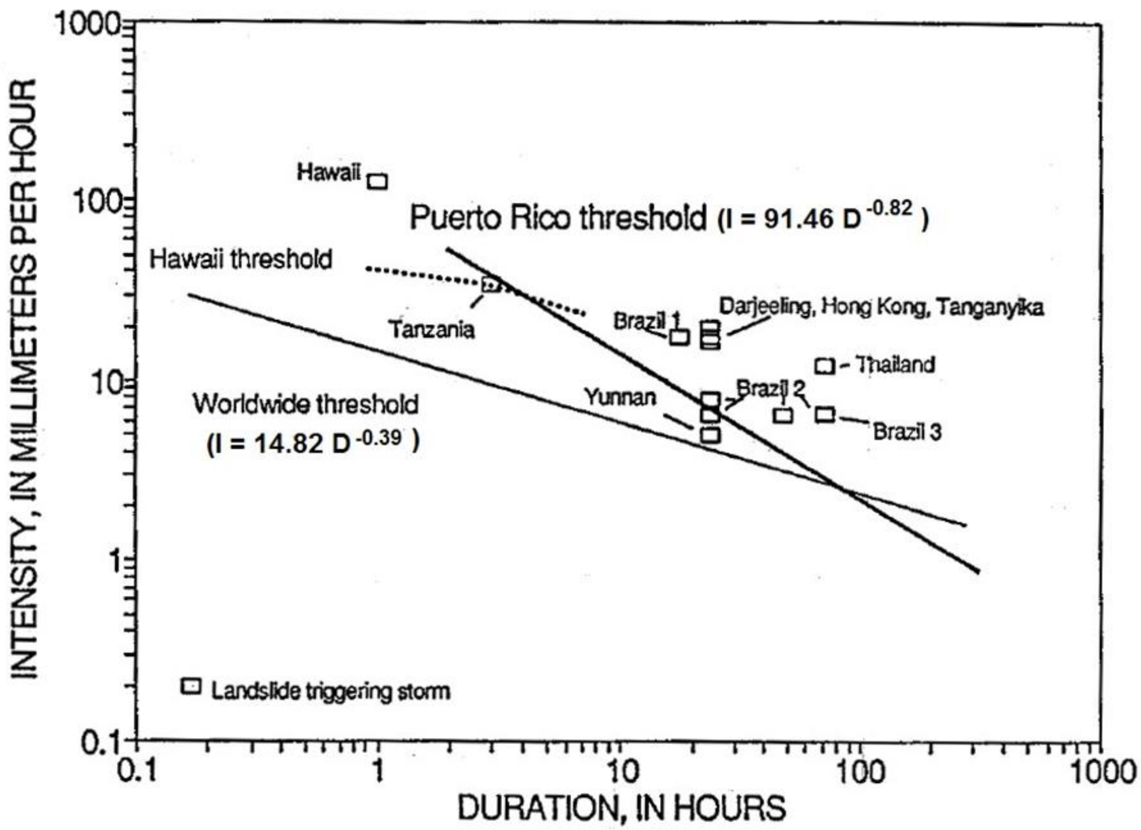


Figure 8-28. Rainfall intensity-duration thresholds for storms triggering landslides (adapted from Larsen and Simon 1993)

(d) High sediment yield conditions from extreme events can transition into debris flows, which transport coarse material as large as boulders into dam facilities. Figure 8-29 shows the Calderas hydroelectric plant near San Carlos, Columbia heavily damaged after an extreme rainfall event in September 1990. Over the course of three hours, 208 millimeters (8.2 in.) of rain fell in the vicinity, initiating over a hundred landslides and carrying boulders as big as 8 meters (26 feet) to the plant downstream (Hermelin et al., 1992). This event was estimated to have a recurrence probability of 0.02% (once in 5,000 years).

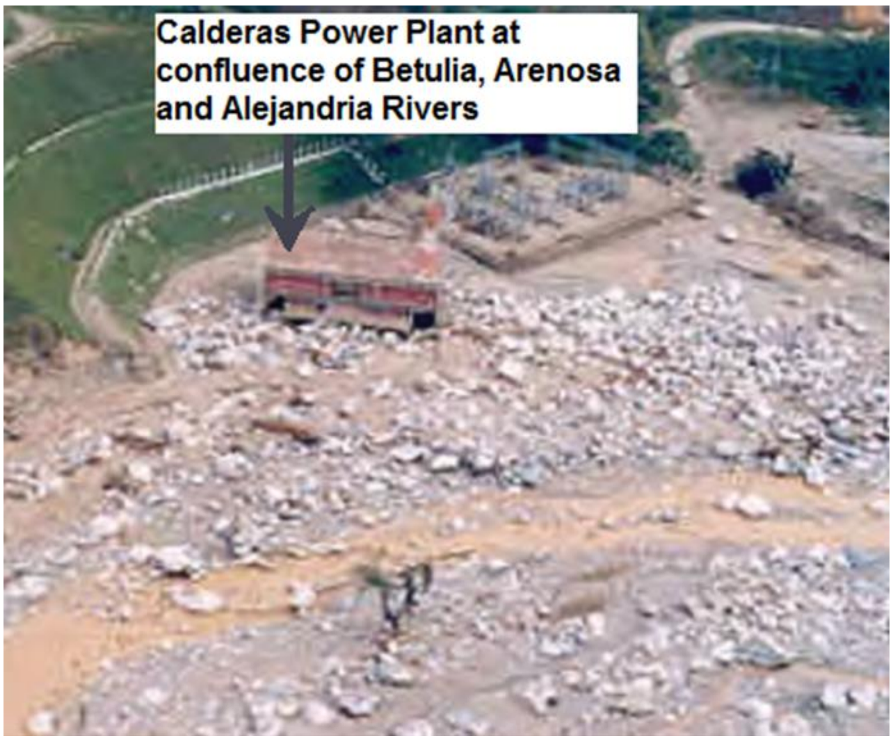


Figure 8-29. Calderas power plant after debris flow of 1990 (Morris 2016)

(e) Figure 8-30 shows the response in the Kulekhani Reservoir in Nepal to a single rainfall event. The main reason for the high sedimentation response was the heavy rainfall event in Simlang (376.8 mm (14.8 in.)) and Tistung (535 mm (21 in.))) over a 24-hour period observed July 19 to 20, 1993. This event occurred over a relatively small area and was characterized as an unusual monsoon storm. This extreme event had long-term impacts as adjustment continued in the basin with higher reservoir capacity loss rates through 1996. Land terracing and slopes may have contributed to the high sedimentation (Sthapit 1995). Refer to paragraph 7-8 and Case Study 10A (Appendix N, paragraph 7) for further discussion on debris flows and extreme events.

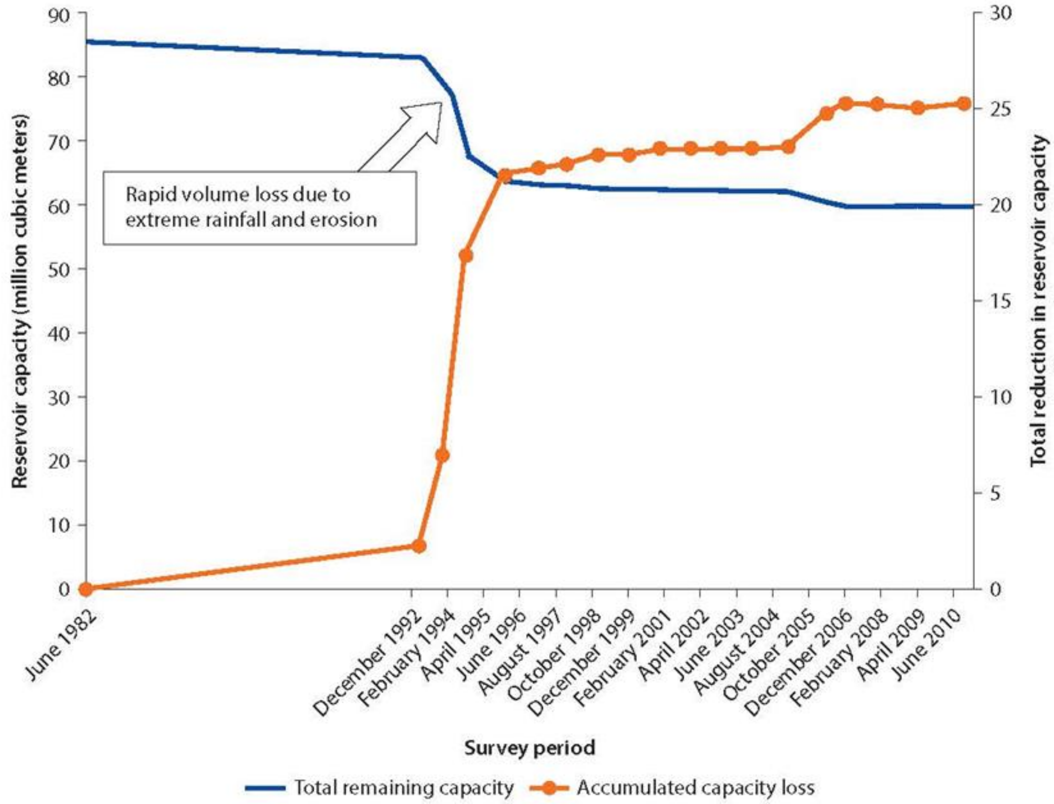


Figure 8-30. Capacity reduction at Kulekhani Reservoir, Nepal after 1993 extreme event (Annandale et al., 2016)

(3) Consolidation.

(a) In reservoirs, sand and gravel deposits attain their ultimate bulk density almost as soon as they are deposited. Consolidation of fine-grained deposits in reservoirs can continue for years or decades before they reach their final, fully consolidated density. Equation 8-3 (Lane and Koelzer 1943) describes the consolidation of reservoir deposits. The average of consolidation of all deposits over T years can be expressed by Equation 8-4 (Miller 1953).

$$\gamma_{dc} = \gamma_{di} + B \log_{10} T \qquad \text{Equation 8-3}$$

$$\gamma_{dc} = \gamma_{di} + 0.434B\left[\left({}^{T}/_{T-1}\right)lnT - 1\right] \qquad \text{Equation 8-4}$$

where:

γ_{dc} = consolidated weight of the deposit
γ_{di} = specific weight of the initial deposit
B = coefficient of consolidation, which varies with size classification (see Table 8-2 for average values)
T = age of the deposit, years

Table 8-2
Consolidation Coefficient for Reservoir Deposits

Operational Condition	Consolidation Coefficient, B in lb/ft^3 (kg/m^3)		
	Sand	**Silt**	**Clay**
Continuously submerged	0	5.7 (91)	16 (256)
Periodic drawdown	0	1.8 (29)	8.4 (135)
Normally empty	0	0	0

(b) When dealing with mixtures of particle sizes, do not use the percent-weighted specific weight in the γ_d terms of Equation 8-3 or 8-4. It does not conserve mass of the mixture. Rather, estimate the specific weight for clay, silt, and sand fractions separately, then calculate the composite-specific weight of the mixture using the following equation:

$$\gamma_d = \frac{1.0}{\left(\left(\frac{F}{\gamma_d}\right)_{clay} + \left(\frac{F}{\gamma_d}\right)_{silt} + \left(\frac{F}{\gamma_d}\right)_{sand}\right)} \qquad \text{Equation 8-5}$$

where F is the fraction.

g. Impacts of Reservoir Sediment Accumulation on Water Quality.

(1) Sediments affect water quality in several ways, including:

(a) Increasing in the phosphorus to nitrogen ratio over time.

(b) Silting or burying fish habitat in the reservoir, upstream of the reservoir, or downstream of a tributary below the dam.

(c) Changing dissolved oxygen content in the water column through turbidity induced stratification or from oxygen demand of organic sediments.

(d) Raising groundwater elevation, which can cause salination or leach chemicals into the groundwater.

(e) Releasing pollutants into the water column during erosional events and dredging operations.

(f) Trapping of pollutants during depositional events.

(g) Releasing clear water downstream of the dam.

(2) Paleolimnological techniques are often used to assess water quality trends in lakes and reservoirs over time. Reservoir sediments record information about organisms that lived in and around the lake and other reservoir processes, including water chemistry, watershed conditions, and climatological trends.

h. Contaminated Sediments.

(1) Sediments that accumulate in reservoirs can be contaminated from a variety of sources. Contaminant sediment sources include agricultural chemicals, industrial areas which may contain heavy metals or toxic organic compounds, mining operations, and those occurring naturally from background contaminants in the watershed. Dissolved chemicals or contaminated sediments can travel long distances until they encounter the slow-moving reservoir pool and either deposit, precipitate, or adhere to reservoir sediments. The reservoir becomes a sink for the contamination in the river system.

(2) Contaminated sediment affects reservoir sediment management or removal options. Reservoirs have been designated as U.S. Environmental Protection Agency (USEPA) Superfund sites because of metal-contaminated bottom sediments.

(3) The Upper Clark Fork River in Montana was contaminated with arsenic, cadmium, copper, lead, manganese, and zinc ores from mining activities. The contaminated area extended from the Butte and Anaconda area to at least 230 km downstream to the Milltown Reservoir. Over time, more than 6 million cubic yards of contaminated sediments accumulated behind the Milltown Dam. The 2008 removal of the Milltown Dam involved the careful excavation of about 2.9 million cubic yards of contaminated sediments. The rest of the sediment was stabilized in place so it would not transport (Woelfle-Erskine et al., 2012).

(4) Contaminated sediments affect benthic organisms in reservoirs, impacting larger organisms through bioaccumulation. As a result, contaminated sediment can affect fish and shellfish, waterfowl, and mammals, as well as benthic organisms.

8-5. Reservoir Sedimentation and Upstream Impacts.

a. General.

(1) Sediment accumulation in reservoirs causes a range of infrastructure and environmental problems upstream of dams. These problems can extend from the dam upstream to above the maximum pool elevation. This chapter discusses some of these impacts.

(2) USACE reservoirs are typically separated into pool zones according to use and operations. The terms used to describe these zones are inconsistent in USACE and have changed over time. Reservoir pool zones also vary due to different authorized purposes, release mechanisms, and operating criteria. The degree of sediment accumulation, and the consequences of sediment depletion, also vary by pool zone. Figure 8-31 shows a schematic of typical USACE terms, as will be discussed in subsequent sections of this chapter. Reservoir pool zones and use at USACE projects are further discussed in EM 1110-2-1420.

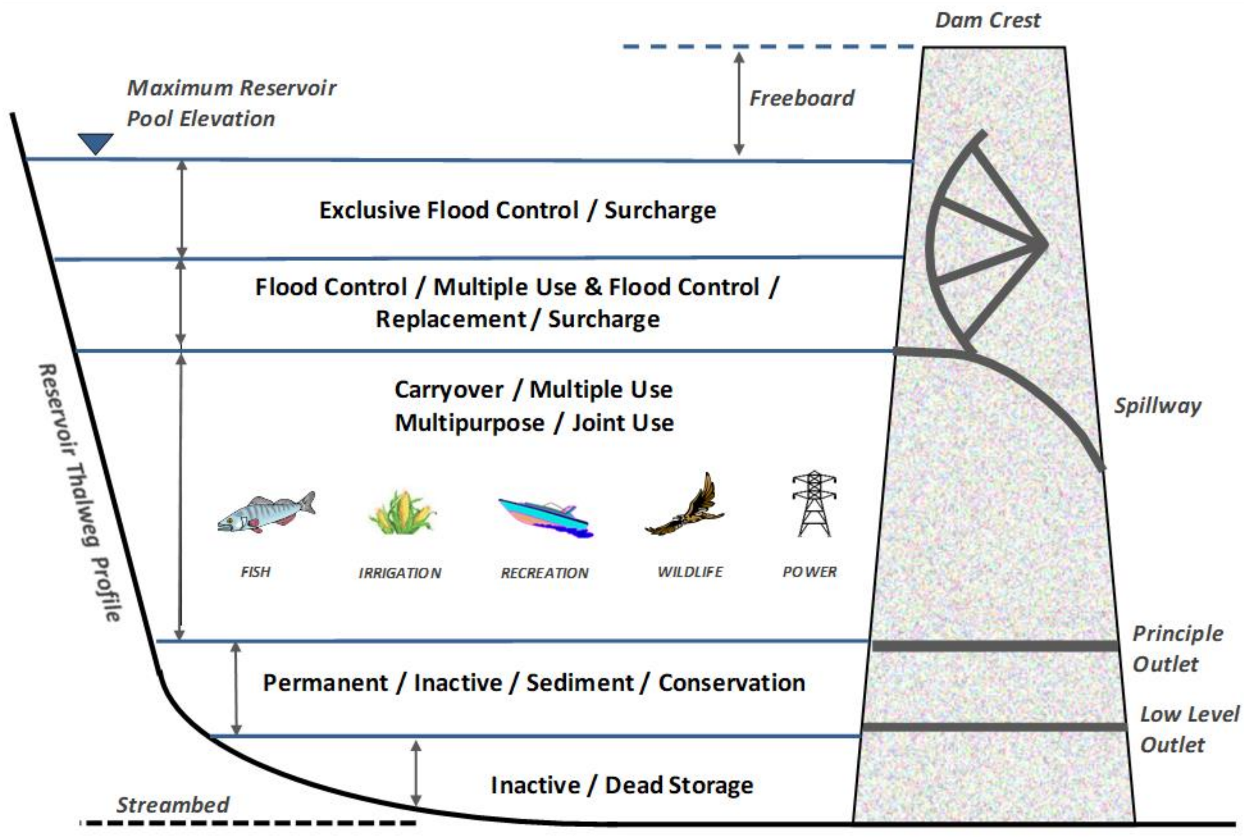


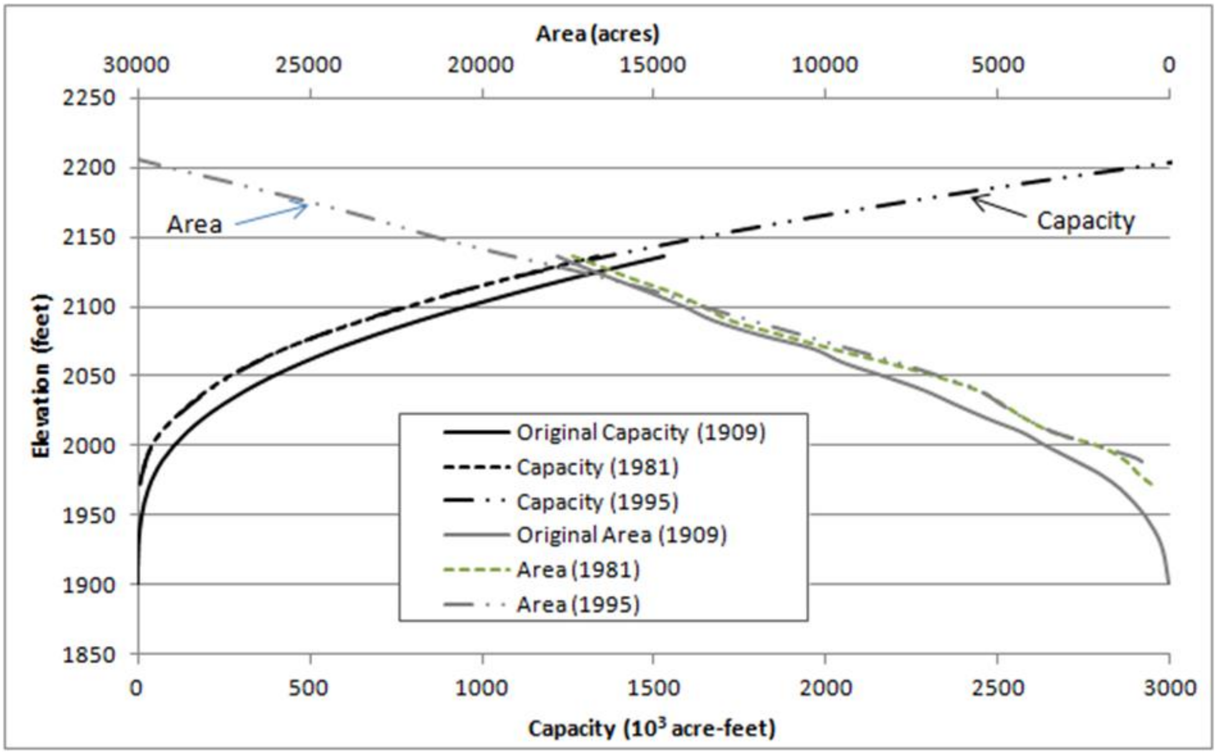
Figure 8-31. Schematic of typical USACE terms for pool zones

b. Reservoir Capacity/Storage Depletion.

(1) Sediment accumulation displaces storage capacity, which impairs the dam's intended operational uses such as water supply, flood control, power generation and navigation. If spillway capacity is based on storage routing through the reservoir, then dam safety can be affected.

(2) Reservoir storage characteristics are often quantified using stage-capacity and stage area graphs. As sediment deposits in the reservoir, the stage-capacity curve shifts to left, reflecting the reduction in storage throughout the low elevations in the reservoir. Data sources: Strand and Pemberton (1987) and Lyons and Lest (1996). Capacity is storage volume (acre-feet), and Area refers to the reservoir surface area (acres), both values shown by elevation.

(3) Figure 8-32 shows this shift for Theodore Roosevelt Lake on the Salt River in Arizona from its original capacity of 1,530 acre-feet in 1909 to a reduced capacity of 1,337 acre-feet in 1981. The original dam elevation was 2,142 feet. Modifications to Roosevelt Dam completed in 1995 raised the dam elevation to 2,218 feet increasing its capacity to 3,496 acre-feet.



Data sources: Strand and Pemberton (1987) and Lyons and Lest (1996). Capacity is storage volume (acre-feet), and Area refers to the reservoir surface area (acres), both values shown by elevation.

Figure 8-32. Area and capacity curves for Theodore Roosevelt Lake

(4) While sediment accumulates in all reservoir pools, as a percentage of existing storage, sediment may disproportionately affect specific pool elevation ranges. For example, in coarse-sediment or large-volume systems, deposition is expected to be concentrated in the delta region, while in fine-grain systems, deposition may transport further downstream with higher impact in the conservation pool. In addition, tributary-dominated load systems generally behave uniquely as driven by the tributary input location.

(5) In many reservoirs, the pool most impacted is the multi-purpose pool, which often has the most consequences on operations. Coupled with increased demand due to population growth, storage loss impacts water supply security. This is the case in the Kansas River basin, where USACE reservoirs provide water supply to approximately 43% the population of Kansas. Rising water demand and storage loss to sedimentation in the USACE reservoirs makes the current state-owned water supply insufficient by 2057. As shown in Figure 8-33, the total available water supply becomes insufficient to protect against drought by 2090.

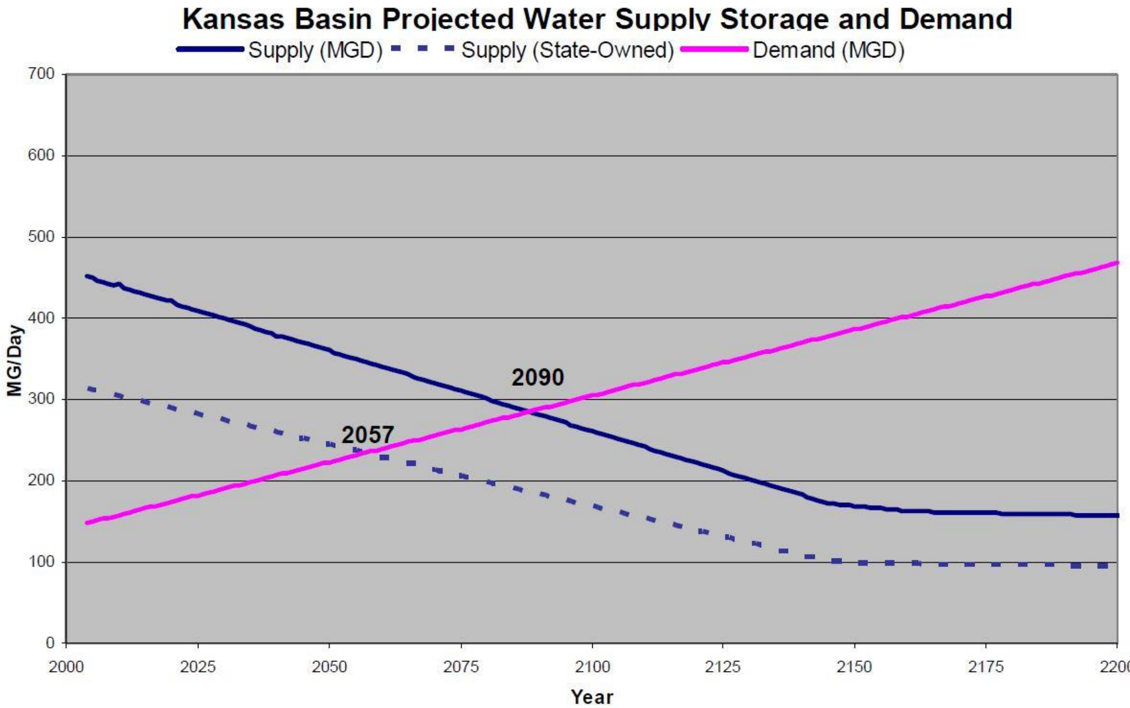


Figure 8-33. Shrinking water supply due to sedimentation and rising water demand in the Kansas River Basin (Kansas Water Authority 2010)

c. Facility Impacts, Gates, and Outlets. Sedimentation and scour can impact reservoir operations by impacts to the release mechanisms such as static loading of the dam structure, gates, and equipment. Another impact is by causing flow changes associated with the presence or absence of sediment at a given location in the reservoir. Finally, sediments may directly impact project features such as hydropower turbines as they pass through the dam facility. These impacts can occur long before complete filling of the reservoir pools with sediment. At Garrison reservoir, deposited sediments within the spillway approach channel have reduced the approach depth for the ogee crest spillway, which has resulted in a loss of spillway discharge capacity.

(1) Operational Failure.

(a) Sediment can block or clog intakes and outlets at dams and can obstruct or otherwise damage gates that are not designed for sediment passage. Sediment impounded against a gate structure can cause excessive eccentric forces on the gate and render it inoperable.

(b) As shown in Figure 8-34, Paonia Reservoir in Colorado lost nearly 25% of the total storage when sediment and debris began clogging the water intake structure, requiring manual excavation and debris removal (Collins and Kimbrel 2015).

Figure 8-34. Clogging of water intake at Paonia Reservoir, Colorado (Collins and Kimbrel 2015)

(c) Loss of upstream storage can reduce the energy available for power generation by steepening the drawdown curve of the reservoir. For hydropower stations constructed in series along a river or adjacent tributaries, backwater from the downstream reservoir delta may affect the tailwater of the next reservoir upstream, reducing available head for power generation and, in extreme cases flooding the upstream facility.

(d) For reservoirs where the spillway design reduces flood peaks, routing and storing flow in the reservoir, sedimentation can reduce the flood peak attenuation, releasing larger flows over time, affecting dam safety.

(2) Abrasion. Sediment abrasion can damage hydraulic machinery, rendering valve and gate seals, outlet works, aprons, tunnels, and spillways useless in relatively short time spans. In hydropower facilities, sediment coarser than 0.1 mm accelerate the erosion of both Pelton and Francis turbine runners and their flow control valves (Figure 8-35). Sediment can also clog generator cooling systems. Angular sediments and high-head operation can cause abrasion with even smaller grain sizes. Abrasion reduces power generation efficiency and generating units are sometimes removed from service for repair (Morris 2016). On steep streams sediment sizes up to boulders can be passed through bottom outlets and tunnels, which results in large scale tunnel abrasion. Figure 8-36 shows impact damage to a steel-lined tunnel that regularly passes material up to the size of small boulders.

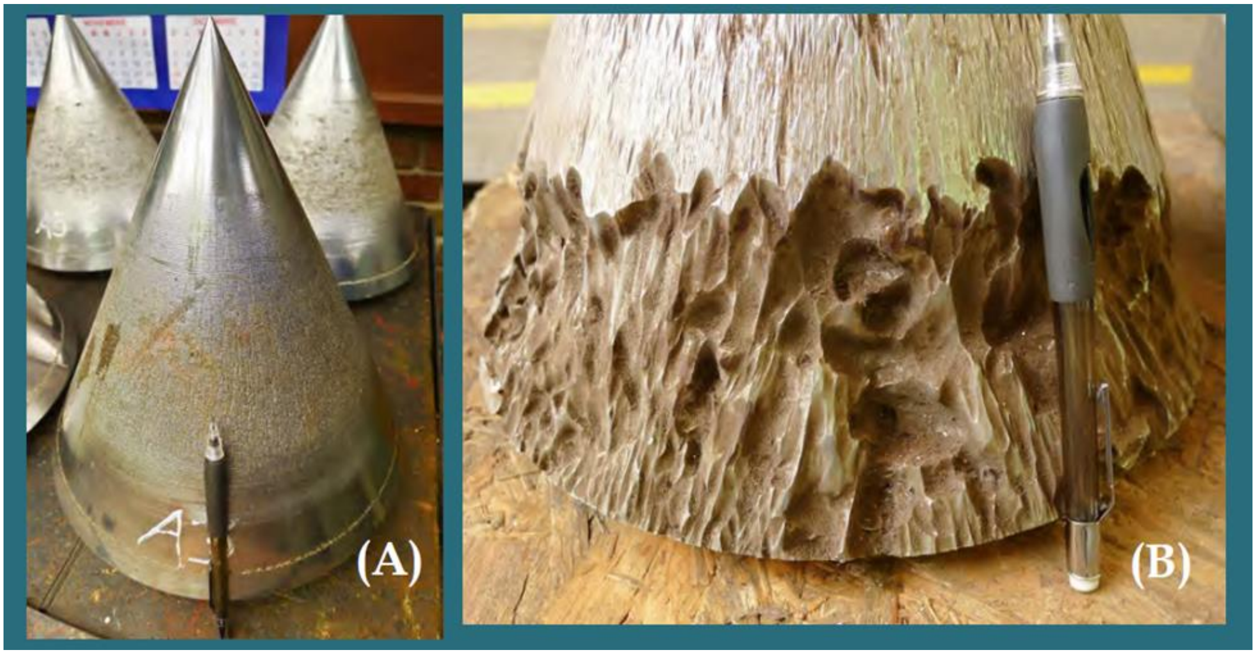


Figure 8-35. Erosion of needle valve on 125 MW Pelton turbine under 800 meters of head: (a) 10,000 hours normal operations, (b) less than 24 hours passing heavy sediment load including sand (Morris 2016)

Figure 8-36. Damage to 9-foot steel-lined tunnel at Mud Mountain Dam, Washington, due to combined effects of cavitation and cobbles, shown by eroded trough of murky water

(3) Upstream Erosion.

(a) Two principal mechanisms cause scour of upstream deposits (considering only the reservoir processes): bank erosion from reservoir operations and wave action on the reservoir shoreline, and scour of deposited sediments. Scour of deposited sediments occurs when deposited sediments are exposed due to lower pool levels and incoming flows have sufficient energy to mobilize deposits (such as high-flow event-deposited sediments followed by normal pool levels, normal pool deposited sediments followed by low pool). Both mechanisms can redistribute sediment in the reservoir and affect downstream waterways.

(b) Sedimentation effects due to the scour of reservoir deposited materials can be significant. The USGS developed a regression equation to predict the sediment scour load transported to the Upper Chesapeake Bay from the three Lower Susquehanna River reservoirs (Langland et al., 2013). They calculated that a flow of about 400,000 cfs (approximately a 5-year recurrence interval) began scouring deposited materials. They estimated that 57 million tons of the 190 million tons of sediment delivered to the Chesapeake Bay scoured from reservoir deposits with the prediction of increasing sediment concentrations and loads to the bay due to the continued loss of reservoir storage.

(c) Local scour leading bank failure can also have negative effects within the reservoir. Lai and Aubuchon (2010) used a 2D model to predict bank failure due to scour on the Rio Grande upstream of the San Acacia diversion dam before designing bank protection. The model predicted scour from 6.6 to 19.2 feet for a 10% annual exceedance probability event.

(4) Sediment Loading on Structures. The sediment and sediment-water mixtures that collect on the upstream face of a dam can significantly increase both the static load on the dam and dynamic earthquake loads against the structure (Chen and Hung 1993). In addition, earthquakes can liquefy sediments deposited near the dam, causing flow toward the dam face and damaging dam equipment, intakes, and outlets. During an earthquake, sediment can modify the frequency of dynamic pressure oscillations (Du et al., 2001). These modifications can change the frequency of long period seiches set up in the reservoir and consequently the hydraulic loading on the dam compared to predictions without sediment.

d. Consequences of Altered Pool Elevations Due to Sediment Loading and Distribution.

(1) In-Reservoir Effects.

(a) Landslides and Shoreline Erosion.

- Landslides and debris flows can partially or completely fill reservoirs and damage the structure of a dam. These events are particularly dangerous shortly after initial loading (filling) of the reservoir. Riemer (1995) analyzed 60 publications concerning landslides in reservoirs and found that 85% of landslides associated with reservoirs are initiated during construction, filling, or in the first two years after their filling. Landslides that occur in reservoirs can reduce reservoir capacity and cause very large waves and seiches.

- An often-cited example of catastrophic failure occurred at the 265-m high Vaiont (also referenced as Vajont) thin arch dam north of Venice, Italy in 1963. On October 9, 1963, a massive 2.6 x 10^8 cubic meter landslide occurred in the reservoir and created a wave that overtopped the dam to a depth of 100 m (330 feet) and killed more than 2,600 people downstream. The failure occurred after smaller previous failures in the reservoir during filling and under conditions of accelerating creep of the slide area (Jansen 1983).

- Many of the landslides that develop along reservoir shorelines are old slides that predate the dam, reactivated by hydraulic forces. Franklin D. Roosevelt Lake, upstream of the Grand Coulee Dam on the Columbia River, has been the site of hundreds of reservoir-induced landslides since filling of the reservoir in the early 1940s, causing tens of millions of dollars in damages. These slides occur in unconsolidated Pleistocene glacio-fluvial materials that constitute much of the rim of the reservoir (Schuster 1979). Schuster (2006) located 254 large (≥33 feet (10 m) high) dams worldwide with landslide impacts.

- Shoreline erosion, although less catastrophic than a landslide, can have significant consequences on operations as previously described (paragraph 8-3e).

(b) Shallow vs. Deep Reservoirs.

- Deep reservoirs (longer retention and higher TE) develop different sediment transport dynamics than shallow reservoirs (short retention times, likely reach a sediment equilibrium). For reservoirs, many of these differences can be attributed to operations and pool elevations and the residence times for water and sediment in the reservoir.

- Deep reservoirs generally have longer residence times for both water and sediment, allowing sediment to settle and developing discrete limnologic zones. Unless reservoir operations pass density currents and draw the reservoir down to run-of-river conditions, the sediment in a deep reservoir may never be transported past the dam.

- Deep reservoirs operated at different pool elevations develop distinct deltas at different water levels. Additionally, deep reservoirs forms thermoclines and reduce light penetration and photoactivity at the lower levels of the reservoir. Thermoclines and light penetration impact water quality directly. For example, phosphorus-carrying particles transported by the upstream river and prosperous generated by reservoir phytoplankton deposit deep in the reservoir.

- The deepest part of the reservoir pool is immediately upstream of the dam, so water quality usually improves downstream, from the shallow delta to the deeper end near the dam. Therefore, deep sluice gates often pass cold clear water and higher sluice gates pass warmer more turbid water.

- Shallow reservoirs are highly affected by the rate of storage water exchange during reservoir operations that can be simply defined as release rate/storage volume. They generally have short water and sediment retention times that depend heavily on upstream conditions and episodic flow events. Sediment deposition rapidly reduces storage capacity in shallow reservoirs, potentially damaging or blocking intake structures and sediment entrainment in power-generating facilities.

- Shallow reservoirs, particularly run-of-river structures, usually achieve a quasi-equilibrium. They deposit during low to moderate flow events and scour during floods. These reservoirs do not affect water quality as much as deep reservoirs.

- Shallow reservoirs increase the potential for event-driven sediment transport downstream, because large events can entrain and scour the sediment accumulated by several years, along with the adsorbed or associated nutrients or contaminants. For example, on the Susquehanna River, approximately 30% of the sediment load transported to the Chesapeake Bay comes from scour of the Safe Harbor, Holtwood, and Conowingo reservoirs (Langland et al., 2013).

- Understanding the relative differences between sedimentation in shallow and deep reservoirs can help in the planning of reservoir systems. Lajczak (1996) examined sedimentation for deep and shallow reservoirs in the Vistula River basin in Poland and concluded that both the location of the planned reservoirs and the construction sequence could reduce sedimentation rates in the reservoirs.

(2) Groundwater and Base Flow Impacts.

(a) Elevated water levels from reservoir operations or from delta backwaters will raise groundwater elevations near reservoirs. Near deltas, reservoirs can raise groundwater elevations significantly higher than the maximum operational pool. In 1971, USACE had to re-locate the town of Niobrara, Nebraska, because the Lewis and Clark delta deposition and rising lake levels also raised groundwater levels (USACE 1970).

(b) Groundwater seepage can also account for a significant percentage of reservoir losses. Raised reservoir water levels causes the surface water to move into bank storage. When water levels in reservoirs decrease, this bank storage returns to the reservoir. Some reservoirs are intentionally sited in high-permeability locations to recharge aquifers. Fine deposition tends to reduce reservoir substrate permeability, which can attenuate seepage from into the surrounding groundwater (Gibson 2014). In a study using 22 groundwater observation wells, Seeboonruang (2012) found that the groundwater level correlated with reservoir levels on the Lower Nam Kam River, and that freshwater displaced the normal saline water conditions at high reservoir levels.

(c) Geological investigations and seepage calculations can predict the interaction between groundwater and reservoir water levels. One of the simplest ways to estimate seepage to groundwater is to develop a function related to reservoir depth. This approach usually assumes that seepage flows are always a net loss from the reservoir, which is likely at most sites.

However, bi-directional flux can be important on some systems, depending on the elevation of the reservoir pool compared to the surrounding groundwater levels (Winter et al., 1998).

- The Nebraska Department of Natural Resources compared annual flow through Box Butte Dam with outflow from a diversion seven miles downstream and concluded that the reservoir added about 300 acre-feet of annual base flow to the downstream reach of the Niobrara River (NDNR 2004).

- Simons and Rorabaugh (1971) found that the aquifer associated with Hungry Horse Reservoir in Montana, which is part of the Columbia River system, returned enough flow to the reservoir at low pool elevations that the reservoir management plan for the Columbia River system had to include bi-directional flux in their water budget.

(3) Impacts Upstream of the Normal Operating Pool on Mainstem and Tributaries.

(a) Sedimentation that increases water levels either by modifying reservoir operations or from backwater upstream of the depositional delta, can have negative effects on facilities in and around the reservoir. Facilities that rely on specified clearance above the water table can fail to function properly, resulting in blocked drainage and malfunctioning infiltration facilities.

(b) Higher pore pressures can contribute to slope instabilities on the shoreline of the reservoir and on the banks of the inflowing river and tributaries.

(c) Rising groundwater levels can damage infrastructure, like piping systems, which were not designed to be submerged.

(d) Legacy contaminants in the unsaturated zone can leach into rising groundwater as the water table raises above historic levels.

(4) Impact of Reallocation on Wildlife Habitat in Delta and Backswamp Areas.

(a) Riparian areas located on the upstream headwaters of both the main stem and tributaries often occupy a narrow band between the river and roads or infrastructure. Riparian vegetation is typically adapted to historic hyporheic groundwater elevations and fluctuations. Increased groundwater levels make riparian vegetation less suitable in marsh and backswamp areas, and it can be replaced by more competitive vegetation that is better adapted to persistently high groundwater levels. If slopes and soils are tolerable, riparian areas may transition to higher elevation zones within the pool.

(b) Reservoirs can also increase phreatophyte abundance. Phreatophytes are deep-rooted plants that draw water from the phreatic zone (zone of saturation) or the capillary fringe above the phreatic zone. Phreatophytes grow fast and are disease-resistant. They provide nesting areas and shelter for fauna, feed livestock, and help to purify these waters, fixing heavy metals and filtering bacteria with their heavy roots.

(c) Phreatophytes also transpire water from reservoirs. For example, Morris and Fan (2010) estimated phreatophytes consumed 3×10^{10} m^3 annually from all the reservoirs in the western United States. In arid zones the transpiration from large areas of phreatophytic vegetation in delta areas is a significant reservoir sink. For example, evaporative losses from the delta of Elephant Butte Reservoir on the Rio Grande in New Mexico were estimated at 1.76×10^8 m^3/yr (Gorbach and Baird 1991). Additionally, phreatophytes increase hydraulic roughness, which can increase upstream water levels and induce deposition. The vegetation itself can also trap sediment, leading to further aggradation.

(5) Impacts of Sediment Deposition on Water Quality. Sediment deposition in a reservoir and delta can drastically change the ecology and habitat distribution in a river. Greathouse et al. (2006) found that in high-gradient streams above large dams in Puerto Rico, migratory fauna such as shrimp and fish were extremely rare, whereas similar sites without large dams had high abundance of freshwater shrimp and migratory fish. Losses of native fishes were summarized by Juracek (2015) for studies of rivers in Colorado, noting that non-native species may have been suited for survival in the slow-moving conditions that now comprise a large portion of the Colorado River and nearby waterways.

(a) In areas of deltaic sedimentation at the upstream end of a reservoir, the lake open water habitat transitions to wetlands and eventually to upland habitat. In addition to sediment, the delta will also accumulate organic debris upstream of the turbidity current plunge point (see paragraph 8-4d). A large portion of the organics, nutrients, and contaminants in this debris are carried in particulate form or become absorbed onto sediments.

(b) Organic matter deposition can affect the reservoir oxygen budget significantly. James et al. (1987) investigated seasonal and spatial differences in sediment trapping rates and found they were related to longitudinal gradients in water quality at DeGray Lake in Arkansas. They found that higher oxygen demand of recently deposited organics in the shallower warmer regions at the upstream end of the reservoir led to a greater region of anoxia earlier in the year, while larger hypolimnetic volumes, cooler temperatures, and diminished deposition further downstream in the reservoir lowered oxygen demand and delayed anoxia.

(c) Many of the chemical impacts of sedimentation on water quality are summarized in paragraphs 8-4g and 8-4h.

(6) Impacts of Sediment Deposition on Ice Jams and Flooding.

(a) Northern rivers can form ice jams from when their ice cover breaks up and transports downstream until the ice floe encounters an obstacle like a structure, a river bend, shallow water, or a thick, competent, ice cover (Figure 8-37).

Figure 8-37. Ice action on bridge piers and flooding caused by an ice jam (USACE 2006a)

(b) Ice jams tend to form upstream of reservoirs. The quiescent reservoir pool forms thick ice cover. The transported ice accumulates behind the thick, static, reservoir ice, causing an ice jam. Reservoir delta deposition can make ice jam flooding worse. The delta decreases the slope and depth of the river and increases the bed elevation, making the ice jam flooding more frequent and more severe (Tuthill 1999).

(c) As shown in Figure 8-38, water levels rise upstream of the reservoir in response to the flow resistance of the rough ice (White 1999), and to accommodate the submerged ice volume. In an ice and sediment modeling study, Gibson et al. (2017) demonstrated that reservoir delta deposition on the Muskegon River increased the probability and severity of ice jam flooding, and that future flood risk would continue to rise as deposition continued.

(d) Jams can form either at winter freeze-up or during spring breakup of ice. Breakup jams are generally more serious with the possibility of ice grounding and rapid downstream surges of water during release. USACE has good modeling tools that apply a steady-state momentum balance to compute the thickness and water surface rise associated with a fully developed ice jam. The modeling tools help integrate ice effects into flood stage-frequency curves (Tuthill et al., 1996). Predicting the timing or modeling the unsteady development of ice jams is more complicated, however.

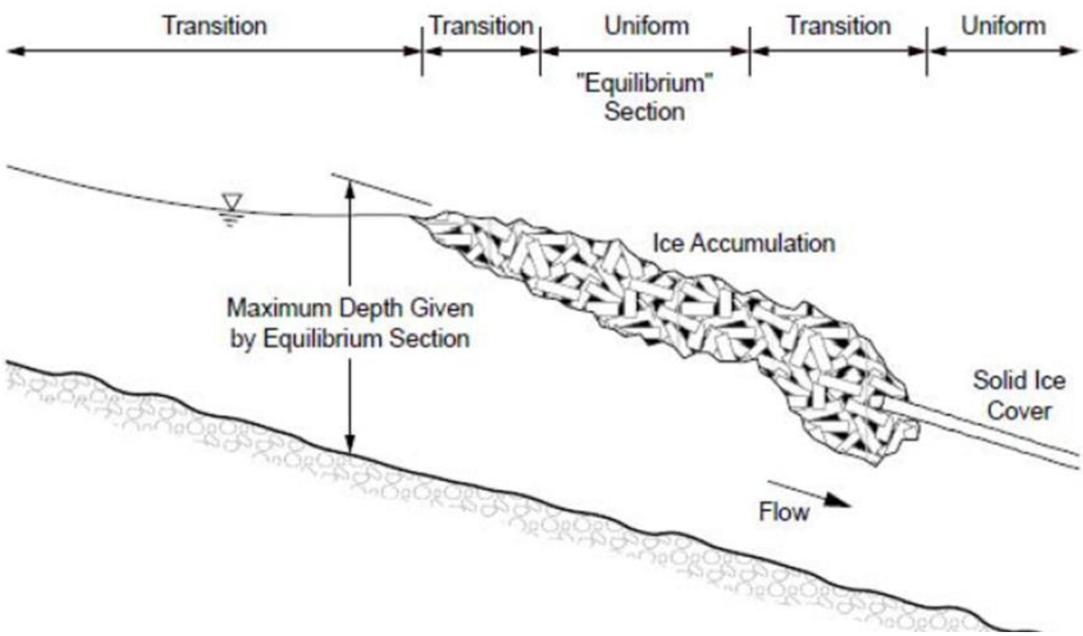


Figure 8-38. Conceptual sketch of ice jam formation from White (1999). Ice jams that form upstream of reservoirs, the solid ice cover which anchors the jam is usually the thick, competent ice that forms on the reservoir pool.

(e) Tuthill (1999) described reservoir operation methods to manage ice problems and reported mixed results for reservoir pool operations to manage ice jams upstream of the dam, where the ice jams tended to form along the delta. He speculated that reservoir drawdowns could drop flood stages, but may also reduce ice conveyance, making the jam worse. Conversely, raising the water level to disrupt the solid ice cover and create ice capacity can expose the structure to ice damage. These operations mainly focused on downstream releases.

(f) Releases during early winter promote rapid formation of a smooth, stable ice cover. Midwinter releases maintain an intact ice cover and avoid premature ice breakup. During the final winter period, releases minimize adverse effects of ice breakup (Ferrick and Mulherin 1989). Jain et al. (1993) found that a pool rise broke up the ice jam upstream of the structure, but it reformed on the dam, damaging the structure and sinking vessels. Gibson (2016) found mixed results in historical accounts and modeling accuracy for predicting reservoir operation on ice jam severity, but also found that spring reservoir drawdowns, which scoured the delta, could decrease ice jam frequency and severity.

(g) Reservoir deposits and ice jams have several other feedbacks summarized as:

- Delta deposits can cause the ice jam to form farther upstream or at a higher elevation. The jams, in turn, can produce sedimentation further upstream than might otherwise be predicted.

- Ice introduces a second rough boundary, which affects the logarithmic velocity profile, increasing near-bed velocity and shear (Ettema et al., 2000b). Therefore, while sediment deposits tend to form ice jams, ice jams tend to scour sediment (Ettema 2008; Gibson 2016). Carr and Dahl (2017) further reviewed the impacts of ice-induced scour on USACE structures.

- While dam removal can mitigate ice jams upstream of the reservoir, removing the historic ice barrier and downstream deposition often causes ice jams to form in new locations downstream (White and Moore 2002).

- Paragraph 7-8f provides additional discussion on ice jams and effects on sediment transport within rivers.

e. Impact on Severity of Wildfires. Reservoirs often change the amount and species of plants and trees growing in the upstream and surrounding areas. Urbanization that occurs following reservoir construction can also lead to increased density of new vegetation. These changes can result in an increase in the combustible material available to burn in a wildfire. Phreatophytes can spread along the shoreline and in deltas on the main channel and tributaries to form substantial areas of new vegetation. When pool levels recede, vegetation in these areas can become fire prone. Wildfire sedimentation effects are further discussed in paragraph 6-7e.

f. Impacts to Navigation. Sediment accumulation upstream of dams can impact commercial and recreational navigation. Navigation projects created by a chain of low-head dams and locks along a river will pass most sediment through the structures at high flow, operating essentially in open water conditions. However, sediment can accumulate in locks, approach channels, and delta regions. Reservoir operations for purposes such as flood control or water supply can exacerbate sedimentation issues for navigation. Recreational access can also be impaired as sediment accumulates at marinas and boat ramps. For reservoirs that provide releases to support downstream navigation, lost storage capacity can impair the capability to meet storage needs and flow releases.

g. Effects of Upstream Reservoir Flushing/Sluicing/Bypassing. Sedimentation in a series of reservoirs can be influenced by upstream sediment management operations that controls the volume, gradation, and timing of sediments released by flushing, sluicing, or other bypass methods. Sediments released by flushing may be finer than those anticipated during dam design and may affect the projected rate of sedimentation. Periodic large flushing events may contribute large amounts of sediment that travels down a channel in waves, causing unsteady sedimentation of the upstream delta and reservoir. Additionally, fine sediments released from an upstream reservoir may affect turbidity and water quality downstream. For these reasons, a system approach to reservoir sediment management is recommended.

h. Recreation and Other Effects.

(1) Reservoir sedimentation can adversely affect lake recreation. Sediment accumulation near boat ramps and marinas can lead to abandonment or necessitate modifications. Debris clearing and storage activities may experience added costs due to higher debris loading from

eroding shorelines. Sedimentation and erosion can also present impacts to the aesthetics of the reservoir if large shoals, mudflats, or scarps and landslides develop in and above the reservoir. The progression of a delta can shrink the overall surface area of the lake suitable for aquatic recreation. At Tuttle Creek Lake, the surface area at multi-purpose pool elevation decreased by 31% from 1957 to 2010 (Figure 8-39).

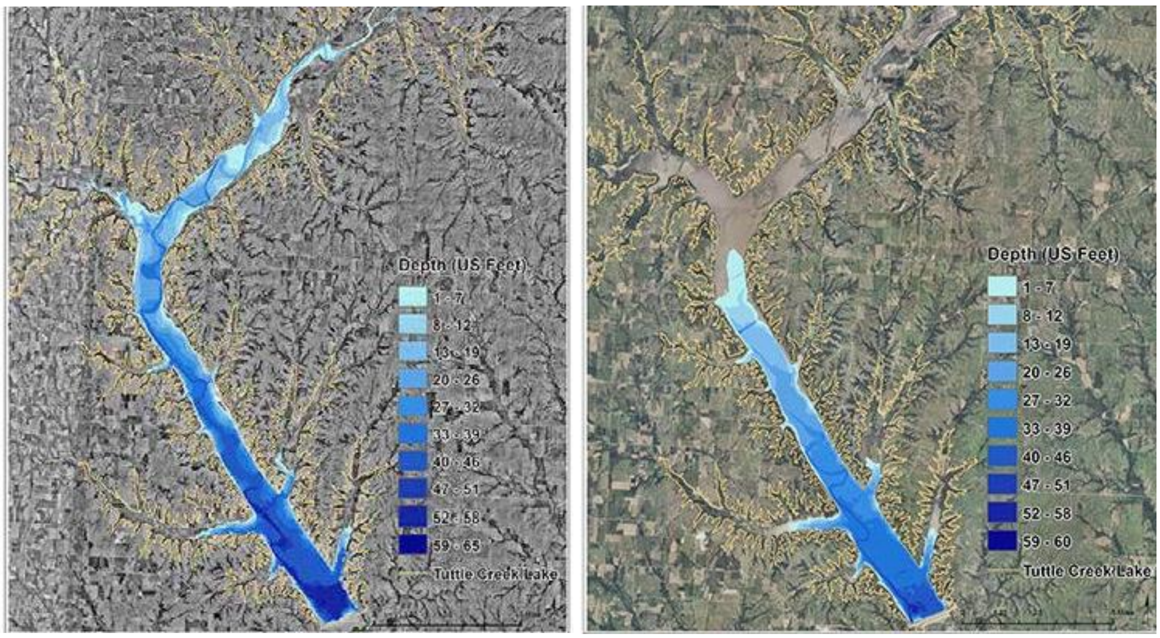


Figure 8-39. Loss in lake surface area at Tuttle Creek Lake, Kansas (Shelley 2017)

(2) On the other hand, delta depositional areas may provide opportunities for duck hunting and environmental wetland benefits.

(3) Sedimentation can affect the value and amount of real estate surrounding a reservoir and upstream near the delta. Shoreline erosion, slope steepening, and soil loss can reduce land area or threaten existing structures. Delta formation and sedimentation upstream can change the value of riverfront real estate over time.

(4) In seasonally empty reservoirs, desiccated deposits of fine sediment can be eroded and transported by wind, creating a nuisance and health hazard to nearby habitat and communities.

8-6. Downstream Channel Processes and Impacts. A dam changes the downstream river by reducing the downstream sediment supply and by reducing intense, large volume floods that are associated with high sediment transport. Williams and Wolman (1984) describe changes in mean channel bed elevation, channel width, bed material sizes, vegetation, water discharges, and sediment loads downstream from 21 dams constructed on alluvial rivers. Kondolf (1997) relates that channels downstream of dams are prone to erode the channel bed and banks, producing channel incision (downcutting), coarsening of bed material, and loss of spawning gravels for salmon and trout (as smaller gravels are transported without replacement from upstream).

a. Sedimentation Effects of Altered Hydrology. Reservoir operations and flow releases can have significant effects on downstream sediment processes. Figure 8-40 illustrates the effects of flow regulation from upstream reservoirs on the Missouri River at Sioux City, Iowa.

(1) Reservoir attenuation of peak flows may result in insufficient high flows to flush fine sediments out of gravel spawning areas. This can also lead to less deposition of coarse sediments onto sand/gravel bars or the floodplain.

(2) Due to the usually nonlinear dependence of sediment transport on flow, decreased peak flows can significantly decrease the overall transport potential of the river, particularly in incised channels, even if the total volume of water releases remains the same.

(3) On the other hand, reservoirs that attenuate low flows can transform flows below the critical thresholds for sediment movement into flows that are competent to move sediment, thus increasing the overall potential for sediment movement.

(4) Sediment rating curves and hydraulic modeling may be used to assess the effects of altered hydrology to evaluate responses.

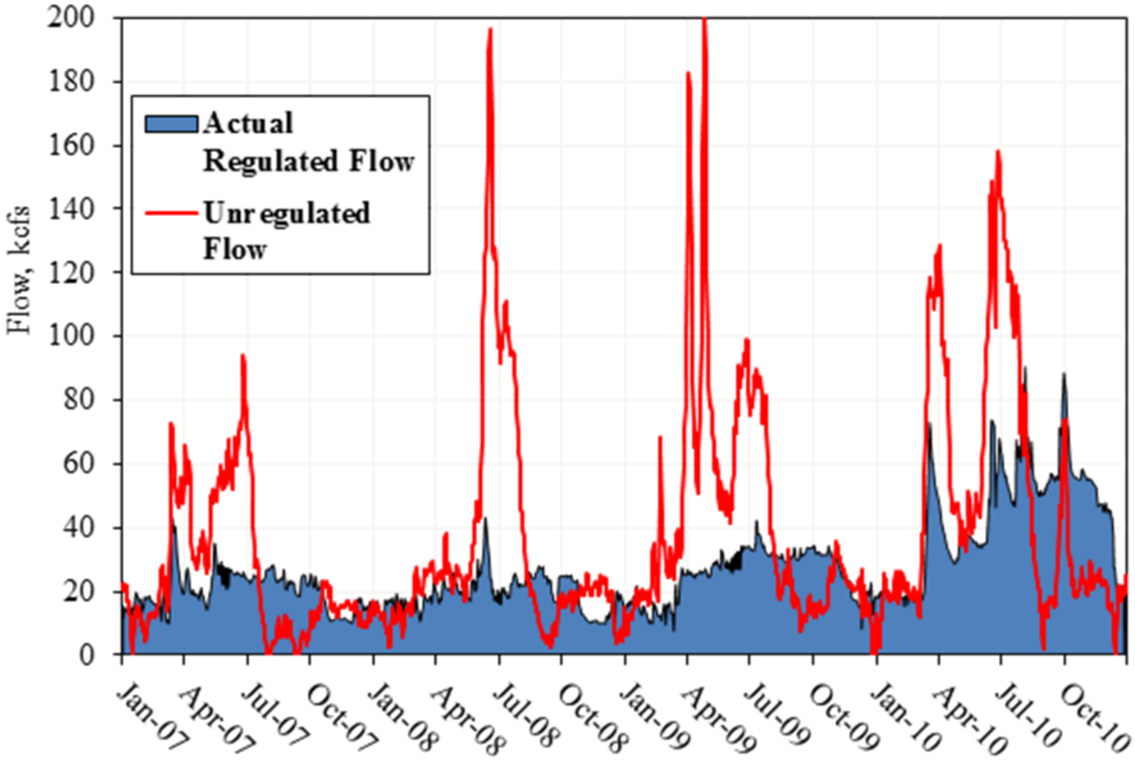


Figure 8-40. Effects of flow regulation on the Missouri River at Sioux City, Iowa

b. Downstream Channel Degradation.

(1) Degradation refers to the persistent lowering of the channel bed elevation over a substantial distance and for an extended period of time. This is different from the local scour that may occur at structures such as pier abutment scour at a bridge crossing. This also differs from temporary fluctuations in bed elevation in response to individual events. Bed degradation can induce water surface degradation, or a persistent lowering of water surface elevation for similar discharges.

(2) The water surface degradation that has occurred downstream from Gavins Point dam since construction in 1955 is shown in Figure 8-41. The figure illustrates the amount of degradation, as measured by the change in water surface elevation, for three different flows. At normal flows, the degradation is about 12 feet (3 m). The degradation rate was initially steeper with a slower rate since about 1980. The degradation also shows some specific event impacts with significant drops in high-flow years like 1997 and 2011.

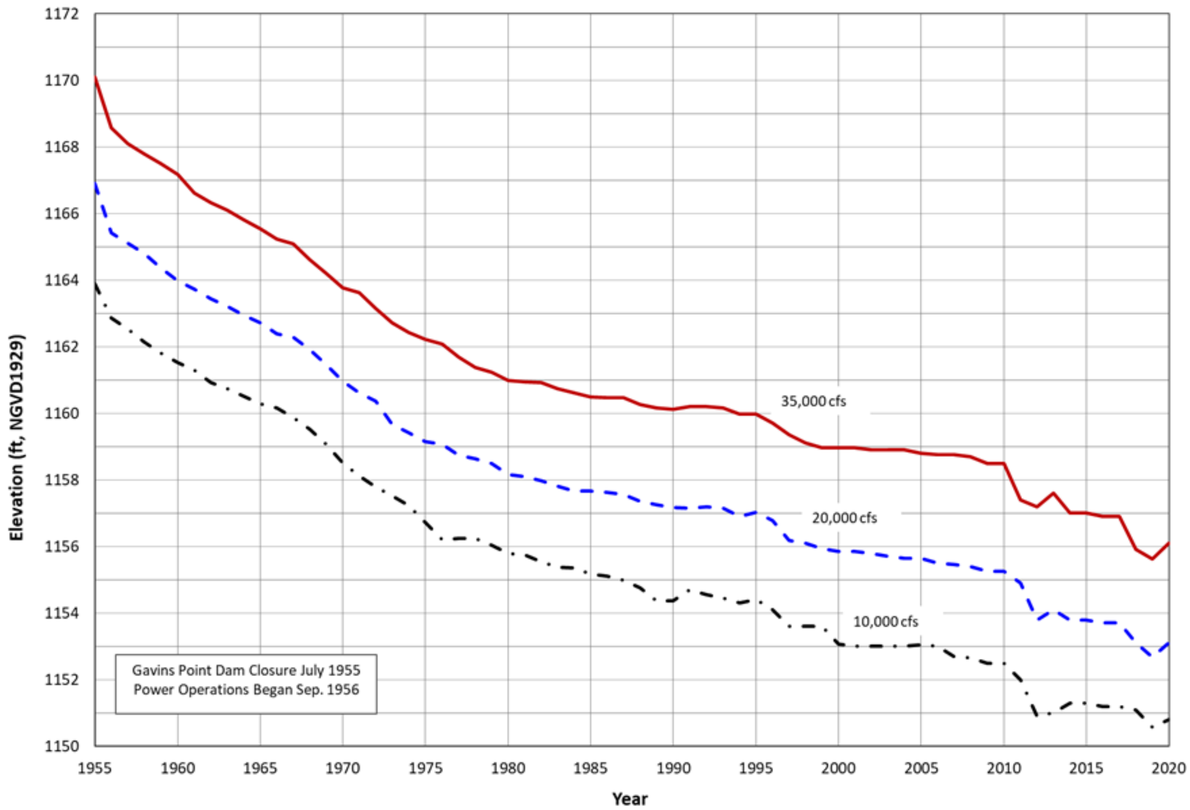


Figure 8-41. Water surface degradation downstream of Gavins Point Dam over time

(3) Degradation downstream from a dam occurs because dam closure causes a dramatic imbalance between sediment supply and sediment transport capacity. The downstream river possesses the width, depth, and slope necessary to carry the pre-dam bed material load. The dam discharge, on the other hand, contains essentially no bed material. The water begins scouring sediment from the river bed for some distance downstream until sufficient load has been entrained such that the rate of settling of bed material out of the water column equals the rate of entrainment of bed material back into the water column.

(4) The rate of degradation is usually greatest at the dam and decreases with distance downstream. As the bed degrades, the channel slope flattens, which lowers shear stress and the transport capacity. Given a steep gradient to the main channel, a tributary may erode at its confluence mouth, known as base-level lowering, which causes degradation to headcut up the tributary.

(5) Downstream from many dams, preferential transport of finer bed sediments plus a decrease in shear stress from that required for transporting the largest sediments leads to the formation of an armor layer. The conditions for the formation of an armor layer are further discussed in paragraph 8-6c. These conditions slow the rate of sediment entrainment and transport and convey less sediment to the channel downstream of the degradation area. The leading edge just downstream from the degradation and/or armored reach receives sediment-poor

water from upstream but has a high transport capacity due to a high channel slope. These conditions propagate the degradation further downstream.

(6) The degradation magnitude, rate, and distance downstream of the dam are influenced by a number of factors that should be considered in an evaluation. Factors include:

(a) Bed and bank material type, size (resistance to erosion).

(b) Amount of armoring (also related to bed and bank material type).

(c) Peak discharge reduction, flow duration changes, base flow increase.

(d) Operations with impacts such as hydropower daily flow variation.

(e) Sediment inflow from tributaries located downstream of the dam, both size and quantity.

(f) Grade control points along the bed such as bars, bedrock, or artificial sills and barrages.

(7) Figure 8-42 shows the change in water surface elevation with distance downstream of Gavins Point Dam on the Missouri River. The plot shows change broken into four different time periods. Note that water surface elevation change is not consistent, indicating that degradation is also influenced by local factors including bed materials, bank materials, and tributary entrance with sediment load, etc. For example, the location from river mile 730 to 750, 60 to 80 miles downstream from Gavins Point Dam (100 to 130 km) had degradation as great as at the dam site. At a distance of 200 miles downstream from the dam (over 300 km) there is still a degradation of 3 to 4 feet (about 1 m).

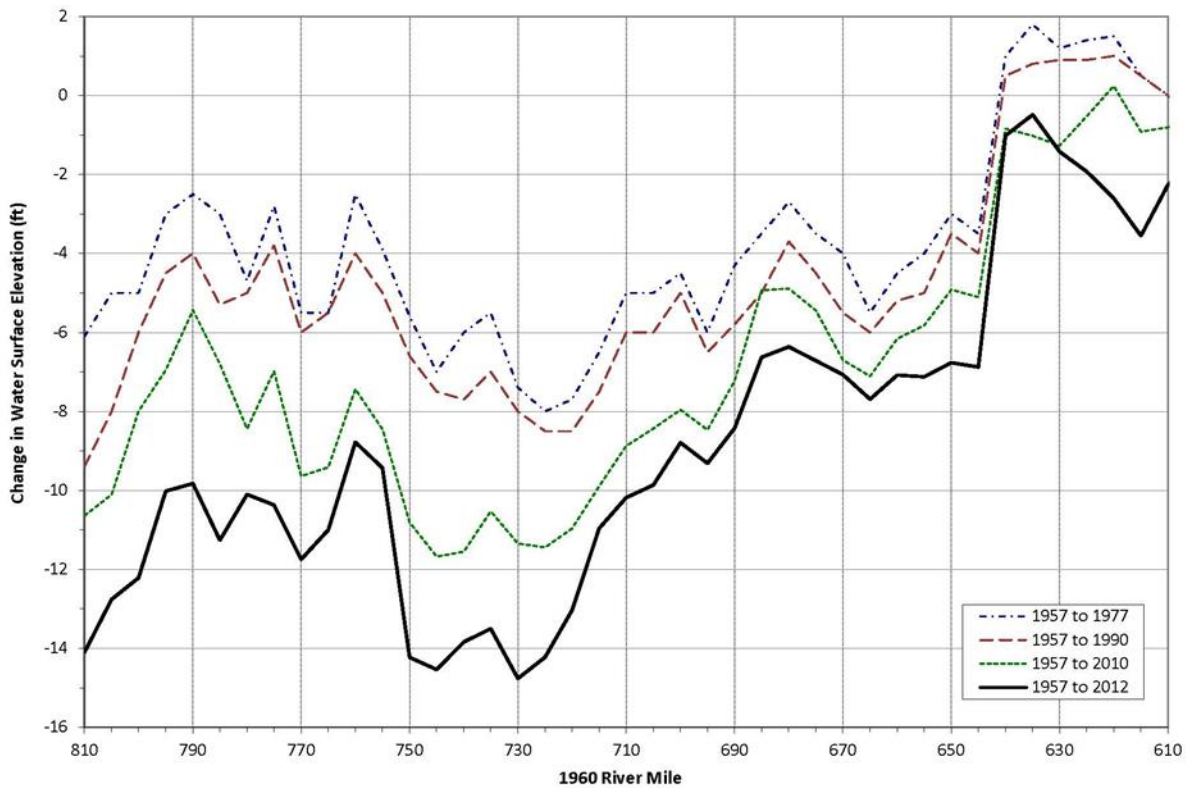


Figure 8-42. Water surface degradation downstream of Gavins Point Dam with distance from the dam; dam is located at RM 811

c. River Bed Armoring.

(1) The bed gradation often coarsens immediately downstream of a dam, a process known as river static bed armoring. A static armor layer forms in response to the altered flow and sediment regime downstream of the dam. Reservoirs release sediment-depleted flow and also attenuate flood flows. These altered flow and sediment regimes selectively erode finer particles during normal flows and small floods, leaving the coarser material behind. In river beds with significant gravel or coarser substrates, the armor layer will consist of a coarse surface layer overlying finer sediment in the substrate. Depending on local factors including bed material gradation and flows, alluvial sand-bed rivers may also exhibit minor armoring. Figure 8-43 provides two examples of bed armoring.

(2) The armor layer protects the finer materials below from excessive scour. However, an armor layer should not be regarded as nonerodable. High flows or moderately high flows of a sufficient duration can mobilize the layer. Mining activities can also remove an armor layer. If the coarse surface layer is disturbed, the underlying finer sediments, which are easily eroded, are exposed, potentially causing rapid scour. Downstream of a reservoir, high releases could mobilize an armor layer that was stable at lower flows and result in significant single-event degradation.

Figure 8-43. Examples of bed armoring
(Advanced streambank protection workshop lecture notes, USACE 2004a)

(3) Figure 8-44 plots the change in bed material size with both time and distance downstream of Gavins Point Dam. The light blue dashed line shows that before dam construction the d_{50} bed material size varied between about 0.2 and 0.5 mm, classified as sand-bed channel. The bed armored following dam closure, especially in the first 10 miles (15 km) below the dam. The d_{50} nearest to the dam increased to the 2 to 20 mm range, indicating a gravel amour layer. Further downstream, isolated samples show coarsening, which could be a remnant of the 2011 flood, but the river bed sediment sizes are generally consistent with the pre-dam sizes.

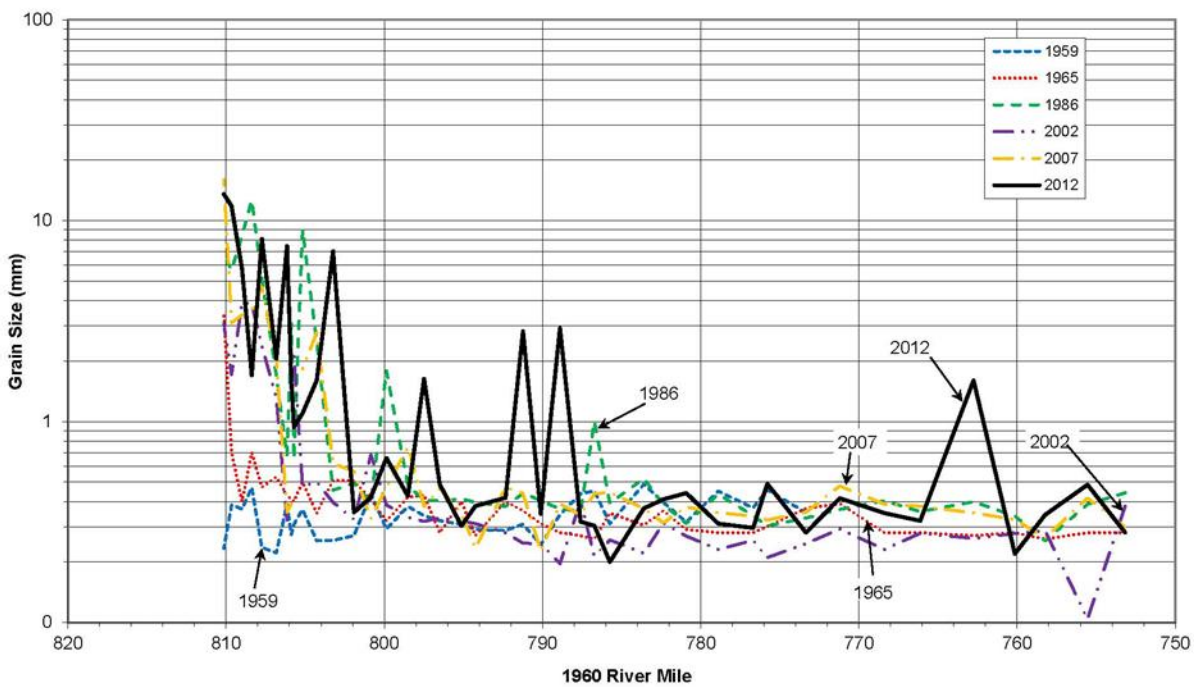


Figure 8-44. Bed material d_{50} size change downstream of Gavins Point Dam

d. Predicting the Slope of the Downstream Channel.

(1) Numerical modeling (typically 1D) is the preferred framework for assessing the bed elevation change in the downstream channel. In most channels, slope flattening and armoring occur simultaneously as the channel degrades. In addition, channel widening (see next section) and planform adjustment can be significant.

(2) Additional methods are available for simple cases. The slope method, adapted from the USBR (Pemberton and Lara 1984), assumes that the slope of the downstream channel will flatten to reach equilibrium with the upstream sediment load. This method solves a bedload or bed material load transport formula for slope, assuming zero transport. The second, the Shield's diagram method, assumes the channel degrades and armors until the grain size is sufficiently coarse that transport capacity is zero. Appendix M gives additional details on the historic methods.

e. Channel Change and Bank Erosion.

(1) Channel cross-sectional geometry change and large-scale bank erosion are common responses to an upstream dam. Flow change and relative erodibility of bed and bank material are critical factors that influence downstream channel response. Cohesive banks may have high erosion resistance and result in channel degradation if bed materials are erosive. Conversely, large bed material compared to noncohesive bank material may lead to rapid channel widening. In addition, channel bedrock/hard points may limit degradation resulting in higher bank erosion rates. Reservoir operations such as peaking (and rapid drawdown of water levels) can induce higher bank failure rates. Paragraph 7-4 provides additional information regarding geomorphic analysis.

(2) While bank erosion and channel widening are common initial responses to dam closure, channel narrowing is quite often the long-term response. Channel incision and attenuated peak flows lead to less frequent inundation of floodplain areas, reduction in the occurrence of channel bar scour, and vegetation encroachment (Petts 1979; Friedman et al., 1998). New vegetation with high-flow resistance such as willows may trap fine materials that accelerate processes to further reduce flow area. In a study of 36 large dams, Graf (2006) noted that regulated rivers had larger low-flow channels, but smaller high-flow channels and active floodplain areas compared to unregulated segments upstream of the dams.

(3) Channel changes can be evaluated from historic surveys and aerial photos. For some analyses, quantification of downstream bank erosion based on degradation range surveys or other cross-sectional data may be sufficient. Figure 8-45 illustrates the cross-section response downstream of Gavins Point Dam on the Missouri River. The cross section is located about 25 miles (40 km) below the dam. The plot illustrates cross sections at dam closure in 1955, about 25 years after closure, and then about 50 years after closure. The channel at this location continues to both widen and deepen. Analysis using repetitive surveys at multiple cross sections can be used to estimate reach wide bank erosion rates and identify rate changes over time.

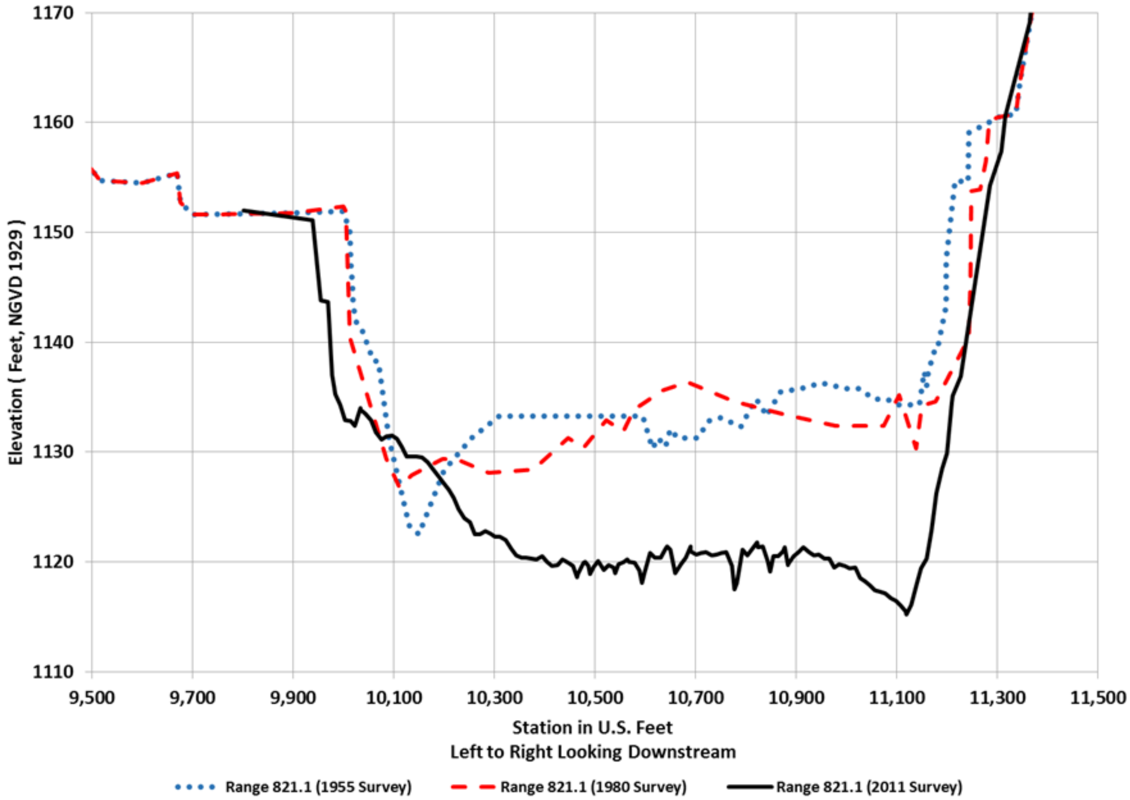


Figure 8-45. Cross-section change downstream of Gavins Point Dam, Missouri River

(4) Computational Analysis of Bank Stability.

(a) A number of published methodologies compute bank failure rates. These methodologies span a spectrum from basic angle of repose methods that require very few parameters, but simplify bank processes considerably to geotechnical slope stability models that require a full suite of geotechnical parameters yet lack a framework for hydraulic toe erosion.

(b) The Bank Stability and Toe-Erosion Model (BSTEM), developed by the National Sediment Laboratory, USDA-ARS, is a physically based model that accounts for the dominant stream bank processes, but requires an intermediate level of complexity and parameterization (HEC CPD-68B; Simon 2000; Langendoen 2008; Simon 2010). As the name suggests, BSTEM includes two major, interacting components:

- Bank Failure. A geotechnical bank failure model that computes failure planes through the bank to determine if the gravitational driving forces exceed the frictional resisting forces (and the interactions of pore water pressure). BSTEM can compute bank failure by the “Layer Method” (Simon et al., 2000), or by the “Method of Slices” shown in Figure 8-46 (from Langendoen and Simon 2008).

- Toe Scour. A toe erosion model that computes the progression of bank undercutting by hydraulic forces. As the toe scours, the bank becomes less stable, so toe scour can initiate bank failure.

(c) A BSTEM analysis requires channel geometry, hydraulic parameters, geotechnical parameters (friction angle and cohesion) erodibility parameters (critical shear stress and erodibility), and water table elevation. At the time of this writing, BSTEM is available as a stand-alone spreadsheet, as a module available within HEC-RAS version 5.0 (HEC 2016c) and more recent versions (Gibson et al., 2015), and in Sedimentation and River Hydraulics – Two-Dimension (SRH-2D) (Lai et al., 2012).

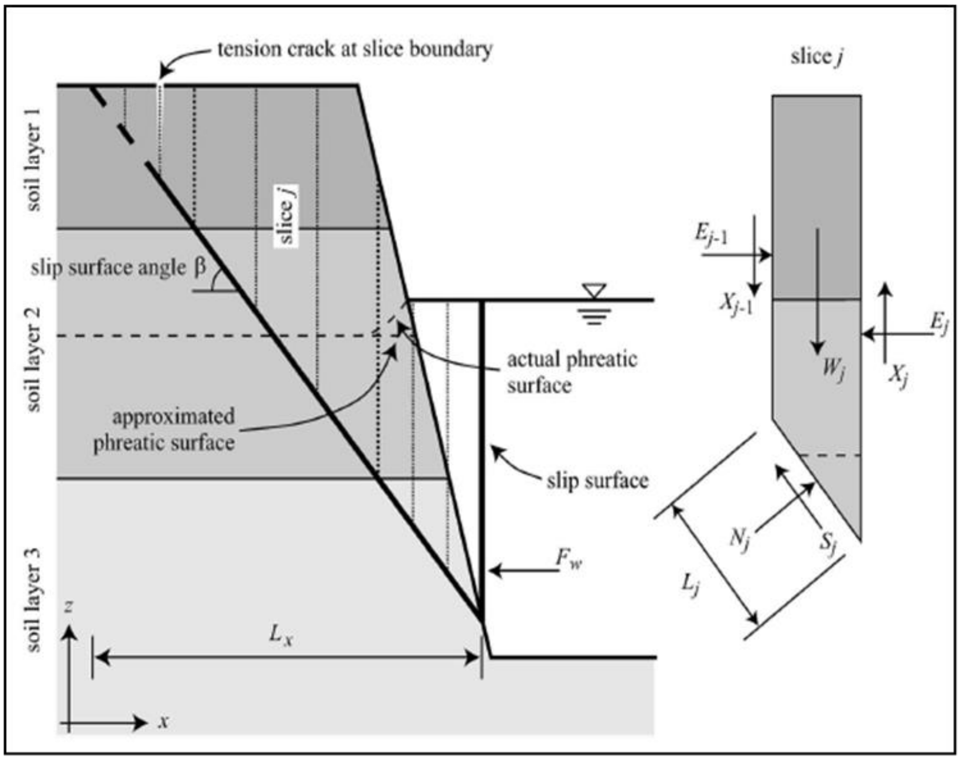


Figure 8-46. The Method of Slices option in the USDA-ARS BSTEM (HEC 2016c)

f. Aggradation. While degradation is the typical response to dam construction, the effects of flow regulation may induce aggradation due to incoming sediment load deposition that more than offsets the degradation effect of upstream reservoir sediment trapping. Numerical modeling is recommended where these effects are important.

(1) Downstream of Fort Randall Dam, the attenuated Missouri River flow and high sediment load of the Niobrara River results in aggradation at the confluence. In this sense, tributary fans can serve as a form of grade control on a mainstem downstream channel.

(2) Xia and Liu (2003) state that operating the Naodehai reservoir to sluice sediments has enabled the reservoir to maintain 80% of its original capacity after 60 years of operation. However, sediment sluicing has resulted in downstream channel deposition and a decrease in the channel capacity.

g. Types of Downstream Impacts.

(1) The physical processes described in the previous section have the potential to induce infrastructure and environmental damage downstream of a dam. This section provides examples of infrastructure and environmental impacts that have been observed downstream of major reservoir projects or related to the processes that occur downstream of major reservoir projects.

(2) Methods for evaluating the impact of these sedimentation processes differ by feature type. For example, evaluating the effect of bed degradation on levee stability requires an estimate of bed degradation plus a geotechnical analysis. An assessment of the ecological effects of decreased channel/floodplain interaction requires estimates for bed and water surface degradation, hydrologic analysis on the frequency of inundation, and ecological analysis. The analysis team will most likely be interdisciplinary. A technically competent sedimentation engineer should perform the sedimentation analysis. Sediment management at reservoir projects that pass sediment to the downstream channel can help alleviate some of these impacts.

(3) Bed Degradation and Channel Incision Impacts. As previously explained, sediment trapping in reservoirs results in bed degradation downstream. Infrastructure/environmental impacts that have been observed with bed degradation include the following. Note that these examples relate to bed degradation that can occur due to many causes, not just due to sediment trapping in dams:

(a) Greater risk of flood scour-related revetment failure resulting in emergency maintenance (USACE 1996) (see Figure 8-47).

(b) Increased maintenance and risk of failure of bridge piers (Kondolf and Swanson 1993; Thair 1990; Lauchlin and Melville 2002) (see Figure 8-48).

(c) Levee or floodwall slope instability (USACE 2009).

(d) Exposure of utility lines.

(e) Decreased frequency of channel or floodplain interaction and other floodplain effects.

(f) Increased bank erosion and failures.

(g) Loss of habitat diversity.

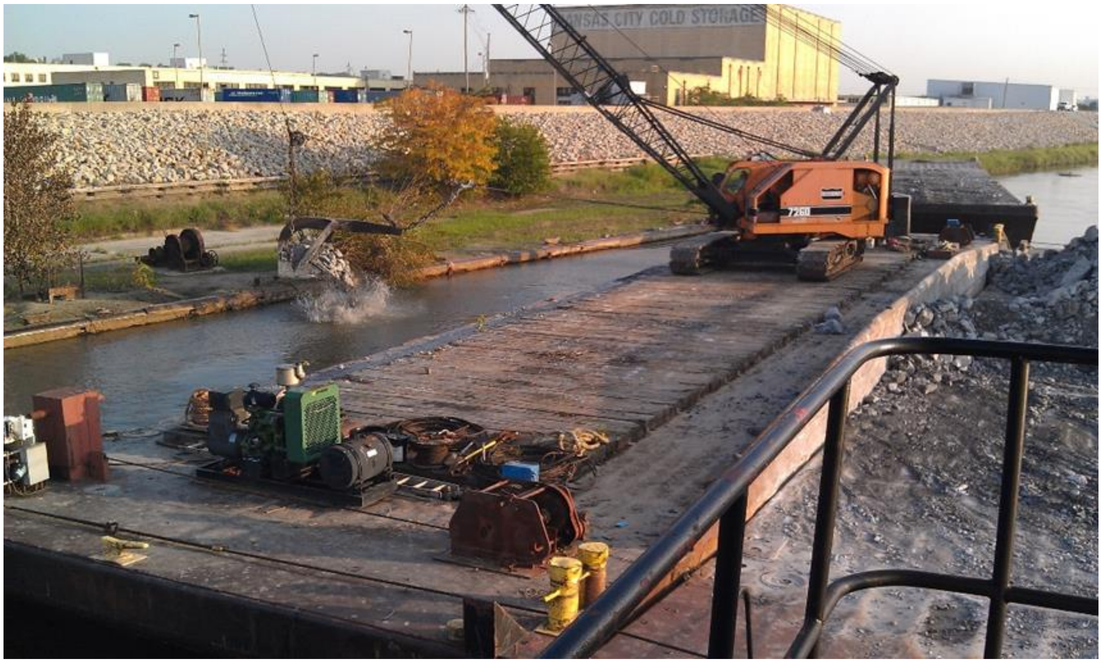


Figure 8-47. Emergency rock placement to protect a levee and floodwall slope in a degraded reach of the Missouri River at Kansas City

Figure 8-48. Degradation threatening bridge piers at Line Creek, Missouri

(4) Impacts of Lower Water Surface Elevations. When the river bed degrades, water surface elevations drop. However, changes in channel width, slope, or roughness also affect the water surface elevation, so the stage-discharge rating curve may shift. Therefore, the water surface elevation may not decrease at the same rate as the bed. The following impacts have been observed from water surface elevation decrease:

(a) Water levels drop below supply or power supply water intakes (USACE 2009).

(b) Abandoned boat docks.

(c) Decreased water depth and frequency of overtopping for river training structures, requiring structure modifications.

(d) Decreased frequency and depth of overtopping for water control structures (weirs, etc.) requiring structure modifications.

(e) Conversion of aquatic habitat to terrestrial habitat; draining of riparian wetlands; lowered groundwater table.

(f) Increased groundwater pumping costs (USACE 2009).

(5) Ecologic Impacts. As dams control flow and sediment, post-construction sediment process changes may impact many critical habitats located in vulnerable areas. Habitats are impacted by the changed flow-sediment balance and also by special operation practices such as sediment flushing, hydropower peaking, and similar activities. Bank erosion/deposition may impact critical bank and nearby riparian habitats. Other impacts include losing sediment-dependent wetlands, changing water levels that alters habitats of sensitive species, reducing nutrient and sediment inputs to riparian areas, losing floodplain connectivity, and losing ocean delta that had been replenished by river sediments. Examples of ecologic impacts are provided in the following sections.

(a) Vegetation Impacts. When scouring of the downstream channel occurs, dams tend to cause more stationary sediment deposits at medial bars and downstream tributary junctions. Regulated flows may have less scouring of established vegetation which limits bare soil locations for new seed recruitment. When the regulated flow regime no longer has the capacity to transport sediments delivered by tributaries, sediment deposits can also impact ecosystems (bury spawning gravel, increase channel uniformity, etc.). If the base flows are increased, the available stream energy at low-flow periods also increases. The increase in base flows may encourage vegetation encroachment, increasing the stability of medial bars, further reinforcing a positive feedback loop on vegetation establishment.

(b) Impacts from Decreased Turbidity. Clear water releases, caused by the trapping of fine sediments, can negatively impact species dependent on high turbidity.

- In the Colorado River downstream of Glen Canyon Dam, Humpback Chub numbers have decreased substantially, and they are now federally protected. One primary reason is the turbidity decrease in the Colorado River. Pre-dam the river was over 1,000 FNU (Formazin Nephelometric Unit, a measurement of turbidity). After construction of Glen Canyon Dam, the river is usually below 50 FNU. The small chub became easy prey for trout species in clear water. Figure 8-49 illustrates the change in turbidity from 1,000 FNU to 50.

- A decrease in turbidity due to USACE dam construction in the Kansas River basin has led to 70% reduction in turbidity (National Research Council 2011) and a decline or extirpation of native, turbidity-dependent species such as shoal chub, plains minnow, flathead chub, river shiner, carmine shiner, silver chub, sturgeon chub, and western silvery minnow (Gido et al., 2010; Shelley et al., 2016). These species have been replaced by fish more dependent on sight feeding (Bonner and Wilde 2002).

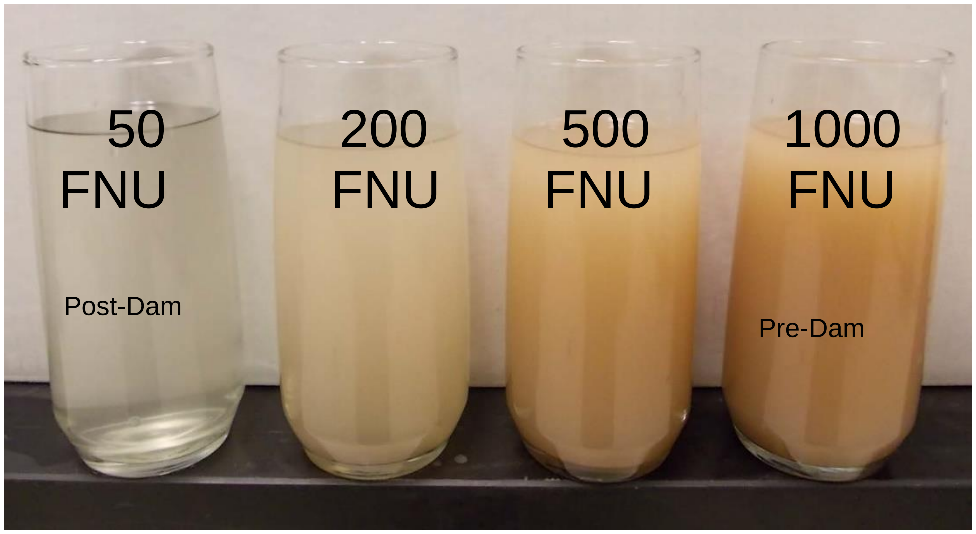


Figure 8-49. Colorado River turbidity downstream of Glen Canyon Dam decrease from over 1,000 FNU to 50 FNU (Ward et al., 2016)

(c) Reservoir sediment trapping can reduce or eliminate coarse substrate supply that maintains riffles and other important habitat in the downstream channel. The loss of coarse substrate can lead to dramatic population decline in salmon and other fish species that require coarse substrate for spawning.

- Modified flow regimes can lead to sand deposition downstream of tributaries below dams, where the river lacks the capacity to transport the load. Tributary deposits can fill or bury coarse substrates required for substrate spawners.

- Lack of coarse sediment input on the Lower Yuba River downstream from Englebright dam reduced the availability of spawning habitat for the Central Valley steelhead and the spring-run Chinook salmon, requiring gravel augmentation (USACE 2010).

- Similar impacts to threatened and endangered salmonids on the Green River in Washington State below Howard Hanson Dam caused USACE to implement annual gravel augmentation in 2003, which has reversed some of the downstream armoring and increased available spawning habitat (Corum et al., 2022).

(6) Abutment Scour. The potential of local scour exists at the dam structure itself. Abutments are the weakest zone and should be designed to either prevent flow from short-circuiting the overbanks and cascading down the tie between the structure and the channel bankline, or to accommodate such a flow path. Abutment scour likely contributes minor sediment volume to the downstream channel. However, abutment scour can lead to the release of additional downstream flows and a potential dam safety issue.

(7) Emergency Spillway. Emergency spillways are usually designed for infrequent use, often unlined, and may be susceptible to high scour rates during operation. Spillway erosion in the vicinity of the dam embankment can be a serious dam safety issue. Impacts can be severe with loss of pool or even dam failure due to excessive erosion. At Oroville Dam, located on the Feather River in California, continued operation in 2017 of the primary spillway, even though damaged and eroding, was necessary to lower pool elevations since the emergency spillway was also eroding and threatening a catastrophic failure. Erosion should be analyzed seriously, particularly when failure consequences are high. More information on spillway erosion analysis can be found in paragraph 8-15.

(8) Changes in Downstream Channel Capacity. Bed degradation may increase the downstream channel capacity, which could have positive effects for flood conveyance. However, in rivers with high sediment loads, land accretion may accompany degradation and offset any flood conveyance benefits. In a study of 36 large dams, Graf (2006) notes an increase in the size of the low-flow channel size and a decrease in the size of the high-flow channel and active floodplain (Graf 2006) in rivers below dams compared to unregulated segments upstream. This could translate into a lowering of the low-flow water surface and an increase in the high-flow water surface for equivalent discharges.

(9) Reservoir Sediment Management Operations. Reservoir operations for sediment management (described in more detail in paragraph 8-13) can alter downstream channel sediment processes. During reservoir sluicing operations, the sediment transport rates are roughly equal to those of natural flows entering the reservoir. During flushing operations, the rates are equal or higher than those of natural flows. As a result of these operations, increased sediment levels are likely to be above the sediment transport capacity of the river.

(a) Deposition in the downstream channel can affect infrastructure, water intake operations, flood control, etc. Figure 8-50 shows flushing effects at Spencer Dam on the Niobrara River in 2014 with increased turbidity and downstream sandbar formation.

Figure 8-50. Spencer Dam flushing release (a) and sandbar developed downstream (b)

(b) The effects of flushing operations can continue a substantial distance downstream. Figure 8-51 illustrates the sediment plume from Spencer Dam flushing at the Niobrara and Missouri River confluence, located approximately 40 river miles downstream from the dam.

Figure 8-51. Niobrara River sediment plume entering Missouri River

8-7. Dam Removal.

a. Federal, state, and local governments, as well as private entities and nonprofits, have removed hundreds of dams in the last 20 years (Service 2011), and the removal rate is steadily increasing (USBR 2017, Figure 8-52). Dam removal often has ecological and habitat benefits but is usually driven by the liability associated with old and obsolete dams (Born et al., 1998). Most dams removed in the United States have been small with a height less than 15 feet, and in northeastern and midwestern states (Service 2011). However, several large dams (such as

Marmot, Elwha) have been removed, and USACE is involved in several other large dam removal studies.

b. Key concepts and detailed guidance regarding sedimentation dynamics that occur post-dam removal are available from ASCE (2011) and the Advisory Committee on Water Information (ACWI), Subcommittee on Sedimentation (USBR 2017).

c. Dam removal studies involve a multi-disciplinary team to adequately address objectives and potential issues. The information provided in this document primarily pertains to sediment impact analysis.

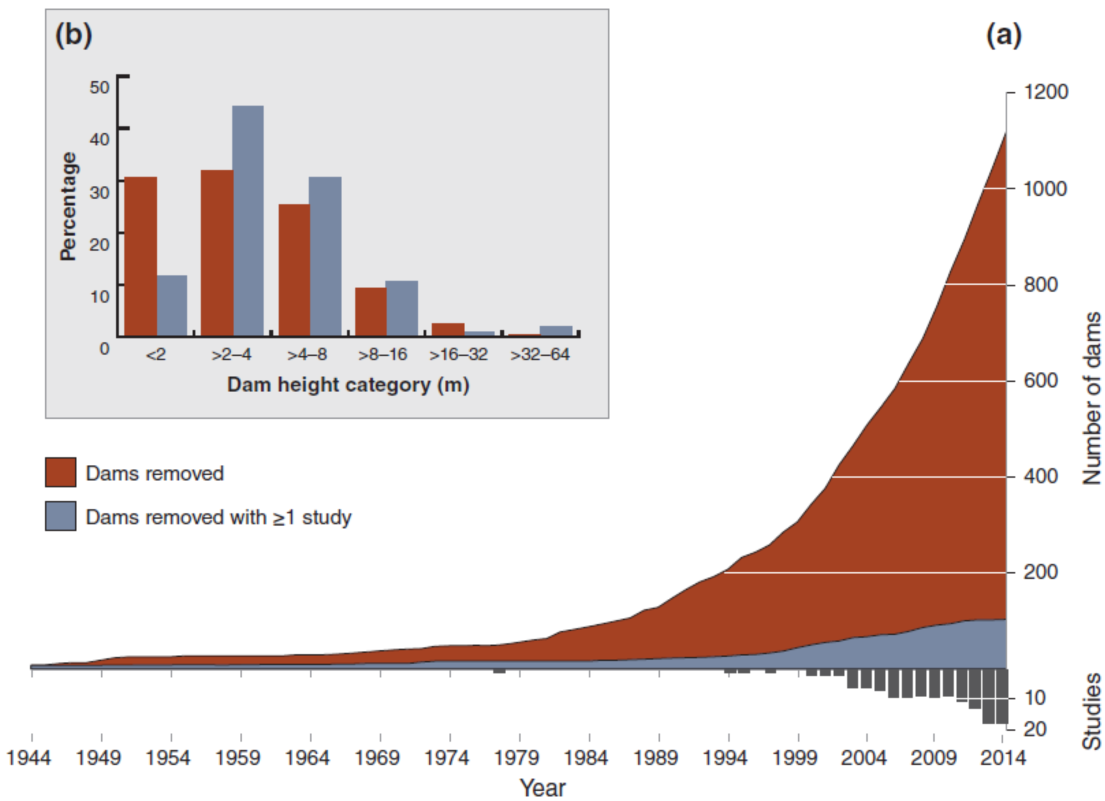


Figure 8-52. Compilation of dams removed (USBR 2017)

Compilation of dams with at least one published study on the physical or ecological response to dam removal (a) by dam height and (b) by the cumulative number of dams removed by year (Bellmore et al., 2017; data from Bellmore et al., 2015; and American Rivers 2024)

d. Dam removal analysis should address the downstream effects of released reservoir sediment. Critical impacts include flood risk and ecological impacts that are associated with sediment deposition and increased turbidity.

(1) Removing deposited reservoir sediments is often prohibitively expensive and reservoir deposits may be left in the channel. Therefore, there is a critical need to predict the downstream effects of the sediment release following dam removal.

(2) Technical challenges with evaluating dam removal include determining the safe and efficient dam removal method and rate, managing stream flow during dam removal, providing any required fish passage, estimating the extent of the dam removal and related facilities that should be removed to achieve objectives and conform with applicable dam removal policy and regulations, managing reservoir sediment, and developing adaptive criteria to deal with the uncertain and changing conditions that will occur during and shortly after the dam removal (USBR 2017).

(3) The risk of downstream sediment impacts depends on the amount and rate of reservoir sediment erosion, transport capacity of the downstream channel, and proximity to the dam site (Randle et al., 2015). Dam removal analysis will generally involve the application of conceptual, numerical, and physical models. Figure 8-53 and Figure 8-54 show a longitudinal profile and aerial view of the Matilija Dam on the Ventura River in California (USBR 2006b). Originally constructed in the 1940s with a capacity of 7,018 acre-feet, the dam capacity in 2012 was only 500 acre-feet and was projected to reach zero capacity by 2020. A principal focus of the Matilija Dam Ecosystem Restoration Project between the Ventura County Watershed Protection District, USBR (2006b), and USACE (2004) was how to manage almost 7 million cubic yards of fine and coarse sediment that had accumulated behind the dam since its construction (Ventura County 2016). Sediment management is usually the critical component of dam removal studies and the most common reason these projects are abandoned or postponed.

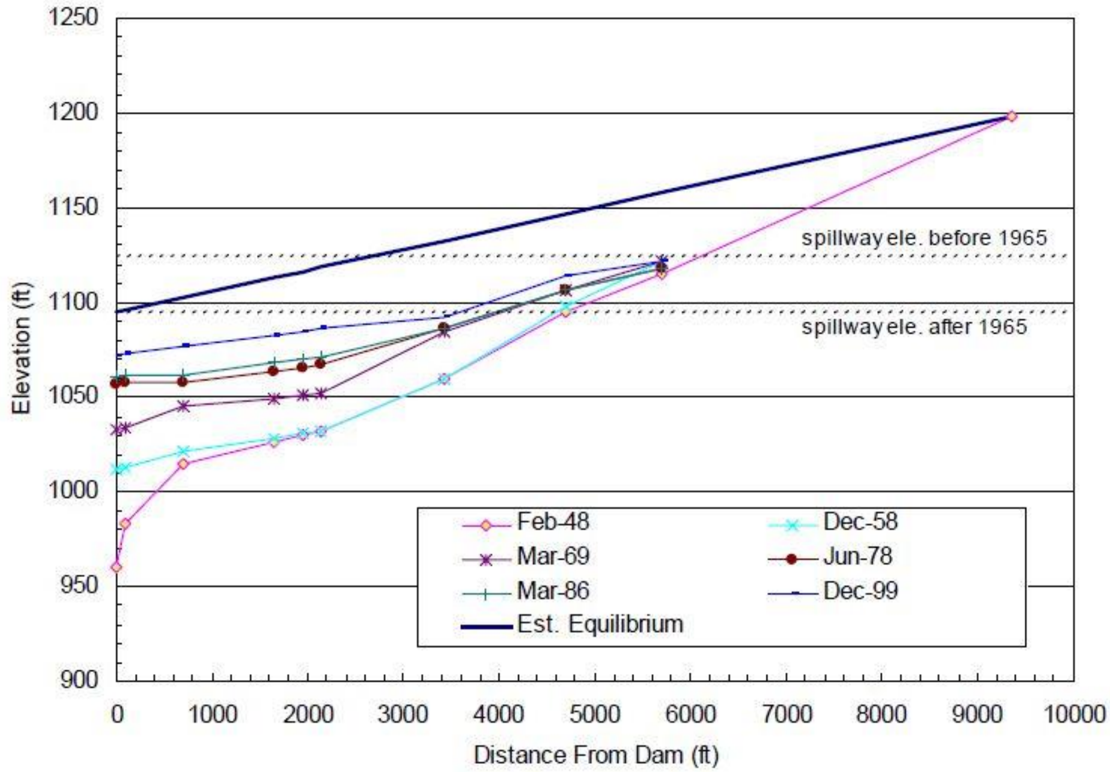


Figure 8-53. Matilija Dam profile of deposited sediments (USBR 2006b)

Figure 8-54. Sediment trapped behind Matilija Dam during drawdown July 2003 (USBR 2006b)

e. Between 2011 and 2014, the U.S. Department of Interior removed the Elwha and Glines Canyon dams on the Elwha River in northwest Washington State. Removal was conducted to restore fish passage, honor federal trust responsibilities to the Lower Elwha Klallam Tribe, and connect the downstream river to the pristine upstream watershed in Olympic National Park in Washington State.

(1) After nearly two decades of planning, river erosion was selected as the only economically viable option and phased dam removal was utilized over three years to manage the large sediment release volumes. An estimated 27 million cubic yards of sediment was impounded behind the dams before their demolition.

(2) Concurrent dam removal began in September 2011. The removal of Elwha Dam (Lake Aldwell) was completed in one year (by April 2012) while the removal of the upstream Glines Canyon Dam (Lake Mills) was completed in three years (August 2014). As of September 2016, 72% of the sediment had been eroded from Lake Mills and 50% had eroded from Lake Aldwell.

(3) Sediment concentrations were high during dam removal with peak concentrations reaching 10,000 mg/l. Nearly every downstream river pool was temporarily filled with sediment. New gravel bars formed along the channel that had been absent while the dams were in place, inducing bank erosion and increased river sinuosity in unconfined alluvial reaches. Flood stage for the 2- to 10-year floods increased by about 2 feet (0.6 m).

(4) Despite the deposition in the downstream channel, about 90% of the sediment eroded from the reservoirs was transported to the coastal estuary and enlarged the coastal delta 1,500 feet (460 m) into the sea.

(5) The Elwha River dam removal project (Figure 8-55) required several infrastructure upgrades associated with sediment releases to address aggradation impact on flood risk, the temporary degradation of water quality, detailed monitoring during the removal phase, and controlled notching of the dams informed by observed reservoir sediment erosion rates. (USBR 2017).

Figure 8-55. Lake Aldwell Delta upstream from Elwha Dam prior to removal in 2011 (left), during dam removal in 2012 (center), and after dam removal in 2015 (right) (USBR 2017)

(6) Reservoir delta deposit erosion processes vary according to many factors. Figure 8-56 illustrates how erosion widths can be quite wide through coarse sediment that lack cohesion as was the case for the upper layer of delta in Lake Mills behind Glines Canyon Dam (USBR 2017).

(7) Figure 8-57 shows the sediment plume at the mouth of the Elwha River in Northwest Washington State during removal of Glines Canyon and Elwha Dams in 2012. Dam removal water quality mitigation included cash settlements to affected landowners, relocation of fish hatcheries, and construction of a new river intake and water treatment plants designed to filter out very high sediment concentrations. Although sediment was allowed to flow into the Strait of San Juan de Fuca, a nearby water treatment plant required repairs from the clogging of intakes by debris and fine organic matter released from the reservoir bottom. The clogging resulted in frequent plant shutdowns and impacts to the dam removal schedule.

Figure 8-56. Looking upstream at extensive lateral erosion of Lake Mills Delta (upper coarse layer) near Port Angles, Washington (USBR 2017)

Figure 8-57. Mouth of Elwha River, April 14, 2012 (Photo John Felis, USGS) (https://www.usgs.gov/media/images/turbid-coastal-plume-elwha-river-washington-1)

f. The sediment-related impacts associated with dam decommissioning can occur in the reservoir, in the river channel, both upstream and downstream from the reservoir, and can affect reservoirs lower in the watershed. Depending on the local conditions and the decommissioning alternative, the degree of sediment impact can range from negligible to very large (Randle et al., 2010). The Interagency Subcommittee on Sedimentation, composed of representatives from 11 federal agencies, universities, and professional agencies, recommended the following steps before proceeding with dam removal:

(1) Reconnaissance of dam history, watershed context, and sediment concerns.

(2) Characterization of reservoir sediment deposits.

(3) Identification of contaminant concerns.

(4) Determination of the amount of coarse and fine reservoir sediment volumes.

(5) Selection of an initial dam removal and sediment management plan.

(6) Evaluation of reservoir and downstream sediment impacts (including impacts to reservoirs lower down in the watershed).

(7) Assessment of confidence, impact probability, and risk.

(8) Development of a monitoring and adaptive management plan.

g. The interagency report (Randle et al., 2010) addressed the level of sediment analysis appropriate for different dam removal conditions. Detailed sediment modeling may be too costly and unnecessary for small mill dam removal, but critical for larger removals with substantial potential downstream impacts like the Elwha and Matilija. Randle and Bountry (2015) provided guidance to choose the appropriate level of analysis based on the ratio of the sediment stored in the reservoir to the average annual sediment load as shown in Figure 8-58. Note that Figure 8-56 assumes noncontaminated sediments; finding contaminated sediments may significantly impact risk.

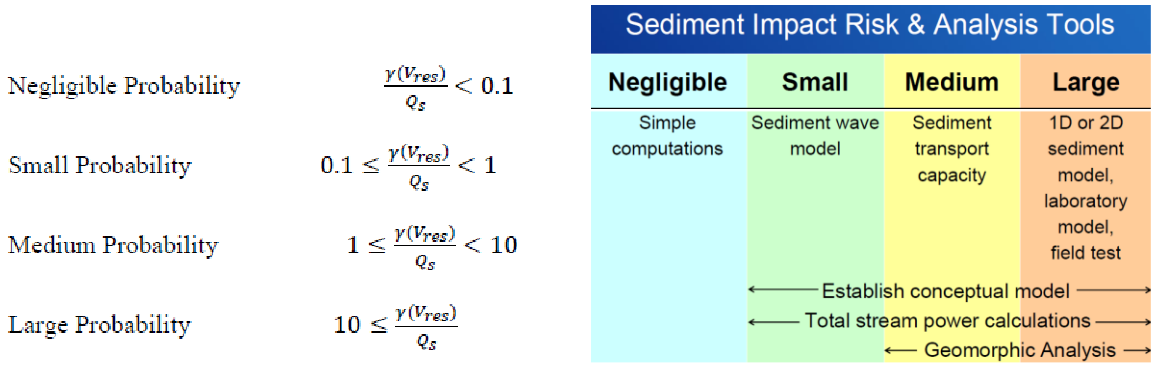


Figure 8-58. Criteria for impact probability (redrawn from Randle and Bountry 2015) (function of the ratio of reservoir sediment mass (γ*V_{res}) to the annual sediment load (Q_s) in tons per year and the recommended analysis based on these risk categories)

h. The Dam Removal Analysis Guidelines for Sediment (USBR 2017) suggest an iterative analysis approach, starting with readily available information and revisiting or repeating analysis steps as more data become available. The guidelines identify a series of 10 steps:

(1) Identify sediment concerns and benefits.

(2) Collect reservoir and river data.

(3) Evaluate potential for contaminated sediment.

(4) Determine relative reservoir sediment volume and probability of impact.

(5) Refine potential sediment consequences and estimate risk.

(6) Develop dam removal and sediment management alternative.

(7) Conduct sediment analysis based on risk.

(8) Assess uncertainty of predictions.

(9) Determine if sediment impacts are tolerable and, if needed, modify sediment management plan.

(10) Develop a monitoring and adaptive management plan.

i. Dam removal in northern regions should also consider the impact on the river ice regime. Dam removal may have significant impacts on the ice regime and result in increased frequency and severity of downstream jams. Lowering of water levels in impoundments containing sediment deposits may result in more frequent or longer duration ice-induced scour and erosion of bed and bank material (White and Moore 2002).

j. To geographically search for case studies of completed dam removals that may provide relevant information for a particular study area, see the USGS Dam Removal Information Portal: https://www.sciencebase.gov/drip/.

8-8. Reservoir Sediment Observations. The combination of reservoir surveys and careful observation can provide insight into historical and possible future reservoir changes.

a. Field Reconnaissance/Procedures for Reservoir Sedimentation. Before any reservoir site visit, an investigation of data readily available in the office should be conducted. Knowledge of historical, hydraulic, and sediment parameters will make the field investigation easier and more efficient. Field reconnaissance is often in response to site-specific issues. Appendix D includes a check list of typical tasks and observations to be made during the field reconnaissance that pertain to typical field reconnaissance and study objectives. Adapt the checklist as needed to meet site-specific project objectives.

b. Sediment Observation Methods. Field methods for sediment observations can be informative and often performed quickly and efficiently.

(1) Test Pits. Test pits are open excavations performed with large mechanical equipment such as an excavator. Multiple pits can be opened in a delta to provide a means for visual observation of temporal sediment variation within reservoir deposition areas. Test pits can be cost effective compared to sediment coring, but require dry conditions for operation.

(2) Check Ranges. Check ranges can be installed and frequently surveyed in critical areas to quickly assess sediment conditions. Survey frequency should be sufficient to establish long-term rates, specific event response, and other critical items such as variability to upstream sediment source and seasonal variation.

(3) Reservoir Profile. Reservoir bed elevation profiles can be collected to track the progression of sediment deposition in the reservoir pool. Temporal profiles are an efficient method to long-term delta progression rates and response to specific events. Due to the possibility of multiple thalwegs through a delta, a reservoir profile may need to be collected at multiple flow paths for comparison.

(4) Deposit Thickness over Event Horizon. Lacking reliable original bathymetric data, sediment thickness over a dateable horizon can also be used to determine sedimentation rate. Cesium 137 is a manmade isotope created by atmospheric testing of thermonuclear devices that can be used to determine sedimentation depths overlying this event horizon both in reservoirs and in natural lakes (Morris et al., 2008). Horizon-dating methods have important limitations including mobilization and reworking of sediments during operation, as well as generally uneven levels of sediment deposition.

(5) Sediment Coring. Sediment cores may be collected and analyzed for size gradation, contaminant and nutrient levels, critical shear stress, erodibility, etc. These measurements parameterize numerical models.

8-9. Reservoir Capacity and Sediment Surveys. Sediment surveys are a critical aspect of evaluating current and future operational issues for existing reservoirs and determining needed information for new projects. Implementation of a comprehensive survey program requires a thorough evaluation of objectives, methods, and available resources. Reservoir capacity is typically performed by pool zone. (Figure 8-31 previously showed typical USACE terms for the various pool levels.)

a. Objectives. Reservoir sedimentation investigations should be planned to achieve the following basic objectives:

(1) Functional Objectives. Recognizing that each reservoir site is a limited national resource and that a reservoir sedimentation investigation program is an expensive endeavor, consider the functional objectives of the nationwide effort when planning the survey program. Take into account the probable nature and magnitude of existing reservoir sediment problems, anticipated future issues, and the proximity of other reservoir projects in which similar investigations are in progress. Determine functional objectives in terms of benefits to the specific project, benefits to other USACE projects, benefits to the watershed, and benefits to the national investment in the development of water resources.

(2) Operational Objectives. Evaluate the reservoir survey needs for existing projects in terms on ongoing and projected issues. In all cases, provide a minimum recommended survey program to answer questions associated with reservoir operation. Such needs will vary from project to project, but in general they include the following:

(a) Criteria for the construction and operation of boat docks, recreational facilities, and other structures within the reservoir limits must be initially determined and updated to include current sedimentation knowledge.

(b) If sediment yield is large in proportion to storage capacities for multi-purpose reservoirs, sediment deposition characterization will be needed for planning storage reallocation and revising reservoir regulation rules to ensure optimal use of remaining reservoir storage space.

(c) Make accurate estimates of the current reservoir storage capacity. Advancement in data collection technology has resulted in the ability to enhance capacity estimates.

(d) Use repetitive surveys to determine storage capacity depletion rates that are needed for forecasting future storage. Surveys are needed far enough in advance to permit the evaluation of reservoir management options for possible implementation. If sediment management is not feasible, then consider future operation scenarios such as decommissioning older facilities and the planning/construction of replacement facilities.

(e) Acquire and maintain data for evaluation related to monitoring or modifying intakes, outlets, water supply intakes, and other facilities adversely affected by sediment accumulations in the reservoir.

(f) Acquire and maintain data for tracking rates of reservoir bank erosion.

(g) In many instances, accurate information on the effects of sedimentation on water surface profiles has been used to settle legal claims arising from the operation of the dam. This includes sedimentation effects on the mainstem and tributaries.

(h) Surveys for observing channel changes downstream from the dam are needed to evaluate project relationship to bed and bank instability.

(i) Perform surveys and evaluations to assess changes in river water levels that result from channel degradation and which impact hydropower head or tailwater rating curves that affect project energy dissipation structures.

(j) Take sediment surveys, which are valuable for determining impacts to physical habitats for flora and fauna.

(3) Planning and Design Objectives. Sedimentation investigations established to meet operational objectives will also provide information useful for planning and design of sediment management options and possible future reservoir projects. However, in many cases, the baseline or original reservoir survey may be inadequate for new evaluations to address current issues. It is likely that additional site-specific surveys, more frequent resurveys, and a substantially more intensive analyses effort will be required to meet planning and design objectives:

(a) Sediment surveys that satisfy comprehensive objectives should be provided in regions where planning and design are the most active, where sediment-related issues are occurring, and where needs are not being satisfied with data from existing investigations.

(b) Information needed on the planning and design of a future reservoir involves practically every phase of reservoir sediment investigations (sediment deposition location, volume depletion rates, predicted sediment distribution during project life, probable channel changes downstream resulting from reservoir-induced retrogression, etc.).

(c) Field data are critical for projects located where sediment problems are known to be severe.

(d) It is important to establish the forecasted severity of sediment problems. For example, demonstrating in project reports that serious sedimentation problems will not occur is as important as forecasting the magnitude of the problem. Therefore, plan sediment investigation programs that will provide the necessary information. The field data collected from these programs are sufficiently comprehensive to meet nationwide needs.

(e) Survey methodology has evolved considerably since most USACE projects were constructed. A thorough assessment of project accuracy, cost, and historic survey accuracy should be performed to ensure that project objectives are met. It is suggested that when implementing a new survey methodology, the old methodology also be used to understand the differences in collected information (paragraph 8-9h).

(4) Timeliness and Location of Information. The sediment investigation should consider the need to provide timely data to evaluate ongoing issues, establish trends, and monitor critical areas. Partial surveys or expansion of a previously conducted investigation may be necessary to address current and future critical sediment issue areas.

b. Reservoir Capacity Surveys. Following the sediment survey objectives, reservoir surveys are primarily conducted for two purposes: (1) to provide an accurate assessment of current reservoir capacity, and (2) to compute capacity depletion rates by comparing current capacity to previous capacity estimates. Limited-extent surveys may also be conducted to provide data required for site-specific analysis.

(1) Survey Methodology. Complete reservoir survey methods are typically classified as either contour or sediment range (Morris and Fan 1998; USBR 2006a). Reservoir sedimentation surveys require a combination of hydrographic and topographic survey methods. Hydrographic surveys are performed to determine the underwater topography. Topographic methods are performed to map the areas above the pool. While general terms are discussed regarding reservoir surveys in this document, USACE has several guidance documents related to survey techniques and accuracy that should be reviewed before determining survey methodology. Primary references are as follows:

(a) EM 1110-1-1000, Photogrammetric and LiDAR Mapping.

(b) EM 1110-1-1002, Survey Markers and Monumentation.

(c) EM 1110-1-1003, NAVSTAR Global Positioning System Surveying.

(d) EM 1110-1-1004, Geodetic and Control Surveying.

(e) EM 1110-1-1005, Control and Topographic Surveying.

(f) EM 1110-2-1003, Hydrographic Surveying.

(2) Survey Types and Methods. Reservoir survey methods employed today have evolved significantly since the principal USACE dam construction era from 1950 through 1980. Survey technological advancements have generally increased data accuracy, precision, and data density, and have provided variable cost savings dependent on many site-specific factors. Remote sensing capabilities continue to evolve with new techniques that may become standard in the future:

(a) Rapid technological advancement and the use of new survey techniques will likely continue in the future.

(b) As survey technology has advanced, data use should recognize the variability in both coverage and accuracy when performing evaluation that relies on data comparison and trends.

(c) Practical considerations often result in the use of topographic survey data available from non-USACE sources. Before using this data for engineering evaluation studies such as reservoir capacity and depletion trends, an assessment of data accuracy and suitability is required.

(d) Consider survey objectives when selecting the appropriate method, extent, and accuracy.

(e) Water levels are a critical factor in survey cost. Dry dams or reservoirs with fluctuating water levels may be effectively surveyed via remote sensing methods. Swamp or marshy areas of broad expanse with variable shallow depth can require significant resources with specialized access collection equipment.

(f) Selected survey methods and density should consider known reservoir construction projects that included substantial grading. For instance, large-scale environmental enhancement projects that include bank reshaping and constructing habitat structures in the reservoir pool can significantly impair comparison to historic surveys. Increased hydrographic survey density in these areas may be necessary to reflect the topographic change.

(g) Topographic. Topographic survey capabilities have evolved significantly since most USACE projects were construction in the 1950s and 1960s. Survey accuracy from different eras can impact computations, especially when trends and rates are determined. Refer to available topographic survey guidance for additional information regarding past and current survey capabilities (USACE 2007, 2015d).

- For reservoirs constructed before about 1980, reservoir land topography was likely based on surveyed sediment ranges using transit and plane table surveys. These methods continued in USACE into the 1980s, followed by a transition to optical total station methods (USACE 2007). This later progressed to electronic total station over the next decade. Land survey capability further progressed to using Global Positioning System (GPS) technology since the late 1990s.

- For most of the past century, aerial photogrammetry has been used for stereo-compilation of 2D planimetric maps and 3D topographic maps including contours. Since the early 1990s, digital orthophotos have become the most popular product of aerial photogrammetry.

- Topographic mapping using remote mass point collection and GIS terrain modeling has progressed from using aerial photogrammetry methods to LiDAR technology. LiDAR is a remote sensing method that uses light in the form of a pulsed laser to measure distances to the Earth. After a transition period starting in the late 1990s, LiDAR data collection has emerged as the most popular and accurate technology for all forms of digital elevation modeling.

(h) Bathymetric.

- Until the 1960s, the primary method in USACE for measuring water depths used manual techniques. Methods included lead lines, sounding poles, and topographic surveys (levels and transits). Depth measurements were positioned using a variety of visual survey methods such as transits, sextants, and tag lines.

- These manual and visual survey methods were gradually superseded when acoustic depth measurements (echo sounding) were implemented in the 1950s and GPS positioning in the 1990s. Single-beam acoustic system equipment has evolved significantly in capability and accuracy since the original echo sounders.

- In the 1970s, multiple transducer sweep systems were developed employing multiple single-beam transducers on a boom array. Acoustic multibeam systems evolved in the early 1990s (USACE 2013a). Other improvements such as GPS positioning equipment and vessel motion correction equipment (pitch, roll, etc.) have also greatly enhanced hydrographic survey data collection capability and accuracy. Refer to available topographic survey guidance for additional information regarding past and current hydrographic survey capabilities (USACE 2013a).

- Multi-beam surveys at the micro-project scale can illustrate individual structure features. A limitation of multi-beam data collection is the high data density and accompanying computer hardware and software needs that may be problematic in large surface area lakes. Shallow depths also limit the data collection swath width. Tightly spaced single-beam hydrographic surveys may be an effective alternative to multi-beam data collection in some circumstances. Figure 8-59 shows an example of a multi-beam mapping product.

- Airborne LiDAR bathymetry (ALB) is a surveying and mapping technology that uses blue-green laser from low-altitude aircraft to measure underwater elevations. Use is primarily in moderately clear, near shore coastal waters, shallow rivers, and lakes, although technology refinements and new technology may offer additional capability in the future (USACE 2015d).

- Before conducting a bathymetric survey, evaluate the bottom coverage density required for the specific project. Single-beam echo sounders are suitable for cross-section surveys. Full bottom coverage surveys are usually obtained by multiple transducer systems or multi-beam sweep systems. High-density single-beam data can be a reasonable alternative for some projects. Differences in data acquisition and processing are substantial between the different methods. Consider survey objectives to determine the most appropriate method.

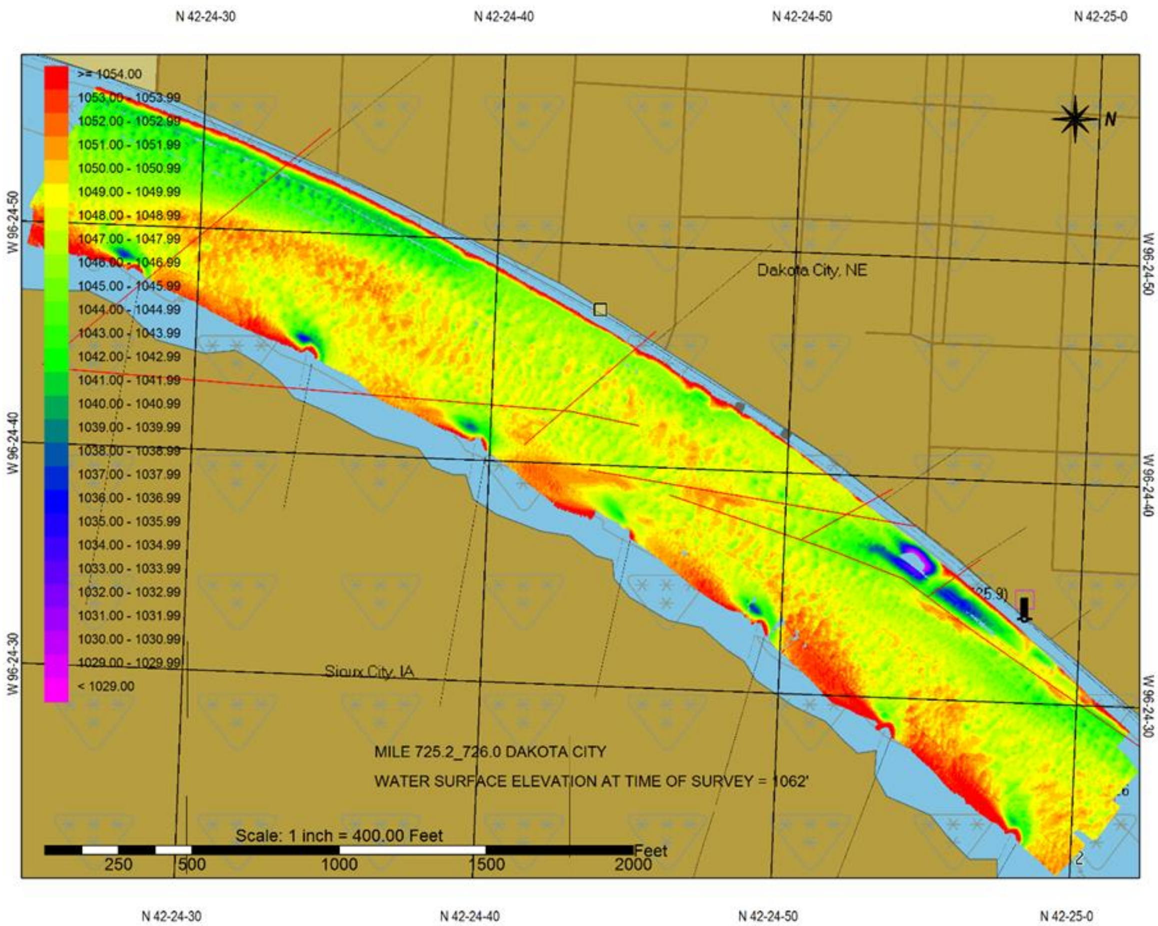


Figure 8-59. Example multi-beam survey mapping product, Missouri River near Dakota City, Nebraska

(i) Sediment Range. A sediment range refers to a fixed line across a reservoir, stream channel, or flood plain along which elevations are measured. Sediment range surveys require both topographic and bathymetric data collection. Although the use of sediment ranges has curtailed somewhat with the advancement of survey technology, sediment ranges still perform a vital function at many USACE projects. Sediment ranges provide a relatively low-cost method to monitor delta advancement into the pool and the rate of reservoir bank erosion.

- The sediment range surveys provide a rapid method to demonstrate reservoir geometry change with time and provide an established survey line to evaluate topographic mapping survey data accuracy. Partial surveys using sediment ranges at select key locations can be employed when a complete reservoir survey is not feasible.

- Sediment ranges are likely the least costly alternative to monitor sedimentation processes on large reservoirs, such as those on the Missouri River, that have large reservoir surface area in the range of 200 to 400 square miles at normal pool.

• Figure 8-60 shows an example sediment range layout at a USACE project. Appendix G provides a detailed discussion on USACE sediment range line methodology.

Figure 8-60. Sediment range layout at Bear Creek Dam, Denver, Colorado

(j) Transition from Historic Survey Methods. To provide both high-quality products and cost savings, future tributary reservoir survey methodology for many USACE projects will likely rely heavily on remote sensing data collection methods. Using LiDAR survey data collected by non-USACE agencies can provide a significant cost savings. However, transition from past practices to new methods and data sources should carefully consider data collection methods, accuracy, and project needs.

• Transition from historic methods to new remote sensing techniques can result in significant cost savings when data collected by non-USACE agencies is available.

• Follow USACE mapping standards for accuracy. Many external LiDAR data sources may have such a coarse grid spacing that reservoir capacity computations are erroneous.

• Evaluate all survey data accuracy and incorporate data checks, as necessary.

- Perform land surveys of selected sediment ranges for verification of survey accuracy and for comparison of both historic surveys and depletion rates when necessary.

c. Sub-Bottom Surveys. Sediment coring and acoustic surveys, previously discussed in Chapter 4, are methods to determine deposited sediment thickness. A sub-bottom profiler determines sediment deposition depth over the original bottom by using a higher frequency sonar signal for bathymetric mapping in combination with a lower frequency signal. The lower frequency signal penetrates finer sediment and is reflected from the underlying denser layers corresponding to the original bottom, which then provides the sediment deposition thickness to be mapped.

(1) Equipment measurement accuracy, variability in sediment source and material size, and spatial variability in sediment thickness may limit application in determining reservoir deposited volume.

(2) For locations where pre-dam construction information is not available, sediment coring and acoustic surveys provide an option to define sediment deposition rate.

(3) The use of age-dated sediment cores to describe water quality trends has a long history (Davis 1990).

d. Accuracy of Surveys.

(1) The objective of the reservoir survey is to measure the highest quality data possible that is needed for the study objective. The accuracy of a hydrographic survey is generally more difficult to monitor relative to conventional land-based surveys and remote sensing data collection methods. Refer to EM 1110-2-1003 for further discussion of applicable survey methods and survey accuracy standards.

(2) When evaluating current survey accuracy needs, the accuracy of historic surveys performed with significantly different methodology should be considered. The error of a map produced with a plane table and alidade varies across the map as the error in stadia measurements varies with distance. Horizontal errors may have ranged from 0.2 foot at 300 feet, to 10 feet or more at 1,000 feet. The plane table survey resulted in a “field-finished” map product with all quality control and quality assurance performed in the field by the party chief/surveyor (USACE 2007).

e. Reservoir Sedimentation Survey Plan Development. The term “sedimentation survey” refers to remote sensing data collection, field measurements, laboratory analyses of sediment samples, data processing and analysis, and reporting. Regular sediment surveys are critical for tracking and predicting storage loss and impacts to the authorized reservoir functions.

(1) Identifying Consequences.

(a) Many USACE projects are reaching the stage in project life, or will in the near future, where sediment issues impact project functions. Under recent budget conditions, USACE operation and maintenance priorities have not often funded sediment surveys and studies at the necessary frequency, quality, and extent. The lack of regular sediment surveys introduces operational uncertainty and project performance risk. Project operations, particularly beyond the project design life, must respond to developing sediment constraints that requires timely and accurate data.

(b) Missed sediment data collection opportunities are irreplaceable. Infrequent sediment surveys can distort projections because they miss dynamic supply trends and event dependence. Determining depositional trend shifts can be a critical component of evaluating USACE project impacts and operation issues. In these cases, responsible USACE resource management requires identifying minimal data collection needs, identifying consequences, and communicating those needs to decision-makers. Consequence identification steps include:

- Review aggradation/degradation trends.
- Identify sedimentation impacts from these trends.
- Develop supporting survey plans.
- Communicate to USACE resource providers the consequences of inadequate data collection for future analysis needs.

(2) Field Measurements. Sediment sampling methods have been previously discussed in Chapter 4 and are also presented in Appendix D and E. A robust reservoir sediment survey program includes the collection of sediment samples for multiple purposes. Other data collection needs such as contaminant levels and water quality related studies can significantly affect reservoir sampling needs. General reservoir sediment sampling and measurements will include:

(a) Reservoir Bed Elevations. Survey of established sediment ranges or grids and/or collection through contour survey or remote sensing. The preparation of topographic maps of special areas, etc. is used to determine elevations and calculate depths of sediment accumulations. These are the most important data for long-term sediment management and planning.

(b) Specific Weight of Deposits. Measurements necessary for computation of sediment densities and sampling required for pertinent determination of material properties.

(c) Longitudinal Sediment Gradations. Deposition profiles illustrating spatial variation (which influences the timing of impacts, see paragraph 8-4e) and appropriate management options depend upon the gradation of reservoir sediment. Since reservoir sediment is longitudinally variable (fining downstream), sediment gradation data that captures the distribution of sediment by grain class is important.

(d) Aerial Photographs. Georeferenced aerial photographs should be captured periodically. This data is more likely to be included with remote sensing, and could be cost prohibitive if only collecting imagery to accompany a sediment range survey. Extending the photography extents and collaborating with others often makes the collection of aerial photography more cost effective.

(e) Ground Photographs. Observations, probings, and other pertinent measurements not related to established ranges, such as photographs and pertinent data on delta areas, etc. are also used to capture changes over time. Documenting the location and direction of the photographs is also invaluable information, allowing for repeat photography of the same view.

(3) Preservations of Field Measurement Data. Some reservoirs surveyed are separated by decades. Infrequent data collection makes the data that are available more valuable. Survey notes and data must be organized properly and placed in suitable form for storage and future use. Survey information should be prepared and consolidated for retention in the District and/or Division office. Provide protection against possible loss by fire or other causes using fireproof storage or create duplicates in a different location. All files should be stored electronically in the District-approved database for survey records. The file storage media should be reviewed periodically to ensure it is still accessible and durable.

(4) Resurveys.

(a) Schedules. Reservoir survey frequency depends on multiple factors and usually have numerous stakeholders. Due to the relatively high cost and challenge to meet an infrequent budget need, scheduling of surveys is necessary. In addition, opportunistic data collection should be included in the planning process. For instance, high or low pool levels can dramatically affect cost at reservoirs depending on site-specific factors that affect survey methodology and cost. Table 8-3 lists considerations and recommended survey frequency.

Table 8-3
Reservoir Survey Frequency Guidelines

Factor	Guidance
General	For reservoirs with permanent or seasonal pools, survey on 5- to 10-year interval. Schedule surveys with funding requests following USACE guidance. Within the general survey interval, opportunities for cost savings data collection should be considered, including pool level impacts, partnering opportunities, and combining survey efforts at multiple projects. Reservoirs without permanent or seasonal pools may not require as frequent surveys.
Specific Need	All or partial reservoir surveys are frequently required to address specific needs such as evaluating delta progression, habitat impacts, water use allocation, recreation access, water quality issues, storage volume for dam safety studies, etc.
Sediment Yield Change	• Forest fire (initially and after period of high sediment yield, typically 3 to 7 years). • New dam or sediment basins. • Land use, stream channelization, or other factors that substantially alter basin sediment yield. • Operational changes to upstream dams that affect sediment passage downstream to project.
Event/Need	• Large flood or high sediment yield event occurrence. • Reservoir drawdown. • Operational change that affects pool level, releases, delta formation, etc. • Dam raise or release structural change. • Detailed evaluation is required (real estate impact, groundwater flooding, etc.).

Factor	Guidance
Storage Loss (most frequent of all factors for setting survey interval)	• Adjust the storage loss-based survey interval to occur more frequently as needed for high-risk conditions. • When deferring surveys for longer period, evaluate storage loss with partial surveys and observations to verify that dam safety is not impacted. • Preferred method for storage loss has three surveys (original plus two more) to set depletion rate, adjusted if needed for sediment yield changes (see above). • Must consider both total storage loss and loss by pool zone when setting interval.
	Small flood control reservoirs (0–100,000 acre-feet of storage): • 5-year maximum interval. • 5% maximum storage loss between surveys. • Minimum of three surveys to set depletion rate.
	Medium flood control reservoirs (100,000–500,000 acre-feet of storage): • 10-year maximum interval. • 7.5% storage loss between surveys. • Adjust to occur more frequently for high-risk conditions. • Minimum of three surveys to set depletion rate.
	Large flood control reservoirs (>500,000 acre-feet of storage): • 15-year maximum interval. • 10% maximum storage loss between surveys. • Minimum of three surveys to set depletion rate.
Accuracy	Original capacity estimates are missing or have questionable accuracy: • Conflicting depletion rate estimates for successive surveys. • Known issues with previous surveys based on results, depletion rates, spot check comparison between surveys, etc.). • Change in survey methods.

(b) Reconnaissance will be conducted at appropriate times to determine the extent of resurveys needed to conform with objectives. The reconnaissance information should be summarized in a concise memorandum to serve as a basis for planning the resurvey. This should include schedule and budget for collection, data processing, analysis, and reporting.

(c) When pool capacity analysis with sediment ranges is desired, complete surveys of all ranges will be required. Reconnaissance and survey planning should be performed to determine whether a complete survey is required or if partial resurveys of limited sections may suffice to address specific areas of concern such as delta progression, navigation channel dimensions, areas near water intakes, critical habitat, and similar. Cost savings of a partial survey should be carefully weighed against the benefit of complete surveys, especially when only minor savings are realized.

(d) Funding of surveys can be difficult. Follow USACE budget procedures to develop funding requests. Develop a survey scope to reach consensus. To the extent practical, plan survey periods to take advantage of favorable weather conditions and reservoir pool levels. Funds required for all tasks including field data collection, observations, data processing, data review, and survey management should be identified prior to survey start.

(e) The completion of computations, analysis of data, and publication of reports on the surveys should proceed as promptly as possible so the results may be used by USACE. As far as practicable, USACE data collection should be coordinated with other reservoir survey needs as well as public sector data collection efforts for economy. Additional funding will be required for analysis tasks.

(f) Results of resurveys will be incorporated in appropriate memoranda or technical reports so the information will be available for engineering applications in the office as well as for regional and national studies. The necessary content of a storage capacity report can be simple when only reporting only capacity. Detailed analysis and reporting vary by project. An example of a simple reservoir storage capacity report that presents basic results for a typical USACE project is included as Case Study 8A (Appendix N).

f. Volume Computations. Computation of reservoir storage volume is a standard USACE task. As discussed in paragraph 8-9h, long-term trends computed from volume estimates should recognize changes due to both computation methodology and data collection accuracy. Accurate estimates of reservoir storage volume and storage depletion rates are of vital importance to project operations and dam safety.

(1) Most USACE projects have historic reports describing methods that were previously used to determine storage volume and depletion rates that should be consulted before performing new volume computations.

(2) Depletion rates will typically vary over time due to change in sediment yield and sediment compaction, and to a lesser extent the potential of reduced trapping efficiency as the reservoir volume storage declines.

(3) USACE project volume computations and depletion rate estimates should consider the need to continuously adjust estimates of depletion rate as new survey data becomes available.

(4) GIS-Based Methods Using Digital Elevation Model Data.

(a) The use of GIS software to compute storage volume from Digital Elevation Model (DEM) data has become the preferred method for many USACE projects. GIS software interfaces well with remote sensing and high-density hydrographic data collection. If adequate topographic mapping standards are followed (USACE 2007; USACE 2015d), GIS-based methods can provide highly accurate reservoir topography contour mapping and computed volumes. However, a concern with GIS computation methods is the relative ease of generating highly inaccurate storage volumes if poor quality mapping products are utilized.

(b) GIS data analysis employs computer software packages that provide a means of organizing and interpreting large data sets. The hydrographic survey data are usually provided in an x, y, z coordinate format conforming to a recognized coordinate system such as Universal Transverse Mercator (UTM), latitude/longitude, state plane, or other systems that represent the Earth's 3D features on a flat surface. The project DEM should combine data from both above and below water reservoir area surveys.

(c) The most cost-effective contour map is usually developed by aerial photogrammetry or topographic LiDAR when the reservoir is empty to allow data collection of the entire reservoir area. For small USACE projects, terrestrial LiDAR or real time kinematic (RTK) surveys may be cost effective. However, the empty pool condition seldom occurs at most USACE projects, and a combination of aerial and bathymetric survey methods are necessary. Evaluate the maximum survey extent for project purposes. Typically, conducting the survey to an elevation above the top of the dam is recommended.

(d) To reduce the time and cost associated with underwater data collection, topographic and photogrammetry data should be collected when the reservoir is as empty as possible, and the high-density bathymetric survey should be conducted when the reservoir is as full as possible, providing maximum overlap of the two data sets. Surveying the underwater portion after the aerial survey with a large overlap reduces the time and cost, since the survey boat does not have to maneuver in shallow water portions already mapped by the aerial survey.

(e) Volume computations using GIS software employ a DEM created from survey data for the entire reservoir area. An accurate DEM can be obtained from surveys using standard GIS procedures. EM 1110-1-1000 provides guidance regarding DEM vertical accuracy. EC 1110-1-110 provides general criteria and policy for elevation data management within USACE.

(f) Before use, review the survey source data for accuracy and spatial extent to ensure compatibility with objectives. Point data types, spacing, and DEM triangulation all have implications on topographic surface quality. For instance, the use of break lines to describe abrupt changes in surface slope, such as those associated with vertical cut eroding banklines in the reservoir pool zone, can be a critical component to include during DEM creation. Qualified GIS professionals must follow the topographic map accuracy standards to meet study requirements. The indiscriminate application of DEM data through powerful GIS software without verifying suitability for study objectives must be avoided.

(5) Average End Area Method.

(a) Historically, sediment range lines were established at most USACE reservoirs to track elevation change and storage volume depletion. Sediment range lines were established at selected points with respect to the irregular reservoir boundary to reflect topography and also to reflect tributary sediment inflow locations.

(b) In the usual historic USACE process, the initial estimate of reservoir storage volume was developed from pre-dam contour mapping using the average end area method following a process similar to the method commonly used in computing earthwork quantities. Volume for each contour interval was computed from the average surface area enclosed by the contour multiplied by the contour interval. Computations were generally separated into segments.

(c) When computing volume from sediment range lines, the cross-sectional areas of the bounding ranges are averaged to determine the average end area. The area is then multiplied by the distance between ranges to compute the segment volume. This method assumes that the bounding sediment ranges are representative of the entire segment. Topographic variations that normally occur make it very difficult to establish sediment ranges that are representative of any given segment. Adding additional sediment ranges can incrementally improve accuracy but adds substantial cost. The reservoir banks and sediment ranges are seldom straight and determining the representative distance between the range lines is difficult.

(d) Many USACE projects employ the modified average end area method (MAEA) to enhance computation accuracy (USACE 1992). After computing storage volume from both the original contours and surveyed sediment range lines using the standard average end area method of volume computation, correction factor tables were developed to match the sediment range computed volume to the pre-dam contour volume. Thus, the MAEA method using sediment range computed storage volume that uses the corrected tables can be a significant enhancement of the traditional average end area method.

(e) MAEA analysis uses a table of factors, which consists of a value by elevation, accounts for the nonuniformity of reservoir shape, slopes, etc. between the bounding sediment range locations. The MAEA method to estimate volume has limitations that should be considered. For instance, grading projects with even minor changes to elevation that occur on a single sediment range, such an installing an elevated roadway or a gravel pit mine, will be overemphasized in the computed storage volume. The recommended practice is to plot all sediment range cross sections and compare to historic sections to verify areas of change. Appendix H provides additional information related to historic capacity computation methods.

(6) Summary. Computation of reservoir storage volume is a standard USACE task. Historically, volume computations using the sediment ranges with correction factor tables, or the traditional average end area method, have been used at many USACE projects. Results of the volume computations should be tabulated in an area-capacity report. Significant storage depletion may lead to additional studies to examine reservoir operations and dam safety.

(a) Common practice is to compute reservoir storage volume by the designated pool storage zones and compare to previous volumes. Tracking change by pool level highlights potential impacts to reservoir operations and future problem areas.

(b) Collect information regarding reservoir construction projects that affected elevations and consider the impact on survey data and volume computation methods.

(c) New remote sensing survey techniques and GIS-based computation methods provide both a cost-effective and highly accurate alternative. Caution is emphasized when switching computation methods and determining storage volume change and depletion rates as discussed in paragraph 8-9h.

(d) GIS DEM creation must adhere to USACE map accuracy standards (USACE 2015d).

(e) The sediment range volume computation method used historically by many USACE projects provides reasonably accurate results and may be the most practical for large surface area lakes.

g. Reservoir Depletion Rates. Reservoir depletion rates refer to the rate at which storage capacity is lost due to sediment deposition in the reservoir. The most commonly used method for estimating lost volume is by subtracting the resurvey capacity from the original capacity. Heinemann and Rausch (1971) stated that reservoir sediment deposits may change in average density because of compaction between successive surveys and could give erroneous capacity depletion rates. Variation in sediment source could also affect sediment density. Computation of reservoir capacity differences and corresponding depletion rates should include adjustments to the sediment density.

h. Switching Data Collection and Volume Computation Methods. Reservoir survey data collection methods at USACE projects often switch from sediment range cross sections to remote sensing combined with high-density hydrographic surveys. Volume computations using the new survey data are performed with GIS-based computer programs. This can affect both the reservoir volume estimate accuracy as well as sediment depletion rates.

(1) Sediment depletion rate estimates that are determined from two surveys using different data collection or computational methods will likely introduce a capacity change solely due to methodology rather than actual capacity difference. For some reservoirs, advances in technology have made this difficult or impossible to assess, because the accuracy of the current survey is better than the original collected data. In some cases, the accuracy difference is very pronounced.

(2) At the Theodore Roosevelt Reservoir (Lyons and Lest 1996), the 1995 survey used GPS and aerial data collection to develop 5-foot contours. The original contour map was completed in the early 1900s using plane table techniques at 10-foot increments. Due to the method differences of the two surveys, comparing the results from the two contour surveys did not provide an accurate value of the total sediment deposition. For that study, the range line method would have provided a more accurate estimate of the sediment deposition since dam closure (USBR 2006a).

(3) Morris et al. (2008) also provides guidance that when updating from the range method to contour surveying, compute the reservoir volume using both methods to determine how much

of the apparent inter-survey volume change is attributable to differences in methodology. Further information regarding USACE studies on volume computation methods is provided in the following sections.

(a) Volume Computation Comparisons.

- An assessment of estimated storage volume using GPS and sediment range was performed at two USACE projects by Huntington District (Miller 2001). Both computation procedures used survey data obtained from the use of GPS. At Fishtrap Lake, Kentucky, the depletion rate of 391 ac/ft/yr determined using GIS computations was nearly identical to that determined with the average end area method of 398 ac/ft/yr. However, at Dewey Lake, Kentucky, the depletion rate of 42.4 ac/ft/yr determined using GIS computations was about 2/3 of that determined with the average end area method of 63.5 ac/ft/yr.

- The Omaha District at Bluestem Lake performed a second assessment. This study determined a variance between the sediment range capacity estimate and the GIS capacity estimate of about 15%. Details of the assessment are provided as Case Study 8B (Appendix N). Figure 8-61 shows the difference in depletion rates between the two methods.

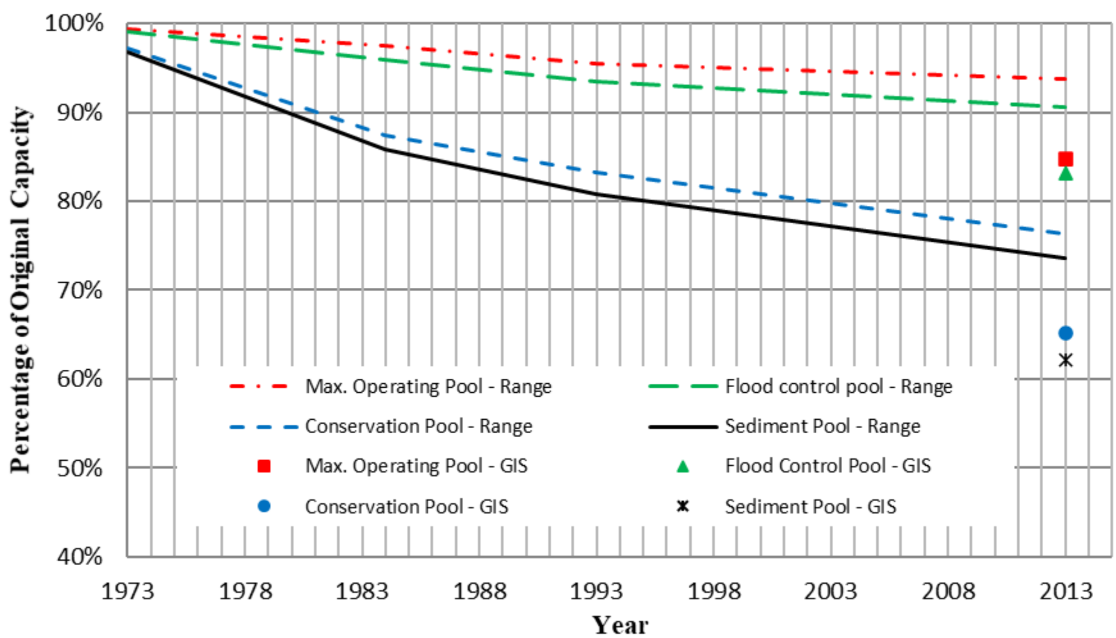


Figure 8-61. Bluestem Lake comparison of capacity methods

(b) Cost and Value Comparison.

- Survey data collection and analysis methods at USACE projects should be compared on both a cost and project value basis. Survey value includes an assessment of project-specific issues such as ongoing sedimentation problems, degree of capacity reduction, and impacts of capacity lost. Project-critical needs may dictate collection and analysis method requirements that have priority over cost. At locations with a lower level of sedimentation impacts, a reasonable combination of accuracy and cost may be warranted. Perform a cost comparison between data collection and analysis methods to inform decision-makers.

- The field surveys and computations necessary for sediment range surveys are often a comparatively low effort on a small reservoir. LiDAR data sources may be available at no cost from outside USACE agencies. This no-cost data can be offset by increased computational costs, since DEM assembly and computation time often requires additional effort. Future efforts should carefully consider the advantages of both methods.

(c) Recommendations for Selecting Survey and Computation Methods.

- The following guidelines are provided to help select the most appropriate survey method and to minimize difficulties in comparing survey methods.

  - Consider that new survey methods may have greater accuracy than the historic data collection.

  - Select a single method that provides the most accurate capacity estimate. In almost all cases, the GIS capacity computations using a DEM of the entire reservoir storage area should provide greater accuracy of current reservoir capacity than the sediment range MAEA method, provided the DEM meets accuracy standards.

  - A disadvantage of the sediment range method is that topographic changes that occur only to a small area at the sediment range, such as that from roadway fill or gravel mining, are magnified by the average end area computation process.

  - Comparison between GPS surveyed sediment range sections and LiDAR data at Bluestem Lake illustrated a surprisingly wide variation. The greatest difference occurred in areas with heavy tree and brush cover near the normal reservoir pool level. When using LiDAR surveys, it is highly recommended to survey all or a portion of the sediment ranges with conventional survey methods to evaluate LiDAR DEM accuracy (USACE 2015d).

  - A USACE project site-specific assessment of both cost and value is recommended when selecting the data collection and analysis method.

- Guidelines specific to evaluating sediment depletion rates:

− Sediment trends using historic capacity data cannot be accurately performed when the method used to compute capacity is changed. At Bluestem Lake, capacity difference between the methods was demonstrated to vary by elevation. The difference at normal pool levels may be small if only minor variation occurs in lake bed elevations.

− It is strongly recommended to perform the necessary surveys and compute capacity with both the sediment range and GIS method to evaluate differences when switching methods.

− Potentially, the capacity difference between survey methods may be used as an offset to establish capacity depletion trends. However, use of an offset should recognize variation by pool level and any large-scale grading projects that impact the accuracy of the sediment range method.

− When capacity differences are large, strongly consider performing future capacity surveys with both methods to provide additional information regarding the variation in capacity due to a change in survey methodology. The acceptable difference magnitude will vary by project and relative impacts. However, survey results that change depletion trends by more than 10% or 20% would likely merit additional duplicate surveys in the future.

− Performing surveys and computations with both methods does not alter the capacity computed with the single most accurate method. However, it may alter the depletion rate.

(d) In summary, when converting volume computation methods, numerous considerations are required to inform the accuracy of current capacity estimates and sediment depletion rates. Evaluate project specific-needs to develop recommendations for future surveys and analysis methods. Performing both historic and new methods to provide a volume estimate from both methods in order to quantify the volume difference that is due only to methodology change is strongly recommended.

8-10. Reservoir Sedimentation Data: Storage and Reporting. Sedimentation investigations pertaining to specific reservoir projects may continue intermittently over many years. Personnel engaged in the studies probably will change long before any particular investigation is completed. The information obtained from surveys will be of immediate and future interest to many engineers not directly involved in the investigations. To ensure that data are easily accessible and have been subjected to the rigor of quality assurance review, reservoir sedimentation data must be posted to the USACE standard Reservoir Sedimentation Information (RSI) Database.

a. RSI Database.

(1) Data capture of reservoir survey and area-capacity data from multiple agencies has been conducted in the past in association with the USGS Reservoir Sedimentation (RESSED) database consistent with the Advisory Committee on Water Information, Subcommittee on Sedimentation. The RESSED database provides access to sedimentation survey data for selected

U.S. reservoirs from USACE and other agencies with reservoir responsibilities. USACE issued a data call in 2008 to collect reservoir sedimentation data from USACE-managed reservoirs and information including sediment management practices, general hydrology, land use, and obstacles to sediment management practices (for example, regulatory, liability, chemical contamination of sediments).

(2) Improved knowledge about impacts such as climate to reservoir sedimentation prompted a change to the geospatial RSI database consistent with established USACE enterprise databases, such as the CorpsMap and the NID. This centralized RSI system supports efficient dissemination of information with reduced needs data calls and improved response time.

(3) The RSI data portal was developed to facilitate entry of reservoir sedimentation data by USACE Districts and provide a comprehensive summary of USACE reservoir conditions. The data portal populates an Oracle database that interfaces with CorpsMap and other enterprise databases used in USACE. Existing data was harvested from the most recent version of the RESSED database, and data from RSI can be fed into the RESSED database. RESSED will remain publicly available for an uncertain future time. The RSI data portal will be the primary data storage location for all reservoir sedimentation information for the foreseeable future.

(4) The RSI system stores and displays reservoir information to assist with evaluation of sedimentation trends and reservoir life expectancy, particularly with respect to a changing environment. The RSI portal is designed to allow credentialed users to add and modify sediment-related data for a reservoir. The reservoirs identified and used in the portal originate from the NID database are operated and maintained by USACE. For each reservoir, the user can add years of a survey and metadata such as survey type, start and end dates, vertical datum used, and a comment field. Furthermore, area-capacity tables can be uploaded by year of survey for a reservoir.

(5) The outputs include a summary of surveys and project metadata, as well as basic trend analysis and projections of future sedimentation using sediment yield variability. The RSI data portal can also produce an Engineering Form 1787 Report (Appendix I) suitable for use in USACE reservoir reports.

(6) Each USACE District has an assigned District data manager for the RSI database. The data district manager is responsible for adding and editing their District's data in the database as well as the quality control procedures for the data. They are also the primary point of contact of reservoir data in the USACE District offices.

(7) Please contact your RSI District data manager to access the most current reservoir storage data. Currently (2019) the database is accessed through CorpsMap (Enhancing Reservoir Sedimentation Information for Climate Preparedness and Resilience (RSI) at https://corpsmapz.usace.army.mil/.

b. Reporting of Reservoir Sedimentation Data and Analysis. To standardize the reporting of reservoir sedimentation data, a systematic series of memoranda and technical reports are essential to the proper accomplishment of program objectives. Reporting requirements are stated in ER 1110-2-8153. The scope and format of these individual reports will vary with problems and circumstances involved, but, in general, include the following:

(1) Proposed Reservoir Sedimentation Ranges and Investigations (for New Projects). This memorandum will be prepared as soon as construction of a particular reservoir is ensured. All information pertinent to formulation of the basic plan for sedimentation ranges, grids, and related facilities will be included, with a clear statement of objectives and circumstances governing specific proposals. Appropriate maps, photographs, and background information should be presented, as discussed in preceding paragraphs. Appendix G presents a suggested general outline for developing the layout of sediment range lines.

(2) Reservoir Sedimentation Survey Data.

(a) The agencies represented on the Subcommittee on Sedimentation, Interagency Committee on Water Resources, have collaborated in preparation of a set of instructions for compiling reservoir sedimentation survey data to promote a uniform assembly of the data and to facilitate future publications of the information in bulletins issued by the subcommittee. Appendix I presents these instructions, entitled "Instructions for Compilation of Reservoir Sedimentation Data Summary" and a sample copy of ENG FORM 1787 to be used in this connection.

(b) Once the area-elevation and capacity-elevation analysis is completed and loaded into the RSI database, an ENG FORM 1787 should be populated. Promptly following completion of ENG FORM 1787, a narrative report describing pertinent features of the resurvey will also be prepared and forwarded for review. In addition to describing soil types and other pertinent features, the report will include appropriate maps, maps, charts, photographs, and tabulations in which data from field notes are presented for use in subsequent analyses.

(3) Analyses of Reservoir Sedimentation Survey Data. Technical memoranda and reports will be prepared promptly following each comprehensive resurvey or important partial resurvey. These reports will include relatively detailed analysis of new data and pertinent correlations with data obtained from previous surveys of the same project or other projects located in nearby areas of generally comparable characteristics.

(4) Sediment Activities Annual Report. Notes on sedimentation activities are to be furnished as previously described in paragraph 2-6. These activities include reservoir surveys.

8-11. Sediment with Climate Change Overview.

a. General.

(1) Watershed hydrology and sediment processes depend on many factors, including climate, topography, vegetation types, geology, land use practices, soil properties, and their interactions (Tomer and Schelling 2009). Climate change phenomena including temperature and precipitation pattern change have potential to significantly alter these processes.

(2) Reservoir sedimentation occurs primarily in response to atmospheric precipitation, a component of weather. While the evaluation of sedimentation often includes simulation of specific (episodic) weather events (along with other things like period interval measurements such as range survey changes), longer-term projections typically involve aggregation of these event simulations to form representative averages. The accumulation of weather events to form averages constitutes climate.

(3) Release of the President's Climate Action Plan (June 2013) along with Executive Order 13653 (November 2013) required federal agencies to enhance climate preparedness and resilience. USACE has developed specific policy guidance, Engineering and Construction Bulletin (ECB) 2018-14, in support of this requirement (USACE 2018a). The ECB provides guidance for incorporating climate change information in hydrologic analyses per the USACE overarching climate preparedness and resilience policy and ER 1105-2-101. USACE policy continues to evolve. Check for current guidance for use with USACE project studies.

(4) Current guidance states that a qualitative analysis is required for all hydrologic studies for inland watersheds. The level of effort of this analysis is scalable to the project complexity, its consequences, and the sensitivity of the alternatives and/or project to climate variability and change. Climate change information for hydrologic analyses includes direct changes to hydrology through changes in temperature, precipitation, evaporation rates, and other climate variables, as well as dependent basin responses to climate drivers, such as sedimentation loadings.

(5) Anticipated changes in hydrologic and vegetation conditions due to climate change have the potential to affect sediment supply to reservoirs. Those changes in sediment supply, in turn, have the potential to affect project storage relationships, and therefore, the performance, operation, and management of USACE reservoirs, and must be evaluated in support of this agency policy. While the guidance in ECB 2018-14 is for performing qualitative assessments, Appendix B provides a preview of quantitative analysis guidelines which could impact sedimentation studies.

(6) Reservoir sedimentation projections for climate change scenario(s) are clearly needed to help identify and rank priorities, improve water resources management knowledge, and understand and enhance infrastructure resilience. The need for meaningful collaboration for scenario development and evaluation is evident, which provides the added benefit of knowledge improvement, as well. Case Studies 8C and 8D (Appendix N) illustrate climate change reservoir sedimentation evaluations.

(7) The Climate Preparedness and Resilience Community of Practice is developing additional guidance and web tools to assist in the analysis of climate change impacts to sediment yield[9]. The latest available methods should be used in evaluating both qualitative and

[9] for USACE staff, available through: https://maps.crrel.usace.army.mil/projects/rcc/portal.html
externally available through: http://www.corpsclimate.us/pubtools.cfm

quantitative changes to sediment yield, in-channel sediment routing, or reservoir sedimentation due to changing climate.

b. Regional Climate Change Affecting Sediment Yield.

(1) The National Climate Assessment (Melillo et al., 2014) divides the United States into eight regions (Figure 8-62), and tabulates observed and projected climate changes by region (Table 8-4). While increased precipitation and the frequency of heavy precipitation events generally equates to a higher sediment yield, these may coincide with differences in precipitation timing and vegetation establishment. These regional changes indicate that changes to sediment yield will not be uniform through the USACE reservoir system and that preclude generalizations about the impact of climate change on sediment yield. However, project evaluation should recognize that historic rates may not be representative of the future. Climate change impact assessments must be made at the individual project level.

Figure 8-62. Regions of the United States, as defined in the National Climate Assessment (Melillo et al., 2014)

Table 8-4
Projected Climate Changes by Region with Summarized Effects on Sedimentation

Region	Projected Change
Northeast	• Increased frequency of heavy precipitation events. • Average precipitation: Increased. • Annual runoff and related river flow are projected to increase. • Reduced snowpack accumulation. • Earlier and/or rapid melting of snowpack, resulting in earlier snowmelt-related streamflows.
Midwest	• Increased frequency of heavy precipitation events. • Average precipitation: North gets wetter; south dries a little in most models. • Upper Midwest: Annual runoff and related river flow are projected to increase. • Upper Midwest: Reduced snowpack accumulation. • Upper Midwest: Earlier and/or rapid melting of snowpack, resulting in earlier snowmelt-related streamflows.
Great Plains	• Increased frequency of heavy precipitation events. • Northern – average precipitation: Increased. • Central – average precipitation: No change (mixed modeling results). • Southern – average precipitation: Decreased. • Increased annual streamflow in Missouri River Basin. • Upper Great Plains: Reduced snowpack accumulation. • Upper Great Plains: Earlier and/or rapid melting of snowpack, resulting in earlier snowmelt-related streamflows. • Late summer/fall hydrologic drought (decreased streamflows) possible due to changes in mountain snowpack.
Northwest	• Increased frequency of heavy precipitation events. • Average precipitation: Increased in winter, spring, and fall; decreased in summer. • Annual runoff and related river flow are projected to increase. • Reduced snowpack accumulation. • Earlier and/or rapid melting of snowpack, resulting in earlier snowmelt-related streamflows. • Increased incidence of wildfires. • Increased hydrologic drought (decreased streamflows) due to changes in mountain snowpack.

Region	Projected Change
Alaska	• Increased frequency of heavy precipitation events. • Average precipitation: Increased. • Annual runoff and related river flow are projected to increase. • Shrinking glaciers. • Melting permafrost. • Increased incidence of wildfires. • Higher summer temperatures, loss of glaciers, and changes in permafrost may lead to reduced streamflows, particularly in the south.
Southeast	• Increased frequency of heavy precipitation events. • Drier west, especially in the southwestern portion of the region. • Increased chance of hydrologic drought (decreased streamflows) across southern portion. • Northern, Eastern: Increased precipitation. • Annual runoff and related river flow are projected to decrease. • Increased risk of hurricanes and other extreme events.
Southwest	• Increased frequency of heavy precipitation events. • Northern – average precipitation: Increased in winter and fall, decreased in spring and summer. • Central and Southern – average precipitation: Decreased. • Annual runoff and related river flow are projected to decrease. • Increased incidence of wildfires. • Reduced snowpack accumulation. • Earlier and/or rapid melting of snowpack, resulting in earlier snowmelt-related streamflows. • Complete loss of snowpack in New Mexico below 36° south. • Impacts to summer monsoon precipitation are unclear.
Hawaii and Pacific Islands	• Increased frequency of heavy precipitation events. • Average precipitation: Decreased or no change.

(2) Climate change science is continually evolving at a faster rate than it can be transferred into publications such as this. Engineers are encouraged to use the best current science when assessing climate change impacts for sustainability, starting with the most current version of the National Climate Assessment. Evaluation of project vulnerability to changes from historic trends is necessary.

8-12. Reservoir Sustainability Planning for Future Conditions.

a. Definition of Sustainability.

(1) The concept of sustainability is a central component of the USACE vision statement: "A GREAT engineering force of highly disciplined people working with our partners through disciplined thought and action to deliver innovative and sustainable solutions to the nation's engineering challenges."

(2) Sustainability is defined as "meeting the needs of the present without compromising the ability of future generations to meet their own needs" (WCED 1987). Morris and Fan (1998) summarized reservoir sustainability as maintenance of three characteristic water resource parameters at equal or higher than current (or historic) conditions. These parameters are water quality, water quantity, and diversity.

(3) In the long term, without active intervention, reservoir sedimentation will result in a nearly complete loss of storage volume and associated benefits for all reservoirs. The current "trap and store" strategy for sediment management is not sustainable.

(4) Dams are unique structures made obsolete not by the age or condition of the engineered works, but by the geologic processes of erosion and sedimentation. The engineered infrastructure can be repaired or even reconstructed, but the forces of geologic time march on (Morris et al., 2008). This context casts dams and their reservoirs into a somewhat different category from other civil infrastructure in terms of their economic life. As Annandale (2013) effectively illustrates, the optimal sites for constructing dams and reservoirs were selected first because they provided the greatest measure of storage for the least cost. Adding additional storage to restore depleted capacity becomes proportionately more expensive.

(5) Effective stewardship of these public facilities will, therefore, require careful consideration of such things as aging reservoir retirement/replacement costs, intergenerational equity, demographics of reservoir population, and how USACE reservoirs fit within this context in the reservoir population (size, age, purpose, etc.). While there are examples of dams filled by sediment to states of obsolescence, control of sedimentation can extend useful dam life well beyond many other types of engineered infrastructure, and examples of ancient dams with operational periods exceeding 2,000 years have been catalogued (Morris et al., 2008).

(6) Planning for reservoir sustainability with respect to sedimentation is an important component of ensuring that USACE reservoirs continue to meet their authorized purposes effectively into the future. Portfolio-wide prioritization of USACE reservoirs with respect to benefits and costs can inform data collection efforts, analysis, writing of reservoir sustainability plans, and implementation of sustainable actions. Similarly, sustainable operational strategies must be implemented at the limited number of new projects that may be constructed in the future.

b. Sediment Sustainability.

(1) USACE reservoirs were typically designed to trap and store incoming sediment in an area designated within the reservoir for sediment storage that was referred to as the “inactive, dead, or sediment” storage zone. The adequacy of this zone was often evaluated based on an economic project life of 50 to 100 years. As constructed reservoirs age, additional sediment accumulation displaces storage relied on for flood control, water supply, hydropower, and environmental benefits. In addition, continued sediment trapping in reservoirs can cause upstream and downstream economic and environmental effects, including flooding, in-reservoir water quality impairment, recreational impacts, and downstream degradation and displacement of turbidity-dependent native species.

(2) Without active sediment management, all reservoirs will eventually fill with sediment until an equilibrium condition is reached, meaning that all incoming sediment is passed downstream. In an unmanaged system, this will occur when there is little to no storage capacity left in the reservoir. This condition of dynamic equilibrium has been reached on the Conowingo, Safe Harbor, and Holtwood Dams in the Susquehanna River basin (USACE 2015g).

(3) Sustainable reservoir sediment management seeks to achieve this condition of sediment equilibrium while still maintaining the storage capacity and beneficial uses of the reservoir. This is accomplished by removing or passing incoming sediment at the same rate that sediment accumulates. More limited sediment management goals may include slowing (but not stopping) the rate of accumulation or may focus on ameliorating a single impairment (such as removing sediment near water intakes) rather than on overall sediment sustainability.

(4) Reservoir sustainability, with regard to sediment, is achieved when the volume of sediment that enters the reservoir is removed from the reservoir at an equivalent rate, resulting annually in a net zero loss in storage. In most cases, this can be accomplished to the greatest environmental benefit and at the least cost by passing the sediment downstream. In general, the closer the timing, volume, and gradation of outflowing sediment match the inflowing sediment (taking into account the altered hydrology of the reservoir), the better from an environmental perspective.

(5) New reservoir designs should consider long-term sustainability, regardless of the economic or planning time horizon used to justify the project.

c. Reservoir Life.

(1) Reservoir life was traditionally conceptualized as an estimated period during which the usable storage of the pool was filled by trapped sediment, presumably followed by abandonment of the structure (Morris et al., 2008). This life was often evaluated for an economic performance period that was typically 50 to 100 years and that may or may not have included the effects of sediment depletion on project benefits.

(2) Observations and studies (Morris et al., 2023; Morris et al., 2008) point out that sediment deposition will seriously interfere with design function long before the entire storage volume is depleted, and propose that a half-life metric is a more useful indicator of the period of effective reservoir operational life. This indicator is defined as the period required to fill one-half of the original capacity using the estimated sediment load.

d. Defining Sustainability at the Project Level.

(1) Sustainability at the project level is defined by the project goals associated with it. Sustainable reservoir operation maximizes long-term beneficial use by adjusting operations and reconstructing or re-purposing physical infrastructure to minimize sedimentation impacts without compromising environmental integrity, but understanding that the sustainable long-term benefits may differ in both type and magnitude from the project's original purpose (Morris et al., 2023).

(2) For each reservoir, sustainability will have a different operational definition. That definition will be driven by the current and future operational goals of that reservoir. What should be common with all USACE reservoirs is a reservoir sustainability plan to assess the current and future sediment and water supply conditions at each reservoir. This assessment should include a plan to either: (a) manage perpetually with the same or modified purposes, (b) manage to extend the useful life with the same or modified purposes and prepare for end-of-life decommissioning, or (c) manage under the current plan and prepare for end-of-life decommissioning.

e. Developing a Reservoir Sustainability Plan.

(1) In assessing ways to manage sediment, and therefore to extend the useful life of a reservoir, many management strategies have been developed. Those are discussed in paragraph 8-13, and the appropriate strategies should be included in the reservoir sustainability plan.

(2) A reservoir sustainability plan should include most or all of the following activities:

(a) Assessment of historical reservoir operation and storage loss rate.

(b) Determination of the impact of current storage loss on operations.

(c) Predict when sedimentation will affect project purposes in the future, considering both storage loss due to sedimentation and changing storage needs due to increasing demand and climate change.

(d) Assess what changes to reservoir operations can be made to minimize sedimentation impact or re-establish sediment balance through the project.

(e) Determine the impact to the future reservoir condition due to reservoir operations change (from a safety, economic, and project benefits perspective).

(f) Determine the impacts to the upstream and downstream channel due to reservoir operations change.

(g) Predict the level of sustainability (extend life, perpetual life, current life) of reservoir operations changes.

(h) Develop a future monitoring plan to measure the impacts of reservoir operations changes.

(i) Implement all sustainability plan components.

(3) While any change to reservoir operations and sediment management is a large procedural and regulatory undertaking, it must be considered. As USACE reservoirs continuously lose storage and operational flexibility, the need to allow for agile, sustainable management increases daily.

f. Climate Change and Reservoir Sustainability. Reservoir sustainability plans should consider future climate change (Pinson et al., 2016; USACE 2018a). Global changes facing USACE reservoirs include increasing water demand and the potential for increased sedimentation rates, both of which impact key reservoir functions including flood risk management, water supply, and recreation, among others.

(1) USACE reservoir project design normally included sediment yield determinations and allowed for a designated sediment deposition volume over the project life. However, these design estimates assumed that historic patterns of temperature, precipitation, and drought provided a reasonably accurate model of future regional conditions over the project lifetime.

(2) Water resources planners now recognize that this interpretation is not correct (Brekke et al., 2009): climate change may result in future patterns of temperature, precipitation, streamflow, and sedimentation that, in many regions, may differ significantly from those conditions in the historic period. Consequently, USACE has recognized the necessity of using the best current, actionable science on climate change impacts to water resources in evaluating reservoir sedimentation impacts (Darcy 2014; Brekke et al., 2009). Case Study 8D (Appendix N) presents an evaluation of reservoir sustainability with climate change impacts at Coralville Dam, Iowa.

8-13. Sediment Management in Reservoirs. This section discusses some of the sediment management strategies that may be applicable at USACE reservoirs.

a. Sediment Management Strategies.

(1) The range of available management strategies and their effectiveness will vary from project to project, due to the wide variability in physical as well as end-user environments. The strategies available are grouped into two general categories. The first deals with active management of the sediments themselves, and can be divided further into three general subcategories (Morris et al., 2023):

(a) Reduce sediment inflow to reservoir.

(b) Route sediment through or around the reservoir.

(c) Remove accumulated sediment.

(2) The second general category relies on adaptive modifications to manage sediment accumulation. These can be subdivided further into two subcategories (Morris et al., 2023):

(a) Structural modification of the project, such as increasing the crest elevation or relocation/replacement of water intakes.

(b) Operational modifications, from reallocation of pool components through retirement of the project.

(3) Some methods and guidance for selection of potentially effective strategies will be presented, as well as some guidance in evaluation of their efficacy. Particle size matters in both the impacts of reservoir sedimentation and the management of those impacts.

b. Reallocation. A pool reallocation increases the storage volume in one pool at the expense of another. Typically, the multi-purpose pool elevation is raised, which increases the water supply storage, but decreases the flood control storage. A pool reallocation is a means to redistribute the deleterious effects of continued sediment accumulation among various reservoir uses. While it does not preserve overall reservoir storage or lead to sustainable sediment management, it can defer costly or unacceptable impacts by years or decades and could form a part of an overall sediment management strategy.

c. Dam Raises and New Reservoirs. A second alternative to reservoir sediment management is to provide new storage space by raising the height of the dam or by building new reservoirs. The Texas Water Board found that new reservoirs could be constructed at less expense than dredging to recover capacity in existing reservoirs (Alan Plummer Assoc. 2005). However, lack of suitable reservoir locations, budget limitations, and environmental considerations make the construction of new reservoirs untenable in many locations. New reservoir design should consider long-term sustainability, regardless of the economic or planning time horizon used to justify the project. New reservoirs could also include provision for a larger storage volume to account for sediment.

d. Sediment Yield Reduction.

(1) Various attempts have been made to reduce sediment inflow into the reservoirs through changes in land, notably reforestation and altering agricultural practices to emphasize contouring and other erosion control approaches. While these methods offer many benefits, such as maintaining soil productivity for food security, increasing infiltration, and reducing storm runoff, their benefits in reducing sediment yield have not been clearly demonstrated (Kondolf et al., 2014; Annandale 2011). San Francisco-based Pacific Gas & Electric Company invested in watershed restoration and erosion control projects in the catchment above their dams on the

North Fork Feather River until concluding that, other benefits aside, they could not justify the cost in terms of reduced maintenance or greater generation (Kondolf and Matthews 1991).

(2) Eroding streambanks have been found in many locations to be a significant, even the dominant, source of sediment leading to sediment accumulation in downstream reservoirs (Trimble 1997; Simon et al., 1996; Juracek and Ziegler 2007). To have a measurable benefit for reservoir sedimentation, bank stabilization or other watershed treatments must be widespread and significant. However, targeting bank stabilization to the most erosional reaches can be an efficient first step.

(a) The Kansas Water Authority (KWA 2009) estimated that roughly half the annual sediment load to John Redmond reservoir could be accounted for by bank erosion at 19 erosional hotspot reaches. Bank stabilization has been installed at erosional hotspots upstream of Tuttle Creek Lake that were estimated to be more cost effective than removal by dredging of an equivalent volume of sediment.

(b) Additional information on streambank stabilization options can be found in Chapter 7 of this document and multiple references (Copeland et al., 2001; FHWA 2009).

(3) Widespread implementation of watershed BMPs to reduce sediment loss from land sources beyond erosional hotspots may not be the most cost-effective measure for managing sediment. At the Conowingo reservoir in Maryland, watershed sediment management was found to be 5 to 10 times more expensive than dredging with upland placement (USACE 2015g).

(4) As sediment transport in rivers is a natural phenomenon, bank stabilization or other watershed treatments will slow, but not halt the sediment accumulation and must be used in conjunction with other management strategies to achieve long-term reservoir sustainability.

e. Sediment Trapping Methods.

(1) Sedimentation basins, farm ponds, check dams, and grade control structures can trap sediments upstream of the reservoir in areas where sediment is less detrimental or easier to remove. A sufficient number of sediment trapping structures can significantly influence the sediment accumulation rate in a downstream reservoir. However, upstream trapping provides only a temporary benefit, as the traps will eventually fill with sediment. For long-term sustainability, sediment traps could be placed where there is a beneficial use for the sediment; for example, near a metropolitan area to provide sand and gravel aggregate.

(2) Grade control, engineered rock riffles, and check dams used in series reduce the average river slope, which reduces the sediment transport capacity and sediment yield to the reservoir. These structures can also prevent headcutting and attendant bank failures from migrating upstream, which can reduce the future potential sediment yield of the watershed.

(3) Successful implementation of these methods to provide long-term sediment reduction can be challenging. Multiple check dams to intercept sediment above the Saignon Dam in southern France were deemed failures by Chanson (2004) because both check dams and the

reservoir had filled within two years of construction. Likewise, Ran et al. (2004) found that nearly all check dams built in four Yellow River tributaries had filled with sediment within 25 years, but concluded they had been successful in reducing sediment delivery to the downstream reservoir (Kondolf et al., 2014). Information on grade control design can be found in Chapter 7, Case Study 7C (Appendix N), and EM 1110-2-1418.

f. Sediment Routing Strategies. Sediment routing encompasses sediment management strategies that keep the sediment in motion as it passes either around the reservoir (sediment bypass) or through the reservoir (sediment pass-through). Routing strategies use the fact that the majority of the sediment enters a reservoir during high flows, with more than half of a reservoir's annual sediment loading typically entering during the highest 5 to 10 days of flow (Meade and Parker 1984). More information about typical temporal patterns for sediment delivery can be found in Morris and Leech (2013).

g. Sediment Bypass. Sediment bypass seeks to bypass sediment-laden flow around the pool while capturing clear water in the pool. This can be done by routing clear water into an offstream reservoir or by routing sediment-laden water through a bypass tunnel or channel, as illustrated in Figure 8-63.

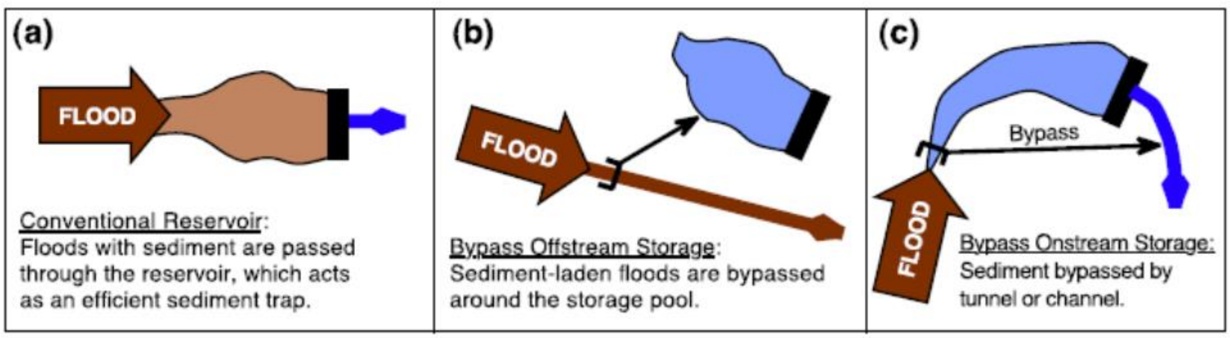


Figure 8-63. Conventional vs. sediment bypass reservoirs (from Kondolf et al., 2014)

(1) Offstream Reservoir for Sediment Bypass.

(a) An offstream reservoir design provides an efficient means of storing relatively clear water while routing sediment-laden flood waters downstream. An offstream reservoir operates by having an intake system with limited inflow capacity. Higher, sediment-laden flows are excluded from the reservoir because they exceed the inflow capacity. Alternatively, intakes are raised to exclude the bulk of the larger gradation sediment load, which typically occurs in the lower portions of the water column. This technique cannot usually be employed on an existing reservoir and is not appropriate for reservoirs with flood control functions.

(b) Per the dam portfolio conducted in 2015, USACE does not own or operate any offstream reservoirs. However, offsite reservoirs have been built and operated with success in Taiwan (Wu 1991) and Puerto Rico (Morris 2010). In Puerto Rico, an offstream reservoir was built on the Rio Fajardo, which began filling in 2006. A schematic illustrating an offstream reservoir is shown in Figure 8-64. This reservoir captures 26% of the flow, but less than 6% of the suspended-sediment load and none of the bedload, which greatly reduces the need for maintenance dredging (Morris and Leach 2013).

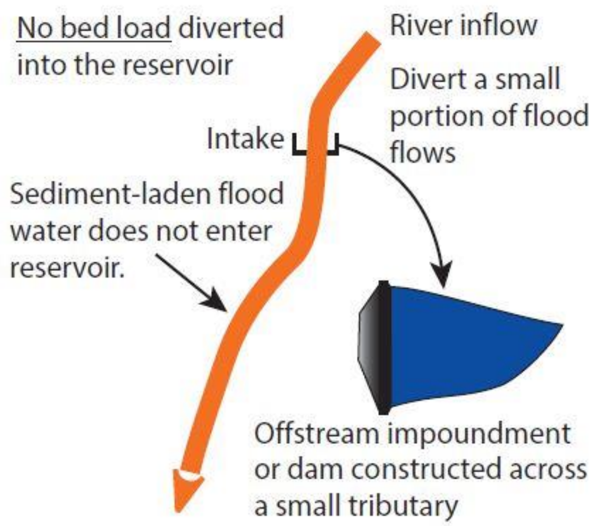


Figure 8-64. Off-channel storage reservoir (from Annandale et al., 2016)

(2) Sediment Bypass of Onstream Reservoirs. A second option for sediment bypass is to construct a channel or tunnel that short-circuits the reservoir during high flows. This technique can be accomplished where the onstream reservoir is located on a stream meander and the bypass conduit is constructed to cut across the meander, or in mountainous regions to cross areas where the slopes are sufficiently steep.

(a) Per the dam portfolio conducted in 2015, USACE does not own or operate any onstream reservoirs with sediment bypass. However, this technique has been employed with success on the Nagle Dam in South Africa; Nunobiki, Asahi, and Wima Dams in Japan; and the Palagnedra, Pfaffensprung, and Runcahez Dams in Switzerland (Annandale 2013). Examples from the Nagle and Miwa Dams are shown in Figure 8-65 and Figure 8-66, respectively.

(b) Passing high flows around the reservoir may not be appropriate where flood control is a major purpose of the reservoir. Where measured sediment inflows are sufficient to create a flow/sediment rating curve, a sediment budget approach may be sufficient to estimate the quantity of sediment that would bypass the reservoir.

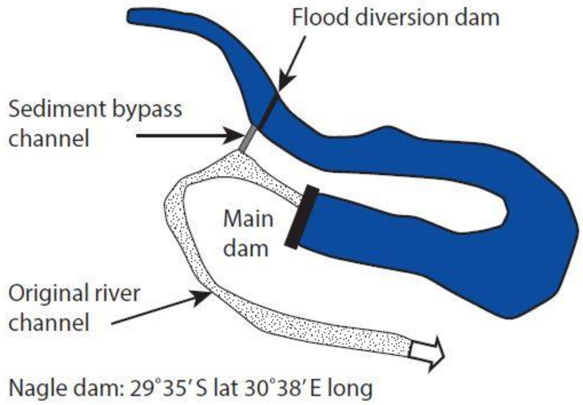


Figure 8-65. Sediment bypass at Nagle Dam, South Africa (from Annandale et al., 2016)

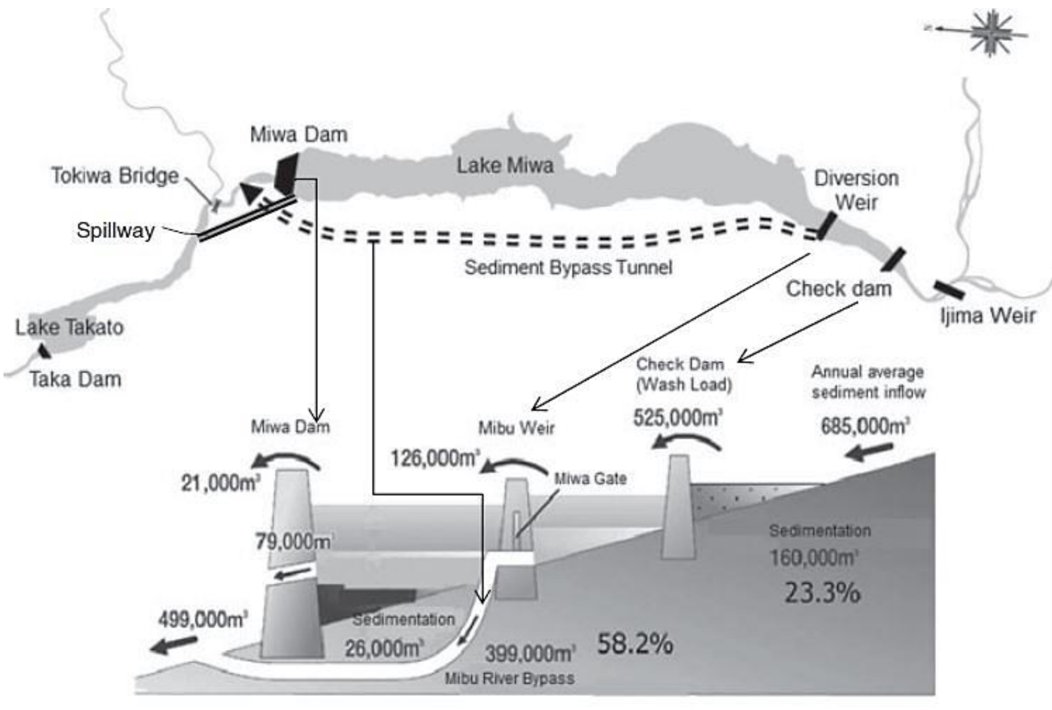


Figure 8-66. Sediment bypass tunnel at Miwa Dam, Japan (from Kondolf et al., 2014)

h. Sediment Pass-Through.

(1) As the name suggests, sediment pass-through (also known as sediment sluicing) routes the sediment through rather than around the reservoir. In a non-impounded river, sediment is maintained in motion and suspension by shear stress and turbulence, both of which are associated with high-velocity flows. When the river reaches the backwater from an impoundment, the velocity decreases significantly and the sediment begins to settle out. Sediment pass-through strategies seek to prevent sediment accumulation by maintaining high velocities during large sediment inflow events.

(2) Sediment pass-through is achieved by drawing down the reservoir to the lowest feasible level before a high inflow event, passing the sediment-laden inflow hydrograph through the reservoir at the highest possible velocity, and refilling the reservoir with less turbid water. This would typically occur on the falling limb of the inflow hydrograph or in the later part of the inflow season. This method is most appropriate for reservoirs with small storage relative to the annual streamflow, a high length-to-width ratio, and an incoming sediment load composed of fine sediments.

i. Pass-Through by Seasonal Drawdown.

(1) Seasonal drawdown empties the pool before the start of a seasonal wet season, then refills the reservoir toward the end of the wet season. Per the dam portfolio conducted in 2015, USACE does not manage sediment on any reservoirs through a seasonal drawdown. However, seasonal drawdown has been used with success in China on the Sanmenxia dam and Three Gorges dam (Morris and Fan 1998). One-dimensional numerical modeling may be sufficient for assessing sediment pass-through under different pool height and release scenarios for a seasonal drawdown.

(2) Drawdown by Hydrograph Prediction.

(a) In hydrologically small reservoirs in watersheds with less predictable, prolonged, or seasonally dependent inflows, a drawdown may be targeted to pass the "first flush" of a single inflow event rather than the early part of an entire season. This is accomplished in the following steps. First, the reservoir is drawn down just before a sediment-laden flood. Second, the flood is passed through the reservoir. Third, the reservoir is refilled on the falling limb of the inflow hydrograph.

(b) One-dimensional numerical modeling may be sufficient for assessing sediment pass-through under different pool height and release scenarios using historic data. Managing a reservoir in this fashion requires hydrologic predictive ability so the reservoir drawdown can occur before the incoming flood, and reservoir releases can stop sufficiently soon to allow refilling the reservoir. This may require additional rain gages and calibrated hydrologic models.

(c) An analysis for John Redmond Reservoir, a large USACE reservoir in southern Kansas, found that increasing early reservoir releases could decrease overall sediment accumulation by 3% without violating current flood control targets (Lee and Foster 2013). This analysis included increased releases early during each inflow event, not before the event. The nearly circular configuration of John Redmond is not conducive to efficient sediment pass-through. Pass-through would be more effective if used in conjunction with relaxed flood control targets or geometric modifications to the reservoir (Morris and Leach 2013).

j. Venting of Turbid Density Currents.

(1) Turbidity currents (see paragraph 8-4d can be vented from reservoirs by opening a low-level outlet at the dam, and at some reservoirs it has been possible to release more than half

the total sediment load in an individual flood by venting the turbidity current. Successful venting depends on properly located low-level outlets, which are opened in time to release the current using a discharge rate that matches the turbidity current inflow. However, it will not be possible to vent turbidity currents in many reservoirs, and the efficiency of releasing turbidity currents can decline over time as deposition that fills the submerged channel changes the reservoir bathymetry and impedes propagation of the turbidity current (Morris and Fan 1998).

(2) A turbidity current will flow only as long as there is a continued input of turbid water at the upstream end of the reservoir, and it will stop as soon as the inflow ends. Thus, the portion of the current within the reservoir at that point cannot be vented. The greatest amount of turbidity can be released when the discharge capacity of the outlet approximately matches the flow rate of the turbidity current reaching the dam. When a turbidity current reaches a dam and is vented, water will be aspired from levels both above and below the outlet level. There is the potential to entrain clear water from the layer above the turbidity current, releasing clear water when it is not necessary.

(3) Relationships are available for the application of turbid density current venting, including plunge point location, turbid current velocity, and aspiration height (Morris and Fan 1998; USBR 2006a). Numerical modeling of turbidity currents is challenged by the computing cost of 3D modeling within a reservoir. A methodology using a double-layer averaged model is presented by Cao (2014), future advances in numerical modeling are likely as computation methods evolve. Morris and Leach (2013) provide a more complete discussion on turbid density currents, including some ways they can be targeted for routing.

k. Sediment Focusing. Sediment focusing includes techniques for controlling where sediment deposition occurs. Sediment may be focused away from areas experiencing adverse impacts (such as boat ramps or water intakes or at the mouths of tributaries) or toward areas more conducive to sediment removal. Geometric changes to the reservoir bottom via dredging or via placement of training structures can focus flows and influence locations of sediment accumulation. Raising or lowering the pool elevation can strongly influence the longitudinal location of coarse sediment deposits (Teal and Remus 2001).

l. Sediment Flushing.

(1) Sediment flushing involves completely draining the reservoir to allow free flow to scour previously deposited sediments. Given the right circumstances, flushing can be highly effective at removing sediment.

(a) Flushing on the Mangahao reservoir in New Zealand removed 75% of over 4 decades of sediment accumulation in a single month (Jowett 1984).

(b) Fall Creek, a USACE reservoir in Oregon with storage capacity of 115,100 acre-feet, has been managed by annual flushing since 2012. The December 2012 flushing event lasted 6 days and removed 50,300 tons of sediment from the lake (Schenk and Bragg 2015). A similar flush during the winter of 2013 removed only 5,200 tons due to insufficient flows following the drawdown. Preliminary analysis shows the winter 2014 flush removed 20,000 to 35,000 tons (Schenk and Bragg 2015).

(c) Spencer Dam, a small hydroelectric facility in northern Nebraska, has managed sediment by flushing twice a year since the 1940s. Each flushing event at Spencer Dam involves the complete drawdown of the reservoir, lasts multiple weeks, and removes several hundred acre-feet of fine to medium sand from in front of the powerhouse and gates. Accumulated sediment in the reservoir floodplain does not mobilize during these flushing events, and the amount of storage maintained is a small fraction of the original storage capacity.

(2) Atkinson (1996) lists additional reservoirs that have been successfully flushed, including Baira (India), Gebidem and Palagneda (Switzerland), Gmund (Austria), Hengshan, Honglingjin, and Naodehai (China), Mangahao (New Zealand), and Santo Domingo (Venezuela). Atkinson (1996) recommends the following empirical procedure for estimating flushing effectiveness.

(a) Step 1. Calculate the flushing width, W_f, as the minimum of either the flushing width computing by Equation 8-6 or the wetted top width at the flushing discharge.

$$W_f = 12.8\, Q_f^{0.5} \qquad \text{Equation 8-6}$$

where Q_f = the representative discharge passing through the reservoir after the reservoir has been fully drawn down.

(b) Step 2. Determine the empirical parameter φ:

φ = 1600 for fine loess sediments
φ = 650 for $D_{50} < 0.1$ mm
φ = 300 for $D_{50} \geq 0.1$ mm
φ = 180 if the flushing discharge is low (Aktinson (1996) gives 50 m^3/s as a suggested threshold, but does not provide a rationale for this estimate)

(c) Step 3. Compute the consolidation factor, c:

c = 1 for annually flushed, unconsolidated sediments
c = 3 for consolidated sediments

(d) Step 4. Compute the sediment load during flushing, Q_s:

$$Q_s = \frac{\varphi}{c}\left({Q_f^{1.6} S^{1.2}} \middle/ {W_f}^{0.6} \right) \qquad \text{Equation 8-7}$$

where:

S = channel slope

(e) Step 5. Compare the computed flushing amount to the incoming sediment load.

(3) Empirical procedures such as suggested by Atkinson (1996) may be useful for reconnaissance-level analysis. More robust equations and procedures, including 1D or multidimensional modeling, may be warranted depending on project purposes and risks.

m. Pressure Flushing. Pressure flushing involves opening low-level gates while the reservoir is still full in an effort to scour sediment. Pressure flushing does not flush large quantities of sediment from the reservoir as does typical drawdown flushing, but it may be an effective means to clear out sediment in the immediate vicinity of the gates. Pressure flushing is used at select USACE reservoirs to keep sediment deposition from hindering gate functionality. Two examples are Cherry Creek Dam near Denver, Colorado, and Blue Springs Dam near Kansas City, Missouri.

n. Hydraulic Dredging.

(1) Hydraulic dredging for removal of accumulated sediments offers the most control over the location, type, rate, and timing of sediment removal and tends to use less water than other options. Dredging may be undertaken for small-scale, tactical removal of accumulated sediment or for more extensive restoration or maintenance of storage capacity. Refer to EM 1110-2-5025 for additional information regarding dredge types, production rates, and disposal techniques.

(2) Tactical dredging is the removal of accumulated sediments that are impacting specific features such as water intakes, gates, boat ramps, or upstream channels. Tactical dredging need not completely remove the sediment from the pool to be effective. However, tactical dredging may simply result in a hole that quickly refills with sediment (Loehlein 1999). Tactical dredging may be combined with sediment focusing for a longer-term solution to localized sediment accumulation impacts.

(3) Dredging has been undertaken with success for restoring lost storage capacity. However, dredging is often the most expensive option for the recovery of lost storage capacity. A major component of the expense is the cost to dispose of the dredged material. Dredged material from John Redmond Reservoir, a USACE reservoir in southern Kansas, is planned to be spread on nearby agricultural fields for 5 years, after which the land will be reclaimed for agricultural use (KWO 2014).

(4) Dredging only temporarily reduces the volume of accumulated sediment in the reservoir and must be repeated for long-term reservoir sustainability. The cost for upland disposal of dredged material can be expected to increase over time, as the nearby, less-expensive disposal sites are filled.

(5) In many situations, the sediment could be reintroduced into the channel downstream from the dam rather than transported and held in upland areas. Recharge of sediment to the downstream channel should be considered in every reservoir dredging project, though it may not be feasible due to environmental or other reasons.

(a) At Conowingo reservoir in Maryland, dredging with downstream recharge of sediment was found to be possible at 11% to 34% of the cost of dredging with upland placement (USACE 2015g).

(b) In addition to cost savings, there may be infrastructure and environmental benefits to the downstream channel from the reintroduction of sediment. For example, dredging with downstream recharge of sediment was analyzed for Lewis and Clark Lake on the Missouri River for the specific purpose of increasing sand loads in the downstream channel to benefit endangered species.

(c) Alleviating downstream bed degradation is a project purpose for reservoir dredging with downstream recharge of the sediment at Prado Dam in California (OCWD 2015). Even downstream discharge of fine sediments may yield an environmental benefit in historically turbid streams (Shelley et al., 2016).

o. Hydrosuction and Inlet Extension.

(1) Hydrosuction uses the available head in the reservoir to siphon sediment up and over the dam or spillway. Hotchkiss and Huang (1995) present a method for computing the effectiveness of this option for removing accumulated sand. They also demonstrate that, for a small dam, the hydrosuction system was able to pass sand downstream at the annual rate of sediment inflow, effectively restoring bed material sediment continuity. For large dams, the height of the dam may exceed the limit for a functioning siphon, in which case a booster pump would be needed and the hydrosuction operation would resemble dredging with downstream recharge of sediments, as discussed previously.

(2) For large dams with multiple inlets, the siphon pipe could be attached to an inlet, which would allow the sediment to pass through (rather than over) the dam. McFall and Welp (2015) estimate that a single, 2-foot diameter pipe at Tuttle Creek Lake would be sufficient to pass between 2 and 4.2 million cubic yards of sediment per year at Tuttle Creek Lake without any increased water use over that already released for environmental flows. This equates to a 34% to 72% reduction in the rate of sediment accumulation in the multi-purpose pool.

p. Modifying Reservoir Geometry. Both sediment pass-through and sediment flushing are more effective in narrow reservoirs that convey flows at higher velocities. In a wide reservoir, a submerged revetment or series of training structures could be used to induce higher velocities and focus sediment-laden high flows through a narrow corridor, as illustrated in Figure 8-67. This would significantly improve the effectiveness of both sediment pass-through and sediment flushing. At the time of this writing, no USACE reservoir has implemented a reservoir modification strategy of this type for sediment management.

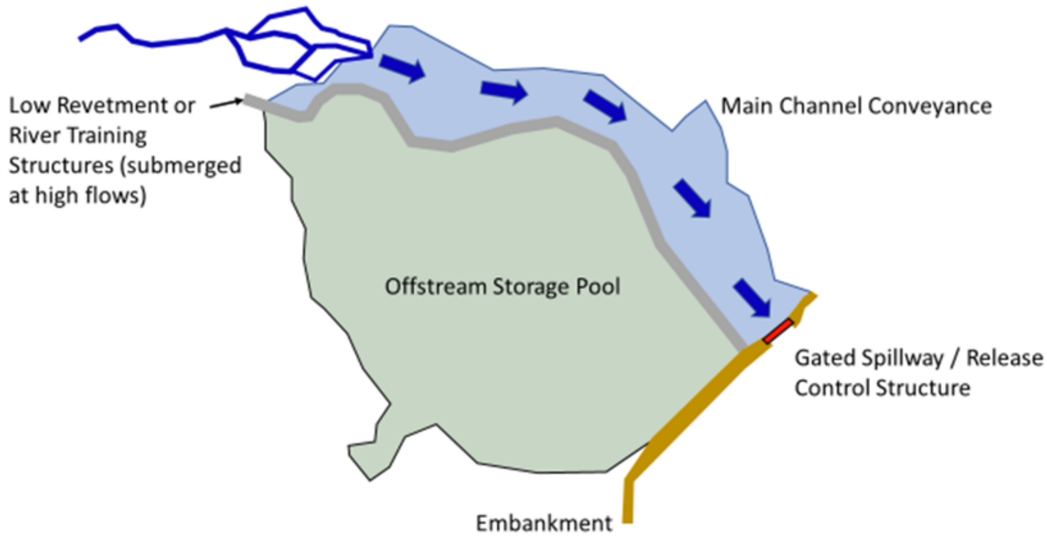


Figure 8-67. Conceptual diagram of reservoir geometry modification

q. Selecting a Management Option. While the appropriateness of any reservoir sediment management option depends on many site-specific factors, two ratios can offer general guidance. The first is the ratio of reservoir volume to volume of mean annual storage loss (CAP/MAS). The second is the ratio of reservoir volume to mean annual water inflow (CAP/MAR).

(1) Figure 8-68 summarizes reservoir sediment management methods employed at various international reservoirs with respect to these ratios. As reservoirs accumulate sediment, they move down the y-axis and left on the x-axis, suggesting that some management options that are not sufficiently effective to sustain a reservoir's current storage volume may be a viable means in the future at sustaining a reduced storage volume.

(2) The chart shown in Figure 8-68 has limitations, including the fact that it does not distinguish between sizes of sediments (silts vs. sands), which may respond more or less favorably to management options. Figure 8-68 may be used for general guidance, but does not substitute for more detailed engineering and economic analysis.

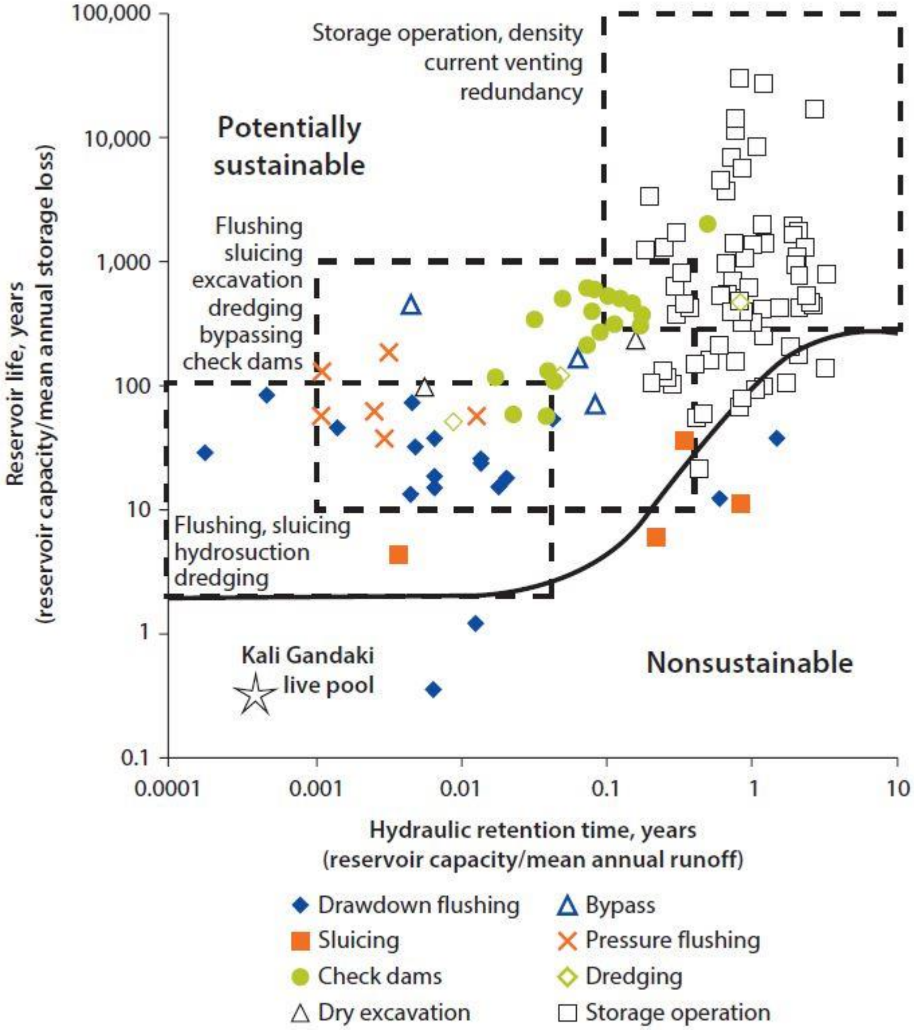


Figure 8-68. Applicability of sediment management techniques based on hydrologic capacity and sediment loading (from Annandale et al., 2016)

r. Reservoir Sediment Management Conclusion. Without active management, the impacts of sediment accumulation to USACE reservoirs will continue to increase with time. A range of options that is available for consideration to manage sediment accumulation in new and existing reservoirs was presented. Sediment management actions are often feasible that prevent sediment from accumulating in the reservoir, remove sediment from the reservoir, prolong the useful life of reservoirs, or extend their benefits. Appropriate actions can be selected that minimize the associated upstream and downstream impacts listed in paragraphs 8-5g and 8-6g.

<u>8-14.</u> <u>Debris Basins</u>.

a. A debris basin is used to trap coarse sediment before it enters a downstream channel, the performance of which would likely be adversely affected by the debris material. In this use, debris refers to the assortment of sand, gravel, cobbles, boulders, logs, and other large material objects that may deposit in the downstream channel, causing flood flows to spill out before design conditions are reached. In addition, the passage of large debris loads through downstream project features such as concrete lined channels can result in costly erosion damage. Figure 8-69 illustrates debris basins used on USACE projects.

Figure 8-69. Debris/sediment basins at USACE projects (a) upstream end of Iao Stream, looking downstream at lined channel in Maui, Hawaii after cleanout (2019); (b) looking upstream before cleanout (2016); (c) Santa Barbara County, California, Romero basin debris removal January 2018; (d) Santa Barbara County, California, Montecito basin debris removal, February 2018

b. Unlike sediment detention basins, which can be designed for multiple objectives (such as to improve water quality, create wetland habitat, and trap sediment from all runoff events), a debris basin is designed to trap only the coarse material or hyperconcentrated load associated with debris flows. Refer to paragraphs 3-4 and 7-8 for further discussion on sediment concentration, hyperconcentrated flows, and analysis methods.

c. To efficiently trap only the desired large size fraction of the total load, debris basins are small with very low storage capacity volume to design event inflow volume ratios. Since debris basins have limited sediment storage capacity, the accumulated sediment must be removed if they are to provide long-term protection. To minimize removal frequency and cost, the design should trap the target grain size while passing the finer sediments that can be transported in the downstream channel.

d. Generally, debris basins are often constructed in historic deposition areas and where sufficient storage is available. Typical locations are at abrupt slope transitions where a stream exits from mountainous or hilly areas to enter a flood plain, or in the headwaters of flood control channels.

e. Guidance. Due to differing climate hydrology and geologic formations, the nature of concentrated sediment flows and applicable debris basin design methods varies considerably by region. Guidance from constructed debris basin projects in the desert southwest may be obtained from the Los Angeles District Corps of Engineers (USACE 2000; USACE 1994b) along with numerous other sources including the Los Angeles County (California) Flood Control District (Los Angeles County 1979), Los Angeles County Public Works (Los Angeles County 2006), and the Clark County (Nevada) Regional Flood Control District (Clark County 1999). Debris basin evaluation is provided in Case Study 8E (Appendix N).

f. Design Overview. Debris basin design, to attain typical USACE objectives in a flood damage reduction project, includes determining the maximum sediment size of the inflowing sediment load that can be passed during the design event without impacting the downstream project. Debris sizing requires determining the grain size distribution and volume of the inflowing load. This information is used to find the basin hydraulic characteristics required to trap the target grain size and minimize the trapping of finer material. Additional design considerations are summarized as:

(1) Determining the maximum allowable grain size to pass the basin will usually require downstream channel sediment transport studies. Application of empirical relationships derived from functioning debris basins in similar hydrologic and geomorphic areas may be feasible.

(2) Project safety requires not only design flood considerations, but also the proper consideration of antecedent conditions to a design flood.

(3) The debris basin should be designed to function such that during an extreme event, which exceeds the design flood, the project will not make conditions worse than would have occurred without the project.

(4) Maintenance with removal of sediment and periodic inspections of the basin structure are imperative for basin function and must be considered in selecting basin location.

(5) Consideration should be given to allow for larger-than-design debris storage due to antecedent debris conditions if post-storm maintenance is unlikely to occur in a reasonable time frame.

(6) Staff gages or similar monitoring methods should be included to easily observe debris accumulation in the basin.

(7) Analysis should identify a minimum removal level to maintain capacity for future events. Unless substantial excess capacity is provided, it is often necessary that the basin debris accumulation does not exceed 10% to 20% full without requiring immediate removal of debris after an event in order to meet design event objectives. Maintenance can be a large and recurring cost that must be thoroughly evaluated and included during USACE project formulation. Maintenance costs are affected by numerous factors including sediment size and volume, access to the basin, and the disposal area location.

(8) Debris basins are usually small; however, the size is not arbitrary. It must be justified by project economics and available sites. Some basins are sized for only one or two major storms. Others may have excess extreme event capacity. Constructed basins are often a bowl-shaped pit excavated in the surface of an existing debris cone or fan.

g. Sediment Yield.

(1) Sediment yield estimates for debris basin design should include at least two types of hydrological events: the long-term record and design flood events. The design flood events should consider fire impacts and include a range of precipitation events. Include snow cover when applicable. Choice of the governing event for sediment yield is determined from the driving yield conditions for the system that are often similar in a region and specific post-fire conditions. Computations should consider both total sediment yield and the debris production.

(a) Multiple methodologies can be used to determine sediment yield, including Los Angeles District guidance (USACE 2000), and in other guidance (Los Angeles County 2006; Morris and Fan 1998; HEC 2016a; HEC 1995; NRCS 2016). Refer to paragraph 6-7e for further discussion on wildfire hydrology and sediment yield.

(b) Site-specific or similar regional methodologies are usually best, where regression equations are based on local information. In lieu of regional studies, models such as those developed using HEC-HMS can be used to determine sediment yield for either single-event or long-term sediment loading. HEC-HMS is implementing new post-fire sediment yield regression equations and algorithms for debris basin design that fall into two basic categories: (1) rapid response equations that are not tied to hydrology and can estimate post-fire loads based on precipitation and topography data; and (2) more robust, flow-based equations that can be integrated with known hydrology.

(2) The current version of HEC-HMS (HEC 2017) includes three methods for debris estimation: the Los Angeles District debris method equation 1, the Multi-Sequence Debris Prediction Method, and the USGS Long-Term Debris Model. Refer to HEC-HMS guidance (HEC 2017) for information regarding applicability of each method.

(3) Debris and sediment production rates vary throughout the country and depend on many factors. The occurrence (or non-occurrence) of outlier events such as historic fires within the watershed should be included when developing debris loading estimates from historic

records. NRCS (2016), Los Angeles District (USACE 2000), and Los Angeles County (2006) provide guidance on methods for modeling debris yields based on recent fire severity.

(4) Extensive construction, strip-mining operations, intensive agricultural use, and timber cutting operations are only a few examples of land uses that can have a profound local effect on sediment production, and thus determine the type of sediment control provided by a debris basin that is necessary. Formulation of a sediment control plan and the design of associated engineering works depend to a large extent on local conditions.

(5) The following conditions are peculiar to the Los Angeles area but provide insight into sediment yield debris basin design considerations for other areas:

(a) Phenomenal urban growth in the desirable land area of the lower alluvial fans makes selection of debris basin locations more difficult.

(b) Extensive and relatively frequent brush and forest fires increase sediment yield production.

(c) Hot, dry climate over a large portion of the year, which inhibits vegetative growth, increases sediment yield potential.

(d) Sudden torrential rainfall on precipitous mountain slopes during a short rainy season creates conditions for increased sediment yields.

(e) Unstable soil conditions subject to voluminous slides when saturated create conditions for debris flows.

h. Debris Deposition.

(1) Debris deposition patterns are an important consideration in basin design. Initially, flow entering the basin is 3D; however, the rapid deposition of sediment typically results in 1D flow patterns within the basin. Therefore, 1D and 2D numerical models such as HEC-RAS and AdH are usually suitable for design purposes and to investigate depositional patterns in the basin. Recent developments in HEC-RAS include the ability to model non-Newtonian debris flows.

(2) The inflowing water-sediment mixture will not expand instantly. Deposition will occur quickly for larger material, including sands and gravels, starting near the basin inlet at the slope break and flow area expansion. Deposition of larger size material will first fill the channel under the expanding jet until the loss in conveyance causes the inflow jet to deflect to one side or the other. Consideration of debris ramps building up against a shallow sloped embankment must be made and possibility mitigated with steeper side slopes. Multiple inlet chutes at the upstream end of the basin can be used to accommodate high-inflow sediment loads and prevent excessive streambed changes upstream of the debris basin.

(3) In a sediment-filled debris basin, when the delta deposits extend to the embankment, the spillway will define the pivot point for the deposited material slope. The material slope is a primary factor in determining the volume of sediment that can be stored.

(a) Since delta deposits extend upstream at some slope greater than horizontal, a fully filled debris basin stores more sediment than water. This can be an important consideration in estimating the amount of sediment than can be trapped in a debris basin.

(b) For preliminary design, the delta topset slope can be estimated to start at the spillway or conduit control elevation with a slope roughly equal to about 50% of the original streambed slope (USBR 1987).

(c) The Los Angeles County Sedimentation Manual (2006) illustrates the debris deposition as a cone slope (Figure 8-70) with the design cone slope set at one-half of the average natural slope of the stream, not to exceed 5%. The likelihood of momentum overflow, when a significant amount of sediment overflows the spillway before the basin is full, is reduced if the cone slope is limited to a maximum of 5% and the level capacity is large enough to accommodate at least 50% of the debris event.

(d) Because a wide range of debris slopes can occur under different conditions, adopting a prudent design limit may be best practice. Numerical modeling may also inform the topset cone slope estimate during a detailed sedimentation study.

(4) Determining the TE of the basin is necessary to evaluate the capability of the basin to capture the inflowing sediment load. The TE of the basin can be estimated using observations from similar basins in the region or with empirical relationships such as those previously described in paragraph 8-4. Numerical sediment models such as HEC-RAS may also inform trap efficiency estimates.

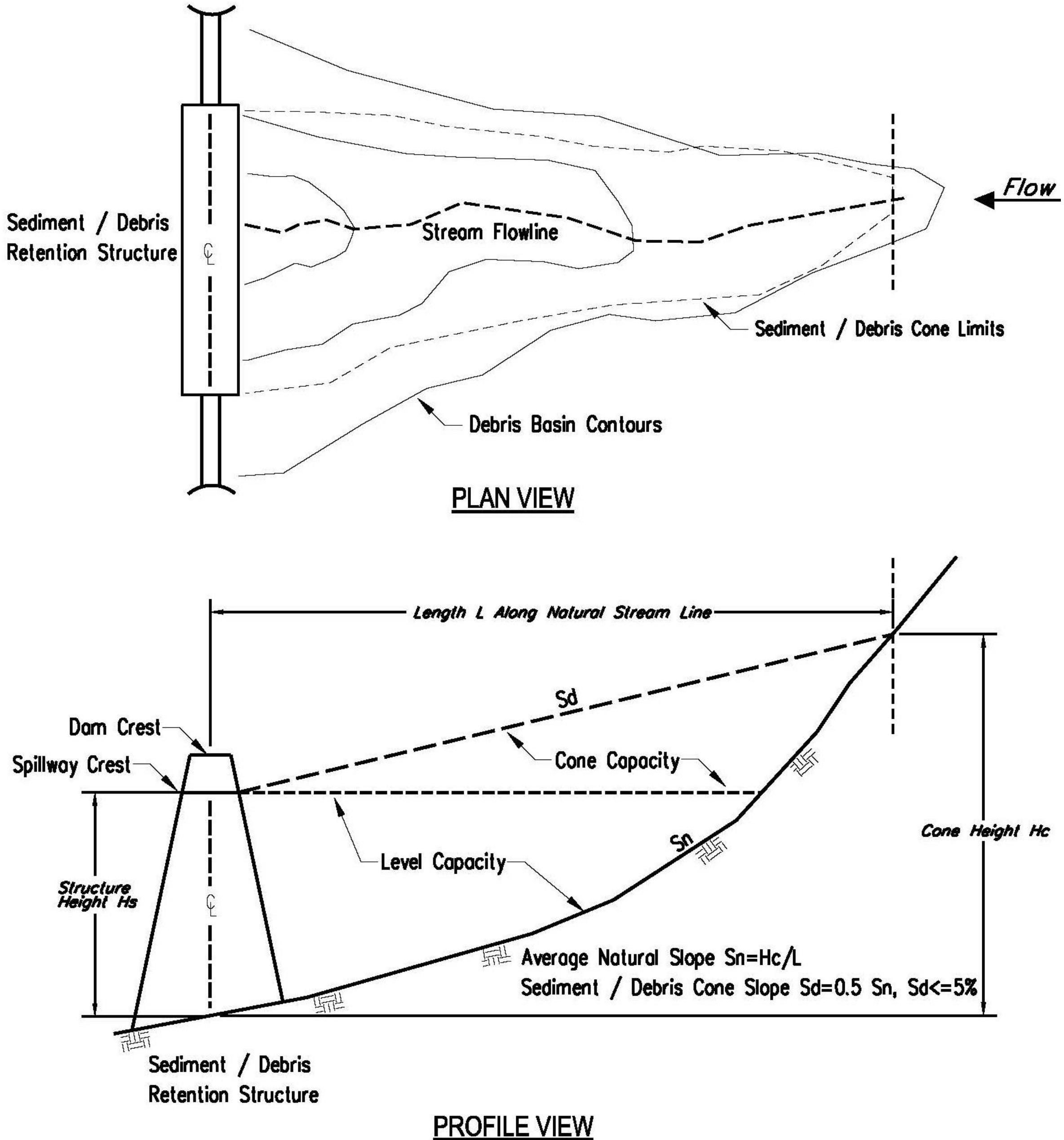


Figure 8-70. Schematic of sediment/debris basin parameters

i. Debris Basin Design and Geometry Considerations. The following debris basin design and geometry considerations are derived from practical experience developed by the Los Angeles District as a result of constructing debris basins in the desert southwest. The design criteria for debris basins in the Los Angeles area should be used only for general guidance because of large differences in geology, precipitation patterns, land use, and economic justification in different parts of the country.

(1) Embankment. An embankment is designed based on site topography to meet debris storage objectives. The embankment is usually U-shaped in plan, with termination at the hillside on each end. Construction using site material is preferred to minimize cost.

(a) The embankment height is determined from the desired flood conveyance within the debris basin for the maximum debris deposition condition. The height of the spillway crest and embankment are established in conjunction with any upstream excavation to provide the design volume for sediment deposition below the spillway crest. The maximum debris condition is determined for the selected design event plus any antecedent debris loading condition to reflect maintenance policies.

(b) Embankment height is calculated using a top slope of sediment into the basin and the distance from the spillway to the end of embankment. Height should include velocity head as well as a risk-based design provision for freeboard. The embankment side slopes should include adequate slope protection to ensure slope stability.

(c) An option to reduce embankment size may be feasible using a flow-by basin, which is intended to pass an extreme post-fire event condition over the entire embankment in addition to the spillway. A designed overflow embankment should include a detailed study to determine overtopping areas and slope armoring requirements along with adequate toe protection.

(2) Spillway and Outlet.

(a) The spillway is designed to ensure safe conveyance of the design flood and larger extreme flood conditions that consider the post-fire hydrology. There are numerous alternatives for the design of the spillway. Some spillway alternatives commonly used include a broad-crested weir, labyrinth, side channel, or stepped spillway.

(b) Analysis should be conducted on the design alternatives to maximize the hydraulic performance of the spillway. For example, the Los Angeles District has employed converging chute spillways on projects in Nevada. These pass the 1% annual chance exceedance event as well as the probable maximum flood (PMF) into a concrete spillway chute, which transitions into a concrete channel downstream.

(c) A debris barrier, which can be constructed using bollards or similar, may be necessary to limit debris from entering the spillway and exiting into the downstream channel. These barriers are designed for hydrodynamic forces and debris loading. The Los Angeles County District of Public Works have set such barriers at a 6-foot distance from the spillway with 4-foot on-center spacing between members. Actual debris barrier geometry should be evaluated for the site based on debris material size, downstream channel properties, and basin flow characteristics.

(d) An outlet drain is preferred to prevent water storage. The outlet drain is usually set as a perforated vertical intake structure with a lower drain pipe perpendicular to the embankment. The Los Angeles County District of Public Works recommends setting the top of the inlet at 1 foot above the design water surface elevation and a bottom drain sloped greater than 5% to prevent siltation. Normally, drains are designed to be at least 36 in. diameter.

(e) Case Study 8E (Appendix N) includes a general design plan for a debris basin. Note that the case study is for an unburned watershed condition. Adjustment of the fire factor used in the case study is necessary to evaluate debris basin size for a burned condition. The basin shape, the inlets, and the outlet should be located so that the debris completely fills the basin before debris discharge occurs over the spillway. Case Study 10A (Appendix N) includes a discussion of debris flow and observations.

8-15. Spillway and Embankment Erosion Assessment. Erosion of rock and soil (earth) are major areas of concern when dealing with earthen embankments, unlined spillways, and other features of a water control project. Many embankment (dam and levee) projects were not designed to be overtopped by a storm event, so any amount of overtopping flow becomes a concern. Unlined spillways are designed to experience flow and are usually expected to suffer some erosion damage; erosion becomes a dam safety issue if it becomes so extensive that it destabilizes structures, or if it enlarges or breaches through the hydraulic control section, thereby allowing an uncontrolled release of the reservoir.

a. Purpose.

(1) The analysis of erosion is a complex topic, requiring a comparison of the hydraulic forces produced by the flow and the resisting physical properties of the rock or soil. Both can be difficult to characterize, leading to significant uncertainty. EM 1110-2-1602 provides guidance for energy dissipation and downstream channel protection for outlet works to prevent channel degradation. This guidance also applies to channels downstream from spillways. The degree to which project features are damaged by extreme events and the expression of project performance risk is a difficult, but necessary, task to inform risk-based decisions.

(2) Erosion of rock and earthen materials will occur at USACE constructed projects. The task of the erosion assessment is to identify the degree of damage related to frequency of occurrence. The erosion assessment should include the identification of critical performance thresholds for project features. The outcome of the analysis will inform consequence analyses and risk-based project decisions.

b. Process.

(1) Once the dam or levee has begun to overtop, or the spillway has started to flow, in general, the most erosive flow occurs on the downstream slope (Figure 8-71). Hydraulic attack is normally highest on the downstream slope, and the slope itself can make it easier to dislodge and transport particles. On dams that have been overtopped by floods, severe erosion has often been observed to begin where sheet flow becomes turbulent flow. Erosion can also initiate where flow encounters an obstacle or discontinuity, such as a structure, trees, shrubs, groins, bare patches of earth, or a change in embankment slope. Figure 8-71 shows erosion locations.

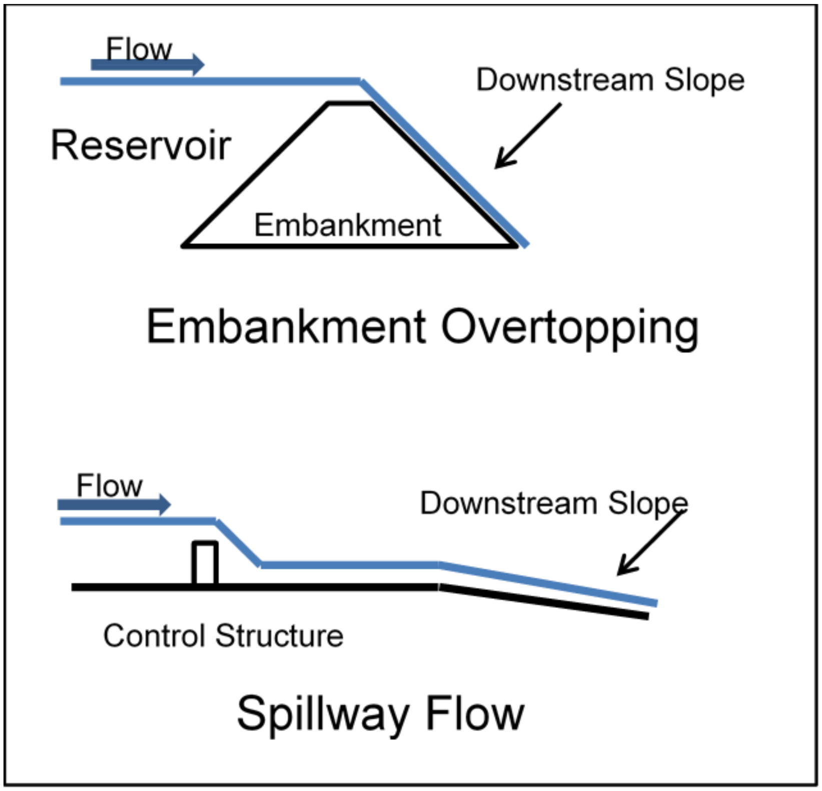


Figure 8-71. Downstream slope locations for embankment overtopping and spillway flows

(2) Erosion of an embankment or spillway occurs in four distinct phases. These phases are:

(a) Surface erosion (removal of vegetal cover or other protective layer, such as riprap).

(b) Concentrated flow erosion (headcut formation).

(c) Erosion progression (headcut advance).

(d) Breach formation.

(3) There are analytical methods and numerical models that simulate these erosion processes and that can be used to assist in the determination of significant impacts or failure of the dam due to overtopping or a spillway flow event. It should be noted that embankments constructed with cohesionless material may not require headcut formation to complete the breaching process.

c. Hydraulic Factors.

(1) Shear Stress.

(a) Shear stress can be used to determine if a spillway or embankment will erode due to the water flowing across it. This calculation is also important when it is desirable to determine if flow along the toe of an embankment or, in some cases, levees would be events of concern. Shear stress, τ_b, in an open channel can be calculated using:

$$\tau_b = \gamma R_b R S_e \qquad \text{Equation 8-8}$$

where:

τ_b = normal shear stress acting on the bed
S_e = energy slope
R_b = hydraulic radius of the bed
γ = the unit weight of water

(b) Once the shear stress for a given flow in a system is known, then it is possible to compare it with the critical shear stress for a material, and a determination can be made for the erodibility of a material. This method can be used for both cohesive and noncohesive materials. Figure 8-72 (Briaud 2008) illustrates one method proposed for erodibility based on either shear stress or average velocity.

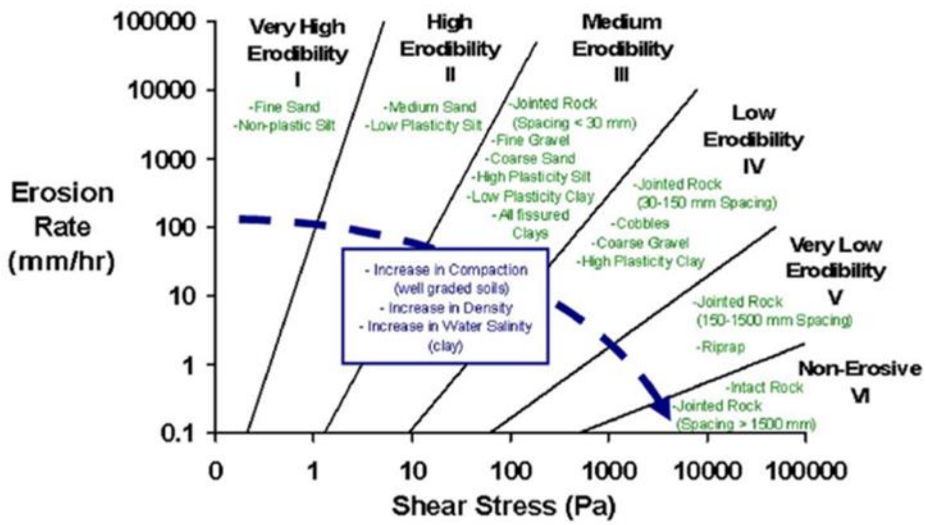


Figure 8-72. Proposed erosion categories for soil and rocks based on shear stress (from Briaud 2008)

Used with permission of ASCE, from Journal of Geotechnical and Geoenvironmental Engineering, Briaud, J.L., Vol. 134, ed. 10, 2008; permission conveyed through Copyright Clearance Center, Inc.

(2) Stream Power.

(a) Although detailed hydraulic studies should be performed to estimate stream power if erosion becomes a critical issue, some simplifying conservative assumptions can be made to determine stream power for initial screening evaluations. Bagnold (1966) introduced the stream power concept for sediment transport based on general physics and defined stream power as the power per unit bed area which can be used to transport sediment. For flow down a slope, the rate of stream power per unit of surface area (P) is a function of the flow depth, flow velocity, and the energy slope:

$$P = \gamma U h S \qquad \text{Equation 8-9}$$

where:

γ = unit weight of water
U = flow velocity
h = water depth
S = hydraulic energy grade line slope

(b) The rate of energy dissipation is small as the flow just comes over the crest and increases as the water velocity increases. The analysis of erosion stability is performed at the location where the value of energy dissipation is the highest. The energy slope is assumed to be approximately equal to the bed slope and flow depths are taken to be equal to the normal depth computed for steady-state flow conditions.

(3) Rock Erosion.

(a) The analysis of rock erosion is a complex topic, requiring a comparison of the hydraulic attack produced by the flow and the resisting physical properties of the rock. The most common approaches to the problem today rely on technologies that were originally created for the mining and excavation of rock. Barton's Q-System (Barton et al., 1974) was developed for the characterization of rock for tunneling activities in mines. Kirsten (1983) adapted this approach to establish a ripability index that helped the excavation industry determine the appropriate equipment needed to rip a specified rock. The primary rock properties determining the index are the joint alteration, joint roughness, joint orientation, compressive strength, and size of individual rock blocks.

(b) The ripability index was adapted for the analysis of soil erosion and described as a headcut erodibility index by Moore et al. (1994) and Temple and Moore (1997). The index was used to establish both thresholds and rates for headcut advancement in various soils.

(4) Headcut Erodibility Index.

(a) The concept of using a rock mass index to correlate with the power it would take to remove the rock was originally developed by Kirsten (1983) to characterize the ripability of earth materials using mechanical equipment. This was extended to examine the removal of soil and rock by flowing water, and at that time the term "erodibility index" was coined. This index was correlated empirically to the erosive power of flowing water, or the energy rate of change, termed "stream power." Data from the performance of unlined spillways in both soil and rock were used to calibrate the method for erosion potential. Thus, this method can also be used for either soil or rock, but this section focuses on its use for estimating rock erosion.

(b) The stream power-headcut erodibility index method can be used to estimate the likelihood of initiating rock erosion. The headcut erodibility index represents how erodibility of the foundation material. Relatively simple to calculate, it can be used for an initial evaluation. The stream power represents the erosive power of the overtopping flows and is much more complicated to compute.

(c) This method provides an indication of the likelihood that erosion will initiate, but if so, additional judgment is needed to evaluate how quickly erosion will occur and whether it will progress to the point of initiating a failure mode (spillway breach, dam instability, or dam breach). This requires evaluating the likelihood of erodibility at various depths and locations. The duration of overtopping flows should also factor into the judgment on the potential for reservoir breach.

(d) The headcut erodibility index, K_h, is calculated as follows:

$$K_h = M_s K_b K_d J_s \qquad \text{Equation 8-10}$$

where M_s is the mass strength, usually defined as the unconfined compressive strength (UCS) for rock (expressed in MPa) when the strength is greater than 10 MPa, and $(0.78)(UCS)^{1.05}$ when the strength is less than 10 MPa.

(e) K_b defines the particle or fragment size of rock blocks that form the mass, which can be determined from joint spacing or rock mass classification parameters. The simplest and most straight forward relationship is $K_b = RQD/J_n$, where J_n is a modified joint set number and RQD is the rock quality designation (USDA 2001).

(f) K_d is the interparticle bond shear strength, and is usually taken as J_r/J_a, where J_r and J_a are the joint roughness and joint alteration numbers, respectively, based on joint surface characteristics defined by Barton's Q-system. Typical reference values are available (USDA 2001).

(g) Plucking and cyclic loading introduced by turbulence, most probably the dominant processes in scour of earth materials (Briaud et al., 1999), act in addition to shear stress to scour earth material. Materials mainly held together by gravity bonds scour principally because of fluctuating forces developing over individual particles, as would be the case for cohesionless granular soil. The fluctuating forces pluck the soil particles out of their positions of rest. In the case of uniform cohesive soil, the cyclic loading introduced by the plucking forces weakens the soil, resulting in scour as the soil gradually yields (Colorado Department of Transportation Report No. CDOT-DTD-R-2000-9).

(h) The relative shape and orientation of the blocks is accounted for by the J_s parameter. This represents the ease with which the water can penetrate the discontinuities and dislodge the blocks. Tabulated values for J_s are available (USDA 2001).

(i) More information on the procedure to estimate the various parameters to estimate K_h are found in National Engineering Handbook 628 Chapter 52 (USDA 2001) and Chapter D-1, Erosion of Rock and Soil (USACE 2019).

d. Erosion Potential.

(1) The chart shown in Figure 8-73 combines the headcut erodibility index with the stream power estimate and can be used to estimate the erosion potential. The dashed black line is the initial erosion threshold proposed by Annandale (1995). Annandale (1995) reviewed about 150 field observations from spillway channels and plunge pools to develop a curve defining the threshold for erosion as a function of applied stream power and the headcut erodibility index. Based on stream power (y-axis) and headcut erodibility index (x-axis), a best-fit line separating cases of erosion and no erosion was determined. It should be noted that the selected "No Erosion" threshold had some points above the line and that other researchers have proposed thresholds for specific materials that differ from those shown in Figure 8-73.

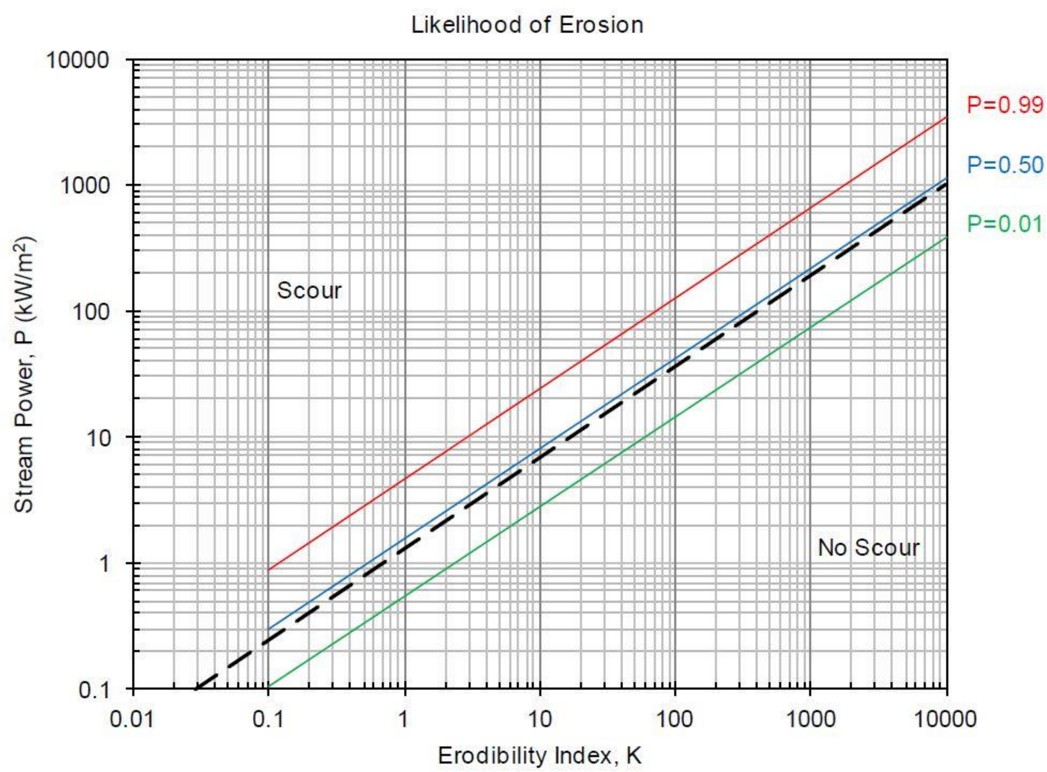


Figure 8-73. Comparison of the Annandale (1995) and Wibowo (2005) threshold lines (USACE 2019)

(2) Figure 8-73 (USACE 2019) also shows the logistic regression results obtained by Wibowo et al. (2005), using the same data analyzed by Annandale (1995). The upper (blue line) represents a 99% chance of erosion initiating. The lower (black line) represents a 1% chance of erosion initiating, and the middle (red line) represents a 50% chance of erosion initiating. The likelihood of erosion initiation can be interpolated between these lines.

(3) If erosion is predicted, but the character of the rock or hydraulic characteristics changes with depth, then an iterative procedure can be employed whereby the rock is assumed to erode to a certain depth, and then the stream power and the headcut erodibility index are recalculated for the new geometry and geologic conditions, and re-plotted on the empirical chart. Due to uncertainties in obtaining input parameters, it is often necessary to look at a range of conditions. For the analysis of a jet plunging from the crest of a concrete arch dam onto downstream canyon abutment walls, the jet stream power level will vary at different abutment impact elevations.

(4) Judgment is required when applying these methods. The results can be sensitive to Kb, which is somewhat difficult to assess for rock. In addition, materials will be more easily eroded on an abutment slope where there are more degrees of freedom for movement than in the bottom of a plunge pool where only the top of rock blocks are exposed. Cross jointing, if not present, can also increase the erosion resistance of the rock. These issues are not directly accounted for in these methods. Key block theory can be helpful in these situations to identify whether there are potentially removable blocks. A combination of the erodibility threshold graphs produced by Annandale and Wibowo can provide a range for the likelihood of embankment and spillway headcut progression.

e. Placed Rock Erosion.

(1) If it is necessary for the risk analysis to account for the ability of an embankment or spillway to sustain some small level of erosion without failure, the analysis should begin with a consideration of whether the embankment slope protection will fail.

(2) If the downstream slope protection is cohesionless and has d_{50} larger than 4 in., the chart from Frizell et al. (1998), shown in Figure 8-74 (USACE 2019), can be used for guidance on the flow at which erosion would initiate. In the figure, S is the embankment slope (V/H), and C_u is the coefficient of uniformity (d_{60}/d_{10}), which can be taken as about 1.8 for typical clean uniform cobbles or boulders as an initial estimate if actual values are unavailable. Note that the units are metric; 1 foot of overtopping corresponds to a unit discharge of roughly 2 $ft^3/s/ft$ or 0.2 $m^3/s/m$. Site data points plotting on the lines represent about a 20% probability of erosion beginning (not the probability of the dam breaching). Site data points plotting further below each line would indicate increasing likelihood of erosion.

(3) It is critical, however, to understand that this figure was developed from experiments on carefully placed, uniformly sized angular riprap in the ideal conditions of a straight-sided flume, not on a dam embankment with irregular groins, protruding structures, etc., that would cause local disturbance of the sheet flow. Furthermore, slope protection with an infilling of finer material may behave differently because much of the flow in the experiments occurred within the riprap void space, rather than over it, which may not be possible if infilling has occurred. Additional references that have examined riprap sizing on various structure types and alignments may also be reviewed for site applicability (Abt et al., 2016; USBR 2015).

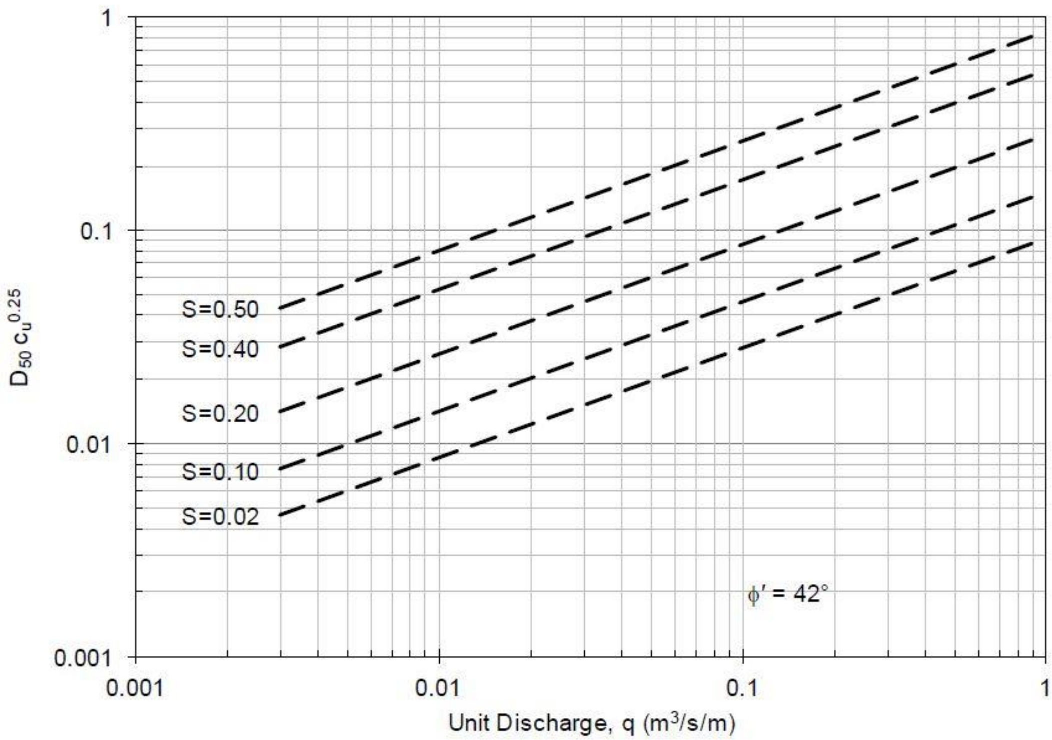


Figure 8-74. Embankment erosion initiation chart (USACE 2019)

f. Soil Erosion.

(1) Erosion of soil in embankments and spillways also requires a comparison of hydraulic attack and erosion resistance to determine whether erosion damage will occur and the rate at which it will progress. Multiple variables must be considered, including flow depth, shear stress, flow velocity, soil material type, geometry, armoring, and vegetation.

(2) A dense cover of turf-type grass, as seen on many dams in the eastern United States, can provide excellent protection against high-velocity sheet flow until the cover is removed, assuming the growth is even and well established. When the cover is removed or sheet flow is disrupted, concentrated flow can form, initiating the headcut formation process, and all benefits of cover are lost.

(3) Generally, the most erosion-resistant soils are plastic clays. The most erodible soils are non-plastic silts and sands. Removal of particle size is dependent on specific velocities required for transportation of various sized coarse material. For a given particle size, the slope has a major effect on the flow required to initiate erosion for cohesionless materials.

(4) If the downstream slope is composed of cohesionless particles with no vegetal cover, little overtopping flow would be required to initiate erosion, and it may be reasonable to assume that erosion would begin at the onset of overtopping or spillway flow. If the soils present contain high clay content, then it would take significantly more time or flow to initiate and progress the erosion process to a point where the safety of the system becomes a concern.

(5) Given that erosion will initiate at a specific flow depth for a system, duration should be considered as the next important variable in determining progression to the dam crest or spillway control. Once a headcut has initiated, the material properties comprising the embankment or spillway floor become important in determining the rate and extent of erosion. Erosion models simulate the processes of headcut formation and headcut advance that can occur after failure of the slope protection material.

(6) Estimation of soil erodibility involves many complex processes, especially for cohesive soil types (Briaud 2008). For that reason, it is recommended the erodibility be measured in the field. One parameter to estimate soil erodibility is the detachment rate coefficient K_d. Reference USDA (1997, 2001) for more information on measuring or estimating K_d and how it can be used to estimate erosion rates. K_d is often estimated from the slope of the line with erosion rate plotted on the y-axis and applied shear stress on the x-axis (Briaud 2008) allowing the various erosion measuring apparatus to be used to measure K_d.

g. Erosion Process.

(1) In general, the most erosive flow occurs on the downstream slope, where the velocity is highest and where the slope makes it easier to dislodge particles and move them away. On embankments that have been overtopped by floods, severe erosion has often been seen to begin where sheet flow on the slope meets an obstacle such as a structure, a large tree, or the groin; where a break in slope occurs; there is a change in material type; or where vegetation is not uniform or bare soil creates local turbulent flow.

(2) Based on the four-phase erosion previously stated, areas where vegetation has been removed or sparse, the erosion will proceed to attack the soil directly until a headcut or overfall is formed. Erosion generally continues in the form of headcutting, in an upstream progression of deep eroded channel(s) that can eventually reach the reservoir. For embankments made from cohesionless material, a headcut may form or concentrated flow will erode a gully more uniformly.

(3) In the case of an embankment dam, erosion of the soil comprising the embankment can ultimately lead to dam failure. For cohesive soils, the failure mechanism is typically headcut initiation and advance. A small headcut is typically formed on the downstream slope of the dam and then advances upstream until the crest of the dam is breached. For cohesionless soils, the failure process typically initiates as a result of tractive stresses from the flow removing material from the downstream face, but then progresses as headcut advance once a surface irregularity is formed. Predicting whether breach initiation and formation will occur can be a complicated procedure.

(4) Pavement on the crest may be of some value in slowing uniform erosion of cohesionless materials once the gullies reach the crest, but should not be expected to affect initiation. Depending on the depth of the headcut, the headcut can actually undermine pavement, leading to a mass wasting of the pavement material as cantilevered sections collapse into the headcut.

(5) If a parapet wall is provided on the embankment dam crest across the entire length of the dam, dam overtopping will initiate when the reservoir water surface exceeds the elevation of the top of the parapet wall. Parapet walls are typically designed to contain waves that might overtop the dam and may need to be evaluated for a sustained water load (considering instability of the wall and blowout at the toe of the wall for loads part way up on the wall). If a parapet wall overtops, the impinging jet from overtopping flows may erode the dam crest and undermine the parapet wall. If the parapet wall or a section of the wall fails, the depth of flows overtopping the dam crest will be significant and breach may occur quickly.

h. Application.

(1) Spillway and embankment erosion analysis studies are typically conducted as part of new project analysis, O&M studies, and dam safety studies. Specific study goals are highly variable and directly influence the conducted analysis. The analysis methodology is relatively new and will evolve. Available references and experts should be consulted to determine the best available analysis techniques and programs.

(2) Due to the widely variable site-specific goals and objectives, acceptable spillway and embankment performance can be difficult to define. USACE design objectives would often include provisions for spillway channel stability measures for project operation during normal events and evaluation of project performance during extreme flow events.

(3) Earth and rock materials are subject to damage during extreme flow events. The degree to which project features are damaged by extreme events and the expression of project performance risk is a difficult, but necessary task to inform risk-based decisions. The outcome of the analysis should include identification of critical performance thresholds for project features.

(4) The fate of eroded material should also be considered. At many locations, the sediment transport capacity within the spillway channel is substantially larger than the downstream channel. Although spillway is likely associated with an extreme event, spillway channel erosion and material impact downstream should be considered.

(5) Three case studies are presented in 8F, 8G, and 8H (Appendix N). Case Study 8F represents a typical USACE project, while Case Studies 8G and 8H are included to illustrate the broad range of challenges in applying spillway erosion at specific projects.

(6) Hydrologic. Analysis is typically based on normal and design event flow hydrographs. A detailed evaluation that examines a wide range of flow events is usually required. A range of events should be selected to correspond with analysis outcome, such as the onset of spillway damage, headcut advancement rate, and the final headcut location.

(7) Geometry. Spillway channel survey data are required to allow analysis of hydraulic flow parameters. For existing projects, spillway surveys must be of sufficient quality to illustrate existing headcut and slope break points. Typical guidance for hydraulic modeling should be followed when assembling site geometry. Typical analysis has employed 1D hydraulic analysis.

The presence of site-specific conditions that could contribute to additional stress on materials such as sharp channel bends should also be evaluated.

(8) Material Properties. Determination of critical material properties follows procedures previously discussed. Assignment of material properties and estimation of erodibility values is often difficult. Site variation as well as analysis techniques should be considered when selecting values.

(9) Sensitivity Parameters. A sensitivity analysis is recommended to address parameter uncertainty. Hydrologic and hydraulic sensitivity analysis should follow risk-based procedures included in EM 1110-2-1619. Hydraulic sensitivity analysis should include a variation of roughness values and consider geometry factors when model outcome is especially sensitive to channel slope break points and existing headcut advancement. Material properties are often difficult to determine with a high degree of accuracy and should be included in a sensitivity analysis. The outcome of the sensitivity analysis is a component of a risk-based assessment of project performance for a range of future conditions.

i. Consideration for Return of Spillway Discharge to River.

(1) Emergency spillways are usually designed for infrequent use and flow is left to seek a path of return to the channel. Ensuring that the path is as long and tortuous as possible can help reduce flow energy before it re-enters the river. Two emergency spillways, Saylorville and Grapevine, experienced moderate flow located near major metropolitan areas. Although the discharge peaked at only 10% of the spillway design discharge and flow continued for a limited duration, extensive erosion of the land occurred as flow sought a return path to the channel. At Grapevine Lake, the flow episode produced dramatic enlargement and downcutting of the creek channel over a 3,000-foot length (USACE 1994).

(2) The spillway for Saylorville Lake, located on the Des Moines River about 12 miles north of the city of Des Moines, Iowa, was designed as an emergency spillway. The first flow event occurred in 1984 with a flow of 17,000 cubic feet per second (cfs) and a depth of flow over the spillway crest of 5.3 feet. This flow event lasted for 14 days, and headcutting and erosion progressed at the rate previously calculated. However, subsequent design changes and resultant loss of reservoir capacity changed the spillway flow frequency significantly. A detailed review of the spillway bedrock profile determined that a questionable layer of shale existed 13 feet below the spillway. A series of rock anchors and a concrete cutoff was constructed to prevent undercutting of the spillway structure (McCully and Mech 1993).

8-16. Staged Sedimentation Studies for Reservoir Projects. Staged sediment studies and the associated work plan (SSWP) are discussed in detail in paragraph 2-5. Once study objectives have been identified, it is up to the engineer to select an appropriate evaluation procedure. ER 1110-2-8153 requires that a sediment impact assessment be prepared for all projects. A staged sediment studies approach should be followed in which contingency factors are assigned and revised as more data and analysis are available to decision-makers. Typical components of the staged study, as applied to reservoir sediment studies, are as follows:

a. Stage 1 – Sediment Impact Assessment (Reconnaissance):

(1) Routine support for existing projects. If sediment problems are determined to be negligible, the sediment impact assessment can be the final stage.

(2) Examples include:

(a) Routine support for existing reservoir projects.

(b) Aerial photo analysis and extrapolation of reservoir delta progression.

(c) Real estate and groundwater impact in reservoir headwaters.

(d) Downstream channel degradation predictions from extrapolating trends.

b. Stage 2 – Detailed Sedimentation Study (Feasibility):

(1) Sediment impact assessment predicts a sedimentation problem.

(2) Similar, existing project is experiencing sedimentation problems.

(3) Detailed numerical modeling.

(4) Examples include:

(a) Sediment bypass/flushing studies.

(b) Delta future progression and localized impacts.

(c) Reservoir operations change.

c. Stage 3 – Feature Design Sedimentation Study (Preconstruction Engineering and Design):

(1) Extension of the Detailed Sedimentation Study to increase accuracy.

(2) Examples include:

(a) Turbidity and density current sediment passage.

(b) Redesign of reservoir outlet works for sediment flushing.

<u>8-17.</u> <u>Report Requirements</u>.

a. Basic Background Information:

(1) Basin and site maps.

(2) Project purpose and life.

(3) Design details for dam.

(4) Reservoir storage allocations.

(5) Stream bed profiles.

(6) Rationale for study area boundaries.

(7) Survey and sediment sampling data collection summary.

(8) Sediment properties of channel.

b. Analysis of Reservoir and Watershed Parameters:

(1) Trap efficiency and volume depletion.

(2) Specific weight of deposits.

(3) Estimated depletion of reservoir volume by pool elevation.

(4) Estimated elevations for real estate requirements.

(5) Predicted effect of sediment deposits on future river stages upstream.

(6) Turbidity impacts in reservoir.

(7) Bank erosion potential in reservoir.

(8) Density current potential.

c. Analysis Downstream from the Dam:

(1) Selection of geometry.

(2) Hydraulic roughness.

(3) Modified stage duration curve at the dam.

(4) Degradation of the channel bed.

(5) Predicted future tributary degradation.

(6) Bank erosion rates.

d. Geomorphic Change (planform, slope, sinuosity, etc.)

(1) Sediment inflow.

(2) Bed material gradation.

(3) Tributary data.

(4) Hydrologic data.

e. Analysis Upstream from the Dam:

(1) Pool elevation frequency curves.

(2) Delta location, backwater influence zone, and progression rates.

(3) Reservoir and river geometry.

(4) Sediment properties of bed material.

(5) Top of rock profile.

(6) Shoreline erosion locations and rates.

(7) Water inflow hydrographs.

(8) Inflowing sediment concentrations and properties.

(9) Operating rule curve.

(10) Specific weight of deposits:

(a) Particle size.

(b) Consolidation (factors that influence).

(11) Elevation-capacity curve.

(12) Topics not addressed by numerical sediment movement models.

(a) Possibility of turbidity.

(b) Possibility of a density current.

Chapter 9
Modeling

9-1. Purpose and Philosophy of Modeling.

a. Philosophy.

(1) Models start as thought experiments and, depending on the situation, they may grow in complexity to include conceptual frameworks, desktop calculations, numerical simulations, and physical scale modeling (Morris and Fan 1998). Real-world or prototype hydrologic systems are complex. Models simplify processes and will not reproduce natural systems exactly. In engineering analysis, this uncertainty is offset by applying informal or formal safety factors to convert approximate modeling results into acceptable design parameters. The literature on modeling theory and practice is vast. This chapter attempts to distill accepted and applied (rather than theoretical) methods, applicable to sediment and morphological modeling, for the practitioner. Figure 9-1 illustrates chapter content.

(2) A model is a representation of a system or phenomenon that simplifies the scale and/or time of the prototype (the actual system as it exists). In engineered systems, models simplify reality to explain and predict complex physical processes to support management decisions. Sediment models generally fall into two categories: (1) physical models, usually constructed at a reduced scale in a hydraulics laboratory, and (2) mathematical models (numerical and analytical representations of rivers and reservoirs). As computing power grew over the last half of the twentieth century, mathematical models became more common and more detailed, used in conjunction with physical models, or often, replacing them.

b. Tradeoffs in Model Selection and Incremental Uncertainty Reduction.

(1) Models simplify the natural and technological world, identifying sensitive processes and predicting system responses to management alternatives. Therefore, the first question any modeling study encounters is "How much can we simplify this system?" Models are often selected implicitly or accidentally, defaulting to familiar tools or historical approaches. However, the actual process of model selection, like most economic or ecological analyses, requires negotiating tradeoffs. For example, increasing dimensionality (such as selecting a multidimensional model over a 1D model) decreases the theoretical simplifications in a model, a more detailed model costs more. Model selection optimizes these tradeoffs to select the appropriate approach. Gibson (2013) identified at least eight tradeoffs that inform model selection (Table 9-1).

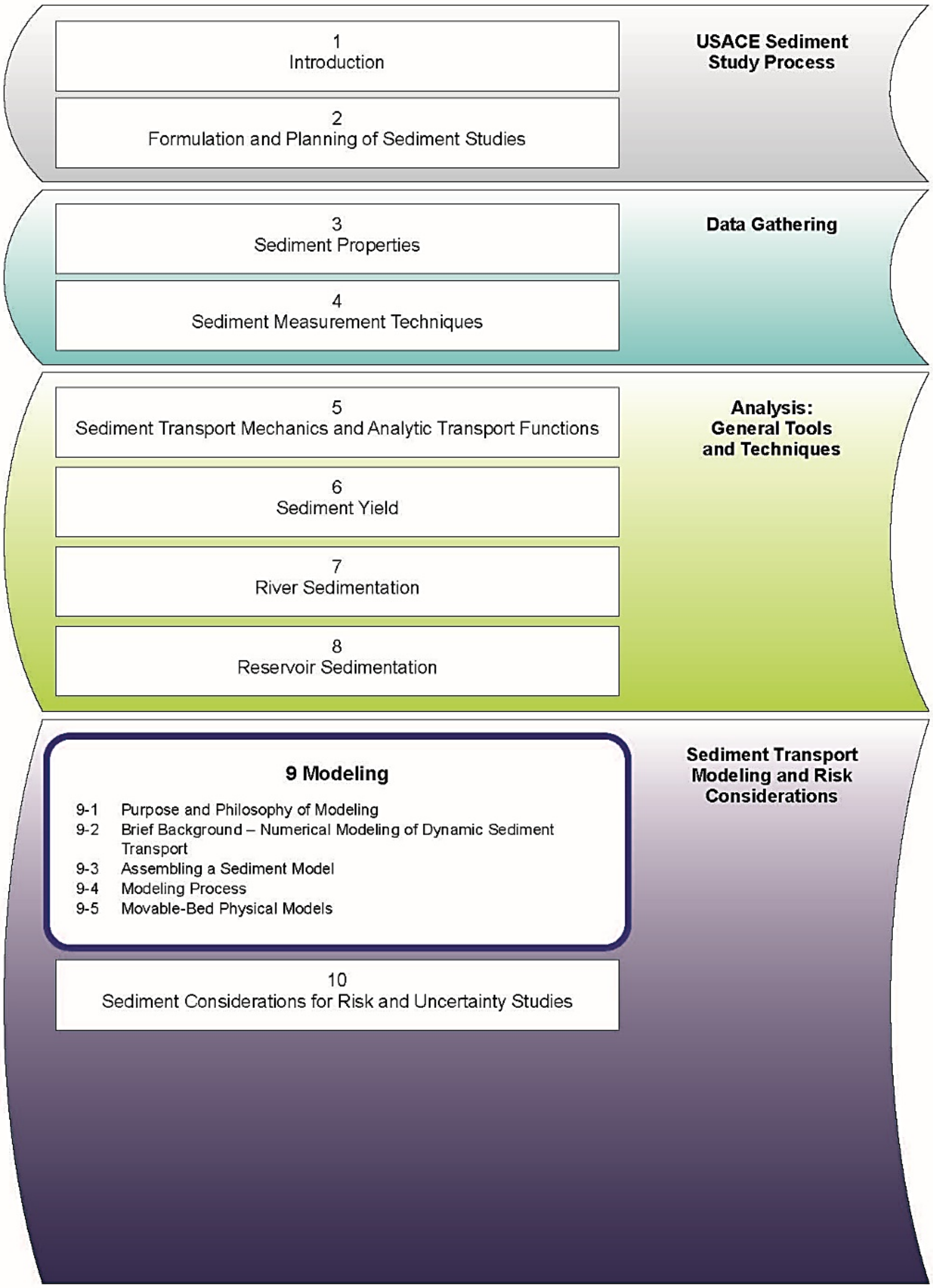


Figure 9-1. Chapter 9 content and general document structure

Table 9-1
Eight Tradeoffs in Model Selection from Gibson (2013)

1	Quality of Fit	Parsimony
2	Structural Validity	Parameter Availability
3	Precision	Cost
4	Extent	Granularity
5	Empiricism	Mechanism
6	Complexity	Transparency
7	Customization	Credibility
8	Heuristic	Predictive

(2) The precision-cost tradeoff is usually the most important. Adding model complexity by modeling more detailed process and increasing resolution usually increases project cost. Increased model costs should purchase precision in model prediction, decreasing uncertainty. Therefore, the model selection should consider "whether the improved quality of the information provided by a more detailed model is worth the added costs of its development and use" (Smith and Vaughan 1980). Freeze et al. (1990) compare the actual study cost of each level of model complexity against the probabilistic costs of failure. They argue that the optimal level of analysis is a tradeoff between the incremental cost of model detail and the potential costs of project failure because of process uncertainty.

(3) Public agencies apply project resources to reduce uncertainty. They turn to modelers to purchase increments of uncertainty reduction. Managers often call this tradeoff "buying down risk." But adding complexity to analysis can have diminishing risk reduction returns. So, the model selection process should determine a modeling and analysis plan that scales with the risks and uncertainties of the project. High-consequence projects may justify very detailed models, where increasing model complexity returns small incremental risk reduction, but large risks make the complexity worth the investment.

c. Models as Predictive Tools.

(1) Most models are designed to compare relative responses to proposed actions. Federal and state agencies commission models to generate concrete, actionable results to inform agency actions. However, the modeling process generates two products, the model, and the modeler (Gibson 2011): a predictive numerical tool and a scientist with an improved conceptual understanding of the system and a grasp on the dominant processes, sensitive parameters, and emerging properties (Kumar 2011) of the system. Careful modeling generates quantitative comparative predictions, but also builds qualitative understanding and intuition about how a river will respond to alternatives.

(2) Bredehoeft (2010) presented a holistic approach to modeling, arguing: "the model is not an end in itself, but rather a powerful tool that organizes my thinking and my engineering judgment." Models are tools that build agency understanding, and organize data to form and test hypotheses about the system. Predictive modeling should interact with scientists and engineers familiar with the system and the processes. Robust modeling practice, including forming hypotheses before simulation and interacting with scientists and engineers familiar with the system, will help modelers design the model and evaluate model results, testing their credibility against what they know about the system and the processes.

d. Model Objectives.

(1) Sediment models developed for USACE project applications should be designed to answer a specific question or a tractable set of well-defined, compatible questions from the start. Every stage of model development, including data collection, sizing the model, selecting the resolution, and designing the complexity, should have the objective question in mind. Modeling to a question will optimize data investments and model complexity, focusing project resources parameters and processes to which the proposed management alternatives are sensitive. This approach recognizes practical model limitations. Models approximate the total system behavior while emphasizing the particular process that requires USACE actions including design analysis, construction, and operation.

(2) For example, when USACE evaluates a dam removal project, the modeler must identify the precise management question to invest data and model development resources effectively. A model designed to investigate depositional impacts on FRM in the downstream channel will require different data to describe a different model configuration than a model constructed to evaluate whether a headcut could put upstream infrastructure at risk.

(3) Modelers must carefully communicate with managers to scope the specific decision(s) the model supports before investing any resources in data collection or model development, and then design the model to answer the specific question(s). Models scoped with the management questions in mind accept uncertainty in places and processes that do not significantly affect either the model outcome or the resulting management decisions.[10]

(4) Sediment models can address a wide variety of management questions encountered in USACE missions. Common sediment modeling questions include:

(a) Reservoir Life-Cycle Analysis. Sediment models can forecast long-term sediment impacts on reservoir capacity, computing decreasing water supply and increasing flood risk as sediment fills the reservoir pool. In areas and regions where new dam construction was/is more common, sediment models are also commonly applied to predict the rate and magnitude of degradation and coarsening that almost always occurs downstream of a new dam.

[10] Blair: Types of Study: (1) process study, (2) comparative studies – compare alternatives (to no alternative)

(b) Sustainable Sediment Reservoir Management Alternatives. Several management strategies are available to increase reservoir life and provide ecologically and morphologically important sediment downstream of dams (paragraph 8-13), including sediment flushing, routing, and bypass. Good "rule of thumb" and screening-level tools can rule out some of these alternatives for particular reservoirs. However, sediment models can quantify relative costs and benefits of these alternatives, once the most appropriate option is selected.

(c) Flood Risk Impacts. Sediment deposition is the most common failure mode of flood control channels. Widening a channel decreases transport and encourages deposition, requiring long-term maintenance. Deposition (Gibson et al., 2010; USACE 1996) and erosion can also complicate FRM in leveed reaches. Sediment models can help quantify these impacts. (See Chapter 10)

(d) Gravel Augmentation. Gravel augmentation manually introduces coarse substrate into the channel to slow erosion and/or provide interstitial habitat for benthic invertebrates and cover for bottom spawners (such as, salmonids and sturgeon). Sediment models can evaluate the lifespan of these projects, predicting how long the river will take to either fill them with fine material, or scour them, transporting the gravel downstream.

(e) Upstream, Impoundment, and Downstream Effects of Dam Removal.

- Many agencies have removed small dams without sediment models and without incident. However, the sediment impacts of moderate to large dams, multiple dams, and even some small dams in high-risk locations, should be modeled before removal. An interagency task force suggests the analysis that should precede a dam removal should scale to ratio of sediment trapped behind the dam to the river's average annual load and the potential severity of downstream impacts (Randle and Bountry 2015).

- Dam removal risks include careful evaluation of potential effects upstream and downstream including questions like:

  - Will downstream deposition increase flood risk or degrade habitat downstream?

  - How far downstream will these impacts be significant and for how long (Echevarria 2014; Healy 2003)?

  - How far upstream will the channel degrade (leaving perched terraces, dropping riparian groundwater levels)?

- The interagency guidelines recommend a sediment capacity analysis if the consequence of downstream impacts is medium and the reservoir impounds between 1 and 10 times the annual load. If the downstream consequences are high or the reservoir impounds more than 10 times the annual sediment load, the guidelines recommend modeling the removal.

- Dam removal models differ from many other sediment models in one important way: they cannot be calibrated. They have no historical precedent under those conditions. Simulating the past (for example, reservoir deposition) does not increase the confidence in the dam removal modeling much. Uncertainties associated with dam removal models tend to be significant. However, they can still provide helpful relative insight to evaluate alternatives (for example, staged removal vs. “blow and go”) and assess risks.

(f) Navigational and Economic Dredging. USACE applies sediment models to evaluate dredging activities. Sediment models can evaluate navigational dredging strategies, improving dredging efficiency and identifying strategic times or locations to dredge, and to simulate the fate of material suspended by dredging processes. USACE Districts have also used sediment models to predict future impacts of economic dredging and gravel mining (Shelley and Gibson 2015) and to design training structures to minimize dredging. One-dimensional models can address cumulative, long-term, longitudinal impacts of dredging and multiple navigational structures, while multidimensional and physical models may be required to evaluate local effects of particular structures.

(g) Building Bars and Wetlands for Habitat. USACE Districts are interested in building sand bars for habitat downstream of dams that have intercepted the historical sediment supply. Sediment models help evaluate potential reservoir operation alternatives to build these downstream bars. USACE modelers have used 1D, 2D, and 3D models to simulate the marsh creation potential of lateral diversions on the Mississippi (Letter et al., 2008; Dean et al., 2012; Brown et al., 2013) and main stem shoaling in response to diversions (Allison et al., 2013).

(h) Long-Term Response to Historical Source Changes. Sediment yields change over time. Walling and Fang (2003) identified a classic trend in historic sediment supply, where sediment loads tend to increase as the watershed develops modern agricultural methods and then decreases after some inflection point as soil management practices are implemented and dams built (Figure 9-2). Rivers aggrade or degrade in response to these trends, and often eventually reach a new quasi-equilibrium. It can be difficult to determine if a river, which has historically degraded or aggraded, will continue to degrade, will degrade at a similar rate, or will degrade more slowly, approaching quasi-equilibrium.

(i) Sediment models sometimes predict long-term future trends (Figure 9-2), accounting for feedbacks to determine whether historic trends will continue, slow, or reach quasi-equilibrium. For example, Sacramento District (USACE 2015c) investigated the long-term response of Sacramento/American River system to the sediment pulse from the hydraulic gold mining.

(5) USACE has applied sediment models to a wide range of management questions, including, but not limited to, those listed above. However, USACE models share an important distinction: they address a specific management question or a limited, related suite of questions. USACE models should be designed and optimized to answer the agency question from the beginning.

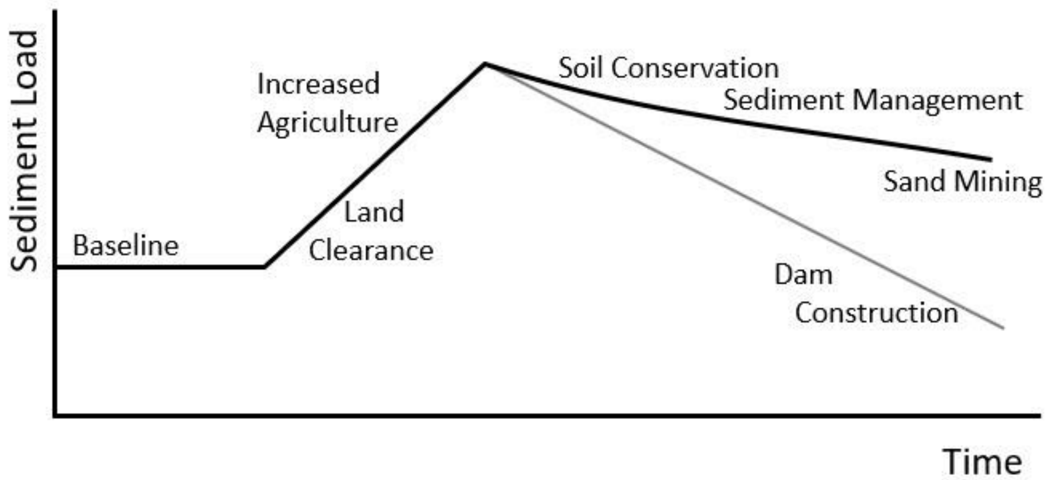


Figure 9-2. Long-term sediment load expectations

e. Developing a Conceptual Model. Modeling advances have made sediment simulations more efficient and powerful. As data becomes more available and the models get more user friendly, modelers can be tempted to start modeling immediately.

(1) Developing a careful scientific hypothesis, before modeling begins, is an important preliminary step. The modeler should organize the data and interview system specialists to identify the location and magnitude of sediment sources and sinks, to predict the processes to which the alternatives will be sensitive, and to hypothesize how the system will respond to alternatives.

(2) Models organize data to test hypotheses. Developing a sediment budget (quantifying the sediment source and sinks) and a rough hypothesis about system response before building a model will help the modeler select the right model, design the right level of complexity, and identify numerical artifacts or model errors when results diverge dramatically from the hypothesis.

f. Physical vs. Numerical Models.

(1) This chapter primarily describes numerical models. However, USACE sometimes uses physical models to meet specific project needs. Physical models are useful when other methods do not provide the necessary level of confidence for project performance.

(2) In some cases, numerical models cannot reduce project uncertainty enough. The free parameters in numerical models, divergent, nonlinear equations, and uncertain future boundary conditions associated with hydraulic structures and natural channels introduce uncertainty into numerical model results. When project costs or consequences are high, these uncertainties may be unacceptable. In these cases, the PDT may use physical models as the primary analysis method or as a complimentary tool to augment other design methods.

(3) Well-designed physical models include less inherent uncertainty than numerical models because they replicate uncertain or chaotic processes implicitly. The modeler does not have to develop equations or algorithms to account for every system response. Physical models are structured using the basic principles of similitude (Pugh 2008) to correlate model and prototype behavior.

(4) Physical model tests are generally preferred where local scour or sediment deposition could endanger the functionality of a hydraulic structure or river modification. For example, mathematical model estimates of local scour at bridge piers are deliberately conservative and restricted to standard pier shapes and configurations (Sharp et al., 2016). Since local scour equations are empirically based, physical modeling is generally regarded as the standard approach for analysis of nonstandard pier geometry.

(5) Physical models evaluate project performance and modifications to obtain the best possible design at minimum cost. Mathematical models are applicable when the sediment behavior can be predicted analytically. Mathematical models generally require more data to calibrate and verify than physical models. However, a developed mathematical model allows relatively simple means to test various modifications and design proposals. Physical and mathematical models should be used to supplement, but not replace, theoretical knowledge, good judgment, and experience.

(6) Physical models are typically limited by scaling considerations to short reaches of river where the nonequilibrium response is dominated by bedload transport and complex 3D current patterns. For example, a physical model would be an appropriate tool for evaluating local responses of the bed surface during major in-channel construction projects. Froude scaling makes them most applicable to coarse bed systems because scaling sand requires very small or very light particles. Numerical models can be applied over larger domains and longer time scales. For example, a 1D sedimentation model could be applied on a regional scale to evaluate reach-by-reach changes in bed profile over the expected life of a project.

g. Model Dimensions (1D, 2D, 3D).

(1) Numerical sediment transport models are available in one, two, and three dimensions. One-dimensional models are more common because they require less input data and computation time. However, growing computing power and multidimensional modeling advances have made 2D and 3D models more common.

(2) USACE commonly uses 1D and 2D models for engineering applications. One-dimensional models tend to be appropriate where the flow is highly channelized and closely follows the thalweg and good lateral mixing processes support 1D assumptions. One-dimensional run times also make them attractive for applications requiring long-term simulations, large spatial domains (Thomas 2014; Shelley and Gibson 2015), many alternatives, or stochastic, Monte Carlo approaches.

(3) Two-dimensional models are appropriate when lateral hydraulic (Gibson and Pasternack 2015) or sediment processes are critical. For example, predicting deposition or erosion in a channel bend or a multi-channel, anatomizing reach require 2D models (Brown et al., 2014; Sánchez et al., 2014). Multidimensional models simulate differential deposition between channel and floodplain better than 1D models (Gibson and Nelson 2016). If the management question requires detailed floodplain or diversion deposition, a 2D model is usually required. Multidimensional models are also essential when evaluating structures that occupy only part of the cross section (such as groins, chevrons, localized gravel augmentation, constructed bars). These structures induce irreducibly multidimensional morphological effects, which 1D models oversimplify.

(4) One-dimensional models often reproduce longitudinal distribution of reservoir deposition relatively well (Gibson and Pridal 2015; Gibson and Boyd 2016; Gibson et al., 2017). Multidimensional models can be useful when the study depends on lateral distribution of reservoir sediments. Spasojevic and Holly (2008) discuss the capabilities and limitations of multidimensional models.

(5) Two-dimensional models of suspended sediments are applicable in regions where the vertical variations of flow and sediment concentration are either relatively small or can be described semi-analytically.

(a) The 2D SSC transport equations vertically integrate the 3D advection-diffusion equation. This vertical integration leads to advection and dispersion coefficients that are treated in a wide variety of ways in literature (Begnudelli et al., 2010; Sánchez and Wu 2011; Brown et al., 2014).

(b) Two-dimensional models use different approaches to averaging sediment concentration vertically and compute dispersive fluxes. The dispersive fluxes due to the vertically nonuniform current velocity and concentration profiles may be modeled by introducing mass flux or advection coefficients in the advection or temporal terms (Sánchez and Wu 2011; Brown et al., 2014). The dispersive fluxes may also be modeled as separate terms in the transport equation (Begnudelli et al., 2010). Either way, estimates of the vertical sediment concentration and current velocity profiles are needed.

(6) Two-dimensional models simulate bedload with 2D equilibrium or nonequilibrium formulations irrespective of whether the suspended sediments are modeled as 2D or 3D.

(a) In equilibrium bedload models, the bed change contribution from bedload is computed as the divergence of the transport rates (Wang and Adeff 1986; Olsen 2003; Buttolph et al., 2006; van Rijn et al., 2007; Warner et al., 2008). These models are most appropriate for suspended load dominated transport or when the grid is relatively coarse compared to the bedload adaptation length.

(b) When the transport is dominated by bedload or the grid resolution is much smaller than the bedload adaptation length, it is better to use a nonequilibrium model for bedload transport (Wu 2000; James et al., 2010; Brown et al., 2014; Sánchez et al., 2014). Nonequilibrium bedload models solve for the bedload concentration (James et al., 2010), transport rate (Wu et al., 2000), or concentration times the bedload layer thickness (Brown et al., 2014). As discussed previously, the nonequilibrium models are generally more computationally intensive, but produce more accurate and stable results.

(7) Three-dimensional Eulerian sediment transport models are usually required for curved and braided channels, or near structures such as spur dikes, bridge piers, or reservoir outlets.

(a) In general, 3D total load sediment transport models separate the sediment into suspended and bedload components. They solve suspended sediments with a 3D advection-diffusion equation with either a concentration or gradient boundary condition at the lower limit of the suspended load layer. The concentration boundary condition assigns the bottom concentration to the equilibrium concentration capacity estimated from empirical formula (Wang and Adeff 1986; Olsen and Kjellesvig 1998; Olsen 2003).

(b) However, this boundary condition is inadequate in simulating nonequilibrium conditions (see Celik and Rodi 1988) and that is not used in any USACE models. The gradient boundary conditions assume the entrainment flux is equal to the capacity under the current flow and bed composition and estimates the entrainment flux as the near-bed equilibrium concentration times the fall velocity. Likewise, the depositional flux is estimated as the actual near-bed sediment concentration times the fall velocity and in some cases times the probability of deposition (James et al., 2010). The gradient boundary condition has been used by many models such as those described in Spasojevic and Holly (1994); Wu et al. (2000); van Rijn et al. (2007); and Brown et al. (2014).

(8) As of 2017, USACE occasionally applies 3D models for river sedimentation. Some Districts have turned to 3D models (mostly CH3DSED) when vertical sediment gradients are critical. For example, ERDC applied a 3D model to compute the sediment concentration and gradation diverted into tidal wetlands (Spasojevic and Holly 1994; Raphelt and Alexander 2001; Raphelt et al., 2001). Sometimes, limited 3D runs can help modelers parameterize larger scale 2D models, applying the more detailed model to develop good empirical parameters for the less detailed model.

(9) Multidimensional hydraulic models can also drive Lagrangian sediment models such as USACE's Particle Tracking Model (PTM) (MacDonald et al., 2006; Davies et al., 2005). The PTM available in the Surface Water Modeling System (SMS) is a Lagrangian particle tracker to simulate particle transport processes. PTM applications to dredging and coastal projects includes dredged material dispersion and fate, sediment pathway and fate, and constituent transport. The model algorithms represent transport, settling, deposition, mixing, and resuspension processes in nearshore wave/current conditions. In addition, it may also be used to track fish larvae, dissolved contaminants, material released at placement sites, outfalls, etc. (Demirbilek et al., 2012).

(10) Increased model dimensionality improves the accuracy of the hydrodynamics and consequently, the transport of sediments. However, many sediment processes remain highly empirical even in multidimensional models that limit the accuracy of the solution. Hydrodynamic models can often improve results by simply increasing the grid resolution. However, the major source of error in sediment models is often associated with the empirical equations or data quality and increasing grid resolution can only do so much to improve the model results.

h. Quality and Quantity of Available Data.

(1) Sediment models have substantial data demands, requiring much more data than hydraulic models. Unfortunately, sufficient high quality sediment data are rare. Before any sediment modeling, a modeler must perform a realistic assessment of available sediment data to decide if they are sufficient to meet study needs.

(2) Determine data gaps before deciding to model, or before selecting a model. The data acquisition plan should be concurrent with the model design. Two general data categories of data will influence model quality: input data and calibration data. Assess both in the process of model selection and study objectives.

i. Data Categories. Two main categories of sediment data are required to build a sediment transport model: input data and calibration data. Table 9-2 summarizes these data, with the commonly collected and most important data marked with bold (red) headings. The study team may have to adjust the study objectives and scope in response to the available data quantity/quality. Additional sediment data collection may be feasible in some cases to enhance model capabilities. Model complexity should scale to data availability. Building a detailed, complex model, the parameterization data are rare, low quality, or infrequent can be counterproductive.

Table 9-2
Summary of Data Required for Sediment Models

Input Data	Calibration Data

Hydraulic Data
• Flow (main stem and tributaries).
  – Calibration period flows.
  – Estimate of future flows.
• Downstream boundary condition.
• Bed/bank surface representative of the start of simulation (1D, 2D, or 3D).
• Roughness and vegetation data.
• Historic gate operations and dredging operations.
	Hydraulic Calibration Data
	• High water marks.
	• Internal gage data (stage time series).
	• Velocity measurements (not common).
	• ADCP measurements to evaluate lateral flow distribution in 2D and 3D models.

Sediment Load at Model Boundaries • Flow-load rating curve. • Suspended load and bedload. • Main stem and tributary loads.	**Volume Change** • Repeated cross sections. • Repeated bathymetry. • Dredging records.
Load Gradation at Model Boundaries • Most sediment models require boundary loads subdivided by grain class.	**Downstream or Mid-Model Loads** • Sediment models can be calibrated against load (suspended, total, or bedload) measured in the model domain. This is not as reliable as calibrating to bed change.
Bed Gradations • Collect enough bed gradations to capture trends and variability. • If the bed is armored, collect separate cover and subsurface gradations.	**Bed Gradations** • If bed gradations changed during the calibration period (for example, reservoir deposition), current bed gradations and trends can be a calibration parameter (Gibson and Pridal 2015).
Cohesive Critical Shear and Erodibility Data • Models designed to predict cohesive erosion must estimate the critical shear and erodibility of the sediment. • These parameters can vary over 5 orders of magnitude and must either be measured or become a calibration parameter.	**Historical Stage and Specific Gage Trends** • In the absence of bed elevation data, models can calibrate to changes in water surface elevations over time, at the same flow.

Input Data	Calibration Data
Bedload and Bedload Gradation • Most sediment load and load gradation measurement (above) are collected with suspended samplers, excluding bedload transport. • Bedload samples are more difficult and expensive and are, therefore, rare. • Bedload can be small (3% to 20% of total load), but is often disproportionally responsible for deposition problems.	**Scour Chain Data** • Measure of net bed change during an event or time window.
Temperature • Sediment models require temperature data. • Finer sediment is more sensitive to temperature data.	**Bank Migration Rates** • Bank pin measurements. • Aerial photograph or LiDAR elevation model comparisons.
Stages • Reservoir models are sensitive to the reservoir stage time series during the calibration and predictions are sensitive to the assumed future reservoir stage time series. • Multidimensional models with limited spatial domains can also be sensitive to downstream stages selected.	–

j. Available Numerical Models.

(1) Models generally fall into two categories: customized research codes and generalized production models. Research code tends to be very detailed, carefully considering complex processes. They also tend to be customized to local conditions and are often difficult for modelers outside the development team to use or even acquire. Generalized production models are destined primarily to be applied by non-developers. While sediment processes are more site-specific than hydraulic models, generalized models try to include functionality that applies to a wide array of systems and processes.

(2) The generalized model category can also be subdivided into public domain and proprietary models. Public domain models are freely available to the public with varying degrees of documentation, support, and training. Private vendors provide proprietary models for a fee. Pricing levels vary with the software, support, and training provided. USACE develops and supports both public domain models and proprietary models, but provides all models to USACE users without fees (see paragraph 6-6).

(3) USACE Models.

(a) As of 2017, USACE actively develops four sediment models applicable to rivers and reservoirs: a 1D model (HEC-RAS), three 2D models or modeling frameworks (AdH, Coastal Modeling System (CMS), and Geophysical Scale Transport Modeling System (GSMB)), and two 3D models (AdH and GSMB).

(b) The HEC-RAS (HEC 2016) includes unsteady and quasi-unsteady, mobile boundary, 1D, sediment routing models. It routes sediment through a network of cross sections, compares supply to transport capacity, and adjusts wetted cross-section nodes in response to sediment erosion or deposition. Bed change is averaged over the entire wetted cross section (Figure 9-3).

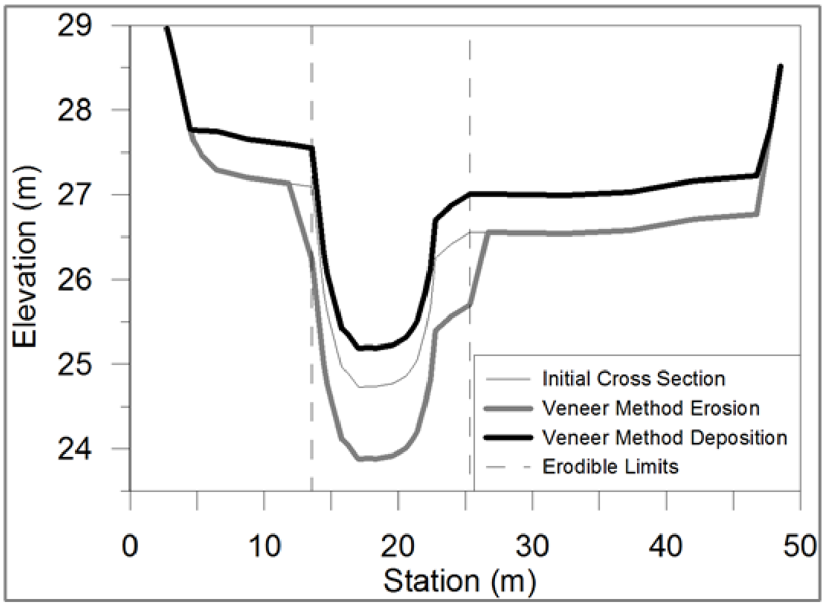


Figure 9-3. Veneer method bed change in a 1D sediment model (from Gibson and Nelson 2016)

(c) HEC-RAS replaced HEC 6, the legacy USACE 1D sediment model, and is based on many of the same modeling principles. However, many USACE Districts also use a model called HEC 6T. HEC 6T is a proprietary model that was built on HEC 6. Thomas (2002) added significant functionality to HEC 6T in the 20 years since USACE stopped actively developing HEC 6. For several decades, HEC 6T was the state of the art in 1D model development, and many USACE Districts maintain HEC 6T models and expertise to evaluate and update existing studies. However, HEC-RAS should be used for new USACE 1D sediment models.

(d) The Adaptive Hydraulics modeling suite (AdH) is a multidimensional 2D or 3D finite-element numerical model (Berger and Howington 2002; USACE 2015a). It simulates multiple physics, including groundwater, Navier-Stokes, and shallow-water physics.

- The shallow-water physics can be simulated in 3D mode, or 2D (depth-averaged) mode (Berger and Howington 2002; Savant et al., 2014). The model also simulates transport.

- The adaptive feature of AdH allows the model to perform mesh resolution refinement and relaxation in response to user-defined criteria derived from local error indicators. This allows the model to allocate computational resources efficiently, while ensuring that the model mesh converges (the model results are not significantly influenced by the resolution of the mesh.)

- AdH is linked to the USACE sediment transport library (SEDLIB) (Brown et al., 2014), which is a multidimensional and multi-grain class cohesive and cohesionless sediment transport module. AdH/SEDLIB can simulate sediment transport processes in rivers and estuaries and applied in both 3D and 2D (depth-averaged) mode. SEDLIB computes erosion and deposition fluxes and bed change at each computational point in the 2D mesh.

- AdH incorporates erosion and depositional fluxes into distinct transport solutions for bedload and suspended load. The mode of transport is determined by the local conditions and the sediment type: in general, sediments move in both modes, with a higher proportion moving as suspended load for higher rates of bed shear.

(e) The Coastal Modeling System (CMS) is a suite of coupled 2D numerical models for simulating waves, hydrodynamics, salinity and sediment transport, and morphology change (Sánchez and Wu 2011; Sánchez et al., 2011; Sánchez et al., 2014). The CMS is often applied to coastal engineering and coastal navigation investigations, simulating coastal processes. CMS helps design and management of coastal inlets, navigation channels, ports, harbors, coastal structures, and adjacent beaches. The CMS has been used for over 70 projects in the United States, including Matagorda Bay, Texas; Grays Harbor, Washington; mouth of the Columbia River, Washington/Oregon; Ocean Beach, California; St. Augustine Inlet, Florida; and Shark River Inlet, New Jersey.

(f) The GSMB is a model framework that includes the 2D deep-water wave action model (WAM); the 2D shallow-water wave models steady-state spectra wave model (STWAVE) and coastal modeling system (CMS-WAVE); the 2D advanced circulation model for oceanic, coastal, and estuarine waters (ADCIRC) hydrodynamic model; the regional scale models CH3D-MB (which is the multi-block (MB) version of CH3D-WES); the MB CH3D-SEDZLJ mixed sediment transport model that uses sediment transport algorithms developed by Ziegler, Lick, and Jones; and the CE-QUAL-ICM water quality model. GSMB has been used to perform water quality and sediment transport modeling studies in several estuaries, coastal seas, and lakes (for example, St. Johns River; Mobile Bay; Dauphin Island – Mississippi Sound; Port of Anchorage – Upper Cook Inlet, Alaska; Duluth Harbor, Minnesota; and Grand Traverse Bay, Michigan – Lake Superior).

(4) Other Sediment Models.

(a) Several model comparison studies are available (Papanicolaou et al., 2008; USBR 2006a; FISWRG 1998). While these studies all predate the state-of-the-art USACE models listed above, they demonstrate the availability and diversity of sediment models available.

- Model complexity and capability varies dramatically. Some models only use quasi-unsteady hydrodynamics, others only apply unsteady flow, and still others offer both.

- These models also integrate a wide variety of sediment algorithms, approaches, and simplifications, including different approaches to bedload/suspended load/total load lumping and separation, sediment exchange processes, of sediment type (cohesive vs. noncohesive), grain class sub-division, bed mixing processes, bank sources and sediment transport functions.

- Table 9-3 lists models included in relatively recent reviews, though many of these models are no longer supported or have been superseded.

(b) These reviews include three basic types of models: (1) generalized public domain models, (2) proprietary models, and (3) research codes.

(c) Most agencies have updated their public domain models since these studies. In addition to the state-of-the-art USACE models covered above, the USBR also supports and develops 1D and 2D models (SRH-1D and SRH-2D) that replaced the GSTARS suite. The USDA-ARS developed CONCEPTS to compute coupled bed and bank migration and occupies an important niche among sediment model options. ARS also develops and supports RVRMeander, a river meander model (Mota et al., 2012). Finally, the USGS actively develops and applies FaSTMECH, a 2D sediment model that is also considered state of the art.

(d) Agency public domain models (Table 9-4) have different strengths and weaknesses than the USACE models, but are publicly available, widely accepted, and may be appropriate in some applications.

Table 9-3
Sediment Models Listed in Model Review Summaries (FISRWG 1998)

<table>
<tr><th colspan="3">Stream Corridor Restoration Principles, Processes, and Practices Working Group (1998)</th></tr>
<tr><td>• CHARIMA
• FLUVIAL-12
• HEC 6</td><td>• USGS D-0-T
• GSTARS</td><td>• MEANDER
• TABS-2</td></tr>
<tr><th colspan="3">Papanicolaou et al., 2008</th></tr>
<tr><td>1D Models:
• HEC-6
• MOBED
• IALLUVIAL
• FLUVIAL-11
• GSTARS
• CHARIMA
• SEDICOUP
• OTIS
• EFDC1D
• 3STD1
• CCHE1D</td><td>2D Models:
• SERATRA
• SUTRENCH-2D
• TABS-2
• MOBED-2
• ADCIRC
• MIKE 21
• UNIVBEST-TC
• USTARS
• FAST2D
• FLUVIAL-12
• Delft2D
• CCHE2D</td><td>3D Models:
• ECOMSED
• RMA-10
• GBTOXe
• EFDC3D ROMS
• CH3D-SED
• SSIIM
• MIKE 3
• FAST3D
• Delft3D
• TELEMAC
• CCHE3D</td></tr>
<tr><th colspan="3">USBR 2006a</th></tr>
<tr><td colspan="2">Federal Public Domain Models
• HEC-6
• HEC-RAS
• TABS-2
• USGS
• SRH-1D/SRH-2D (formerly GSTARS)</td><td>Proprietary or Research Models
• CHARIMA
• FLUVIAL-12
• MEANDER
• D-0-T</td></tr>
</table>

Table 9-4
Federal, Public Domain, Generalized, Production Oriented Sediment Models

Agency	Contemporary Models
USACE	HEC-RAS, AdH, CMS, PTM, GSMB, SIAM
USBR	SRH-1D/2D/3D
USDA-ARS	CONCEPTS, RVRMeander
USGS	FaSTMECH

(e) Several state-of-the-art proprietary codes are also widely used. The MIKE series (1D, 2D, 3D, SHE), and the Delft suite are actively developed and supported and are the most common proprietary codes applied to federal projects.

(f) Several agencies, universities, and firms have developed simplified spreadsheet models or calculators that either compute sediment flux (such as Bedload Assessment for Gravel-bed Streams (BAGS) – Pitlick et al., 2009) or route sediment through simplified cross sections (Parker 2006). These tools can be helpful to build conceptual models of the system and potential project responses.

(g) Model selection is important. However, several models will be applicable to most sediment modeling situations. In most cases, Dawdy and Vanoni's (1986) conclusion that "… the choice of a modeler is probably more important than the choice of a model" is probably still valid, as long as the model selected includes the capabilities required to simulate the sensitive processes.

k. Particle Tracking Models. Most engineering studies do not require tracking the motion of individual particles. Most applications target changes in the bed surface elevation in response to dynamic sediment conditions and to feed those changes back into the calculation of the flow intensity sediment load parameters. However, some models concerned with sediment associated contaminants (such as metals or sorbed nutrients) require tracking individual particles. Lagrangian sediment models (such as PTM) track the path and fate of individual particles (see paragraph 9-1g(9)).

l. Model Philosophy Summary. Sediment study model selection, data collection, and study design should consider a range of factors. These factors include the following:

(1) Real-world systems are complex, models are simplifications of these systems.

(2) Analysis uncertainty is offset by applying informal or formal safety factors.

(3) Historically, modeling studies included mobile bed physical models and simple computational models. Recently, mathematical model use has expanded significantly. Numerical models are used in conjunction with physical models, or often to replace them.

(4) Sediment models should be designed to answer a specific question or a limited set of related questions from the start. Every stage of model development should have the objective question in mind, including the data collected, the design of the model scale and resolution, and the model selection.

(5) Sediment models have substantial data demands. Perform a realistic assessment of available sediment data to determine if it is sufficient to meet study needs.

(6) Two general data categories will influence model quality: input data and calibration data. Assess both in the process selecting a model and designing study objectives.

(7) USACE actively develops four numerical sediment models (a 1D model (HEC-RAS) and three 2D models (AdH, CMS, and GSMB)) and a simplified sediment budget tool (SIAM).

9-2. Brief Background – Numerical Modeling of Dynamic Sediment Transport.

a. Introduction.

(1) For many USACE project design applications, dynamic conditions drive interest in sediment modeling when aggrading or degrading trends exist in the current condition or are projected in response to USACE project implementation.

(2) In past USACE guidance documents, dynamic sediment transport refers to cases where the sediment leaving a reach ($Q_{sed\text{-}out}$) does not equal the sediment flowing into that reach ($Q_{sed\text{-}in}$), where the difference ($Q_{sed\text{-}in} – Q_{sed\text{-}out}$) deposits or erodes the streambed. All five processes of sedimentation: erosion, entrainment, transport, deposition, and consolidation are active in dynamic transport, creating unstable streambed elevations. In such cases, a numerical sedimentation model or movable-bed physical model provides a framework for analysis.

b. Mobile Bed vs. Fixed Bed Transport Evaluation.

(1) The 1D, 2D, and 3D sediment models described above are mobile bed models which update the bathymetry each time step to reflect sediment deposition or erosion. Some projects do not have data to perform dynamic sediment transport modeling or are in the early project stages, where a simpler tool is appropriate to screen the sediment impacts of project alternatives. In these situations the PDT may select fixed bed sediment analyses. USACE tools like SAM (Thomas et al., 2002) or the Hydraulic Design calculator in HEC-RAS (Brunner and Gibson 2005), or other tools like BAGS (Pitlick et al., 2009) compute sediment transport capacity without updating the bed.

(2) SIAM, a sediment budget tool also incorporated into the hydraulic design package in HEC-RAS, is a more sophisticated fixed bed sediment analysis tool. SIAM (Mooney 2006; Little and Jonas 2010) compares annualized sediment loads to reach averaged, annualized sediment capacity to make estimates whether reaches will degrade or aggrade long term. However, SIAM is not intended to simulate time series and does not update bathymetry in response to sediment. It is a fixed bed snapshot of the relative relationship between supply and capacity that can help PDTs to screen alternatives and select a few for more detailed analysis.

(3) These tools use the same sediment transport formulas used in the mobile bed models. However, there are significant differences between mobile bed and fixed bed calculations. Table 9-5 summarizes those differences.

(a) Mobile and fixed bed analyses are sometimes called fixed and dynamic sediment analyses, and were called equilibrium and nonequilibrium, respectively, in previous USACE guidance (but the latter terms have adopted different definitions in sediment transport since).

(b) The river bed is the only source of sediment to a sediment transport formula in fixed bed analyses while mobile bed models consider sediment condition include the bed, upstream reach, tributaries, and bank caving. Additionally, mobile bed models consider sinks, including the bed, long-term bar accretion (planform changes usually associated with bank erosion), controlled or uncontrolled diversions, sediment deposition in the floodplain, dredging, and sand or gravel mining.

Table 9-5
Differences Between Static and Equilibrium Sediment Transport Modeling

Fixed Bed Analysis Sediment Discharge Formula (SAM, or the Hydraulic Design Tools in HEC-RAS)	Mobile Bed Sediment Transport Models (1D and 2D models like HEC-RAS, AdH, CMS, and GSMB)
Require flow intensity, bed roughness, particle density, and bed surface gradation.	Require flow intensity, bed roughness, particle density, both surface and subsurface bed gradations, inflowing sediment load, geometry over long distances, and identification of bedrock outcrops.
Calculate the equilibrium condition.	Calculate both the equilibrium condition and the changes in bed profile due to sediment inflow deficit or excess or apply dynamic disequilibrium transport equations (such as advection-dispersion, erodibility functions, and adaptation lengths).
Only consider bed material load (except for SIAM, which has methods to track wash load).	Can consider both bed material and wash loads.
–	In the case of sand moving over a gravel-bed, models will calculate both the quantity of sediment load moving and bed surface gradation required to sustain it.

c. Background of Mobile Bed Models. Mobile bed models use a control volume approach. The control volume partitions the river into reaches (or lateral and vertical sub-reach sections in multidimensional models). Both the river bed within the control volume and sediment load flowing in from adjacent control volumes and boundary conditions become sediment sources in each control volume. Therefore, the most significant feature of a numerical mobile bed model is its formulation of the sediment continuity equation, which handles the exchange rate between the water column and the bed surface. The model must account for:

(1) Sediment transport by grain class.

(2) Bed roughness, potentially as a function of discharge.

(3) Bed material sorting.

(4) Bed surface thickness and porosity/bulk dry density.

(5) Bed consolidation.

d. Examples of Modeling Limitations. It should be recognized that there are major knowledge gaps related to sedimentation processes and that many of the sedimentation process descriptions employed in numerical models are empirical relationships developed from incomplete data. For example, bed sorting process are still poorly understood, which makes it difficult to formulate a numerical representation of the process. Also, because sediment transports primarily in the channel, mobile bed models are very sensitive to maintaining careful flow distributions between the channel and floodplains.

e. Theoretical Basis.

(1) Equilibrium Equations of Flow and Continuity. Models that apply the Exner equation and sediment transport equations are described as equilibrium (or saturated) models because they assume the transport is equal to the transport capacity (equilibrium transport). (*Note*: Previous USACE guidance called these nonequilibrium models, but this no longer reflects common modeling terminology.)

(a) The 1D partial differential equations of gradually varied unsteady flow in natural alluvial channels are: (1) the equation of motion for the water-sediment mixture, (2) the equation of continuity for water, and (3) the equation of continuity for sediment.

(b) The system of equations for unsteady flow are established by considering the conservation of mass (both sediment and water) and momentum in an infinitesimal space between two channel sections:

(c) Equation of Motion:

$$\frac{\partial(\rho Q)}{\partial t} + \frac{\partial(\rho QV)}{\partial x} + gA\frac{\partial(\rho y)}{\partial x} = \rho gA\left(S_o - S_f + D_l\right) \qquad \text{Equation 9-1}$$

(d) Water Continuity:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q_w = 0 \qquad \text{Equation 9-2}$$

(e) Sediment Continuity:

$$\frac{\partial Q_s}{\partial x} + (1 - n)\frac{\partial A_d}{\partial t} - q_s = 0 \qquad \text{Equation 9-3}$$

where:

A = channel cross-sectional area
A_d = volume of sediment deposited on the bed per unit length of channel
D_l = momentum loss due to lateral inflow
g = acceleration of gravity
n = porosity of the bed deposit (volume of voids divided by the total volume of sample)
Q = water discharge
Q_s = sediment discharge
q_s = lateral sediment inflow per unit length of channel, outflow (-), inflow (+)
q_w = lateral water inflow per unit length of channel, outflow (-), inflow (+)
S_f = friction slope
S_o = slope of channel bottom
t = time
x = horizontal distance along the channel
V = flow velocity
y = depth of flow
ρ = density of the water

(2) Nonequilibrium Transport Equations.

(a) Nonequilibrium transport models differ from equilibrium transport models in that they allow the local sediment transport rate to be different from the transport capacity. Nonequilibrium sediment transport models may solve separate transport equations for the bedload and suspended-load (Brown et al., 2014) or a single transport equation for total load (Sánchez et al., 2014).

(b) The main advantage of combining the bedload and suspended-load is efficiency, since one less transport equation must be solved for each grain class. However, total load models require estimating a transport mode parameter (fraction of suspended sediments) for model closure.

(c) Some models may treat the bedload as in equilibrium while the suspended-load as in nonequilibrium (Buttolph et al., 2006; van Rijn et al., 2007). In equilibrium bedload models, the bed change due to bedload transport is computed as the divergence of the bedload transport rates, while in nonequilibrium models, the erosion and deposition fluxes are estimated using empirical or semi-empirical formulas.

(d) Suspended sediments may be solved using a 3D transport equation:

$$\frac{\partial c_k}{\partial t} + \nabla \cdot [(\vec{v} - \vec{\omega}_k)c_k] = \nabla \cdot (\vec{\varepsilon}_k \nabla c_k), \dots, N \qquad \text{Equation 9-4}$$

where:

c_k = SSC of the kth grain class (M/L^3)

$\vec{\omega}_k$ = sediment particle settling velocity corresponding to the kth grain class (length, L/time, T)

$\vec{v}$ = current velocity (L/T)

$\vec{\varepsilon}_k$ = turbulent diffusion coefficient corresponding to the kth grain class

(e) For further details on 3D sediment models, the reader is referred to Spasojevic and Holly (1994, 2008); Wu et al. (2000); van Rijn et al. (2007); and Warner et al. (2008).

(f) By definition, all 3D models are nonequilibrium models. A zero flux boundary condition is applied at the surface and either a concentration or gradient boundary condition at near the bed. Bedload is in general solved with 2D transport equation. The 2D governing transport equation is generally of the form:

$$\frac{\partial(hC_k)}{\partial t} + \nabla \cdot (h\beta_k \vec{V} C_k) = \nabla \cdot (h\vec{\varepsilon}_k \nabla C_k) + E_k - D_k \text{ for } k = 1, \ldots, N \qquad \text{Equation 9-5}$$

where:

C_k = sediment concentration of the kth grain class (M/L^3)
β_k = advection coefficient corresponding to the kth grain class (-)
$\vec{V}$ = depth-averaged current velocity (L/T)
h = water depth (L).
$\vec{\varepsilon}_k$ = diffusion (mixing) coefficient corresponding to the kth grain class
E_k = erosion rate (mass M/T/L^2)
D_k = deposition rate (M/T/L^2)
N = number of sediment grain classes (-)

(g) There are many variations of the above equation. The major differences occur generally in the state variable definition. In many models, the state variable is defined as a sediment concentration in mass per unit volume as in the equation above (van Rijn et al., 2007; Spasojevic and Holly 1994, 2008; Wu et al., 2000). The sediment concentration may represent suspended-load, bedload, or total load depending on the model. In the case of bedload transport, the state variable may be defined in mass per unit area (see Brown et al., 2014), or in actual transport rate magnitude (Wu 2007).

(h) If the 2D suspended-load is not defined as depth-averaged, and is instead defined as in a layer above the bedload layer, then the bedload transport equation should include both entrainment and deposition fluxes from the bed and from suspended sediments (Wu et al., 2000). In some bedload and total load models, the state variable can be the actual transport rate magnitude (Wu 2007).

(i) The main advantage of solving a total load sediment transport equation instead of separate bedload and suspended-load transport equations in 2D models is that that there is one less equation to solve for each sediment grain class. However, total load models require estimating a transport mode parameter (fraction of suspended sediments) for model closure.

(j) Some models may treat the bedload as equilibrium while treating the suspended-load as nonequilibrium (Buttolph et al., 2006; van Rijn et al., 2007). In equilibrium bedload models, the bed change is computed as the divergence of the bedload transport rates, while in nonequilibrium models, bed change is computed from erosion and deposition fluxes estimated using empirical or semi-empirical formulas. In general, nonequilibrium models are more computationally demanding than equilibrium models, but they provide more accurate and stable results.

(k) Johnson and Zyserman (2002) showed that the equilibrium sediment balance equation leads to higher order harmonics in bed change due to the nonlinear dependence of the bed celerity with bed elevations. They also showed that these harmonics, if not damped numerically, will lead to instabilities. Therefore, an equilibrium model without numerical or artificial diffusion is expected to be unstable.

(l) Equilibrium models should be applied with caution and in temporal and spatial scales over which the assumption of equilibrium is valid. In general, equilibrium models work well for coarse grid models and slowly varying conditions, because over large distances and times, the sediment is, on average, close to equilibrium even though locally, it may not be in equilibrium. For models that require fine resolution grids or have rapidly varying conditions, nonequilibrium sediment transport models should be applied.

9-3. Assembling a Sediment Model.

a. General Project Startup Tasks.

(1) The following project startup tasks are recommended to systematically define study parameters:

(a) Mark project area and study area boundaries on a project map to delineate the area data collection limits. The study team should also identify the lateral study limits and the important tributaries.

(b) Examine bed profiles and cross sections from historical surveys in the project. These historic data are extremely valuable for determining the historical trends that the model must reconstitute.

(c) Collect aerial photographs and aerial mosaics of the project area. Note changes between them and identify important historical trends in channel width, meander wave length, rate of bankline movement, and land use in the basin.

(d) Organize stream gage records. Gage records establish the annual water yield into and out of the project area. They are also useful for establishing the hydraulic parameters like depth, velocity, roughness, and stage-discharge trends in, or close to, the project reach. Limit conclusions to measured data. The extrapolated portion of a rating curve should not be regarded as measured data. Be aware that measured data are also subject to error.

(e) Visit the project reach. Visiting the project reach will help the modeler in determining channel morphology, geometric anomalies, the existence of structures, and sediment characteristics. Include a local geomorphologist and geotechnical and environmental specialists in the field reconnaissance. Document these observations of the prototype in project reports. View as much of the prototype as is feasible and not just bridge crossings. The modeler should spend time on the river before building the model and during the project to evaluate modeling hypotheses. If possible, getting on the river with a boat will provide much more insight than visiting a few road access points that may or may not be representative.

(f) Gather hydraulic data such as measured water surface profiles, historic high water marks, velocities, and flood limits in the project reach. Local action agencies, newspapers, and residents along the stream can provide important information field measurements.

(2) For additional information regarding general analysis tasks and modeling data collection, refer to Appendix B and D.

b. Selecting Model Extent and Boundary Locations.

(1) The model extent will influence both the quality and cost of the model. Most models focus on part of a river system, investing computational and cognitive resources on a particular area of interest where sediment poses a present or potential problem. The model extent must, at least, include the area of interest. However, numerical and data considerations usually drive model extents, leading the modeler to extend the model well upstream and/or downstream the area of interest. Therefore, the modeler draws two boxes on the map, indicating the area of interest and the region required to model that area without problematic boundary effects or numerical artifacts.

(2) When possible, mobile bed model boundaries should be placed at river reaches in quasi-equilibrium. However, USACE project studies rarely model reaches in quasi-equilibrium. Agencies and stakeholders tend to commission sediment models when a river is either chronically eroding or depositing. If the river does not present stable model boundary locations, select model boundaries far enough from the reach or region of interest that they do not affect, or worse yet, specify the solution.

(3) Selecting a good upstream boundary location is particularly important in reservoir models. Reservoir deltas prograde into the pool, advancing systematically downstream. However, as the delta forms, it also aggrades the upstream reach, inducing deposition well upstream of the pool elevation. The upstream boundary for a reservoir model should be substantially upstream of the high pool, beyond the expected upstream base-level effects expected in the project life (Thomas et al., 1982; USACE 1993).

(4) Multidimensional models require particular attention to the model extent and location of open boundaries.

(a) The extent of the model domain is often a compromise between domain size, grid resolution, and run times. The minimum model extent, however, is limited by the physical processes. One common error in multidimensional models is to make the model extent too small and, for example, exclude areas that should be wet. If there is any doubt about the model extent, a grid sensitivity analysis can be performed to determine an appropriate domain extent.

(b) The location of open boundaries should be carefully selected for model accuracy and stability. Open boundaries should be placed away from the area of interest. However, the location of forcing boundaries is limited by data availability. The appropriate distance depends on many factors, including the boundary condition type, intensity and accuracy of the boundary forcing data, general model stability, and the length of the simulation.

(c) Forcing boundaries should be consistent with the model solution; otherwise instabilities may occur. For example, river inflow boundaries should be placed in areas where the flow is consistent with the boundary forcing. If the boundary is computing a local discharge using a conveyance approach, then that flow should be consistent with the internal flow. Inflow boundaries should be placed away from recirculation areas and ideally where the flow is relatively one-dimensional. Depending on the model, the discharge boundary may also need to be oriented to be perpendicular to the flow. For example, CMS has the option to specify the inflow direction (Sánchez et al., 2014).

c. Constructing Model Geometry or Bathymetry.

(1) Hydraulic models require numerical representation of river topography, sometimes called the model geometry, or bathymetry. One-dimensional models represent bathymetry with a series of cross sections, orthogonal to the flow direction. Two- and three-dimensional models extract bed elevations from Digital Terrain Models (DTMs), typically represented as grids or TINs (triangular irregular networks).

(2) Selecting model resolution is a tradeoff between accuracy and run time (or cost). However, it is also an implicit tradeoff between local detail and scope (the Extent vs. Granularity tradeoff listed in Table 9-1), or even between spatial detail and the length of the time series or number of alternatives evaluated. Resolution is usually a relatively fixed resource, which can be spent by building a low-resolution model that covers a large reach, or a high-resolution model of a smaller region.

(3) 1D Model Geometry.

(a) One-dimensional hydraulic models are sensitive to skillful cross-section selection. One-dimensional sediment transport models are even more sensitive to cross-section layout than hydraulic models. Define cross sections perpendicular to flow, in the channel and the overbank. Keeping cross sections perpendicular to flow usually requires “dog-legged” cross sections. Cross sections usually are not straight features, but angled, perpendicular to flow in each zone. Cross-section location should also capture any substantial change in the river including flow constrictions, slope change, roughness change, structures, expansions or contractions, tributaries, or other important features.

(b) Cross sections come from a manual field survey or are cut from a digital elevation model (DEM). With LiDAR and other technologies making DEMs cheaper, high-resolution floodplain 1D geometries are often cut from high-quality DEMs. However, because most remote sensing technologies cannot penetrate water, DEMs usually lack channel data. Therefore, modelers must combine remotely sensed DEMs (like LiDAR) with boat-based multi-beam data, or integrate them with surveyed cross sections (Figure 9-4). Multidimensional models may also require merged bathymetries from surveys and LiDAR, but for these models, the data are generally merged before they are imported into the model.

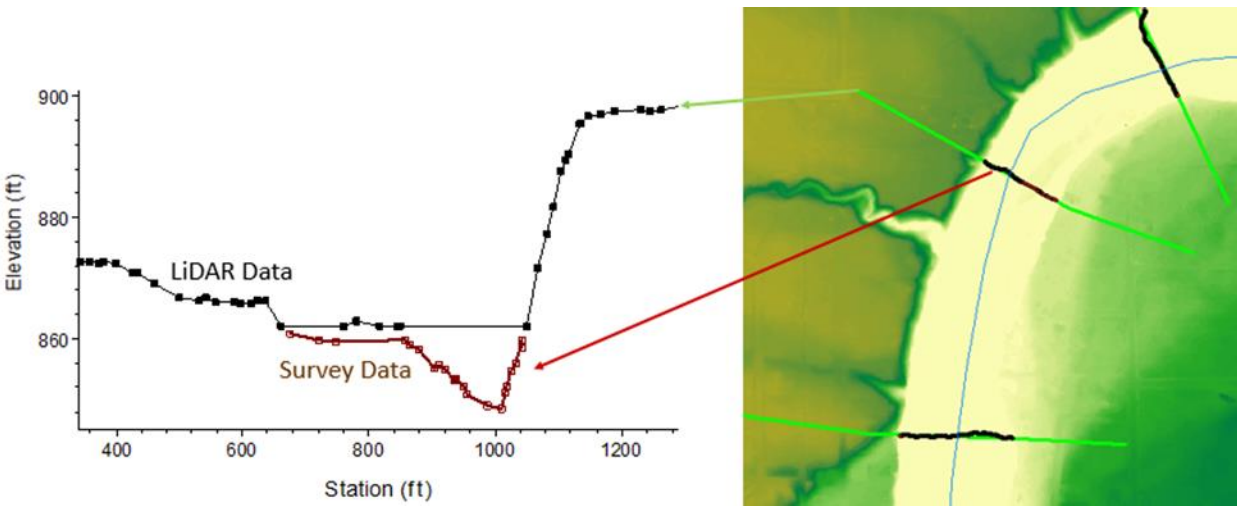


Figure 9-4. Combining digital elevation overbank data with surveyed, in-channel bathymetry

(c) Fixed bed hydraulic models usually assume that contemporary geometry reflects past river geometry or vice versa. However, because sediment modeling assumes the bed changes in time, contemporary geometry is useful only for predictive sediment modeling (projecting future trends). Calibration requires geometry data from a past condition. It is common to mine historical channel surveys (sometimes combined with contemporary overbank data from a DEM) to create a historic geometry for calibration. Modelers also must be careful when using cross sections collected at different times (on-synoptic cross sections) since assigning a date to a mixed bathymetry complicates the calibration.

(d) One-dimensional models do not explicitly account for 2D effects like flow shadows, expansion or contraction zones, backwaters, or floodplain depressions. The entire wetted cross section conveys flow unless the modeler specifies otherwise. Most 1D models include features to specify non-conveying zones in 1D cross sections (for example, ineffective flow areas in HEC-RAS). Carefully considering flow transitions between cross sections and skillfully defining non-conveying zones is essential to good 1D hydraulic modeling. Sediment transport models are even more sensitive to skillfully selecting these features. Multiple levels of conveyance limits may be needed to facilitate accurate reproduction of river hydraulics for a range of flows. Without accurate hydraulics, accurate sediment transport modeling is not feasible.

(e) There is no established maximum or minimum spacing for cross sections. Some studies have required distances as short as a fraction of the river width. Others (such as coarse evaluation models are large rivers) have allowed spacing sections 10 to 20 miles apart. Sediment models often start with the same geometry that was developed for fixed bed calculations. However, most fixed bed models used for FRM analysis are biased toward constrictions such as bridges and deficient of reach-typical sections that are important for long-term river behavior. Models to evaluate large scale sedimentation problems will likely be constructed without sufficient detail to evaluate localized structure effects and vice versa. Consider the study area of interest, data sources, and desired accuracy when evaluating cross-section location and spacing.

(f) Filtering cross sections (a common practice to simplify 1D hydrodynamic models) can impact mobile bed model results. Because mobile bed models adjust wet nodes, multiple nodes along a straight slope can improve the model results. At different water levels, different nodes will move in a mobile bed model.

(g) Finally, bend or pool cross sections can be deeper than run, crossing, or riffle cross sections. Multidimensional forces and secondary circulation usually maintain these cross-section shapes, processes that a 1D model cannot replicate. One-dimensional models tend to artificially deposit in these cross sections. Sometimes 1D modelers leave these cross sections out of the model to avoid unrealistic deposition.

(4) Multidimensional Model Geometry.

(a) Multidimensional models essentially require the same geometry data. In general, the geometry for 2D and 3D models is easier to generate than 1D models because it does not require judgment on the positioning and location of cross sections. Instead, the geometry for multidimensional models is determined mostly by the spatial resolution required to simulate the flow, sediment transport, and morphology change.

(b) In multidimensional models, the orientation of cells does not influence the results significantly. Therefore, the spatial grid resolution and an accurate representation of the bed elevations are the main considerations when generating a multidimensional model geometry. The spatial resolution required depends on the purposes and objectives of the modeling study. Mesh resolutions that are too coarse may underestimate conveyance, and meshes that are too fine waste computational resources.

(c) The AdH model allows for an adaptive grid resolution and therefore, if the user captures the geometry with the initial mesh, the model will refine the mesh necessary to resolve the hydraulic and sediment transport processes (Tate et al., 2006). Brown and Kraus (2007) discuss several tips for developing bathymetry grids for the CMS.

(d) Multidimensional sediment transport models require more attention to the initial bed elevations than multidimensional hydraulic models. If the initial bed elevations have artificial discontinuities, oscillations, or any other artificial morphological features, the sediment model will remove these artificial features, inducing artificial bed change results.

(e) Multidimensional modelers should carefully pre-process model geometry to remove artificial features and morphological structures the model cannot simulate (for example, small sand waves compared to the grid resolution). Modelers sometimes interpret simple bed form smoothing, where the model works out irregularities in the initial bathymetry as system bed change. Misinterpreting model response to minor or artificial initial features is a common multidimensional modeling error.

(f) Running a short simulation to “pre-condition” the bathymetry can remove artificial features from the bathymetry. A preconditioning time period should be long enough to remove the small-scale artificial features, but not long enough to significantly change major morphologic features. If possible, use only the bathymetry from the preconditioning simulation in trouble areas while keeping the original unmodified bathymetry in other areas. However, preconditioning the bathymetry should not be a substitute for good bathymetry.

(g) Bathymetry that is good enough to simulate water levels and flows may not be good enough for sediment transport and morphology change (for 1D or multi-D models). Sediment models require extra care in preparing the model bathymetry. In particular, defining linear features with breaklines will improve sediment model results over simple interpolated bathymetries. Add mesh resolution in areas of interest and rapid change. Multi-beam or LiDAR data are often filtered to reduce the size of the file. Use slope-based or binning techniques instead of simple n^{th} point sub-sampling when filtering these files. Modelers can also filter high resolution bathymetry data by binning it on a regular grid and then smoothing the grid or portions of the grid.

(h) Like 1D models, multidimensional models require careful attention when combining bathymetric data from different time periods and resolutions. In many cases, the datasets will not match at their boundaries, forcing modelers to create transitions between the datasets to avoid abrupt and unrealistic changes in bathymetry. Creating good bathymetry for models is a skill learned through experience and requires not only a good understanding of river and reservoir geomorphology, but also the capabilities and limitations of numerical models.

(i) The regions around structures often lack survey data required to represent them in the bathymetry. The study team may need to collect additional data around structures to represent them correctly. In 2D models, the planview coverage of the structures may be sufficient, but 3D models should include vertical resolution consistent with vertical discretization of the 3D domain.

(5) Boundary Conditions.

(a) Numerical models require boundary conditions. Modelers must define flow and sediment load at the upstream end of the model and a downstream stage series. If the model has more than one upstream reach, the models require flow and load data for each. Depending on model capability, the flow time series can be a steady, quasi-unsteady, or an unsteady flow hydrograph.

(b) A downstream boundary condition is also required, and typically consists of a stage-discharge rating curve or a stage hydrograph. The downstream rating curve can be calculated assuming normal depth if the boundary is in a reach where friction is the control, and the water surface slope is relatively constant for the range of modeled flows. However, since normal depth does not fix a downstream stage, it is applicable only in an equilibrium reach. Applying normal depth at an actively aggrading or degrading cross section will allow the model boundary to move indefinitely. When a backwater condition exists, such as at the mouth of a tributary or in a reservoir, then a stage hydrograph as the boundary condition is preferred.

(c) The boundary condition also requires a sediment time series for each grain class. However, complete sediment time series are almost never available, even on the largest rivers with the best data. Most modelers attach a sediment rating curve to the upstream hydrograph, introducing sediment load as a function (usually a power function) of flow. Modelers usually also have to subdivide this load by sediment size.

(d) Many models offer an equilibrium load sediment boundary condition, which computes sediment load at the model boundary based on the hydraulics and bed gradation of the upstream cross section(s). However, equilibrium assumptions are only appropriate when the upstream boundary is in equilibrium in the prototype. Even when this condition is met, they introduce uncertainty into the model because equilibrium loads are very sensitive to uncertain data and model parameters. Without measured data, equilibrium load can be the best option for sand and gravel using mobile bed hydraulics and sediment transport theory, but there is no comparable theory for the wash load inflow. Wash load is important in studies such as reservoir depletions where wash load volume may be significant.

(e) The upstream boundary associated with an equilibrium boundary condition should be selected very carefully. The upstream boundary should be located at a reach upstream of the project that has a slope, velocity, width, and depth typical of the hydraulics that are transporting the sediment into the project reach. It should also have a bed surface that is in quasi-equilibrium with the sand and gravel discharge being transported by the flow. Evaluate the calculated equilibrium load by particle size for the full range of water discharges in the study plan.

(f) In multidimensional models, model boundaries include several nodes rather than a single cross section. Therefore, multidimensional models must distribute the sediment boundary load across multiple cell faces. Multidimensional modelers must make an assumption about the horizontal/vertical sediment distribution at the boundary, but many models provide algorithms to help make this assumption.

(g) Most numerical models are deterministic, but the boundary conditions are not. Future hydrology and sediment loads are unknown and stochastic. Therefore, selecting boundary conditions that accurately represent the hydraulic and sediment characteristics of the reach can be critical. Some USACE projects require different boundary conditions for future conditions than for the calibration period or for present conditions to reflect changes in sediment processes. Future boundary conditions should reflect expected trends and uncertainty in these boundary loads or particle sizes. Chapter 10 provides more information on integrating sediment results into present and future project conditions.

d. Flow Record.

(1) All sediment models require flow time series data for the entire simulation window. If the river has gage data covering the period of interest, then calibration, forensic, or other historical models can use these historical flows. When historic flow data are not available (a more common condition), hydrologic models can estimate river flows from rainfall data. If a gage recorded historic flows for only part of the simulation window (or a time period outside the simulation window), these flows can sometimes be correlated to a more complete, nearby gage record (Gibson 2016), or they can be used to calibrate a hydrologic model, which can then estimate flows from rainfall data with less uncertainty.

(2) Tributaries are lateral flow boundary conditions. It is important to include water and sediment inflows from all significant, gaged, and ungaged areas within the study. When determining the tributaries that are significant, keep in mind that a sediment transport is nonlinear with flow. Therefore, a 10% increase in water discharge could result in a 20% or greater increase in bed material load transport capacity. If tributaries are not gaged, they will require external analysis. Drainage area ratios can provide rough estimates; however, a hydrologic model of the basin may be necessary to accurately describe model flow distribution. Flow-load curves from the main stem are not generally applicable to tributaries. Smaller drainage areas tend to deliver more sediment per unit flow.

(3) Future conditions simulations pose different uncertainty challenges than calibrations. Future hydrology suffers from “natural variability” instead of “knowledge uncertainty” (that is, aleatoric uncertainty instead of epistemic uncertainty). Modelers must make assumptions about future hydrology that generate an expected average sediment behavior, but also provide information on the range of likely outcomes. Paragraph 10-6 provides additional information for estimating uncertainty.

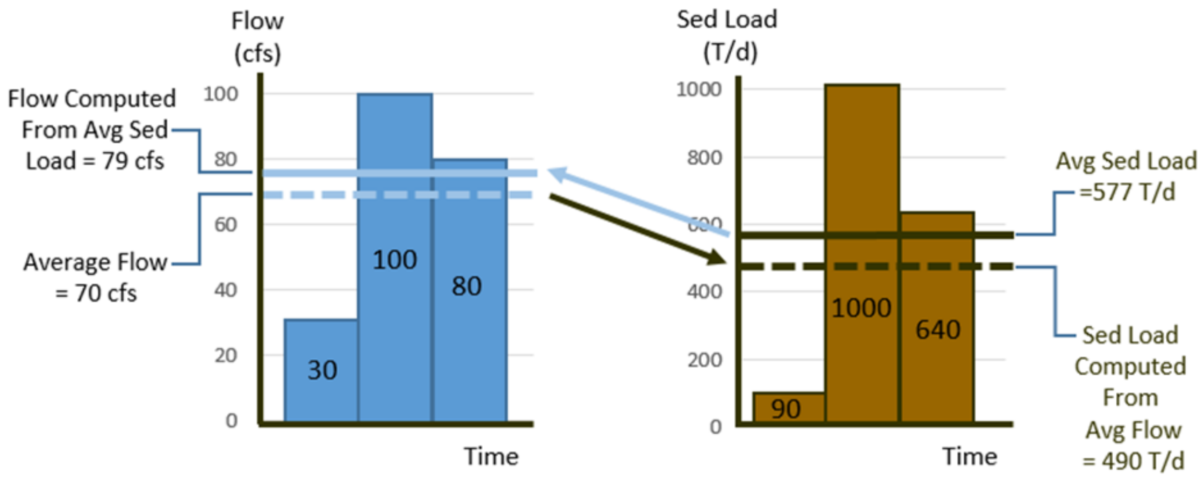
(4) Most one-dimensional sediment models can use unsteady or quasi-unsteady hydrodynamic models. Unsteady flow computes the shallow water (or diffusive wave) equations, conserves flow, and is more accurate. Unsteady flow is particularly appropriate for systems with significant storage (such as reservoirs). However, many modelers prefer quasi-unsteady flow (despite the simplification) because it is more stable and usually runs faster. Both quasi-unsteady and unsteady sediment models can improve run times with variable time steps in the current version of HEC-RAS, but quasi-unsteady is usually stable at larger time steps. An overview of the two modeling approaches is included in Table 9-6.

Table 9-6
Comparison of Unsteady and Quasi-Unsteady Hydraulics Used in 1D Sediment Transport Models

Quasi-Unsteady	Unsteady
More stable	Less stable
Does not account for storage (does not conserve flow).	Accounts for storage (conserves flow).
Larger time steps. Usually runs faster.	Usually requires smaller time steps. Runs slower.
Use for large, long, steep, potentially unstable models with little in-channel or off-channel storage.	Use for high storage systems (such as reservoirs), flow reversals, lateral diversion, or dynamic models with complex hydrographs (such as flushing and sluicing).

(5) The required resolution of the flow data will scale with the size of the watershed and the time of rise of the hydrograph. Because sediment transport is nonlinear with flow, averaging flows underpredicts transport. Therefore, flow data with finer temporal resolution may produce better results in flashy or fast-rising systems. Most large rivers and reservoirs use at least daily flow data during wet seasons. Computational time steps can be much smaller during large flows.

(6) Sometimes, modelers will select coarse (up to a month) time steps during dry seasons to save run time, using a single flow to simulate low-flow, low-transport conditions. Averaging the sediment loads from multiple daily flow records will not compute a representative load because the flow-load relationships are nonlinear. These errors can be minor, but modelers often back-calculate a flow that yields an average transport for the time step rather than simply averaging the flows, which almost always underpredicts transport (Figure 9-5).



Equation:

$Q_s=0.1*Q^2$

$Load=0.1*Flow^2$

Flow(cfs)		Load (T/d)
30	$Load=0.1*(30cfs)^2$	90
100	$Load=0.1*(100cfs)^2$	1000
80	$Load=0.1*(80cfs)^2$	640
Average Flow		
70	$Load=0.1*(70cfs)^2$	490
		Average Load
76	$Flow=\sqrt{(10*Load)}$	577

Figure 9-5. Example of how the nonlinearity of a simple transport equation affects averaging flows

Note: The average flow (70 cfs) will generate a lower sediment load from the rating curve (490 T/d) than the average sediment load computed from the individual flows (577 T/d). Because sediment continuity is often more important than flow continuity in sediment models, the higher flow (79 cfs) back calculated from the average sediment load (557 T/d) is a better representative flow for this period.

e. Sediment Data Requirements.

(1) Like most numerical models, sediment models have two data requirements: initial conditions and boundary conditions. Initial conditions provide model data at each location at the beginning of the simulation and boundary conditions feed the model data at the external nodes or cross sections throughout the modeled time period. A hydraulic model can work out initial condition errors, converging on a correct solution relatively quickly. However, errors in initial conditions can influence sediment model results for the entire simulation.

(2) Sediment model data requirements are listed in the left column of Table 9-2 and summarized in Figure 9-6. Sediment models require initial bed gradation data, at each location (Initial Conditions) and sediment loads, by grain class, at the upstream boundaries and tributaries (Boundary Conditions).

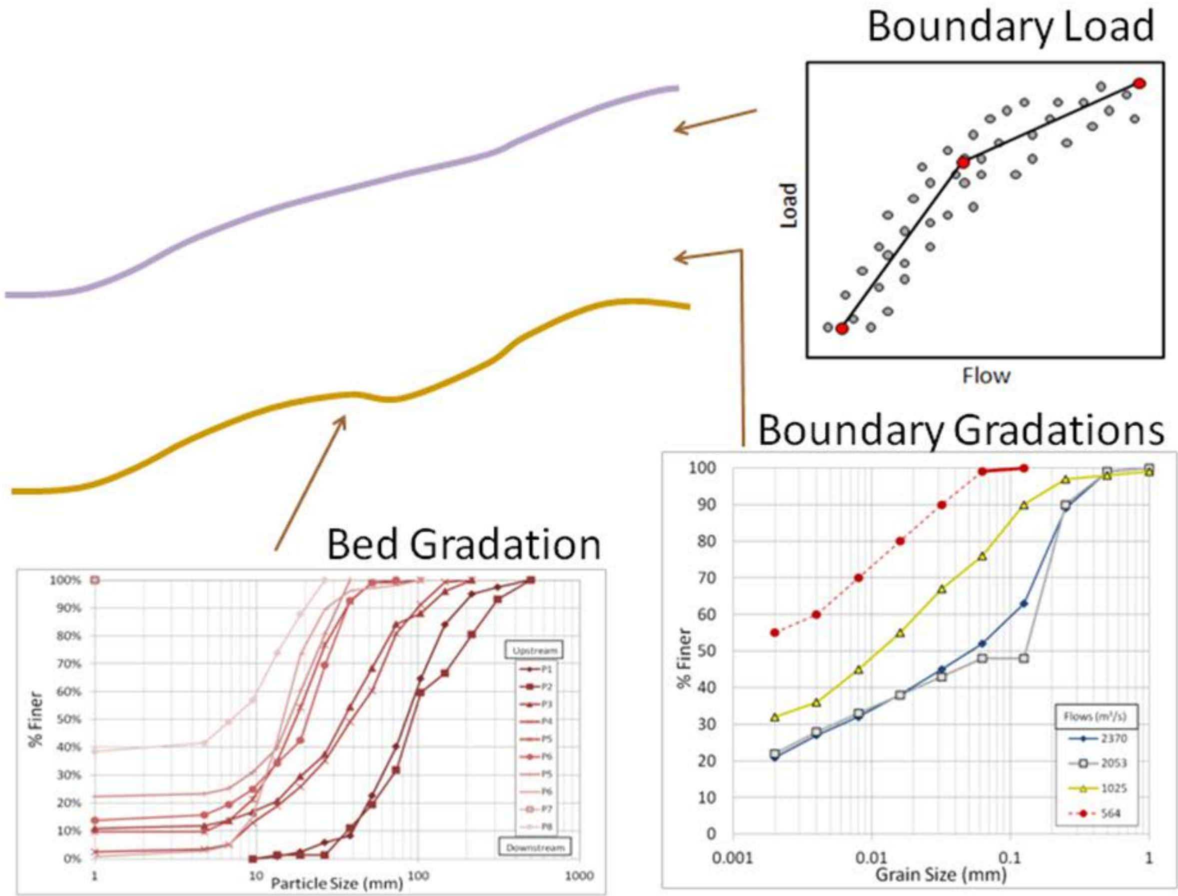


Figure 9-6. Sediment input data required from sediment models

(3) Initial Conditions.

(a) Bed Gradation Data.

• Besides the initial bathymetry, which should reflect the river geometry at the beginning of the simulation, the initial bed gradation is the main initial condition in sediment models. Most models require user-specified bed gradations at each cross section (1D) or computational node (multidimensional). This is also one of the most sensitive parameters, so the following discussion includes detail on sampling considerations, modeling strategies and common errors. Bed material sampling guidelines are also included in Appendix E.

- Bed gradations are spatially heterogeneous. They can vary dramatically in space, and those variations can be random or systematic in three dimensions (laterally, longitudinally, and vertically). This spatial variability can make selecting representative bed samples or sample locations challenging. Bed samples also tend to represent times and locations that are easier or safer to access, introducing data bias. For example, bed gradation samples are often collected from river bars exposed during low flows.

- However, bar gradation can vary dramatically within a bar, generally fining downstream and landward. When selecting a bar sampling location, it is important to select one that represents the bed gradation of the cross section or the reach represented by the cross section.

- Two-dimensional models often require coarser gradations in the thalweg, a phenomena observed in rivers. If the gradational data does not provide enough spatial detail to resolve coarser thalwegs, the modeler may need to coarsen these nodes based on engineering judgment to keep the model from scouring them.

- Bed gradation data represents a reservoir of sediment in the stream bed from which sediment can be eroded or deposited. In a 1D model, this reservoir occupies the entire width of the channel, and in some cases the width of the overbank as well. However, the river sediment may have a zero depth in the case of a rock outcrop or a concrete channel. Likewise multidimensional models can account for revetments with non-erodible elements.

- The gradation used in a numerical model should represent the reach, accounting for the vertical, lateral, and horizontal variations. Bed gradations should also address the engineering question at hand. For example, multidimensional models simulating scour can be very sensitive to coarse thalweg gradations. Alternately, mid-bar gradations are more likely to reproduce reach transport rates.

- The more variable bed gradations are, the more samples the model will require to simulate representative bed processes. Limited samples should capture regional trends and reach scale transition. Do not invest limited samples to resolve local or sub-reach scale features that will not improve the model. The biggest bed sampling challenge is selecting samples that capture important trends or variability while ignoring inconsequential noise. It is often useful for the modeler to accompany the sampling team into the field to adjust sample locations based on observed morphology or processes. Bed sample selection is a modeling decision.

- River bed gradations tend to fine downstream (Paola et al., 1992; Rice 1998; Ferguson et al., 2009). Downstream fining is usually gradual, but a sand-bed can transition to gravel over relatively short reaches (a few hundred meters) in response to slope change, backwater effects, or fine tributaries loads. Downstream coarsening is rare and tends to occur locally (Costigan et al., 2014) in response to regulated tributary loads, though can occur on larger scales in degrading rivers (Frings et al., 2010; Gibson et al., 2016).

- Unlike load data, when bed gradation data are noisy, it is usually better to use all the data than to try to distill it into reach or regional averages (Thomas 2000). Because the transport functions are exponential, longitudinally averaging bed gradation overpredicts transport.

- Bed gradations can also vary vertically. Sand-bed rivers tend to form a coarse layer at the base of bed forms and depositional systems record the gradational history of the system in their stratigraphy. However, bed forms on sand-beds tend to mix sediment enough to make these features temporary and their spatial and temporal scale is too small to influence model results or justify model attention.

- Reservoirs can have more dramatic vertical gradational trends, which can become important when modeling dam removals or flushing operations. Some rivers deliver clay pulses, while others (generally in glacial systems) transport extra gravel and cobbles during flood events, creating clay (Gibson and Boyd 2016) and gravel lenses (Evans et al., 2002) in the reservoir.

- Armoring is the most common vertical variation in river bed gradations. Model results are often very sensitive to these variations, particularly in gravel rivers. Therefore, considering armoring modeling approaches while collecting bed gradation data will produce a better model. In armored beds, where a thin cover layer (~2d_{90}) is substantially coarser than the bulk sediment below it, the modeler has two options: bulk gradations and separate armor/subsurface gradations.

  - Bulk sediment samples that include the cover layer and part of the subsurface layer are easier to collect, particularly in deep or swift water. These bulk samples can help modelers estimate bed gradations, but if the cover layer and subsurface layer are substantially different, they cannot be used directly.

  - Bulk samples are difficult to interpret, because the gradation depends on the relative proportion of armor layer sediments to subsurface sediments, which vary depend on the sample size and depth, which are very difficult to control. They also underpredict initial cover layer gradation and overpredict subsurface gradation.

  - Models that use bulk gradations without interpretive analysis or model warm-up strategies will initially overpredict scour and concentration, but will underpredict both after the initial event. Because bulk gradations do not reflect the actual river mechanics, they become a calibration factor.

  - A sediment model with separate armor and subsurface gradations will perform better. This requires separate samples. Usually subsurface samples are bulk shovel samples, while armor layers are sampled with particle counts either in the field or from photographs. In deep or swift water, bed photographs can help modelers infer surface and subsurface gradations from bulk dredge samples.

- Regardless of armoring or the data quality, most sediment models require the modeler to either adjust the gradations or run a warm up period. The bed gradation is related to the boundary load gradation and the transport function in a sediment transport model. The first thing a model will do is adjust itself to its own frame of reference, adjusting the bed gradation to respond to transport and the boundary loads.

- If the initial model bed gradations represent a quasi-equilibrium condition in the river, then the inflowing load and transport function should be specified to roughly maintain these gradations over the early model run. Most USACE models include a gradational hot start option or an automated warm-up tool to help coordinate the transport function and the bed gradation and to avoid big changes in the first few simulation time steps.

- Preconditioning the bed gradation is related to preconditioning the bathymetry (see paragraph 9-3c(4)(f)). AdH automatically preconditions the bed during a warm-up period. The model simulates bed sorting and updates bed gradations while maintaining constant bed elevations. In AdH, it is possible to set nodes as pass-through nodes in which the bed sorting and gradation is computed, but the bed elevation is not changed (USACE 2017a). CMS can start the bed change and bed sorting computations independently of one another. Preconditioning the bed gradations is important because inaccurate bed gradations can lead to drastically different morphologies.

(b) Cross Section Mobile Bed Limits (1D Only). Multidimensional models compute lateral shear and velocity distributions and can erode or deposit each node individually. One-dimensional models use cross-section weighted hydrodynamic assumptions and adjust wetted cross-section nodes together. Getting physically representative results from a 1D model usually requires subdividing the cross section into movable and immobile portions (or erodible and nonerodible sub-sections). The model can be sensitive to these decisions, particularly when transport functions compute transport per unit width, which are then multiplied by the width of the movable width of the cross sections.

(c) Vertical Scour Limits. HEC-RAS, AdH, and CMS all require vertical scour limits. Maximum scour depths or minimum scour elevations define the maximum vertical scour allowed. Accurately setting the vertical limit is particularly important when simulating sediment transport around structures or when bed rock or resistant substrate limit erosion.

(4) Boundary Condition: Sediment Loads (Rating Curve).

(a) Flow-Load Rating Curves.

- Sediment rating curves are the most common sediment boundary condition. A sediment rating curve defines the magnitude and gradation of inflowing sediment as a simple function of flow. Selecting a flow-load relationship can be challenging, because flow-load data usually include more than an order of magnitude of scatter. (See Figure 9-7 and Case Study 10A in Appendix N.) Gradation data for these loads are usually scarce, and it is often difficult to discern a trend with flow (Gibson and Cai 2016).

- Occasionally, wash load concentrations, expressed as milligrams per liter, are available. These are usually plotted against flow, but often do not correlate strongly with discharge; however, use of such graphs is encouraged when developing or extrapolating the inflowing sediment data.

- When developing a sediment flow-load rating curve, most analyses convert the concentrations to sediment discharge in tons/day and express that as a function of water discharge as shown in Figure 9-7. An order of magnitude scatter is shown at most flows. The scatter in a sediment rating curve can come from intra- or inter-event variability. The rising limb of a hydrograph generally delivers more sediment than the same flow on the falling limb and storm location or timing can deliver different loads with similar flows.

- Load variability can also be non-stationary, changing over time in response to land use changes or river engineering (for example, upstream dam). If the loads are non-stationary, using all the data to develop a flow-load relationship will bias predictive results. Test data stationary split with time series analysis, plotting early and late data separately can determine if the loads changed over time.

- A standard least-squares power curve fit (a linear fit through the logarithms of sediment concentration or sediment load and water discharge) underestimates the mean values of concentration or load. Ferguson (1986) proposed a correction factor for developing an unbiased estimate of the predicted values.

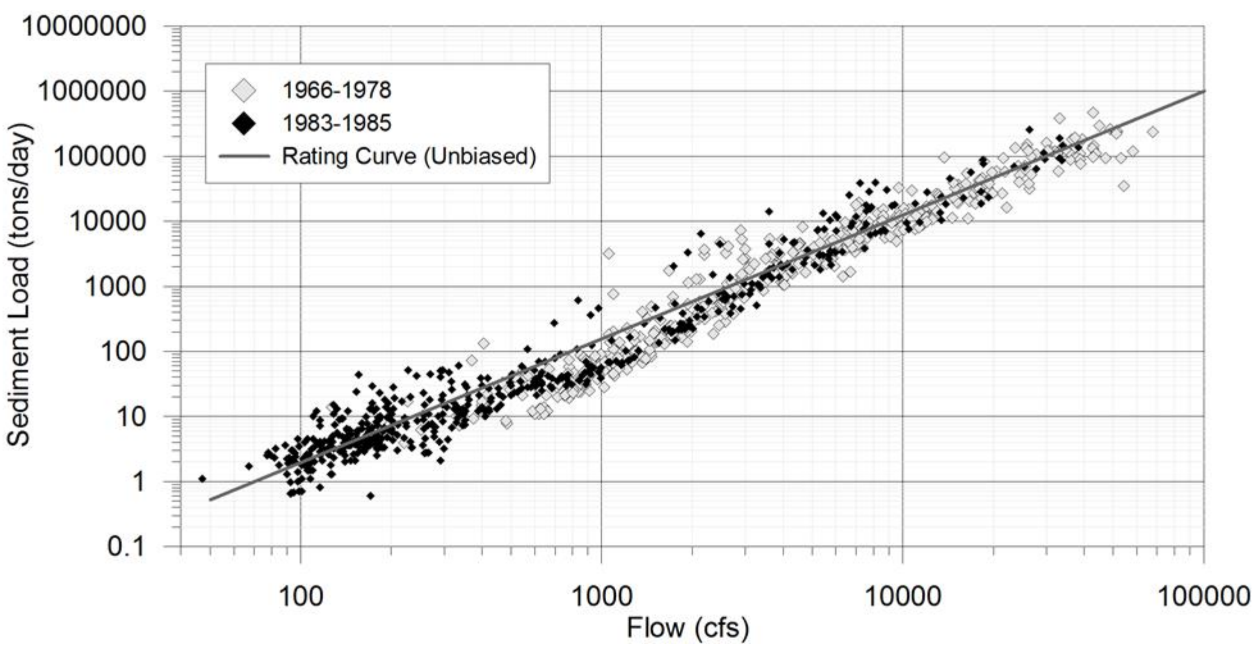


Figure 9-7. Sediment discharge rating curve from Russian River data (after Gibson 2011)

- Only use rating curves to define sediment boundary conditions if flow is coupled to sediment load. Wash load can be supply limited rather than capacity limited, which makes it less correlated with flow. Additionally, river management (such as gravel augmentation or reservoir flushing) can introduce sediment loads that do not correlate with flows.

- Historically, modelers used two different conventions to define upstream sediment boundary rating curves. Modelers can define upstream sediment boundary conditions based on sediment concentrations (mass/volume) or loads (mass/time). Both concentration and load conventions are useful and acceptable for USACE models. Because load is the product of concentration and flow, flow-load curves included flow on both axes. Therefore, flow-concentration curves have more scatter than flow-load curves, but only because the variables are independent. The unbiased corrector will work with either convention.

(b) Sediment Load Systematic Bias – Unmeasured Load. Modeling the unmeasured load can introduce a systematic bias when developing a sediment rating curve.

- First, most sediment samples measure water column concentrations, excluding transport, in an unmeasured zone close to the bed. Therefore, sediment samples usually exclude bedload and sometimes exclude the highest suspended-load concentrations. Even when bedload is a small fraction of the total load, model results can be very sensitive to bedload estimates, because bed erosion or deposition can be disproportionally sensitive to the bedload component. A relatively minor bedload component can drive deposition or erosion.

- Therefore, modelers usually augment measured sediment loads with estimates of unmeasured load, including measured or estimated bedload. There is no universal “bedload percentage” (which, in this section is defined as mass of bedload divided by the mass of the total load). Bedload fractions have often been estimated with “rule of thumb” percentages without much empirical support.

- Ten percent bedload is commonly cited. Turowski et al. (2010) found modelers and scientist often assumed 10% to 20% bedload in general and 20% to 40% for mountain rivers, “often without original data or references.” They traced these assumptions back to Maddock and Borland (1951).

- However, these assumptions are not general. Bedload fraction can vary between 0% and 100% (Lane and Boreland 1951; Turowski et al., 2010) and can vary dramatically between systems and with flow in the same system. Therefore, developing site-specific bedload assumptions based on local or regional evidence will generally improve model performance. Paragraph 4-10 includes more discussion of bedload and bedload measurement.

(c) Sediment Load Systematic Bias – Unbiased Corrector. Modeling the unmeasured load can introduce a systematic bias related to the use of log-transform regressions to develop the sediment rating curve.

- Least-squares regression biases logarithmic rating curves. Log-transform regressions compute the geometric mean instead of the arithmetic mean, which underestimates loads. There are several approaches to computing representative values from log-normally distributed data. USACE studies apply Ferguson's (1986) estimator most often. This unbiased estimator is based on the standard error of the regression. It is a multiplicative corrector in the flow-load or flow-concentration power function such that:

$$\mathrm{C} = \exp(s^2/2) * \mathrm{aQb} \qquad \text{Equation 9-6}$$

where:

C = concentration
Q = flow
a and b = the biased coefficients of the log transformed least-squares regression
$\exp(s^2/2)$ = the correction factor, where:

$$s^2 \equiv \sum_{i=1}^{N} \frac{(ln(C_i(obs) - ln(C_i(measurd))^2}{N-k} = \sum_{i=1}^{N} \frac{(ln(C_i(obs) - ln(aQ_i^b))^2}{N-k} \qquad \text{Equation 9-7}$$

- Alternately, Duan (1983) proposed a "smearing estimator" that is insensitive to non-normal regression residuals, which can cause Ferguson (1986) to overcompensate the load correction (Cohn et al., 1989; Gray and Simões 2008), where:

$$\mathrm{C} = \mathrm{E} * \mathrm{a}Q^b \qquad \text{Equation 9-8}$$

E is the smearing estimator:

$$E = \frac{1}{N} \sum_{i=1}^{N} exp(ln(C_i(obs) - ln(aQ^b) \qquad \text{Equation 9-9}$$

(d) Sediment Load Gradations.

- Most sediment models route sediment by grain class. Several independent variables can influence load gradation, including flow, tributary supply, bed gradation evolution, season, flood duration. The USGS often performs detailed parameter sensitivity analyses to determine the factors that drive gradation dependence. Modelers can use these detailed relationships to generate sediment time series by grain class.

- Rating curve boundary conditions require either constant sediment load gradations or gradations that vary as a unique relationship with flow. Load gradations are often rare, requiring modelers to interpolate, extrapolate, or even apply engineering judgment to fabricate gradational trends.

- Gradation does not trend consistently (Gibson and Cai 2017). Load gradation can fine or coarsen with flow (Figure 9-8), and even systems that have similar trends, can get there by different processes. For example, loads can coarsen with flow by maintaining a constant sand concentration while fine concentration drops, or by maintaining a constant fine contrition as sand concentration increases, or by increasing sand and decreasing fine (Walling and Moorhead 1989).

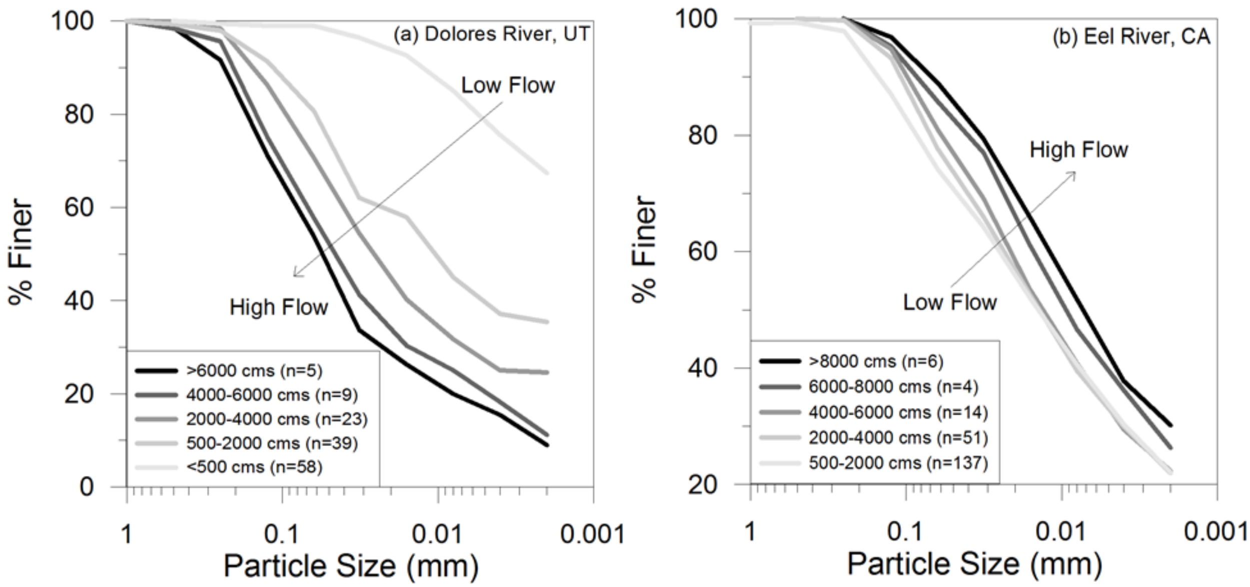


Figure 9-8. Suspended-sediment gradations coarsen with flow on the Dolores River (left) and fine with flow on the Eel River (right) (Gibson and Cai 2017)

- Bed gradation is usually much coarser than inflowing load gradation. Models that use the same gradation for the bed and the inflowing load almost always perform poorly.

- When measured loads are available, they usually exclude bedload. So modelers must augment measured loads with bedload mass and gradations. Bedload usually contains the size classes available in the bed. Lisle (1995) compared bedload to bed material gradations in nine very well graded/poorly sorted streams. He found that bedload gradations were close to bed gradations in some streams, but were substantially finer than the bed material in others.

- Bedload also varies substantially with flow. It is often limited to time flows competent to transport it, making bedload components very small during much of the year. When bedload is competent, its gradation generally coarsened with flow, the median particles size of bedload often approaching the bed gradation at bank flow. However, while bedload magnitude increases dramatically with flow, the fraction transported as bedload usually decreases with suspended sediment concentration and flow (Turowski et al., 2010).

- While suspended-sediment gradation data are more common than bedload data, they are still rare. Gibson and Cai (2017) surveyed over 80 gages, 20 models, and 10 studies, finding load gradation increasing and decreasing with flow. They found that, while the nonlinearity of the transport equations predicts that loads should get finer with flow (finer grain class transport increases faster than coarser grain classes), more gages, models, and studies demonstrate the opposite (load coarsening with flow) suggesting these systems are supply limited. Although data for flows larger than the 10% event is usually sparse, there is some evidence that while higher frequency events tend to coarsen with flow, larger events get finer.

- When data are available, evaluating sediment load by grain size class is recommended.

- Table 9-7 shows a procedure developed for the Clearwater River at Lewiston, Idaho (Gee 1982). The data in this table come from measured bedload and measured suspended load. Figure 9-9 is the graph of that data set. Note that, due to the availability of various size fractions in the bed and the suspended-load gradation for a given flow, the transport rate does not necessarily decrease with increasing particle size.

(e) Tributaries. Tributaries generally have less sediment data than the main stem and boundary conditions are more difficult to estimate. Assess each tributary for sediment delivery potential during the site reconnaissance. For example, look for a delta at the confluence of the tributary and main stem. Look for channel bed scour or deposition along the lower end of the tributary. Persistent changes in bed material gradation in bars along the receiving side of the river from upstream to downstream of a tributary indicate a potential significant influence. Drop structures or other controls designed to stabilizing a tributary is evidence of historic instability. These observations guide the tributary sediment estimates when data are scarce.

(5) Cohesive Data.

(a) Cohesive model calibration deserves special consideration. Cohesive deposition physics are straightforward for individual particles; they apply similar empirical modifications to Stokes law as cohesionless sediment. Flocculation and “apparent particle size” (Walling and Moorhead 1989) complicate deposition, since cohesive particles can form aggregates that make them deposit faster.

(b) Sometimes modelers coarsen the cohesive fraction of the measured, dissociated, grain size distribution to reproduce prototype deposition. Artificially coarsening fine sediment in the model can reproduce aggregates in the field that were disassociated in the laboratory. However, despite these complications, reliable, physically based equations predict cohesive deposition relatively well if the grain size distribution is appropriate. Cohesive sediment erosion, on the other hand, is much more difficult.

(c) Cohesive erosion depends on over a dozen properties of the cohesive soil and the water eroding it. The scientific consensus is that no universal equation describes cohesive erosion. Most sediment models predict cohesive erosion with relatively simple equations that lump all the variability into lump empirical parameters, usually critical shear and erodibility. These parameters can vary by 5 orders of magnitude. SEDflume or soil jets can measure these data, but even then, results can span multiple orders of magnitude.

(d) Cohesive model parameters must either be measured or calibrated (Briaud et al., 2001; Huang et al., 2006; Ravens 2007). Some guidance is available for predicting these parameters (Winterwerp et al., 2012). The model should be designed to replicate historic scour if cohesive parameters are not available. In these cases, other data or parameters must fixed so the cohesive parameters can be the main calibration parameters.

Table 9-7
Distribution of Sediment Load by Grain Size Class (after Gee 1982)

Water Discharge: 35,000 cfs				**Total Bedload, tons/day: 130 Total Suspended Load, tons/day: 1,500 Total Sediment, Load: 1,630**		
Grain Size Diameter, mm	**Classification**	**Percent Bedload**	**Bedload tons/day**	**Percent Suspended Load**	**Suspended Load, tons/day**	**Total Load Column (4) + (6) tons/day**
1	2	3	4	5	6	7
< 0.0625	silt and clay	0.04	0.05	54	810	810
0.0625–0.125	very fine sand	0.10	0.13	10	150	150
0.125–0.25	fine sand	2.75	4.00	13	195	199
0.25–0.50	medium sand	16.15	21.00	19	285	306
0.50–1.0	coarse sand	13.28	17.00	4	60	77
1.0–2.0	very coarse sand	1.19	2.00			2
2–4	very fine gravel	1.00	1.00			1
4–8	fine gravel	1.41	2.00			2
8–16	medium gravel	2.34	3.00			3
16–32	coarse gravel	6.33	8.00			8
32–64	very coarse gravel	23.38	30.00			30
>64	cobbles and larger	32.03	42.00			42
TOTAL		100.00	130.18	100	1,500	1,630

Notes:

1. The distribution of sizes in the bedload is usually computed using a bedload transport function and field samples of bed material gradation. The bedload rate is rarely measured and may have to be computed.

2. The suspended load and its gradation can be obtained from field measurements. The bed material portion of the suspended load may be calculated using a sediment transport function, but the wash load can only be obtained through measurement.

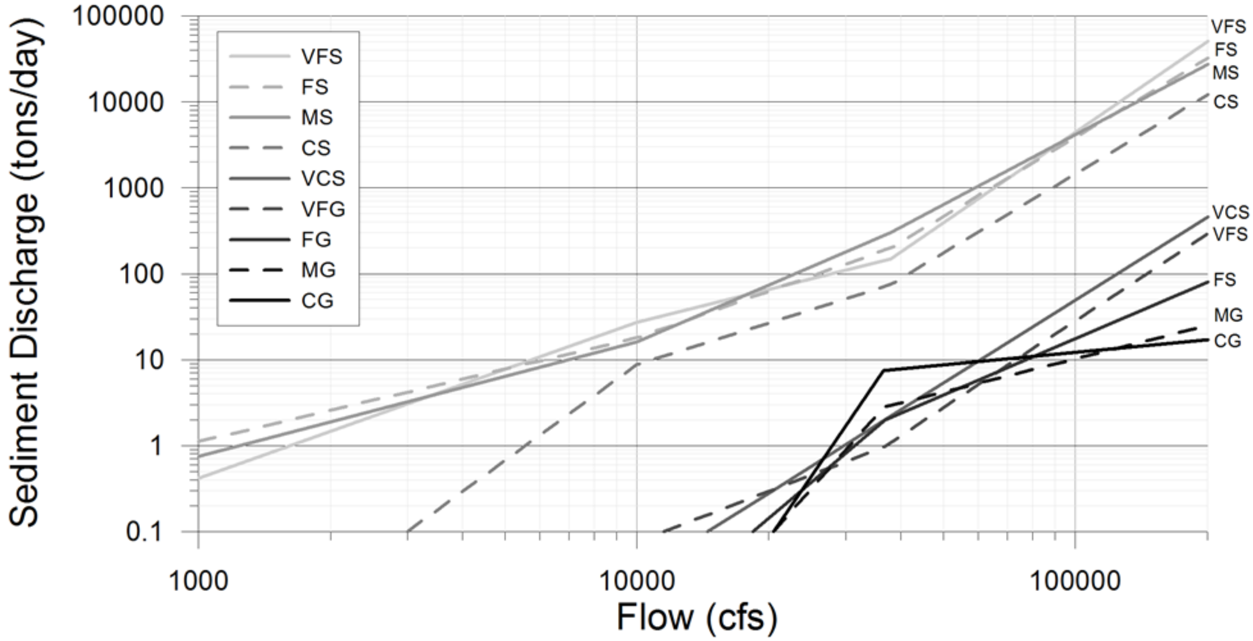


Figure 9-9. Sediment load curves by grain class (after Gee 1982)

(6) Equation and Parameter Selection. Sediment transport models generally combine several equations and algorithms to simulate different processes. These algorithms often include parameters that users can adjust. Each sediment equation or formulation is based on different data and conceptual models. Equations and algorithms include: the fall velocity equation, hiding functions, grain shear partitioning, entrainment rates, temporal limiters, adaptation lengths, bed change assumptions, bed roughness factors/bed form predictors, and mixing algorithms. However, selecting an appropriate transport function, including the grain shear partitioning function and the bed mixing algorithm, are generally the most important algorithm decision modelers make.

(a) Transport Function. Sediment transport functions compute sediment mass fluxes based on hydrodynamics and sediment properties. Because transport is nonlinear and dependent on multiple processes, these functions are notoriously uncertain.

- Two “appropriate” transport functions can differ by an order of magnitude. Selecting a transport function is more a process of ruling out inappropriate transport functions (such as those developed for different transport conditions) and testing the remaining functions to evaluate the one that performs best in the modeled system.

- Transport functions have several empirical parameters based on the best fit to noisy data. In particular, most have a critical transport parameter (such as τ_c^*, τ_{rm}^*, VS_c) that may be adjusted to improve transport function performance for a specific system (see discussion of τ_c^* in paragraphs 3-3 and 5-2). Paragraph 5-7 and Appendix B provide additional background on sediment transport functions.

- Consult additional references for more information on selection of sediment transport functions (Simons and Sentürk 1992; Thomas et al., 2002; Garcia 2008b; Gray and Simões 2008; Stevens and Yang 1989).

(b) Grain Shear Partitioning. Shear stress drives most transport functions. However, only part of the river shear translates directly into particle movement.

- Many transport functions partition the shear stress into the grain shear, which actually transports sediment and other components (for example, form drag) that do not.

- Shear partitioning is difficult and controversial. Partitioning algorithms include simple equations like Strickler and complicated algorithms, like the Larsen-Copeland's iterative partitioning algorithm, which is more complicated than their transport function. Ackers and White (1973) considered computing grain shear too uncertain, and based their function on total shear. Yang avoided the issue with the stream power approach.

- In HEC-RAS, most transport functions have grain shear algorithms built into them. However, different models may give different results with the same transport function if they are using different grain shear partitioning algorithms.

(c) Bed Mixing Algorithm, Active Layer Thickness, and Vertical Gradations.

- Bed processes like mixing, sorting, hiding, and armoring, affect transport of individual grain classes by making them more or less available to the flow field. Transport can be extremely sensitive to the mixing model selected. Bed mixing models stratify bed sediment into layers, allowing surface sediment to fine or coarsen independently from the sub-surface. They fall into two main categories:

  - Lagrangian Mixing Models: These are two- to three-layer models, where the layers change size and elevation in response to bed change. Lagrangian models include "armoring algorithms" (Thomas 2002; Copeland 1992), which subdivide the active layer, allowing a thin cover layer to coarsen independently and slow or stop erosion.

  - Eulerian Stratigraphy Models: These models subdivide the bed into layers of fixed elevation and thickness, allowing users to specify discrete layer materials and models to compute detailed vertical stratigraphy. Detailed stratigraphy requires core data that are rarely available, and Eulerian models generally do not include the armoring algorithms available in the Lagrangian methods, but they keep track of vertical trends more carefully. These models often compute transport with a Lagrangian active layer that samples the vertical Eulerian stratigraphy.

- Bed mixing and armoring algorithms are generally complex and self-contained, without much user manipulation. Selecting an appropriate mixing algorithm for the system and modeling problem can be the most important modeling decision (Thomas 2000). However, some mixing methods are sensitive to user parameters. Active layer methods, for example, are sensitive to the active layer thickness (Gibson and Piper 2007) and many models expose it as a user parameter.

- The most important modeler input into bed mixing algorithms is the bed gradation or gradations. Both Lagrangian and Eulerian bed mixing algorithms subdivide bed sediment into gradationally distinct layers. A well selected bed mixing algorithm will converge to a vertical bed gradation that approximates the actual system stratigraphy. However, allowing the bed to evolve during the simulation can cause artificial scour and transport while the mixing algorithms converge.

- There are two common approaches to avoid spurious results from initializing mixing algorithms in the simulation: (1) collect depth-dependent bed gradation samples (cores or separate cover layer and subsurface gradations), or (2) hot start layer gradations, running an extended constant flow through the model and using the final layer gradations to initialize the actual simulation. The warmup/hot start simulation can double as the robustness test (paragraph 9-4b).

9-4. Modeling Process. USACE follows consistent, systematic, sequential, sediment modeling presented in training document (TD)-13 (Gee 1982) and ASCE Manual No. 110, Chapter 14 (Thomas and Chang 2008). This process starts with a three-phase calibration process to improve model performance and stability. Models must demonstrate that they can replicate past river behavior, over the range of conditions and at the necessary level of detail, before they can predict future conditions or sediment responses to alternatives. Models that reproduce the past are much more likely to reliably predict the future. The modeling process includes four steps before alternative analysis, summarized in Table 9-8.

Table 9-8
Steps in a Sediment Modeling Study

Modeling Stage	Flow	Bed Elevation
Stage 1: Hydraulic Calibration (paragraph 9-4a)	Variable	Fixed
Stage 2: Steady Flow-Mobile Bed Analysis (paragraph 9-4b)	Fixed	Variable
Stage 3: Dynamic Calibration (paragraph 9-4c)	Variable	Variable
Stage 4: Verification (if possible) (paragraph 9-4d)	Variable	Variable
Stage 5: Alternative Analysis (paragraph 9-4e)	Variable	Variable

a. Stage 1: Hydraulic Calibration.

(1) Sediment models are very sensitive to the underlying hydraulic models driving them. Therefore, it is essential to build a sediment model on a robust, accurate, hydraulic model. The hydraulic model should be calibrated to known water surfaces for the full range of flow. Available channel velocity measurements should also be reviewed and compared to model results.

(2) Steps and guidance for assembling and calibrating a 1D hydraulic model are available from a number of sources, including the HEC-RAS Users and Hydraulic Reference Manuals (HEC 2012). Mobile bed modeling will magnify errors in model assembly and hydraulic parameter selection. A calibrated hydraulic model performs well over a full range of flows without abrupt changes in velocity or water surface elevation or overbank flow that are not warranted by physical conditions.

(3) The hydraulic calibration should specify coefficients and boundary conditions that reproduce field measurements with the model within an acceptable error range. Computed results should be compared with field measurements to identify data deficiencies or physically unrealistic values. To improve the agreement between observed and calculated values, modelers can adjust model coefficients and boundary conditions, but only within the bounds associated with their uncertainty. Model calibration should avoid physically unrealistic coefficients. Over-adjusting coefficients or data to force agreement with field data often masks other model problems that should be fixed.

(4) Bed Roughness.

(a) Bed roughness is the most common hydraulic calibration parameter. Sediment models often begin with existing hydraulic models constructed for other purposes. Modelers should evaluate the hydraulic calibration of legacy models, to make sure it is appropriate for the sediment modeling objectives. Hydraulic parameters like bed roughness can be the most sensitive parameters driving the sediment results in the model. Therefore, it is essential to calibrate the hydraulic model carefully before adding the complexity of a mobile bed.

(b) It is also important to note that the roughness value used in hydraulic models may not be appropriate for sediment transport. For example, if the bottom roughness in the hydraulic model includes vegetation (which should be considered separately from bottom roughness), this will lead to excessively large bottom shear stresses and overestimation of sediment transport.

(c) Many hydraulic models target flood flows, and are calibrated for extreme events. However, the most efficient sediment transport flows over time (the channel-forming discharge) tend to be much lower. Therefore, while it will be important to simulate flood hydraulics faithfully, because they transport exponentially more sediment, good hydraulics for smaller recurrence flows can be even more important. Before adding sediment data, calibrate the hydraulic model to multiple water surface elevations (such as base flow, bankfull flow, and flood flow) as depicted in Figure 9-10, or calibrate it to an unsteady flow series.

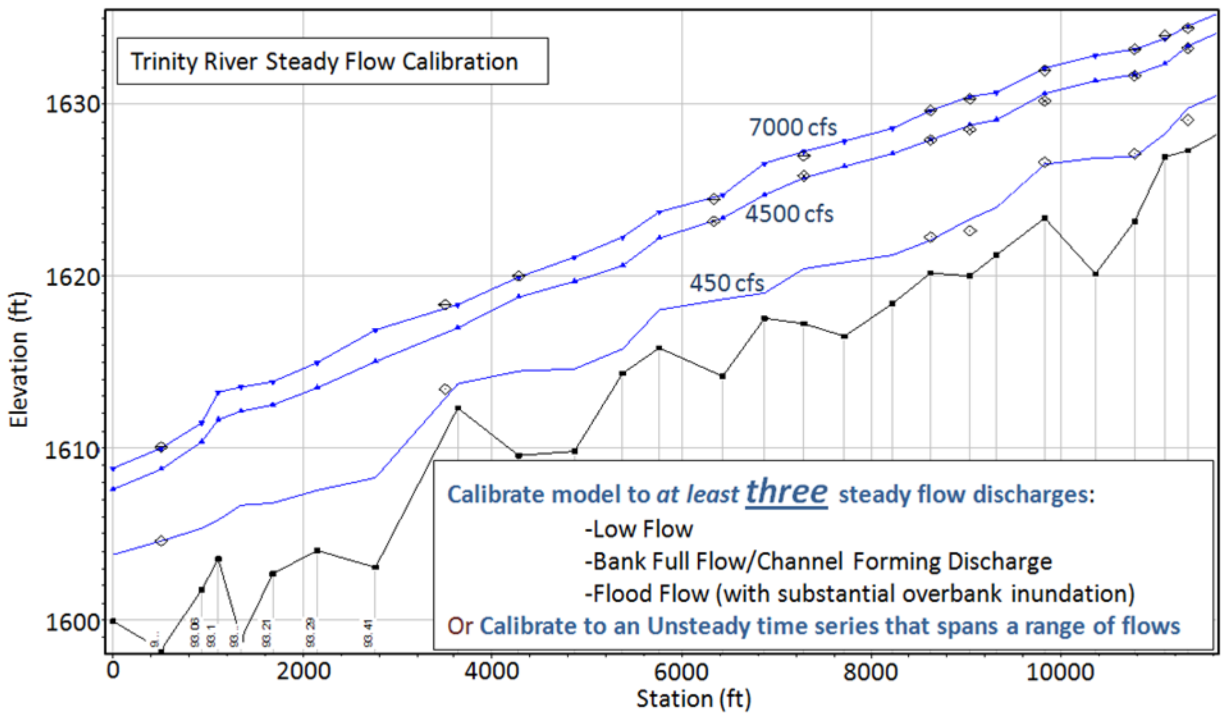


Figure 9-10. Hydraulic calibration, adjusting n-values to match water surface elevations for three different flows

(d) Roughness can vary with flow in complicated ways. For example, bed roughness on the Cowlitz increases with flow until approximately 40,000 cfs, which pushed the bed forms into a new regime, dropping bed roughness dramatically (Gibson et al., 2010). Bed roughness predictors can capture these effects, but can also be noisy and uncertain, and can introduce nonlinear effects as they shift between regimes. Modelers should generally avoid major roughness changes over short distances (for example, between adjacent cross sections). Also, consider that field data are collected from a single time frame and may not be representative of long-term conditions.

(e) When there are no reliable field measurements, bed roughness predictors (like those developed by Brownlie, Strickler, Limerinos, Jarrod, and others) can help estimate roughness parameters (see paragraph 5–2 and HEC 2016b). Other options include review of regionally similar river values and calibrated photographs (Barnes 1967; Chow 1959) for the overbank and channel portions.

(5) Smooth Conveyance Transitions (1D).

(a) In addition to bed roughness, carefully evaluate overbank conveyance and ineffective flow areas in 1D hydraulic models before adding sediment data. Good 1D hydraulic modeling practice includes reviewing overbank flows, making sure that flow gradually transitions from the channel to the overbanks and vice versa. One-dimensional models do not explicitly compute flow transitions between floodplains and channels. Careless modeling can push water erratically from the overbank to the channel or even between overbanks. A simple test is to evaluate a range of flows from normal to slightly above channel capacity. Review channel and overbank flows in the model output, looking for abrupt changes between the channel and overbank.

(b) Sudden and erratic conveyance transitions can cause problems for hydraulic models, but are even more detrimental to sediment transport models. One-dimensional models usually compute sediment transport based on channel hydraulics. If conveyance moves erratically between the channel and floodplains, the channel hydraulics will vary dramatically, over- or underpredicting transport from cross section to cross section.

(c) 1D modelers can use ineffective flow areas and overbank n-values to make conveyance transition gradually from cross section to cross section as part of the hydraulic modeling phase, before adding mobile bed complexity.

(6) Multidimensional, Model-Specific Issues.

(a) The calibrated bottom roughness values from one model may not be appropriate for other models with different dimensionality or different formulations for flow drag due to bottom roughness, vegetation, ice cover, etc. Roughness values calibrated from a 1D model may not be appropriate for 2D or 3D models and vice versa due to the way the geometry and bottom friction are represented in the models. 1D and 2D models commonly use the Manning's friction law to estimate the bottom shear stress. However, the Manning's friction law is not appropriate for 3D models, which typically use a log-law formulation that uses near-bed current velocities and a roughness height. AdH and CMS offer several bottom roughness options and AdH even allows for different friction laws for different regions of a mesh.

(b) Modeling turbulence is important to accurately model flow separation, recirculation patterns, and eddies in multidimensional models. Turbulence affects the sediment mixing/diffusivity coefficient in both 2D and 3D models. In most models, the sediment diffusivity coefficient is estimated as the turbulent eddy viscosity divided by the Schmidt number (see Spasojevic and Holly 2008). Three-dimensional and sometimes 2D models require estimates of vertical diffusivities or include methods that assume vertical diffusivity distributions (Brown et al., 2014; Sánchez et al., 2014).

(c) Calibration of turbulence may be important in 2D sediment models, and it is more important in 3D sediment models. Most 2D and 3D models offer several options for turbulence modeling with varying degrees of complexity and computational burden. Depending on the application, different turbulence models may be necessary. In general, the turbulence model and its coefficients are important model calibration settings and parameters that should be calibrated. In 3D models, the vertical eddy viscosity is very important specifically for suspended-sediment transport since it directly affects the carrying capacity of sediments in water column.

(d) Multidimensional models generally require longer spin-up (preconditioning or warm up) times than 1D models. The spin-up period is the period over which the artificial effects from approximate or inaccurate initial conditions are removed. For example, multidimensional models are often initialized with a constant water surface or a dry bed. The results from the warm up period should not be included in the model calibration. Therefore, properly determining when the model has finished the spin-up period is an important part of model calibration.

b. Stage 2: Steady Flow-Mobile Bed Analysis (Robustness Test).

(1) The second step of the USACE sediment modeling process (Table 9-8) is the steady flow-mobile bed analysis. The steady flow, mobile bed step (constant flow modeling) is an important and often overlooked stage of sediment model development. Hydraulic calibration fixes the bed and varies the flow to simplify the problem and isolate geometry and model structure problems before adding complexity. Similarly, allowing the bed change in a fixed flow scenario adds incremental complexity. This step simplifies the mobile bed problem. It isolates select model components, making model problems easier to identify than they would be when both flow and bed are dynamic (Gee 1982; Thomas and Chang 2008).

(2) The steady flow-mobile bed modeling phase can expose model deficiencies and numerical instabilities. This phase hones two model components in particular: (1) the relationship between spatial and temporal resolution ($\Delta x/\Delta t$) of the model, and (2) the relationship between the inflowing load and gradation and the transport function.

(3) A specific instance of constant flow sediment model evaluation is sometimes called the "robustness test" (Thomas and Chang 2008). The robustness test specifies the channel-forming discharge as the constant flow. The robustness test is built on channel-forming discharge theory that the entire flow regime can be collapsed into a single, constant flow that generates the same morphological change (see paragraph 7-3h(3) and Case Study 6A in Appendix N). Based on this premise, running the channel-forming discharge in a model for an extended period should produce the same results as running the entire period of record. If the study reach, as reflected by the initial model geometry, is in quasi-equilibrium, running the effective discharge for decades should converge on a stable model geometry that does not depart much from the starting geometry.

(4) It is also important to evaluate the model with several long-term, constant flows to evaluate model suitability for both existing and alternative conditions. Additionally, if the system is not in equilibrium (such as a reservoir or degradational reach) no constant flow will converge on the current geometry. Including sensitivity tests at this stage are helpful to illuminate potential model issues. For example, when the boundary concentrations are increased, there should be a deposition trend in the interior of the model.

(5) Models often compute higher sediment transport rates at the beginning of the simulation because they flushed fines as they form the bed layers. It is important to balance the sizes in the inflowing bed material load based on the transport potential and bed gradation. The scatter in measured data usually justifies smoothing these data, but the adopted curves should remain within that scatter.

(6) A successful steady flow-mobile bed test eventually reaches equilibrium. The bed change at each cross section asymptotically approaches a constant elevation without major oscillations or instabilities (Figure 9-11).

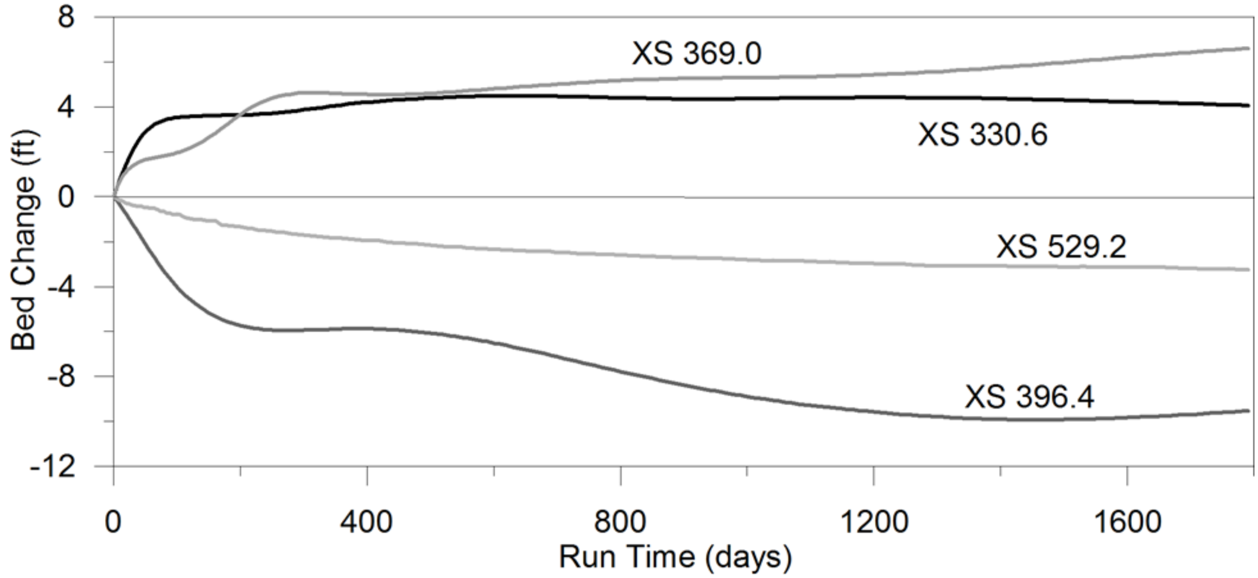


Figure 9-11. Results from a constant flow mobile bed analysis (redrawn from Copeland et al., 2020); the model appears numerically stable because the bed elevations approach constant elevations

c. Stage 3: Model Calibration. All hydrologic and hydraulic models should be calibrated. Calibration adjusts uncertain parameters to reproduce observed river behavior. Models that replicate the past are much more likely to reliably predict the future. Because sediment models have multiple uncertain and sensitive parameters, imbedded in nonlinear equations and algorithms, calibration is particularly important before making management decisions with a mobile bed model.

(1) Calibrated Model vs. Computational Experiment.

(a) Calibration is important for all hydrologic and hydraulic models. However, three factors make calibration particularly important for sediment models.

- Equations are nonlinear, usually power functions, and different equations (that are each "appropriate" for the hydraulic and sediment conditions according to applicability criteria) often generate results that differ by 100% or more.

- Sediment models have more free or uncertain parameters than hydraulic models.

- Input data (such as loads and gradations) often vary by orders of magnitude.

(b) Because of these factors, Thomas and Chang (2008) made a distinction between a "sediment model" and "computational analysis," limiting the former to calibrated simulations. They state: "Many studies will not have sufficient field data to calibrate models … Such studies will be beneficial because they will include the full computational capability of the model. Consequently, they will be called *computational analysis* studies rather than *computational model* studies."

(c) While calibration is essential, it also tends to be the most time-consuming and expensive portion of a modeling project, often requiring over half of the study time and budget. Applying a partially calibrated model to simulate project alternatives can make the calibration process more efficient. Applying partially calibrated models to the project can anticipate model problems before the modeler invests too much time calibrating a model that they will have to change (and recalibrate) to address the project questions.

(2) Selecting Calibration Parameters. Sediment models have several free parameters, many of which vary spatially or temporally. This makes them more like hydrology or groundwater models than hydraulic models, which are often calibrated by adjusting the roughness. To avoid equifinality problems (see paragraph 9-4c(6)), modelers should isolate a limited set of calibration parameters to adjust, leaving the rest fixed. The calibration parameters should be those that are sensitive and uncertain (Table 9-9). Selecting calibration parameters follows a four-step process:

(a) Analyze the sensitivity and uncertainty of the parameters and data.

(b) Fix any parameters or inputs that are insensitive, even if they are uncertain.

(c) Fix any parameters or inputs with low uncertainty (such as based on good data) even if they are sensitive.

(d) Identify one or two parameters most sensitive and least certain parameters to adjust. These are the target calibration parameters.

Table 9-9
Target Calibration Parameters Are Those That Are Both Sensitive and Uncertain

	Insensitive Parameters	Sensitive Parameters
Low Uncertainty Parameters	Fixed Parameters	Fixed Parameters
Uncertain Parameters	Fixed Parameters	Target Calibration Parameters

(3) Calibration Evaluation Data and Techniques. Calibration replicates past river behavior, evaluating the model against historical data. Modelers use a wide variety of historical data to calibrate sediment models. The most common are summarized in Table 9-2 and Figure 9-12. The calibration metric is often driven by data availability. However, the model calibration should replicate processes that drive the project and alternatives (such as models designed to predict bed change should be calibrated to bed change, and models designed to predict

concentration trends should be calibrated to concentration measurements, when possible). The different types of historical, calibration evaluation data are described below.

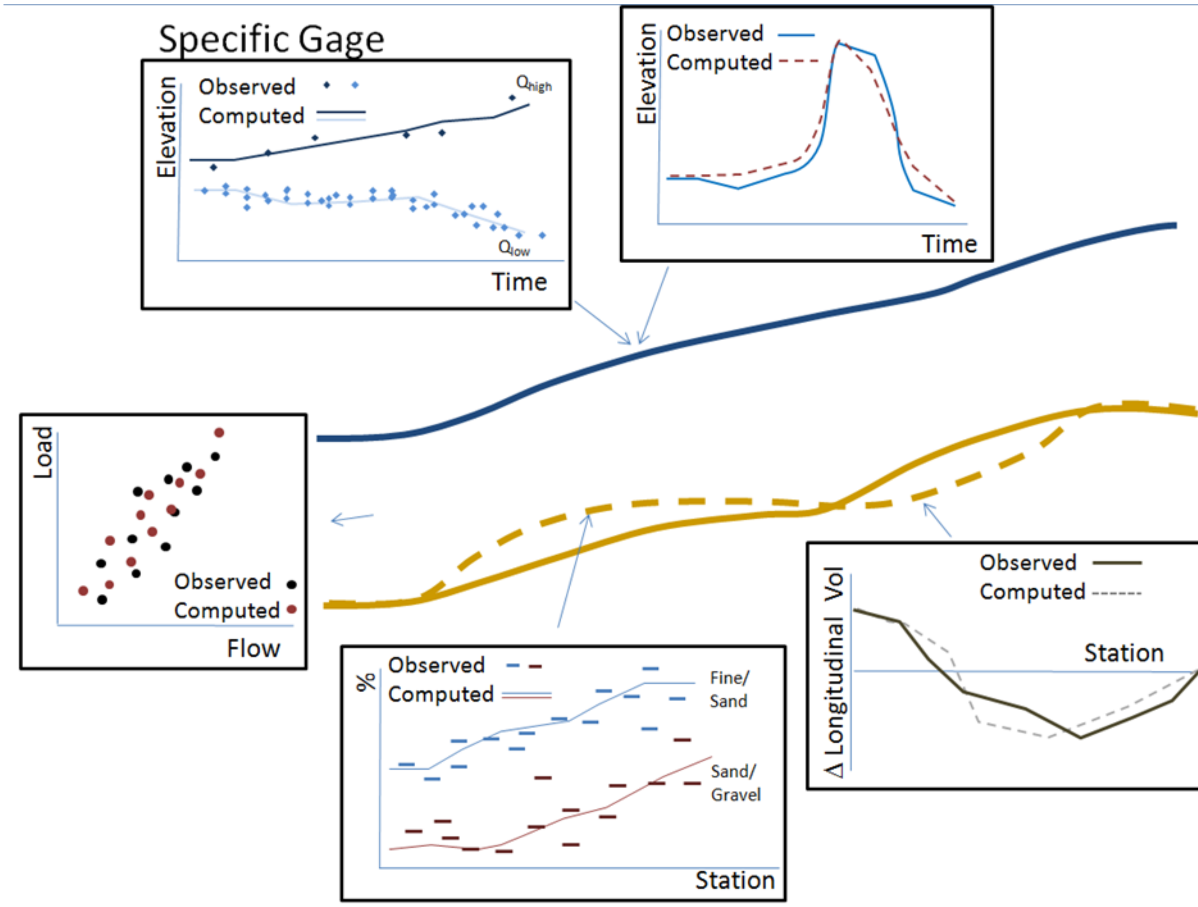


Figure 9-12. Calibration data to evaluate sediment models

(a) Repeated Bathymetry.

- Bathymetry is often the most expensive data to collect. However, repeated bathymetry is almost always the most reliable calibration data if the system is actively aggrading or degrading (long-term bed change exceeds the annual or seasonal bed variations). Bed elevations are easier to measure reliably than concentrations. Repeated cross sections or multi-beam bathymetries provide bed volume change between the surveys, which modelers can compare to simulated volume change.

- Unless the model objective involves isolating the impact of a particular event (such as flood scour or flush concentrations), the more years that elapse between repeated cross sections, the more valuable the calibration data are. Longer calibration periods are likely to average out natural variability from hydrologic, sediment source, and transport process, providing more reliable estimates of long-term volume change trends.

- In addition to volume change, sediment models can replicate bed elevation change measured from repeated bathymetry. Multidimensional models can evaluate the spatial distribution of bed change residuals, the difference between measured and computed bed change. Sánchez and Wu (2011) successfully calibrated a 2D CMS model at Ocean Shores beach just north of Grays Harbor inlet, Washington State. Figure 9-13 shows a comparison of the measured and computed bed changes between May 6 and 30, 2001.

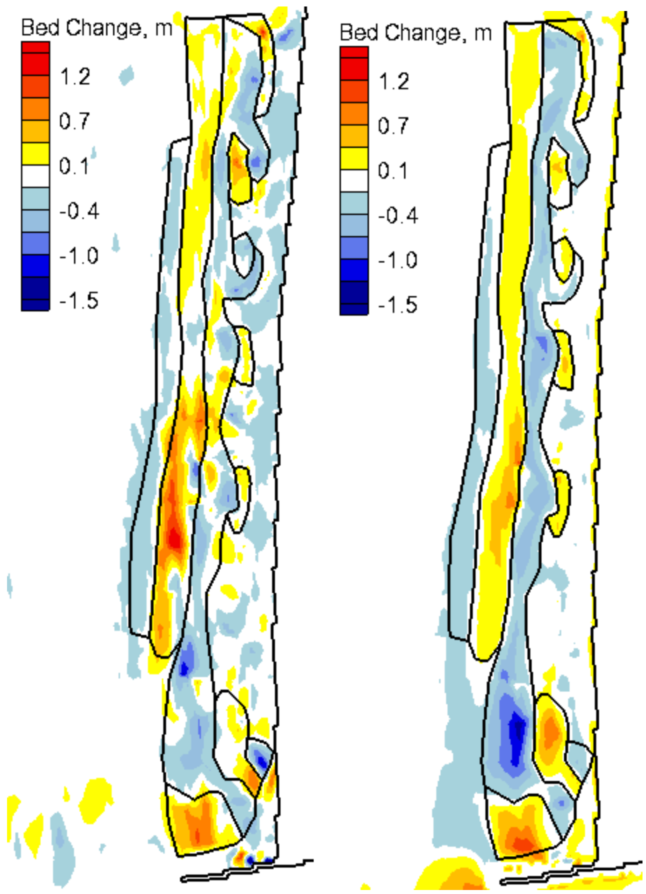


Figure 9-13. Measured (left) and computed (right) bed changes during May 6 to 30, 2001 (from Sánchez and Wu 2011)

- Identifying a representative bed elevation for a cross section can be challenging in 1D models. Some measure of average bed change usually preforms better than thalweg or invert change because the thalweg can deposit more or less than the rest of the cross section. The thalweg can even deposit while most of the cross section erodes, or vice versa (Gibson and Nelson 2016). Computing mass or volume change from repeated bathymetry captures total cross-section change, making them better calibration data for 1D models (Figure 9-14).

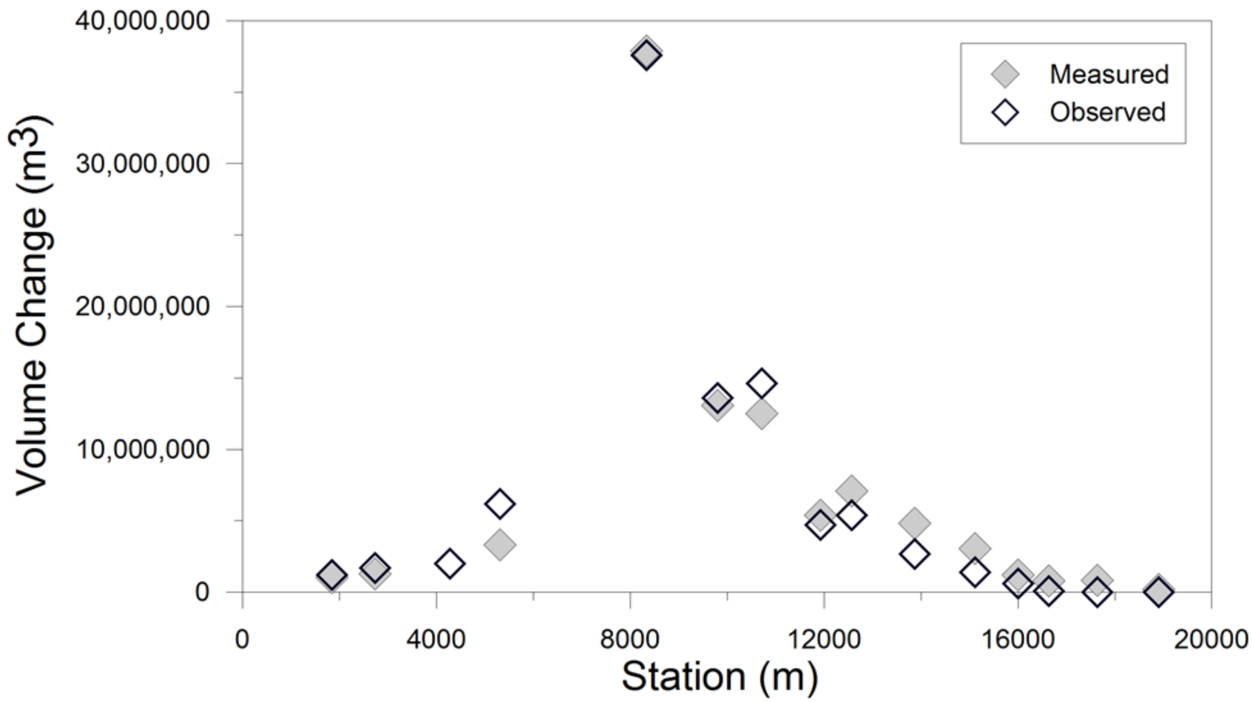


Figure 9-14. Measured and computed volume change for the Arghandab Reservoir from 1952 and 1971 (from Gibson and Pridal 2015)

- 1D calibrations are often evaluated with cumulative longitudinal volume change curves (Figure 9-15). Longitudinal cumulative volume (or mass) change plots sum observed and computed volume change from upstream to downstream. These plots can compare computed and observed volume change when the model uses more cross sections or different cross-section locations than the model.

- 1D models are generally reach scale tools, more appropriate for modeling morphological change on reach scales than reproducing individual cross-section change. Longitudinal cumulative mass or volume plots reflect this analysis scale, smoothing erratic adjustments in observed and modeled cross sections, to compare reach scale trends. In a longitudinal cumulative volume change plot, the value associated with the downstream station reflects the total volume change in the reach. Positive slopes indicate regions of deposition while negative slopes indicate regions of erosion.

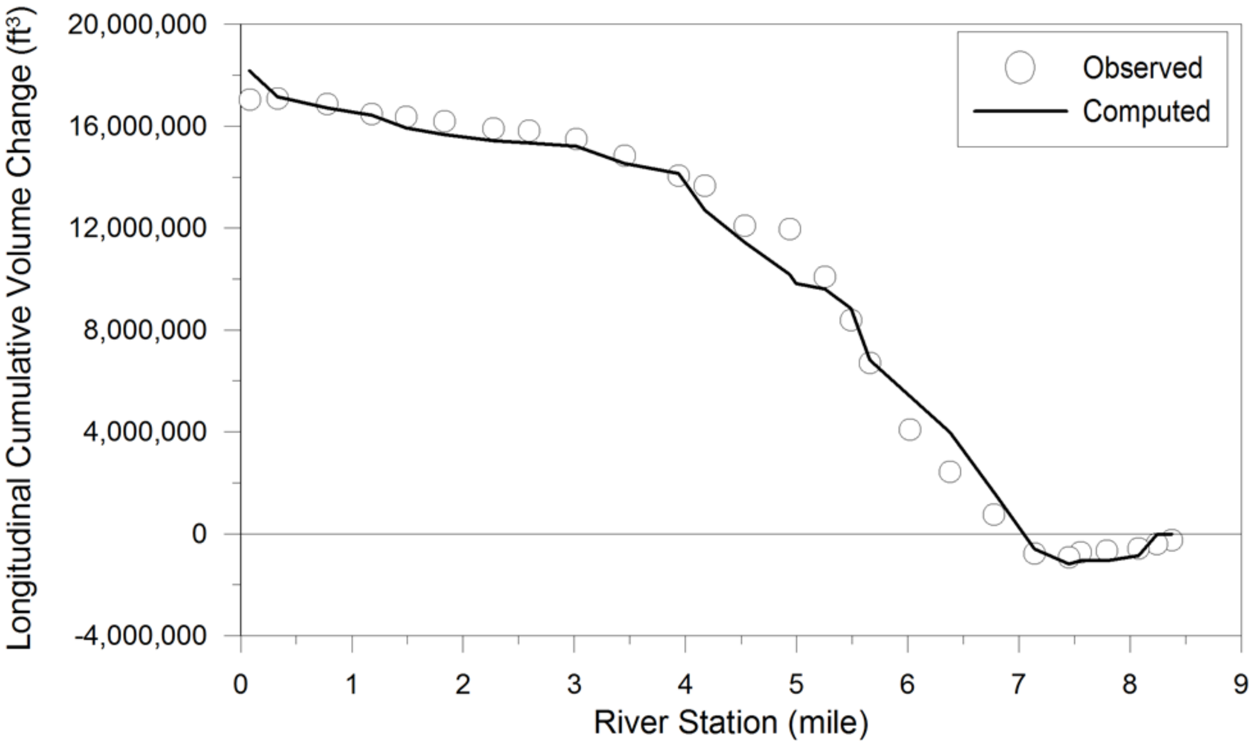


Figure 9-15. Longitudinal volume change plot for the White River (after Gibson et al., 2017); the volume change between repeated cross sections (in 1984 and 2009) was computed for each station, accumulated from upstream to downstream, and compared to the HEC-RAS computation volume change results for the same period

- Repeated bathymetry measurements of sand waves can also estimate bedload transport. The ISSDOTv2 approach (Abraham 2011) is the most common approach for this analysis. For example, Abraham et al. (2015) used repeated bathymetry surveys of the Ohio River, Illinois, to validate AdH bedload transport rates across the channel in Figure 9-16.

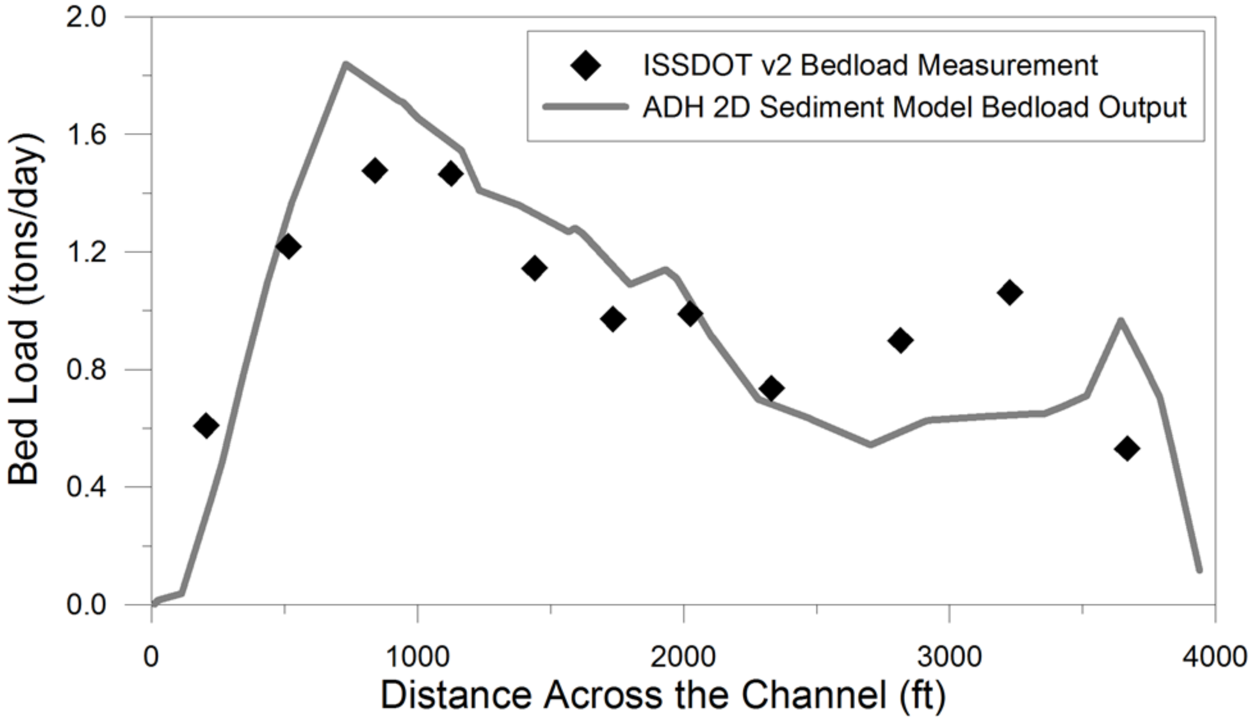


Figure 9-16. Comparison of measured and computed bedload at RM 971 (adapted from Abraham et al., 2015)

(b) Internal or Downstream Concentrations.

- Sediment models can be calibrated to internal or downstream sediment concentrations (Figure 9-17) when bed change data are not available, when historic bed change is minor, or to make a model calibrated to bed change even more robust. When comparing observed and computed loads or concentrations, it is important to compare the same load component. For example, the modeler should adjust model results from a total load transport function before comparing them to suspended-load sediment measurements, or evaluate the difference qualitatively.

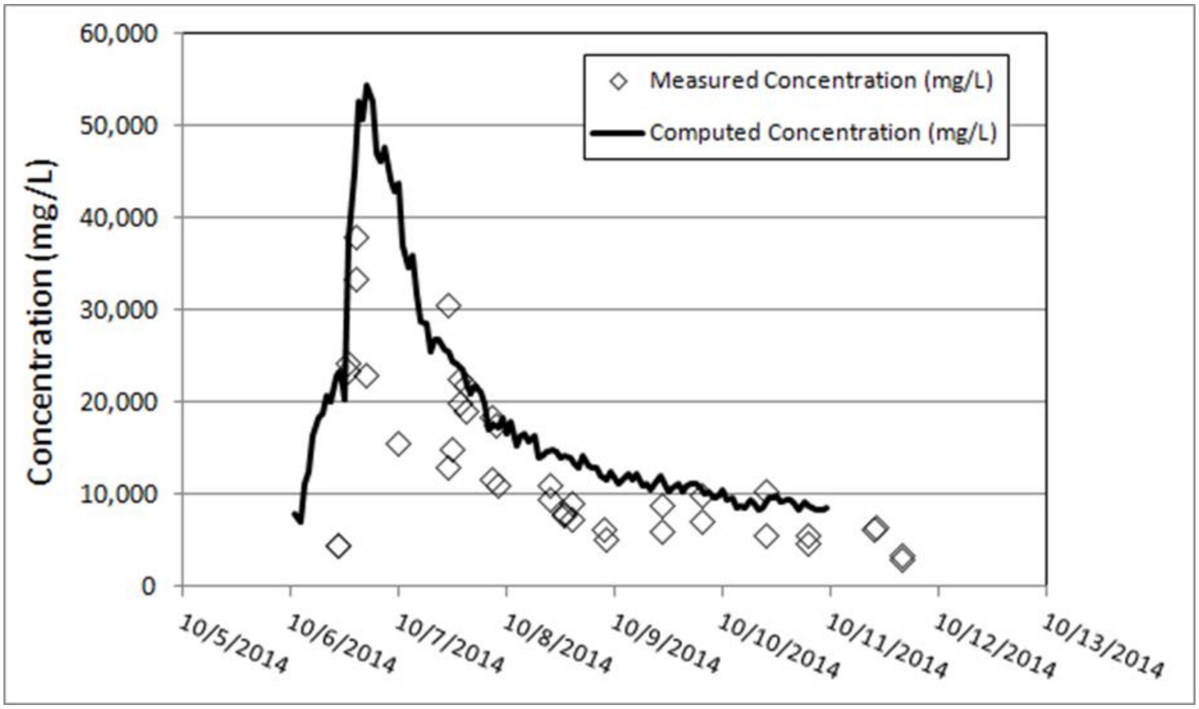


Figure 9-17. Concentration measured downstream of a reservoir flush of Spencer Reservoir on the Niobrara River and the HEC-RAS concentration results (from Gibson and Boyd 2016)

- Multidimensional models can take a concentration calibration one step further. When data are available, multidimensional modelers can evaluate model performance laterally, based on measured loads as shown in Figure 9-16 (Abraham et al., 2016).

- Multidimensional modelers also use vertical concentration data to evaluate 3D and quasi-3D models (2D models with analytical algorithms that distribute element concentrations vertically).

- Heath et al. (2015) measured separate suspended-sediment and bedload transport on the Mississippi River. They estimated suspended sediment with point samples combined with ADCP current velocity and measured bedload with the ISSDOTv2 technique described in paragraph 4-10g (Abraham 2011). Then they compared the separate suspended and bedload measurements with AdH model results (see Figure 9-18). Sharp et al. (2013) compared concentrations from a 3D model (CH3D) to vertical, point vertical, point concentrations measured at multiple locations across Mississippi River cross sections (Figure 9-19).

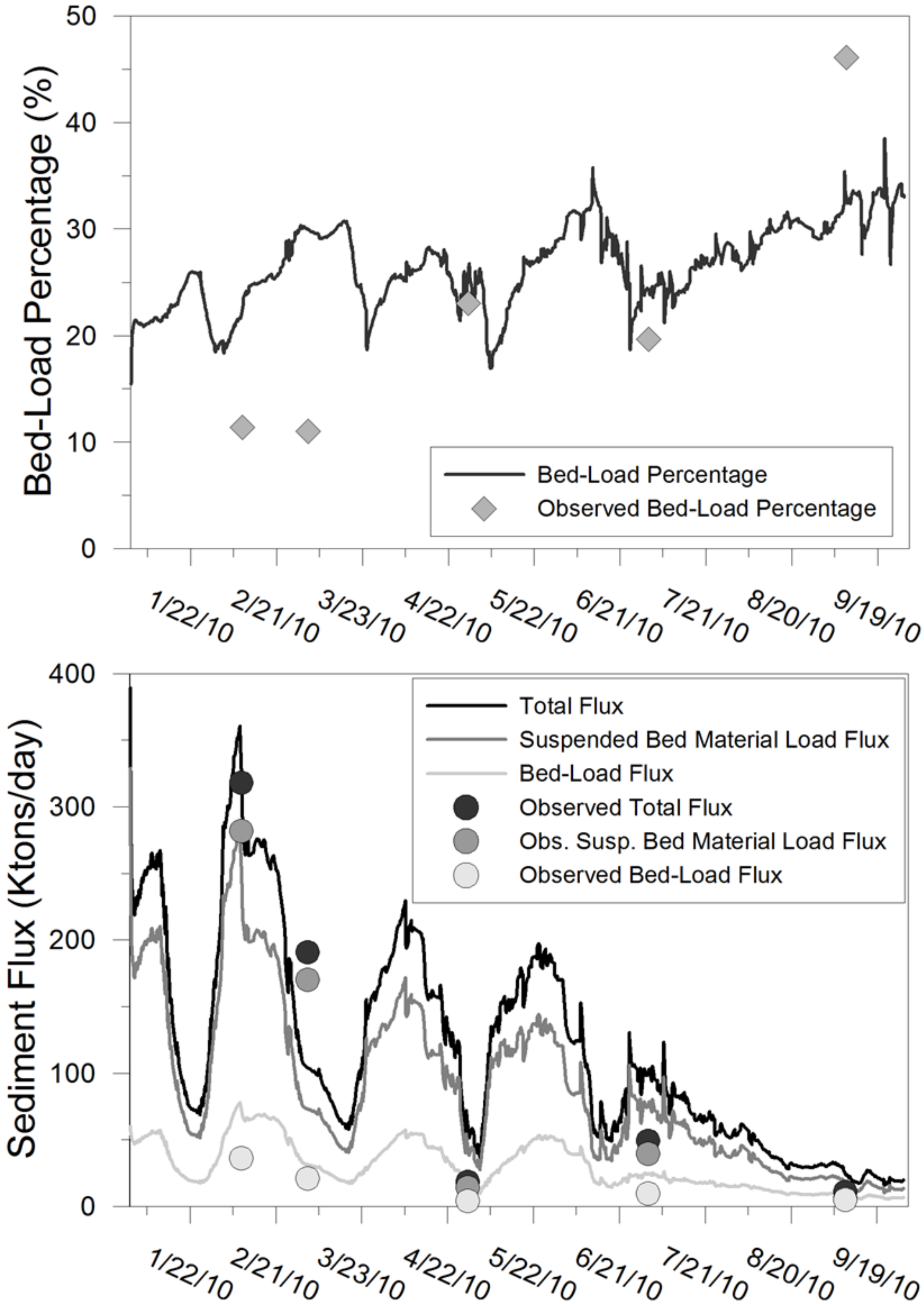


Figure 9-18. Modeled and observed fluxes in the Mississippi River downstream of auxiliary (from Heath et al., 2015)

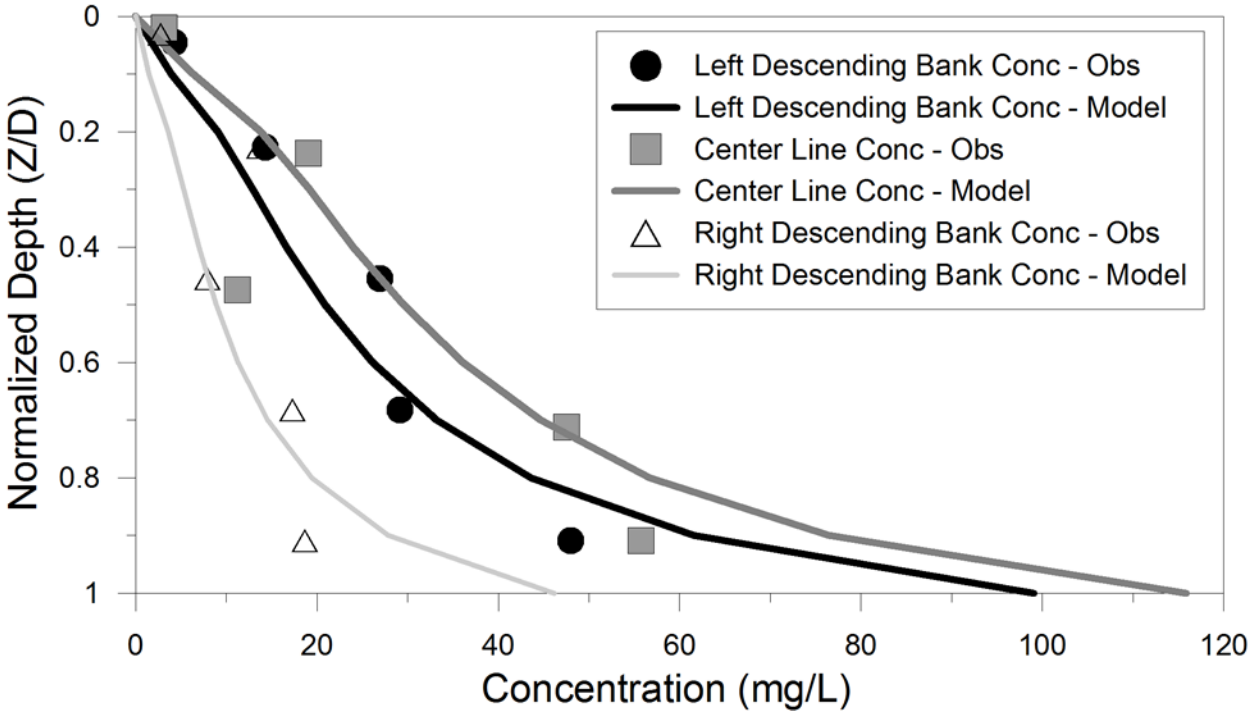


Figure 9-19. Comparison of measured and computed, vertical, SSCs from a 3D Model (CH3D) for Mississippi River Mile 4.5 (after Sharp et al., 2013)

- TSS time series data are more common than SSC data, because the U.S. Environmental Protection Agency mandates TSS. Municipal waste water facilities often measure TSS, sometimes offering intriguing mid-model calibration opportunities. However, Gray et al. (2000) compared over 3,000 paired SSC and TSS data, concluding that they "are not comparable and should not be used interchangeably." They found TSS data tend to be biased low when sand content exceeds about a quarter of the dry unit weight.

- Finally, turbidity measurements are substantially cheaper than SSC or even TSS, and are often proportional to SSC (Rasmussen et al., 2011), making turbidity time series an attractive calibration metric. However, there is no general relationship between turbidity and SSC. Turbidity data require site-specific regression between measured SSC and turbidity to establish a relationship between turbidity and SSC, before a turbidity time series is viable for calibration. Rasmussen et al. (2009) recommend starting with monthly sediment samples for 2 to 3 years, covering the "full range of hydrologic conditions," then evaluating the turbidity SSC relationship. They stress the importance of sampling large flows to avoid extrapolation errors.

(c) Flow-Load Rating Curves. Flow-load or flow-concentration data is useful to evaluate model performance at the downstream boundary or at mid-model gages as illustrated by recent results on the Mississippi River (Figure 9-20). Plotting computed loads or concentrations against modeled flows compared to measured data at the same location allows model performance evaluation even if the measured data is not in the simulation time period. Sediment load results, plotted against model flows, should fall within the scatter of the downstream flow-load curve. If grain class-specific concentrations are available, this method can evaluate each grain class independently, as shown in Figure 9-20. Plotting flow-load results from different transport equations with available flow-load measurements (Copeland et al., 2020) can be used to evaluate transport function performance.

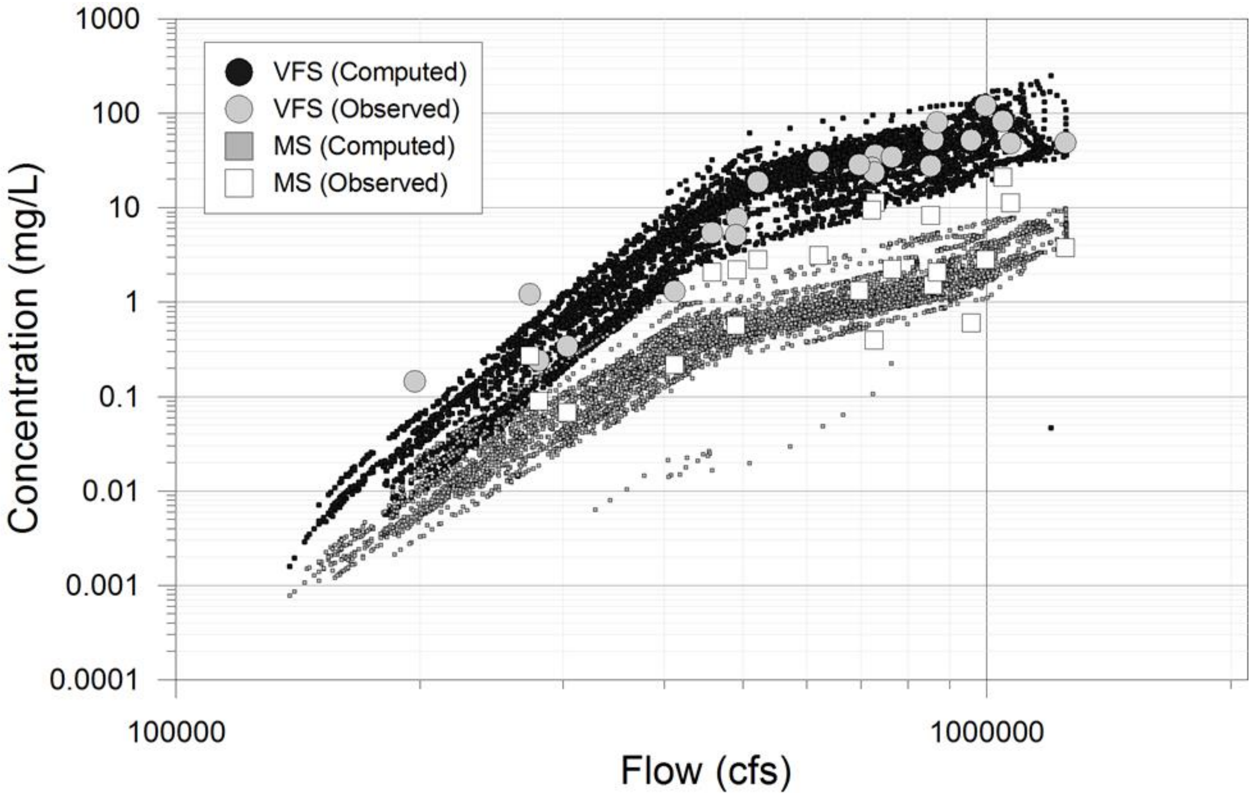


Figure 9-20. Rating curve calibration for two grain classes from the HEC 6T model of the Mississippi (redrawn from Copeland et al., 2020)

(d) Annual Regional Loads. Thomas (2000) recommends comparing average annual model transport with published, estimated, annual sediment yields. This is more of a model check than a robust calibration, but is a relatively easy way to test the plausibility of input data. Dividing total into bed material load and wash load components makes this check more robust when partitioned load estimates are available. In watersheds with no long-term, published, yield values, modelers can compute yield using regional sediment regression relationships (see paragraph 6-4) or information from a comparable watershed in a similar hydro-geographic setting.

(e) Specific Gage Analysis.

- River bathymetry change and sediment concentration measurements can be rare. Where they exist, they tend to be recent. However, historic flow and water surface elevation data are much more common and continuous, often including much older measurements. Specific gage analysis uses historic flow and stage data to infer bed change. Modelers can compare historic flow-stage relationships to simulated flow-stage results to calibrate morphological models over longer time periods. Modelers tend to use specific gage analysis to evaluate long-term models that begin before their sediment data.

- A specific gage analysis (see also paragraph 7-7) parses the flow record, finding the elevations associated with several different flows, and plotting these elevations as a time series (Thomas 2002).

− If the water surface elevation associated with a constant flow changes over time, it indicates bed change trends. A modeler can pause a sediment model at various points along the time series compute the flow-stage relationship with intermediate bathymetries.

− Model water surface elevations for the selected flows can be compared to the measured water surface elevations for the same flows (the Specific Gage curve) over time to evaluate historical model performance (Figure 9-21). Some models will perform this analysis automatically (HEC 2016).

− In Figure 9-21, Thomas (2007) compared model stages associated with specified flows to measured stages from 1963 to 1983 at three gages. This analysis demonstrated that the model reproduced the historic trend, increasing confidence in the predication (the computed results from 1984 to 1997).

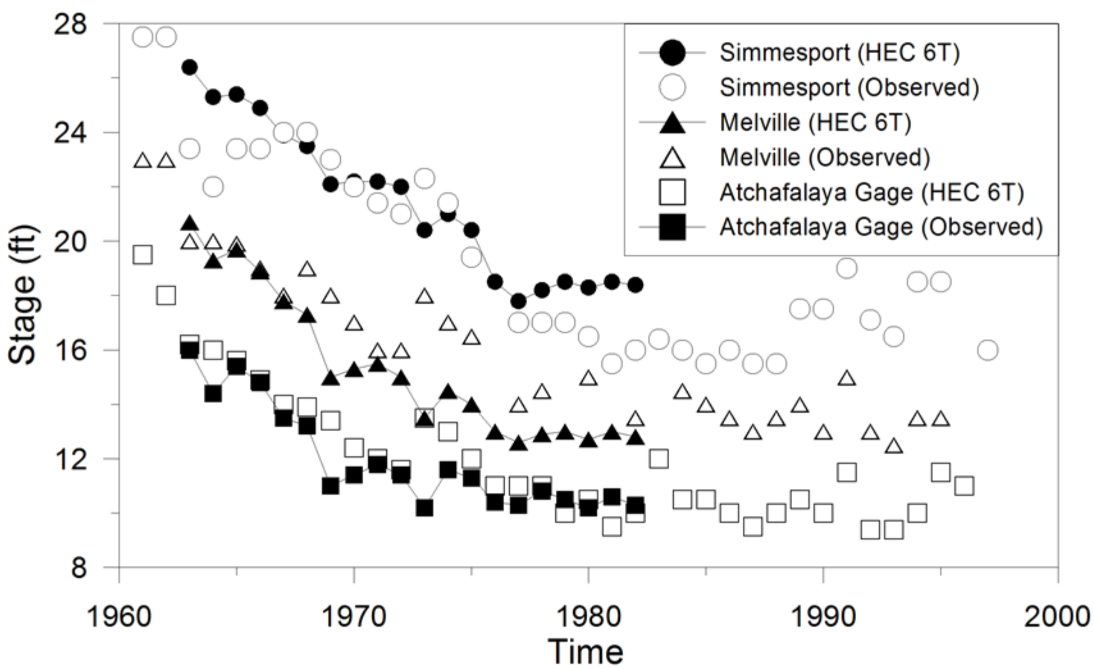


Figure 9-21. Specific gage observations and HEC 6T model results at three gages (Thomas 2007)

(f) Long-Term Stage Time Series. Bed elevation influences water surface elevation. If a river bed is actively aggrading or degrading, a model must simulate bed change trends to reproduce water surface elevations over time. Therefore, in the absence of (or in addition to) bed change data, simulating the water surface elevation time series in a system with a dynamic bed provides indirect evidence that the sediment model approximates historic bed change (Figure 9-22).

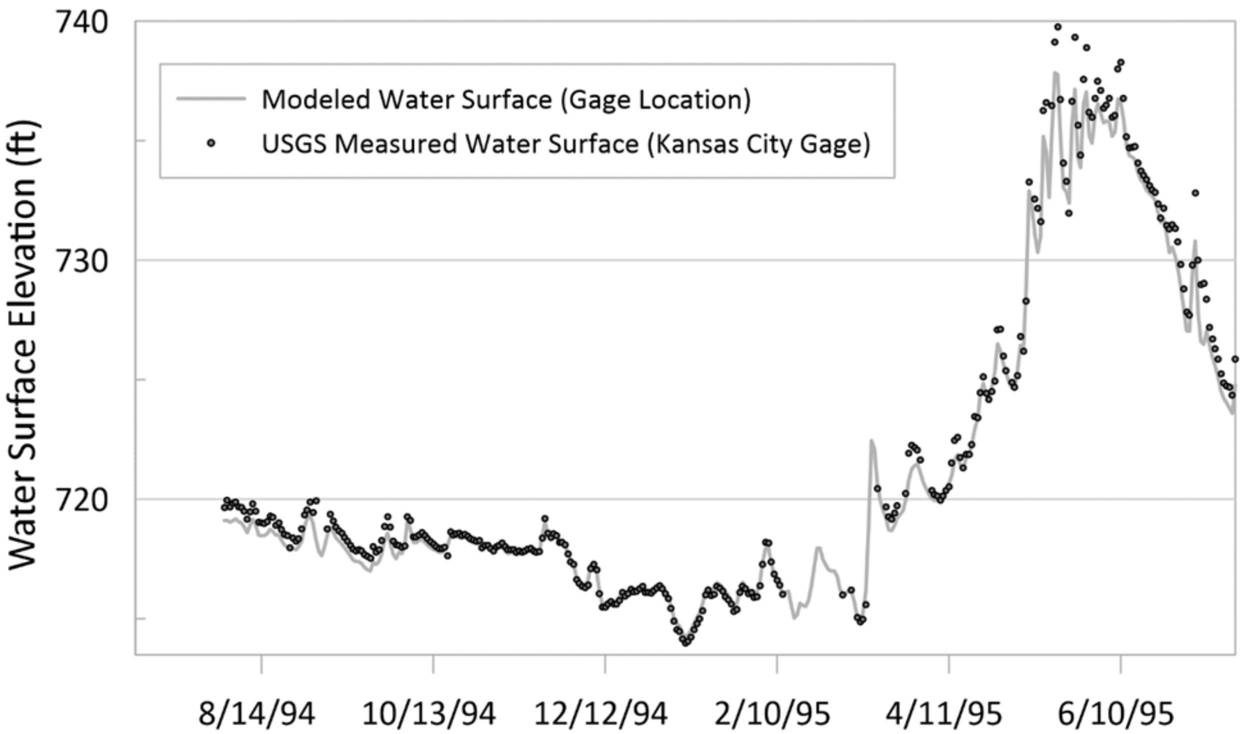


Figure 9-22. Stage time series calibration from the Missouri River at the Kansas City gage (adapted from USACE 2017b)

(g) Bed Gradation. In systems with extreme, monotonic bed evolution (such as deposition in a reservoir or scour downstream of a reservoir) bed gradation evolves predictably over time.

- Calibrating to bed gradation is not usually sufficient to evaluate model performance for most objectives, particularly bed change objectives.

- Bed gradation evolution can help evaluate a calibration with several free parameters, improve confidence in model performance, and avoid equifinality errors (Gibson and Pridal 2015; Gibson et al., 2017).

- If the load and gradation of inflowing sediment load are both highly sensitive and uncertain, the load can be calibrated against bed change and the load gradation against measured bed gradations. For example, Gibson and Pridal (2015) compared the final gradations for a long-term reservoir calibration with bed gradations measured at the end of the calibration period shown in Figure 9-23.

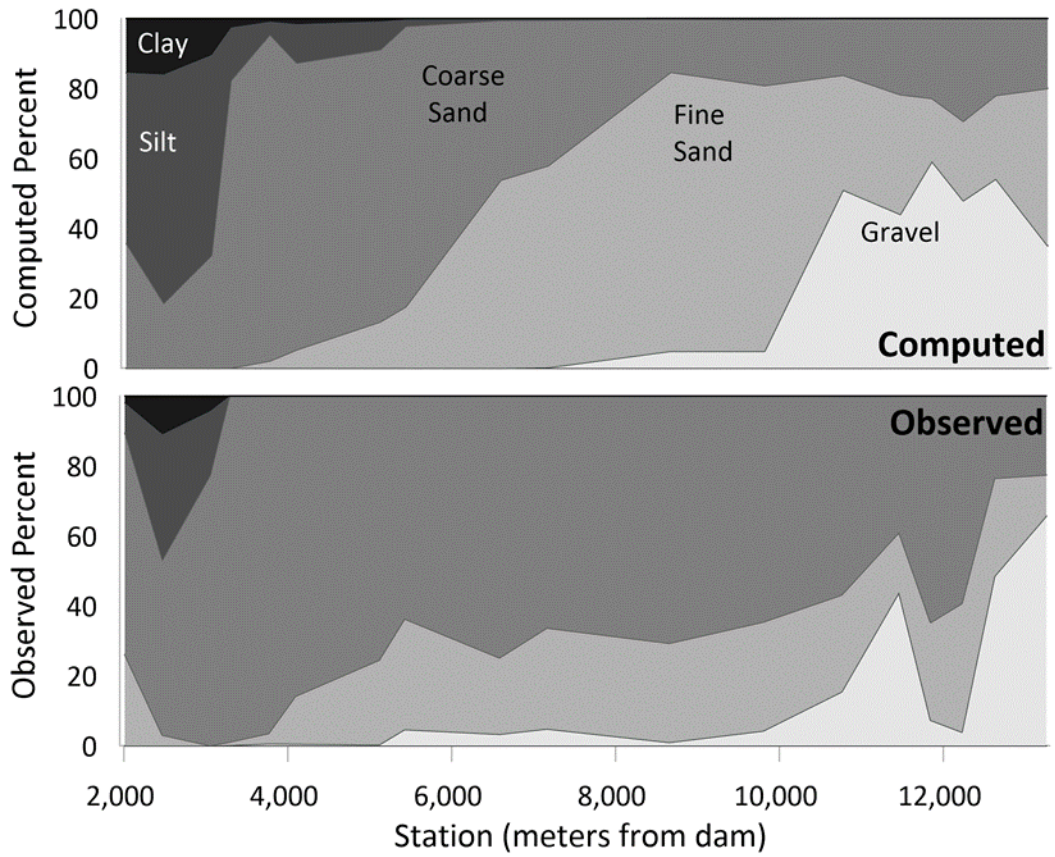


Figure 9-23. Measured (surface) gradations from reservoir deposits on the Muskegon River and simulated deposit gradations from HEC-RAS in the same reservoir (from Gibson et al., 2020)

(h) Other Calibration Evaluation Metrics. Modelers can compare computed results to several other measurements.

- Shelley and Gibson (2013) compared model velocities to those measured during standard and flood flows, to increase the confidence of their hydraulic calibration, which included flow-dependent roughness parameters.

- Scour chains offer episodic bed change measurements.

- Models that include bank movement can be calibrated to repeated cross sections (Gibson et al., 2015), historic aerial photographs, or bank pins. Bank pins are rebar segments driven into an eroding bank. Measuring the exposed rebar in subsequent visits can provide bank recession rates.

- Models can also be calibrated to dredge quantities. In dredging channels where year-to-year deposition is removed and dredge quantities are recorded, a sediment model can replicate dredge volumes to calibrate flux and deposition.

(4) Evaluating Calibrations.

(a) While calibration is a quantitative process, most sediment calibrations are evaluated qualitatively. Modelers plot model results with observed data that reviewers and stakeholders evaluate visually. Sutherland et al. (2004) describe this standard, qualitative, model evaluation practice well: “This comparison is typically subjective, and of the form ‘reasonably good agreement was found.’” The qualitative, ad hoc approach to morphological model calibration (and verification) led Copeland to memorably describe the process as “circumstantiation” (Thomas and Chang 2008).

(b) However, more quantitative methods are available to evaluate calibration results. Modelers can report simple performance metrics like total volume change error, or they can compute more detailed quantitative residual analyses. Quantitative model evaluation metrics are more common in multidimensional, coastal, morphological modeling, but the principles are applicable to riverine and 1D applications.

(c) Sutherland et al. (2004) categorize quantitative model performance metrics into three categories (bias, accuracy, and skill) and offers several metrics and six helpful criteria for selecting a metric. However, the root mean square error (RMSE) is the most common quantitative model performance metric (van Rijn et al., 2003). The Brier Skill Score (BSS) (Brier 1950) is a useful performance metric for evaluating bed elevations. When evaluating bed change, the correlation coefficient or a residual based performance metric such as the RMSE generally works well.

(d) In some cases, particularly in multidimensional models, it may be useful to compare the volume of specific features rather than simple node-to-node elevation comparison. For example, a computed sand bar may have the similar volume as a measured sand bar, but may be slightly offset spatially. When simulating a deltaic or braided system the location of channels might be very different, but the channel morphology may be very similar.

(5) Calibrating Processes Significant to Engineering Analysis. Calibration should replicate the process modeled in the alternative evaluation phase. Models concerned with channel bed change should calibrate to bed change, while models concerned with Total Maximum Daily Loads (TMDLs) should calibrate to concentration when possible. Calibrating to a different metric than the model was designed to predict can reduce model uncertainty, but less than calibrating to the predicted variable.

(6) Equifinality, Overfitting, and Non-Unique Solutions.

(a) All multi-parameter models suffer from equifinality (Bevan 1993; Bevan 2005). A model with even two free parameters offers a non-unique solution. Multiple combinations of the two parameters can produce similar results. More free parameters exacerbate this problem, and sediment models can have 10 or more parameters.

(b) Equifinality or “overfitting” problems arise when a model is finely tuned to a specific condition. Errors in one parameter can offset errors in another parameter (for example, the critical shear stress and the load can both be too high, reproducing the measured deposition, but misrepresenting the river processes). Because an over-fit model does not represent the physics correctly, even though it reproduces calibration trends, it may generate spurious results when simulating a different time series or predicting outside its calibrated conditions. Modelers should identify a small subset of the model parameters that are most sensitive and least certain to use as calibration parameters and fix the rest to the best estimate.

(c) There are two approaches to mitigating equifinality model problems. First, the modeler can evaluate their model against multiple calibration metrics. Comparing model results to bed gradation change, water surface elevation trends, or downstream concentration in addition to bed change will often expose overfitting errors. Second, the modeler can test the model against multiple time series.

(7) Common Methods to Improve Model Calibration. If the model reproduces processes in the prototype, the key evaluation metrics should match reasonably well. These include water depths, measured velocities, measured sediment concentrations within the study reach, and bed gradations. The following suggestions illustrate common problems and methods to improve model calibration:

(a) Position the upstream model boundary at a relatively stable river reach, and verify that the model exhibits that stability (a stable upstream cross section should neither erode nor deposit).

(b) Resolve hydraulic problems within the model from downstream to upstream. Reverse that direction for sediment problems, resolving sediment issues from upstream to downstream. Do not worry about scour or deposition at the downstream end of the model until it is performing well upstream.

(c) Be sure that the model is numerically stable before calibrating by adjusting coefficient.

(d) If the calibration focuses on bed change between two bathymetries (for example, repeated cross sections collected 20 years after the originals), review intermediate bed change output, particularly after large events. Make sure the bed change the model predicts makes physical sense over the whole simulation, and does not just match the end points.

(e) Review how bed gradations change throughout the simulation, considering whether they respond to large flow events and periods of aggradation or degradation in logical ways.

(f) If the model scours more than the prototype, check for stabilizing features like armor layers, stiff cohesive layers, or bedrock outcrops.

(g) If the model deposits more than the prototype, check bank stations and ineffective flow limits. Too much overbank flow typically decreases channel transport rates (in 1D models).

(h) Finally, only after carefully reviewing hydraulic conditions, channel/overbank flow, model data and parameters, and transport equation, adjust the inflowing sediment load and gradation (within the scatter of the data). First use a constant ratio to translate the curve without rotation. If that is not successful, rotate the curve within the scatter of data.

d. Stage 4: Multiple Time Series Calibration (Validation).

(1) Testing a calibration over a second, unrelated time series, can increase confidence in the calibrated parameters and reduce the probability of overfitting or equifinality problems. This process is often called validation or verification, though both these terms have controversial and, often overlapping, semantic ranges (Oreskes et al., 1994; Rykiel 1996).

(2) A classical validation, common in research models, uses the second time series as a blind test, evaluating the calibration. Research validations do not revisit the parameters based on the validation. However, operational models cannot leave data "on the table." Therefore, if a model performs poorly in the validation, agency modelers must revisit the calibration and evaluate the assumptions and parameters, to determine why the results diverged and improve model performance over both time series. Therefore, "split-time series calibration" (Thomas 2000) describes this process a little more precisely. Figure 9-24 illustrates multiple time series calibration of an HEC-RAS sediment model for the Missouri River near Kansas City.

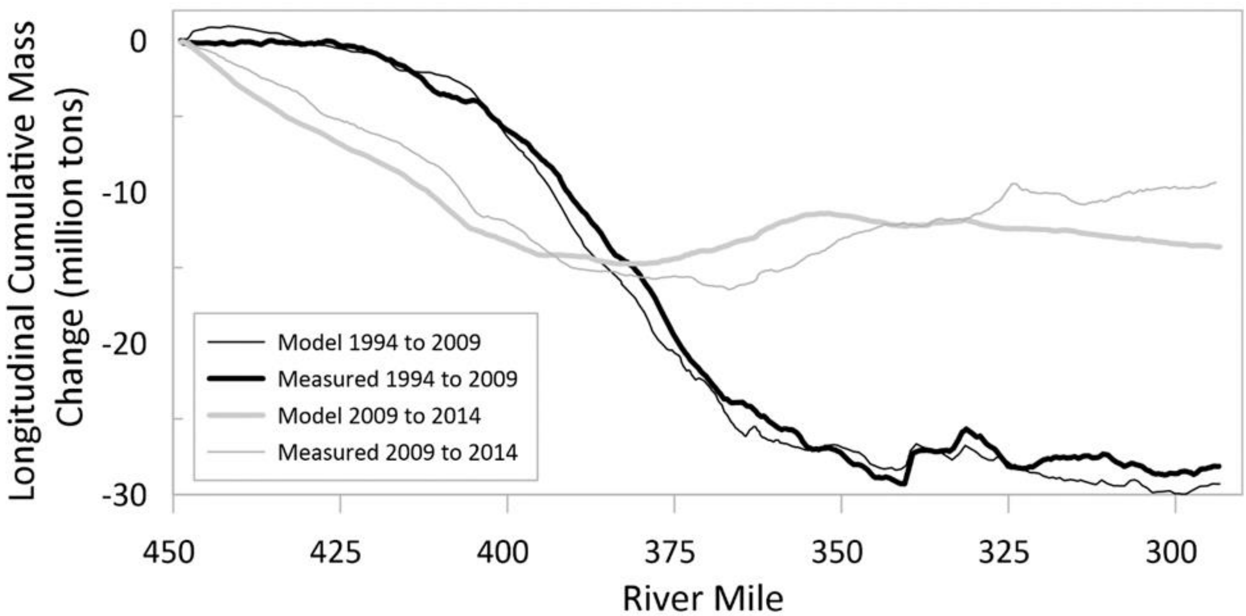


Figure 9-24. Multiple time series calibration (calibration and validation) of the Missouri River sediment model through Kansas City (adapted from USACE 2017b)

(3) Sediment validations are more complicated than hydraulic validations, or even validations in more parameterized models like groundwater or hydrology. Sediment validation require more work than simply splitting the flow or rainfall record in other models. Simulating a new time window with a sediment model requires setting up a new initial bathymetry and bed

gradations that reflect the new starting condition and period of record. However, validation or split-time series analyses can be extremely helpful for three reasons:

(a) Validation exposes overfitting or equifinality problems.

(b) If the second time series is substantially different from the first (for example, including a large flood event or extended drought period or watershed source change like wildfires), it will test how the parameters extrapolate.

(c) A split-time series analysis can reveal non-stationarity in the data or processes, leading the modeler to conclude that one of the time periods (usually the later one) is more representative of future conditions.

(4) If the validation run departs drastically from calibration measurements with input data and parameters from the calibration, the modeler should start a systematic forensic and sensitivity analysis, zeroing in on the sensitive variables that cause the model results to diverge.

(5) Validation or split-time series calibration introduces a paradox. Testing a model against multiple time series can make each calibration worse (higher RMSE, lower BSS, or just less visually compelling), but makes the model better. Many singe-time series (and single-calibration metric) calibrations are over-fit, making detailed adjustments to highly variable and uncertain parameters. These calibrations are more precise than justified. Therefore, even though multiple time series calibrations will force a worse model-observation fit for each time series, it almost always produces a more robust model that can predict model response to future conditions and engineering alternatives more reliably.

e. Stage 5: Alternative Analysis.

(1) With the calibration complete (and usually, well over half the study budget and schedule expended), the model is finally ready to predict with some confidence. However, it can be advantageous to set up rough alternatives and simulate them before the calibration is complete. Constructing rough model alternatives mid-calibration will ensure that the calibrated model structure and assumptions can handle alternatives before investing too much time in the detailed calibration runs.

(2) Stages in Alternative Analysis.

(a) After a sediment model is calibrated, providing the best estimate of past performance, it can evaluate future conditions. If the sediment parameters are stationary, the calibrated sediment parameters can be retained from the calibration runs, projected into the future. The modeler should update the historic bathymetry and bed gradation from the calibration model to reflect present conditions. Often, a recent survey, used as the final condition in the calibration, doubles as the starting bathymetry for the future conditions model.

(b) Estimating future hydrology can be the most difficult part of predictive sediment modeling. While the sediment parameters introduced most of the uncertainty in the calibration, hydrologic uncertainty becomes the most important source of uncertainty in model projections. Because sediment transport is so sensitive to flow, sediment responses to project alternatives will depend on the flows during the project life.

(c) Future flows are unknown and variable. Because the natural variability of future hydrology is one of the major uncertainties associated with sediment modeling, methods for evaluating risk and uncertainty are covered in Chapter 10. There are three major options for constructing future hydrology: (1) repeating the period of record, (2) creating an average annual flow record from a flow duration curve and computed, low-probability events, and (3) constructing multiple, stochastic, project life flow time series from historic data and synthetic, large, low-frequency events that do not occur in the record.

(d) The alternative analysis process has four components:

- Simulate Future No-Change Conditions. To evaluate alternative benefits, economists must compare alternatives to a likely future without action. The first model run addresses the question: "How would the river respond if we did nothing over the proposed project life?" Therefore, the first alternative simulation runs the model into the future with contemporary bathymetry and bed material, the calibrated (or projected) sediment loads, calibrated parameters, and future hydrology. Economists compare alternative damages to the no-change condition baseline from this model to quantify benefits.

- Simulate Alternatives. After simulating the future no-change condition over the project design life, the modeler adds alternatives to the model. Alternatives can take many forms in a numerical model.

− Modelers can change the model bathymetry to simulate channel modification for flood risk management or restoration, levee setbacks, levee raises, or other channel modification.

− Most models include adjustable downstream boundary conditions, and some models can simulate reservoir rule curves to evaluate operational alternatives for reservoir models.

− Modelers can adjust boundary conditions to simulate upstream operations (like sediment traps) or non-stationary sediment loads due to watershed sediment management or targeted bank treatments.

− Most models also accept detailed dredging records to evaluate different dredging schemes, distributing them in time and space to optimize the impact per dredged unit.

− While this phase of the analysis produces the answer, providing the project with actionable insight, it is generally cheaper and quicker than the preliminary modeling steps (such as data analysis, sediment budgeting, model set up, model calibration, and model validation). However, modelers should leverage all of the intuition they developed during the preliminary steps to evaluate the plausibility of the alternative model results.

- Long-Term Analysis. Most USACE projects are analyzed for 50-year design lives. However, once a model is calibrated and set up to evaluate the project life, it is often trivial to run the model for 100, 150, or 200 years. Long-term runs provide decision-makers with insight on long-term sustainability and resilience. Sediment issues that are manageable on 50-year time scales can become expensive, ecologically deleterious, and/or dangerous on longer time scales, which can inform the project risk registry and qualitative considerations of project feasibility.

- Sensitivity Analysis.

  − Small changes in uncertain parameters, like the load gradation or bed gradation, can sometime produce major changes in results. These sensitivities should be investigated as part of the calibration process to establish the calibration parameters. However, it is also useful to adjust these parameters within the reasonable range in the alternative simulations (such as a fixed percentage or a standard deviation above and below the selected parameter) to communicate the model sensitivity to the decision-makers who will act on the model results (Thomas and Chang 2008).

  − Modelers can modify input data by one or two standard deviations and rerun the simulation to test model sensitivity. If results show little change, the uncertainty in the data is minor. If large changes occur, the input data may need to be refined. Refinement should proceed using good judgment and by modifying only one parameter or quantity at a time in order to assess the effect. Sensitivity studies provide sound insight into the prototype's behavior and will lead to the best model description of the real system. High model sensitivity to uncertain parameters should be included in the risk and uncertainty analysis (see Chapter 10).

9-5. Movable-Bed Physical Models.

a. Overview of Physical Models. Modeling a movable riverbed in a physical model requires integration of both sediment mechanics and the application of practical river engineering perspectives. Practical and theoretical experience is a key factor for successful model execution and interpretation of model results. In some cases, where the processes are particularly complex or where the project risks and consequences are high, a physical model is the best way to evaluate project impacts and benefits. Methods for modeling sediment were developed using work by Vanoni (1975, 2006), Einstein (1954), Gessler (1971), Dodge (1983) and others. Recent publications including Appendix C of ASCE Manual No. 110 (Pugh 2008) describe aspects in physical sediment models.

b. Scaling

(1) Models may be either true or distorted scales. A distorted model is a physical model in which the geometric scale is different between each main direction. For example, river models are usually designed with a larger scaling ratio in the horizontal directions than in the vertical direction. Normally, movable-bed models at the ERDC (previously WES) were constructed to horizontal scales of 1:100, 1:120, or 1:150. Most ERDC movable-bed models were distorted in that the vertical scale was larger, 1:60 or 1:80, depending on the movable-bed material used.

(2) A difficulty in movable-bed physical models is to scale both the sediment movement and the fluid motion. The bed roughness becomes a function of the bed geometry and of the sediment transport. In addition, the time scale governing the fluid flow differs from the time scale governing sediment motion. It is important to verify and to calibrate a movable-bed model before using it as a prediction tool. Examples of physical models are provided in Figure 9-25.

Figure 9-25. Example physical models: (a) mud mountain and (b) intake dam

c. Model Scales. Physical models must be designed such that turbulent flow will prevail with the model velocities and depths to preserve essential flow patterns. Model Reynolds Numbers greater than 1,800 are generally required to ensure turbulent flow. Since the model Reynolds number will always be smaller than the prototype Reynolds number, there will be some scale distortion of certain phenomena such as zones of separation, wave dissipation, flow instability, and turbulence in the model. For sediment models, where the gravity force dominates the flow, similitude will require equality of Froude number in the model and prototype.

d. Shield’s Scaling. Movable-bed modeling also requires similarity of intensity of transport and intensity of shear. The work of Taylor and Vanoni (1971) suggests that similarity is possible when the prototype/model ratio of the difference between Shields parameter (dimensionless shear) and the critical shields parameter is equal. This is done by adjusting model particle size, sediment density, and model slope to obtain model values of the Shields parameter that lie on a curve parallel to the critical Shields curve in common with prototype data. This method is often referred to as the Shields parameter offset method.

e. Distorted Physical Models.

(1) Movable-bed physical models of river channels, flood ways, harbor, and estuaries often require a distortion of the vertical scale to ensure movement of the model bed material. Vertical scale distortion also allows for measurable depths and slopes as well as ensuring turbulent flow in the model. Sediment load and sediment transport time must also be distorted to achieve similarity. For an undistorted model, bedload per unit width can be shown to scale by L3/2, where L is the length scale. Imposing distortion of sediment density and grain size also requires distortion of sediment load. Scaling of bedload was evaluated for each model scale using the Meyer-Peter Mueller bedload equation by Gessler (1971).

(2) Similarity may not be possible for all grain sizes, and therefore, judgment must be used to determine where deviating from similarity is acceptable. For example, good similarity of fall velocity for large particles may not be required if the particles are not likely to be suspended or settling times are small in both prototype and model.

(3) To ensure sediment movement at low model velocities, it is often necessary to use a model bed material lighter than sand. The choice of scales and bed materials for movable-bed models is largely based on the experience and judgment of the modeler. Coal dust (specific gravity = 1.3 approximately) and plastics (specific gravity = 1.2) are common model bed materials. Model velocities ranging between 0.3 and 1.0 ft/sec are required to simulate bed material movement. These velocity criteria are used to select a vertical scale. The slope of the model is then determined using the Manning's equation with a roughness coefficient of the model material (Manning's n is approximately equal to 0.018 for coal dust).

(4) It is frequently impossible to preserve similitude with respect to size and weight of bed material in physical models. However, several investigators have concluded that the effect of bed material size on scour depths is insignificant and that bed material size effected rate of scour around a spur dike, but had limited effect on ultimate scour depth.

(a) Vanoni (1975, 2006) concluded that bed material size had an insignificant effect on the depth of local scour at bridges based on a thorough review of available references. Laursen (1962) reached the same conclusion as long as there was sediment transport into the scoured region.

(b) These investigations increase confidence in results obtained from physical models where bed material similitude is not maintained. However, there remains insufficient prototype-to-model comparisons to prove conclusively that bed material size is insignificant in local scour problems and model results should be considered qualitative.

(c) El Kadi Abderrezzak et al. (2014) discuss a scaling approach for multi-grain size mixtures that preserves similarity of initial motion for each grain size class and of the bank stability coefficient between the model and the prototype, but relaxes strict similarity of the Shields and particle Reynolds numbers. This approach may be appropriate when bedload transport near incipient motion conditions is being studied, and allows for larger grain size scales than when full Shields parameter similarity is enforced. This method was applied to scale a Froude number model of a reach of the Old Rhine (France) at an undistorted scale of 40 using sand as the model bank material.

(d) Distorted sediment scaling presents a complex problem and can be a crucial variable to achieve reasonable results. Regardless of the approach selected, calibration with field data is generally recommended.

f. Considerations for Movable-Bed Physical Models. A partial summary of major considerations involved in movable-bed physical models includes:

(1) Similarity of bed form type between prototype and model is likely where both prototype and model conditions fall in similar bed form zones. Predicting bed form resistance and applying to the physical should be considered. Modeling of bed forms as published in Simons and Sentürk (1992) can be consulted.

(2) Vertical distortion may increase the bank slopes beyond the angle of repose so that they will no longer stand. One remedy is to make the banks rigid, but this can only be done if the banks are known to be stable. Scale distortion also increases the longitudinal slope of the river making it necessary to increase model roughness. However, roughness is primarily a function of bed forms and cannot be arbitrarily adjusted.

(3) Vertical distortion also distorts the lateral distribution of the velocity. This creates simulation problems at confluences, bifurcations, and sharp bends. The problems related to vertical distortion generally limit movable-bed models to mild sloped streams where the distortion ratio should be limited to 3. In special cases, the distortion ratio could be as high as 10. In harbor and estuary models, greater distortion is permitted due to the relatively small prototype sand slopes and very mild water surface slopes.

(4) Particular care should be taken in interpreting those effects that are known to be strongly dependent on viscous forces.

(5) The time scale governing the fluid flow in the model will probably be different from the time scale governing sediment movement. This means that the hydrographs applied to the model will have to be reduced by model operation. During the model verification process, adjusted historical hydrographs are run through the model until historical bed changes can be reproduced. The adjusted hydrographs may require different time scales for low discharges than high discharges because of the nature of the model bed material. For instance, coal dust moves rapidly from little movement to violent movement with small increases in tractive force so that the time scale would be increased for low stages and decreased for high scale, to simulate prototype bed movement.

(6) The verification of the movable-bed model is very important due to the absence of quantitative similarity. Once the model and its operations are adjusted so that it accurately reproduces known bed configuration changes, then there is ground for confidence in model predictions of future events.

(7) Consideration should be given to conducting a phased testing procedure to document specific changes for various alternatives. A phased testing procedure will also allow collecting detailed data around structures and channels to determine model response in critical areas. If appropriate, the critical areas can be modified in the semi-fixed bed model before continuation of testing.

(8) USACE modelers gained considerable experience with coal movable-bed physical models at WES (Franco 1978). Specific considerations related to coal include: (1) addressing model water temperature to ensure repeatability over time through various weather seasons; (2) using a coal gradation that does not produce ripples (recommended), but rather coal waves to reduce the possibility of the coal acting cohesively and armoring the model bed; (3) minimizing the re-handling of coal material during model operation to reduce grinding of the coal and altering fine grain fractions; and (4) maintaining the coal model material under water during non-operational periods.

g. Hydraulic Sediment Response Physical Models.

(1) A hydraulic sediment response (HSR) model, formerly known as a micromodel, is a small-scale physical movable-bed model that follows specific procedures to investigate river sedimentation processes. HSR model lengths range length from as small as 6 feet to as long 70 feet, with typical channel widths of 4 to 6 in. The models are operated with light, synthetic bed material (specific gravity ~ 1.5) and with a dominant discharge to simulate prototype bed response.

(2) The HSR model consists of a planform insert, with projected aerial photography, placed within a hydraulic flume with automated and instrumented flow control. Fixed boundary features such as banks, islands, dike structures, rock, and consolidated clay formations are represented with wire mesh or other materials. Figure 9-26 shows a typical model. Rivers modeled with HSR models include the Mississippi, Missouri, Atchafalaya, Brazos, Ohio, White, and Tombigbee. These models include navigation, dredging, and environmental alternatives.

(3) USACE subject matter experts (SMEs) differ on the applicability of HSR models. Davinroy et al. (2011) presented evidence of HSR model accuracy. Maynord (2006) however, argued that micromodels deviate too much from similarity conditions, resulting in too much physical distortion for project design or even alternative screening. Others have pointed out that HSR models do not reproduce relative shear stresses, and that they distort fall velocity and bed mixing and are limited by the assumptions of the dominant discharge theory.

(4) In 2004, the USACE Channel Stabilization Committee (CCS 2004) concluded that HSR models are useful tools for project sponsor and stakeholder engagement, and for evaluating qualitative morphological response of river systems. They also agreed that they could be useful screening tools to make coarse, relative comparisons between alternatives (leading to more detailed engineering analysis of the most promising options) for "basic river engineering problems involving river training structures, navigation channel realignments, and some types of environmental modifications in non-critical river reaches."

(5) However, because of the degree of distortion imposed by the scale, they did not support HSR models for project design or even for alternative screening for sensitive or high-risk projects, particularly when the project poses significant risk to human life and property, unless they are part of a larger analysis strategy including other river engineering practices, tools, and models. Employing an HSR model for concept exploration or as a support tool for more detailed and robust modeling/analysis efforts may be an effective approach for USACE studies in those instances. Constructing numerical or physical models at multiple scales and resolutions can develop experience and familiarity with study reach that is necessary to properly interpret model outcomes and to recommend appropriate design actions.

Figure 9-26. HSR model, Atchafalaya River (Davinroy et al., 2011)

Chapter 10
Sediment Considerations for Risk and Uncertainty Studies

10-1. Risk and Uncertainty Analysis in USACE Flood Risk Management Studies.

a. Introduction. USACE FRM projects are planned, designed, constructed, and operated to reduce risk to people and property from adverse impacts of floodwater. These projects must "contribute to national economic development consistent with protecting the nation's environment, per national environmental statutes, applicable executive orders, and other federal planning requirements" (WRC 1983).

b. Risk is an index that considers the probability and consequence of flooding. As part of risk analysis, USACE policy requires PDTs to analyze the impact of parametric uncertainty on alternative selection. EM 1110-2-1619 describes USACE risk and uncertainty policy and procedures in detail, consulted for the full analysis procedures.

c. This chapter is a supplement to EM 1110-2-1619, focused specifically on integrating results from sediment analyses and uncertainty associated with sediment processes into risk and uncertainty analyses. Figure 10-1 illustrates chapter content.

d. Approach. USACE policy requires an evaluation of all engineering elements with the National Flood Insurance Program (NFIP) levee system probability and uncertainty analysis framework. This approach integrates a probability flood damage model to compute economic risk values. Figure 10-2 illustrates the steps of this computation (from EM 1110-2-1619). The framework explicitly considers several processes for four conditions.

10-2. Analyses – Four Conditions. USACE risk and uncertainty analyses consider four project conditions:

a. Baseline without Project. The baseline without-project condition is the current hydrologic, hydraulic, and economic condition in the study area, without action to modify flood risk. Detailed guidance for defining this condition is provided in ER 1105-2-100.

b. Baseline with Project. The baseline with-project condition includes current hydrologic, hydraulic, and economic conditions in the study area, with the proposed alternative implemented.

c. Future without Project. The future without-project condition is the expected future hydrologic, hydraulic, and economic condition in the study area, without further action to modify flood risk. If conditions in the study area are expected to change at any time during the period of analysis, those changes must be forecast, with probability and other functions developed to represent the changed conditions.

d. Future with Project. The future with-project condition, which comprises the expected future hydrologic, hydraulic, and economic conditions in the study area, with implementation of a proposed alternative.

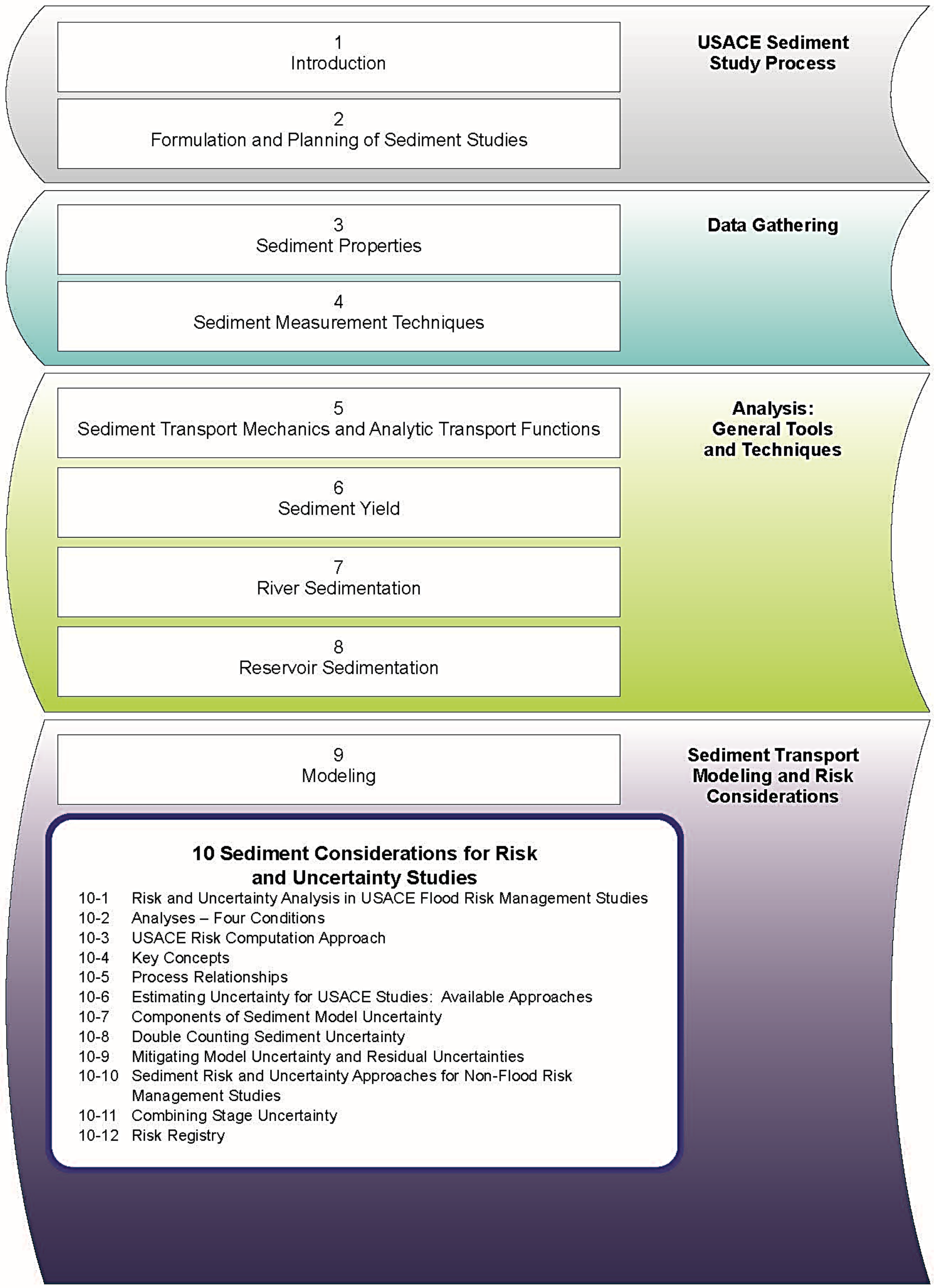


Figure 10-1. Chapter 10 content and general document structure

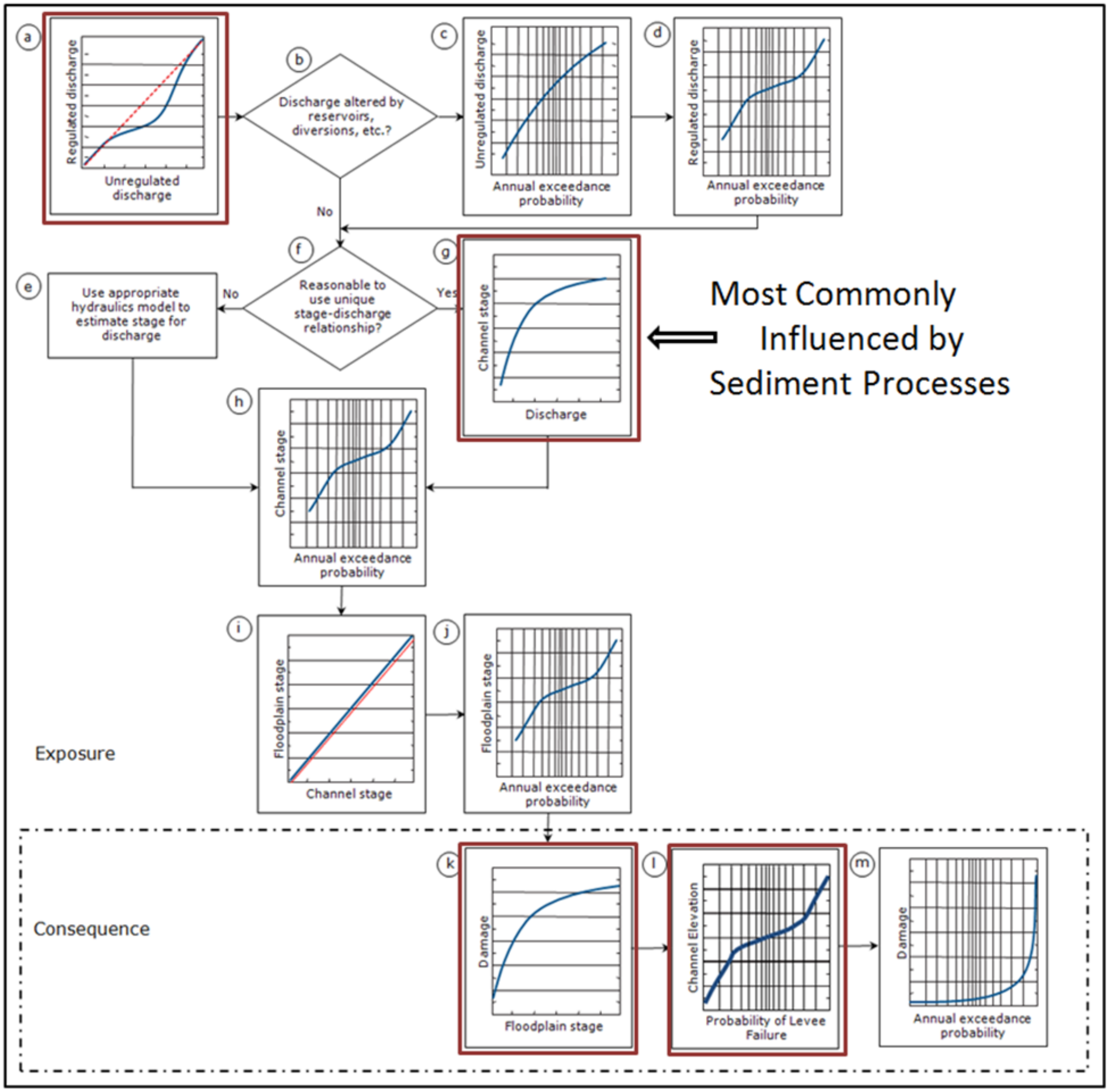


Figure 10-2. Schematic of the integration of risk analysis process relationship curves (redrawn from USACE 1996)

10-3. USACE Risk Computation Approach.

a. Event Timescale. Sediment analyses can directly inform the processes indicated with the red boxes in Figure 10-2. Consequently, following processes are indirectly affected.

(1) If sediment transport affects one of the process relationships in Figure 10-2 on event timescales (such as during a flood), sediment transport analyses could affect the baseline conditions (1 and 2).

(2) In some cases, high flows scour the channel and increase channel capacity at high flows. For example, Copeland (1986), demonstrates that the 1% event scours accumulated littoral sediment at the river mouth, dropping the downstream hydraulic control and creating flow capacity during the flood. A mobile bed model computed lower water surface elevations for large floods than a fixed bed model computed for baseline conditions.

(3) In other cases, a large event may deposit sediment, increasing the stage.

b. Long-Term Impacts. Sediment models (particularly 1D models) are much more reliable on multi-decadal, project time scales than events. Sediment impacts to the curves in Figure 10-2 are often either too small or too difficult to model on the event scale. Therefore, sediment analyses are rarely integrated into baseline conditions (either with or without project). Long-term erosion and deposition, on the other hand, are precisely the kind of system non-stationarity that future conditions (analyses conditions 3 and 4) account for. Long-term deposition or erosion likely to affect long-term performance of USACE projects must be included in future conditions.

10-4. Key Concepts.

a. Key Concepts. The key concepts for incorporating sedimentation processes in risk and uncertainty analysis presented in this chapter include:

(1) USACE standard practice includes risk and uncertainty analysis in project formulation. Refer to EM 1110-2-1619 for risk and uncertainty analysis guidance. Provided guidance is a supplement to that document.

(2) Hydrologic and hydraulic processes are interrelated with sedimentation processes. However, uncertainty estimates (as described in EM 1110-2-1619) are distinct from sedimentation-related uncertainty.

(3) Sediment transport is nonlinear, sediment data tend to be noisy, and sediment processes are often poorly understood. Therefore, sediment impacts on USACE projects must be incorporated in the risk and uncertainty framework.

(4) Sedimentation processes usually apply to the discharge-stage function and can also impact other functions including stage-damage, levee fragility, and project maintenance.

(5) Recognize that sedimentation is only one component of stage uncertainty. However, in studies that require sediment models, it is likely that sediment processes are usually a significant component of the total stage uncertainty.

(6) Sediment models are more suited for longer time scales. Single-event models are vulnerable to natural variability and stochastic fluctuations, which increases the uncertainty of the result.

(7) Qualified engineers can estimate uncertainty for some USACE projects without a mobile bed model. When sedimentation processes are well understood, trends are minor, and potential consequences of project non-performance are minimal, the project team can estimate sediment trends and uncertainty with trend analyses, analytical methods, or engineering judgment.

(8) Extreme events and project consequences should be considered in the risk and uncertainty process.

(9) Deferring sedimentation studies to future design phases via the risk register is not acceptable for USACE project evaluation if sediment processes may affect the selected alternative. Projects with challenging sedimentation processes must be evaluated early in the study process.

(10) Identification of complex sedimentation processes in the risk register will usually require detailed sedimentation studies that should be initiated early in the USACE planning process.

b. Application. Applying risk and uncertainty principles to sediment impacts on USACE projects can be a substantial effort. Sediment model results and other quantitative sediment analyses can be difficult to fit into EM 1110-2-1619.

(1) The project team must avoid double-counting uncertainty due to the overlap within the hydrologic, hydraulic, and sediment processes included in risk and uncertainty.

(2) Risk and uncertainty analysis should keep the final product in mind, an accurate determination of the contribution from sedimentation processes to the discharge-stage and the stage-damage relationships.

(3) However, sediment processes drive the uncertainty in some projects and introduce important failure modes at others, making them important components of the risk and uncertainty portfolio in many USACE studies.

(4) This chapter describes sources of sediment risk and uncertainty and methods for incorporating it in to EM 1110-2-1619.

10-5. Process Relationships.

a. Process Relationships. This section considers morphological influences that can alter the future condition of these process relationships. The USACE risk and uncertainty approach included in EM 1110-2-1619 and illustrated in Figure 10-2 integrates a series of discrete process relationships into a damage-probability curve that can be translated into a cost-benefit metric for National Economic Development (NED) analysis. Since sediment processes tend to affect channel conveyance, study results will influence the future Discharge-Stage (Figure 10-2 (g)) curve and consequently the future Stage-Frequency (Figure 10-2 (h)) curve.

b. However, there are at least four curves in the risk and uncertainty paradigm and an analysis on the cost side of the cost-benefit analysis potentially affected, including:

(1) Probability – Discharge (Figure 10-2 (d)) usually through the Unregulated Discharge to Regulated Discharge relationship (Figure 10-2 (a)).

(2) Discharge – Stage (Figure 10-2 (g)).

(3) Stage – Probability of Levee Failure (Figure 10-2 (j)).

(4) Stage – Damage (Figure 10-2 (k)).

(5) Maintenance Cost (not pictured).

c. Probability – Discharge (Figure 10-2 (d)).

(1) USACE analysis often accounts for the effects of urbanization and climate change on future probability-discharge relationships (described in detail in EM 1110-2-1619, Chapter 3). Natural morphological processes rarely impact flow probability over time and generally do not influence this relationship. However, sediment affects reservoirs, diversions, and other engineered features that regulate flow and transform the probability-discharge relationship.

(2) The probability of the regulated discharge (Figure 10-2 (d)) is a function of reservoir capacity, based on the relationship between Unregulated Discharge and Regulated Discharge (Figure 10-2 (a)). Sediment deposited in reservoirs (Chapter 8) decreases reservoir capacity, increasing the probability of downstream flows, making the system behave more like an unregulated river, particularly for high flows. Therefore, by reducing reservoir capacity (and the regulated discharge curve), sediment can affect the probability-discharge curve (Figure 10-3).

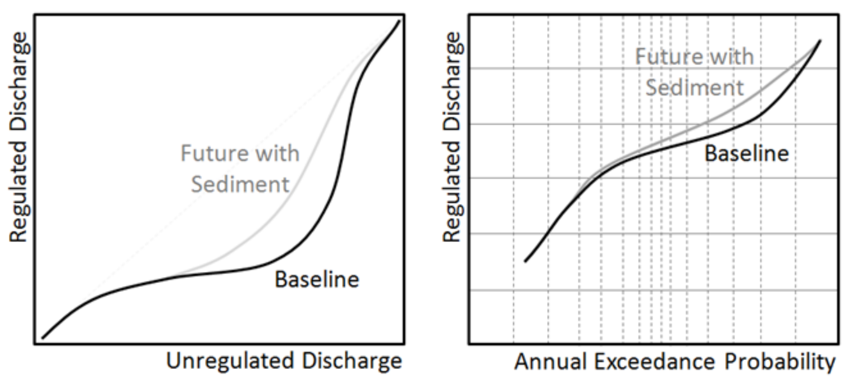


Figure 10-3. Potential impact on the regulated-unregulated relationship due to reservoir deposition and the resulting impact on the probability-discharge curve

(3) USACE (2012a) illustrates the reservoir sedimentation impacts on the reservoir stage-probability curve and the downstream probability-discharge relationship (Figure 10-4). This study explores the impact of future sediment deposition rates under different climate change scenarios at Garrison Dam, showing higher stages and reservoir release flow-exceedance curve.

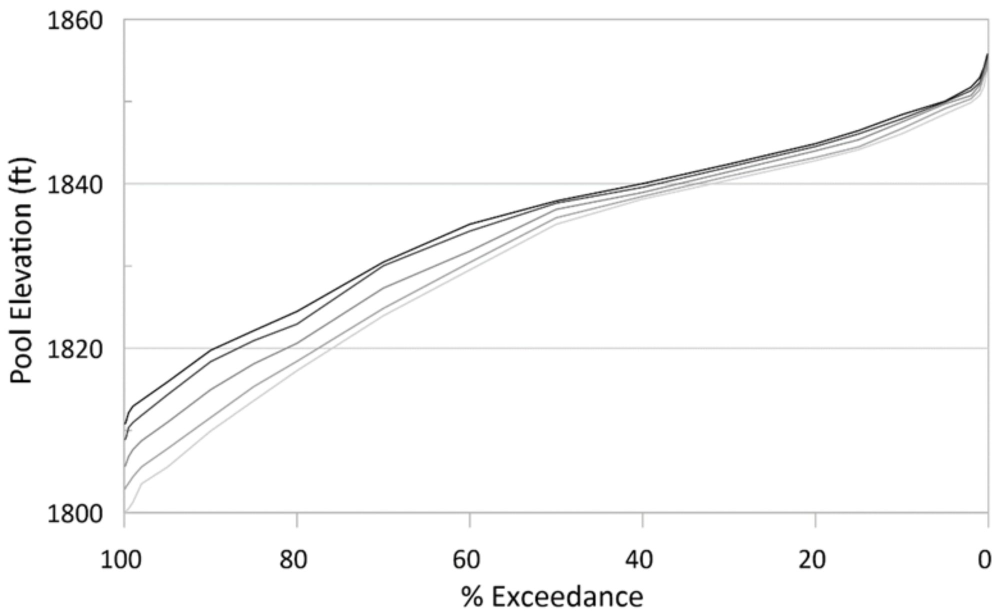


Figure 10-4. Sediment processes impacts on pool elevation; different climate projections deliver a range of sediment loads, which impact the future elevation-probability relationship (making higher pool elevations more likely) (redrawn from USACE 2012a)

d. Discharge-Stage (Figure 10-2 (c)).

(1) Most FRM sediment studies evaluate impacts on the future condition discharge-stage curve. Sediment erosion or deposition affect channel conveyance, increasing or decreasing water surface elevation associated with a given flow (Figure 10-5). Sediment transport models like HEC-RAS and AdH can update the hydraulic model bathymetry to predict the future project condition discharge-stage relationships.

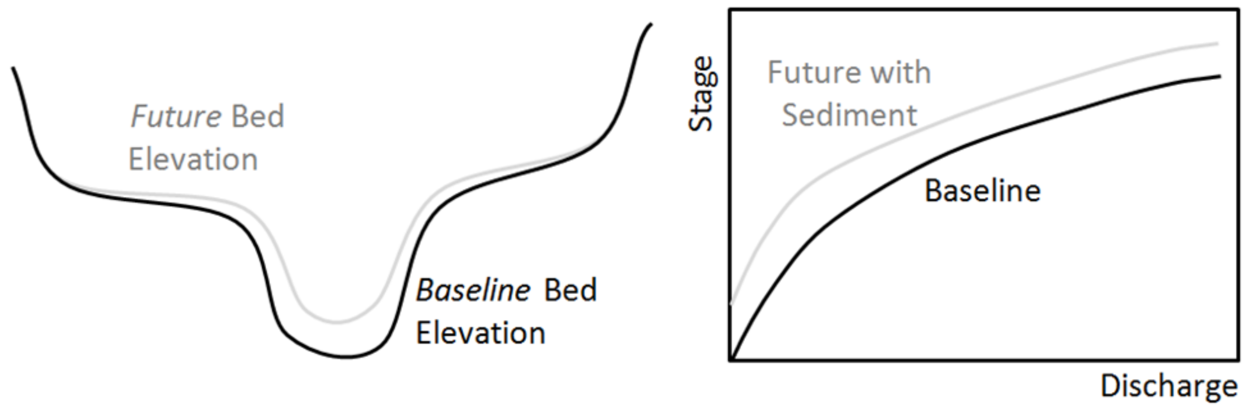


Figure 10-5. Common influence of sediment deposition on the discharge-stage curve

(2) The Cowlitz River, downstream of Mount St. Helens, is one of the most dramatic examples of sediment impacts on the discharge-stage curve. Without mitigation, hundreds of millions of tons of sediment introduced by the 1980 eruption would have deposited inside the Lewiston levees. These deposits would have increased the flood stages and decreased the level of protection. More commonly, the supercritical channels characteristic of USACE FRM projects in the 1970s and 1980s, tend to deposit sediment where the engineered channels transition from supercritical to subcritical, reducing the level of protection unless consistently maintained.

(3) Channel modifications achieve flood risk benefits by lowering the flow-stage relationship. However, sediment analysis must evaluate the resilience and sustainability of these alternatives. Excavating channels to increase conveyance often reduces channel velocity, shear stress, and sediment transport capacity, causing deposition. Deposition reduces conveyance and increases stage, reducing future flood risk benefits or introducing O&M costs. Sediment analysis is a critical component of these studies because sediment processes impact the future condition discharge-stage curve, affecting sustainability of benefits.

(4) Likewise, channel restoration projects that change the slope and conveyance of a channel by introducing meanders or grade control often target the stage-discharge relationship. These measures often try to increase the frequency of floodplain inundation or maintain minimum riparian water tables. Long-term sediment scour or deposition will affect the future condition stage-discharge relationship and the project benefits.

e. Stage – Probability of Levee Failure (Figure 10-2 (j)).

(1) Sediment processes can affect levee fragility curves. Levee failure is non-deterministic and should instead be considered as the response of the system to failure for the entire range of hydrologic loading. While overtopping of the system may approach a probably of failure of 1, it is likely that there is some probability that the system can withstand an overtopping event.

(a) Levees and floodwalls also have non-zero risks of breaching prior to overtopping. These risks can be related to internal erosion, slope stability, floodwall stability, and a host of other potential failure modes. Each potential failure mode, however, shares a similar trend of increasing risk with increasing river stages up to and beyond the top of levee.

(b) Fragility curves quantify this uncertainty with an estimate of the relationship between water surface elevation and the probability of levee failure (Figure 10-2 (l)), where low stages have very low probabilities of failure and overtopping stages approach 100% probability of failure.

(2) Morphological processes can impact these relationships, increasing the probability of failure at various water surface elevations. Sediment processes can increase the probability of levee failure by undermining the toe of the levee, either by regional base-level degradation or localized toe scour (Figure 10-6). For example, many alluvial river floodplains are stratified. Fine silt-clay material deposits overlay coarse sands and gravels. In these cases, a levee encroachment can elevate floodplain velocities, eroding the top silts and clays, which have a lower seepage rate. This preferential erosion can elevate levee seepage rates.

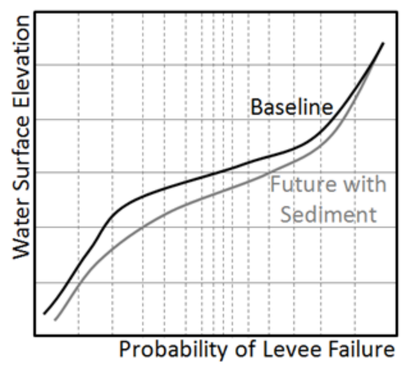


Figure 10-6. Influence of incision or scour on the fragility curve

(3) Walla Walla District applied a Snake River sediment model to estimate the impact of regional morphology on levee fragility. The Jackson reach of the Snake River is degrading because upstream reservoirs intercept historic load, dropping the channel base level. USACE Walla Walla District modeled projected base-level change that was included in the geotechnical levee fragility analyses. USACE Kansas City District has also evaluated the role of Missouri River channel degradation on the reliability of levees through Kansas City (USACE 2017b).

f. Stage – Damage (Figure 10-2 (k)).

(1) Sediment processes are rarely considered in the stage-damage analysis. However, floodplain deposits can damage urban and agricultural areas, introducing cleanup costs. During the 1993 and 2011 floods, the Missouri River deposited so much sediment in the floodplain in some places that removing sediment to restore agriculture was cost prohibitive (Figure 10-7a). In the steep Colorado rivers, 2011 flood damage included sediment processes that led to channel avulsion and large-scale sediment transport through urban floodplain areas (Figure 10-7b), in addition to high flood waters. Refer to Case Study 10A (Appendix N) for additional factors that abruptly shift the stage-damage curve.

(2) Additionally, scour associated with large events, or long-term base-level changes induced by USACE projects or independent of them can strand public works (water intakes or outlets) and undermine infrastructure (bridges on the main stem or tributaries) (Shelly 2017).

Figure 10-7. Examples of sediment processes: (a) sand deposition adjacent to Missouri River, landward of private levee that overtopped, October 2011; (b) flood damage in Longmont, Colorado, due to channel avulsion, September 2013

g. Maintenance Costs.

(1) In the previous four relationships, sediment affects the final probability-damage curve computation, which are integrated to compute a total project benefit (damage avoided) by modifying one of the mechanistic relationships combined to commute the final probability-damage curve. However, NED analysis requires a comparison of project benefits with project costs to develop a benefit-cost ratio. Sediment analysis affects the benefit analysis in the four ways described above. However, the PDT must also consider sediment related long-term project maintenance costs (equal to or greater than the 50-year project life).

(2) Underestimating sediment maintenance costs have rendered some USACE FRM projects unsustainable (Wong and Kondolph 2015). Local sponsors were unprepared to fund sediment maintenance, which proved much more costly than expected. This has been one of the primary failure modes of USACE channel modifications for FRM. The magnitude and cost of future sediment maintenance must be estimated, which often requires careful modeling.

(3) Wong and Kondolph (2015) analyzed nine USACE FRM projects in the San Francisco Bay area. The projects that performed well quantitatively accounted for sediment in their maintenance plans. Those that made simple maintenance assumptions (commonly, that annual maintenance would be 0.5% of project cost) proved too expensive to maintain. Deposition reduced the project life level of protection.

(4) Maintenance uncertainty is associated with both the magnitude and cost of sediment removal. Sediment modeling can reduce the uncertainty of the magnitude, but sediment removal costs are themselves non-stationary. Even if sediment processes are predicted precisely, uncertainty of future cost-per-unit sediment removal can make projects impractical to maintain. Loss of disposal areas for removed material and contamination are examples of issues that affect future cost. Additionally, occasional assumptions that sediment could be sold to offset the cost of maintenance (USACE 1973) have mostly not been demonstrated.

10-6. Estimating Uncertainty for USACE Studies: Available Approaches.

a. Available Methods. Many USACE studies will not require mobile bed sediment models. However, all USACE studies must explicitly address potential sediment transport impacts and implications of morphological change on project objectives (citation). Any study that identifies current or future sedimentation processes that may affect stage-discharge trends must include sediment-induced stage uncertainty in the economic analysis. These studies may quantify sediment uncertainty in the stage-discharge curve with several methods including trend analysis, specific gage analysis, expert elicitation, modeling, or a combination of these methods.

b. Selection of Methods. Selecting a USACE project design without quantifying sediment impacts implies that sedimentation processes are well understood, that the changes in the stage-discharge rating curve are unlikely, and that the consequences of low probability stage-discharge risks are acceptable. Expert elicitation or other qualitative methods are not an acceptable substitute for quantitative methods in USACE projects with high sediment risk or uncertainty.

c. Quantifying Sediment Impacts. If sediment is likely to impact one of the relationships in Figure 10-2, or if the probability of morphological modifications to these curves is low, but the consequences are serious, the sediment impacts and uncertainty around these curves must be quantified. Sediment impacts can be quantified by two methods: trend analysis and mobile bed modeling. Three main criteria drive the decision between trend analysis and mobile bed modeling: stationarity, feedbacks, and data quality. The data in Table 10-1 summarize these considerations.

(1) Trend Analysis. First, trend analysis assumes stationarity; the conditions that caused historic sediment trends are likely to continue throughout the project life.

(a) If sediment load or gradation is expected to change over the project life, a mobile bed model will be required to assess the impact of these changes on project performance.

(b) Hydrologic non-stationarity can also have morphological impacts. Flow regime changes can affect deposition or erosion rates. If future flows are likely to change enough to affect the rate of erosion or deposition (or even switch the dominant process), the study team should model the impacts.

(c) The project itself is the most common and often, most significant, non-stationarity issue that drives morphological modeling. Project alternative changes in the hydrology, hydraulics, or sediment continuity that are large enough to change rate or magnitude of bed change should be modeled.

(2) Mobile Bed Models. Second, the study team should consider mobile bed models any time the sediment processes change the channel conditions enough to alter future sediment processes. Models consider feedbacks between morphological change and future sediment processes explicitly. For example, if deposition increases the slope along the reach, it will increase the slope, shear, and transport capacity, decreasing future deposition rates. In this case, trend analysis would underpredict future conditions. However, trend analyses that ignore feedbacks can be conservative or unconservative.

(3) Data Quality. Third, data quality can drive the decision between trend analysis and modeling. If historic data are abundant and reliable enough to establish historic trends, trend analysis is an option. Modeling, however, will interpolate and extrapolate from sparse data to provide a more reliable estimate if data are scarce or unreliable. The data used to develop trends also must represent the expected flow regime. If historic bed change trends are computed from a short hydrologic record (for example, less than 10 years) it is unlikely to represent the full range of flows. Additionally, if a longer trend is dominated by flood or drought conditions, or otherwise does not represent the likely future flow or sediment regime, trend analysis is not appropriate.

Table 10-1
Summary of Stationarity, Feedback, and Data Quality Considerations That Drive the Decision between Quantitative Sediment Impact Analyses: Trend Analysis and Mobile Bed Modeling

	Trend Analysis	**Mobile Bed Modeling**
Stationarity Considerations	Future sediment loads and gradation are stationary.	Future sediment loads or gradations are non-stationary.
	Future flow regime likely to approximate past flow regime.	Reach flows likely to change over the project life enough to mitigate or accelerate sediment trends.
	Proposed project condition does not change hydraulics in ways that will increase or decrease the future rate of erosion or deposition.	Proposed alternatives change hydraulics or hydrology or sediment continuity.
Feedback Considerations	Predicted bed change will not change the channel hydraulics enough to alter future sediment processes.	Proposed alternatives change hydraulics or hydrology or sediment continuity that will affect the rate or magnitude of future sediment processes.
Data Quality Consideration	The project reach has enough data, of sufficient quality, collected over a representative range of conditions, to establish reliable trends.	Data are not sufficient to establish a historic trend reliably or reflect a time period that is too short or idiosyncratic (dominated by flood or drought) to be representative of future conditions over the project life.

d. Uncovering Uncertainty. Estimating sediment-related uncertainty with and without modeling can introduce a paradox. Intuitively, modeling should reduce uncertainty since the modeling improves understanding of sediment processes. However, mobile bed modeling can generate larger uncertainty bounds than more approximate or qualitative sediment uncertainty estimates. Modeling often uncovers new sources of sediment risk and uncertainty that trend analyses and expert elicitation may ignore. Therefore, if modeling increases sediment uncertainty bounds, the model is not creating uncertainty, but uncovering important uncertainty overlooked by other methods.

10-7. Components of Sediment Model Uncertainty.

a. Total Uncertainty. In addition to predicting the most likely future conditions (such as the flow-stage curve), sediment models also predict the uncertainty associated with future condition estimates (such as the distribution around the flow-stage curve). Sediment processes decrease confidence of future conditions, increasing the uncertainty bounds around the relationships (Figure 10-8). Because sediment transport is nonlinear, sediment processes introduce asymmetrical uncertainty, which skews high. The uncertainty associated with the sediment analysis must be combined with other discharge-stage uncertainty.

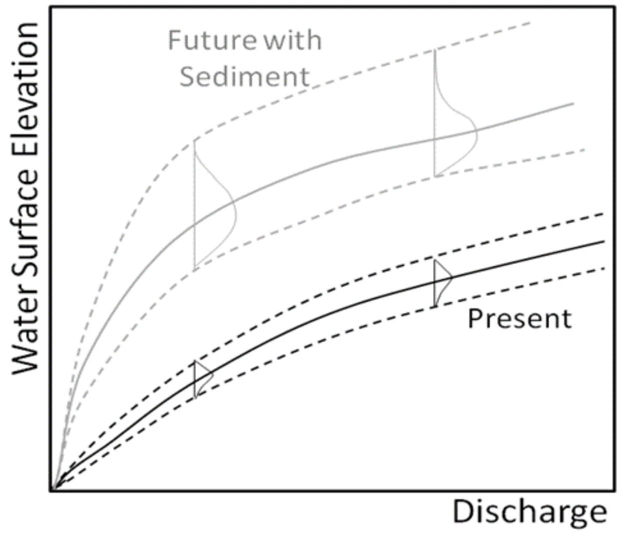


Figure 10-8. Sediment processes not only change the future conditions discharge-stage relationship, but also increase the uncertainty around the curve

b. Sources of Sediment Uncertainty. In addition to the direct impacts of sediment uncertainty (see next section) on the discharge-stage curve, classical hydraulic and hydrologic uncertainties have sediment feedbacks that affect how they are incorporated into a sediment analysis.

c. Hydraulic Uncertainty.

(1) Processes and principles for computing hydraulic uncertainty are included in EM 1110-2-1619. These methods quantify uncertainty due to topographic and bathymetric resolution, downstream stage, and roughness coefficients. Each of these interact with sediment processes (for example, episodic deposition or scour or even bed forms can generate temporal variability in the flow-stage relationship), but the roughness parameter is usually the most important source of stage uncertainty. Uncertainty in the roughness parameter is also interrelated with sediment uncertainty. Sediment processes can influence the roughness parameters. Channel roughness can vary in response to changing bed material or bed form regime.

(2) Bed roughness affects water surface elevations in fixed bed models, making it the primary source of parametric uncertainty in the discharge-stage curve. The hydraulic fixed bed model roughness estimates are more certain than sediment modeling parameters. However, sediment results are often very sensitive to roughness, amplifying uncertainty associated with this parameter in morphologically active systems. Ruark et al. (2011) demonstrated that n-value sensitivity was not only responsible for nearly all of the first order sensitivity in their hydraulic velocity results, but over half of the first order sensitivity in the deposition results for a well parameterized depositional flume model.

(3) Therefore, if sediment processes affect future conditions, uncertainty in the roughness parameter will not only translate into uncertainty in the baseline conditions, but will also amplify stage uncertainty in future conditions.

d. Hydrologic Uncertainty.

(1) Uncertainty is often divided into two categories: epistemic and aleatory. Epistemic uncertainty reflects limited knowledge that could be improved with more data collection or analysis. Projects accept epistemic analysis because they cannot justify the diminishing returns of additional investments to reduce this uncertainty. Aleatory uncertainty, on the other hand, is more fundamental. This type of uncertainty cannot be reduced by more data or analysis. In sediment models, much of the uncertainty is epistemic, due to limited knowledge. However, the uncertainty of future hydrology introduces an aleatory component to sediment uncertainty.

(2) Long-term sediment models generally predict future bathymetry after simulating a hydrologic period of record. Because of the nonlinearity of sediment processes with flow, the selected flow record will influence predicted future condition flow-stage relationships substantially. However, because predictive sediment models simulate a future period of record to evaluate future conditions, the hydrology is unknown. Future hydrology is not only a source of uncertainty, it is also a source of aleatory uncertainty. Therefore, while it is difficult to reduce the hydrologic uncertainty associated with sediment models, it can be (and often should be) quantified.

(3) There are three basic approaches to projecting future hydrology for sediment studies:

(a) Repeating a representative hydrograph.

(b) Repeating the period of record.

(c) Monte Carlo Analysis.

(4) Representative Annual Hydrograph. A representative hydrograph combines measured and statistical flows into a single, annual hydrograph.

(a) The historic period of record is ordered into a flow duration curve, then compressed and temporally normalized to generate a 365-day hydrograph.

(b) Probabilistic flows not captured in the period of record are added to incorporate the impact of these unsampled, large-flow, low-probability events. The 10%, 4%, 2%, 1% and, sometimes rarer events from a flood frequency analysis are added to the hydrograph. The 1% flow, with a probability of 1 day in 100 years, represents 0.0027% of the total flow record. This compresses to about 15 minutes of a representative annual hydrograph.

(c) Flows are sometimes reordered into a symmetrical hydrograph centered on the most likely peak flow date. Because the representative hydrograph compresses the whole flow record into a single year, multi-year flow records can be constructed by repeating them. By repeating the representative annual event, the model builds all the hydrologic information into a long-term simulation.

(5) Repeated Period of Record. Sediment processes are not simply additive. Hydrologic sequence matters in sediment modeling. The same flood event will affect a project reach differently depending on when it occurs in the project life and the ambient sediment conditions left by the preceding flow regime.

(a) Therefore, the representative hydrograph approach can distort results. As flow records grew, predictive sediment models used the hydrologic record, assuming past hydrology approximated the future.

(b) If the project life or prediction window is longer than the historic record, the historic record is often repeated. Repeating the historic record can exclude rare events and may artificially over-represent sampled events. However, it is better than the representative annual hydrograph because it captures the variability, shape, and sequence of the natural hydrograph, and because it is more likely to produce representative sediment simulations.

(6) Stochastic Time Series.

(a) Modeling multiple stochastic time series pairs the benefits of the other two methods and explicitly quantifies the hydrologic uncertainty in the sediment results. Gibson and Pridal (2015) used the following process:

- First, they ordered the years in the Argandab flow record according to the 7-day maximum flow, and associated a probability with each extant year.

- Then they computed Log-Person III distribution with the statistical software package (HEC-SSP) to estimate the 1-day, 3-day, and 7-day peaks for that system.

- Then they modified the peaks of existing annual hydrographs to generate "synthetic" annual hydrographs associated with the years associated with the 4%, 2%, 1%, 0.5%, 0.2%, and 0.1% floods, which were not in the record, though local reports provided anecdotal evidence of flows much larger than those recorded.

- Then the existing annual hydrographs and the synthetic hydrographs of rare events were sampled with a Monte Carlo approach to generate multiple 50-year time series that predicted the most likely result from the recorded and synthetic annual hydrographs.

(b) The approach to model multiple stochastic time series pairs used on the Argandab also quantified the hydrologic uncertainty, illustrating the uncertainty in future reservoir volume (Figure 10-9) based on the aleatory uncertainty of the future hydrology. This approach was also used for studies on the Muskegon River (Gibson 2016).

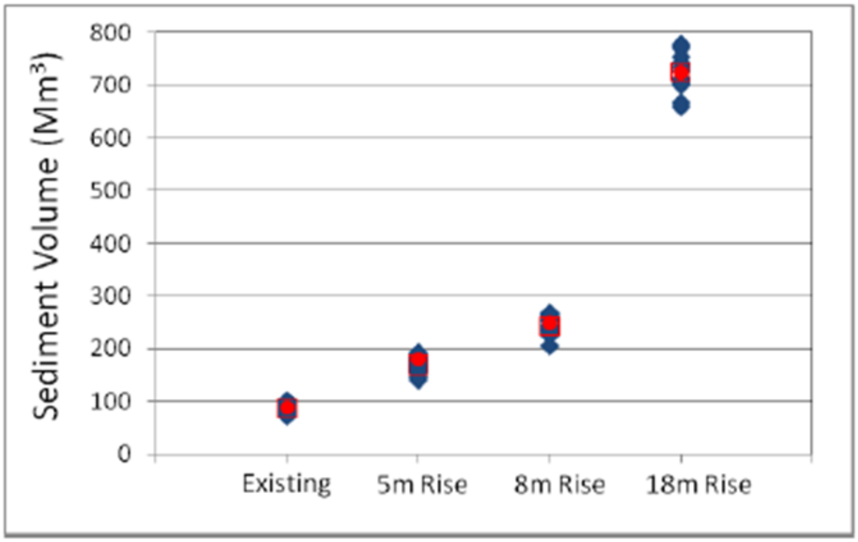


Figure 10-9. The final predicted reservoir water volume (in 2065) beneath the final spillway elevation for three spillway rise alternatives for the predictive sediment simulation random 50-year hydrologic scenarios, which quantify the central tendency and the hydrologic uncertainty in the sediment result (Gibson and Pridal 2015)

(c) Flood studies generally assume that maximum damage occurs at the peak stage, which occurs at the peak flow. These studies rank the period of record by peak flow. Studies that find that maximum stage is not coincident with the peak flow (for example, reservoir pools or backwater areas and ice jam locations), use stage-frequency analysis and rank the period of record by peak stage.

(d) In sediment studies, however, the most important year may not include the highest peak flow. Both flow duration and magnitude matter, in addition to sediment supply and other factors. Therefore, sediment studies should evaluate stochastic time series based on sediment response rather than a peak flow. For example, Shelley (USACE 2017b) transformed each year into an annual sediment load with a bed material rating curve (Figure 10-10). He then computed the cumulative bed material load from 999 randomly sampled combinations of 50 individual flow years to quantify the potential aleatory uncertainty.

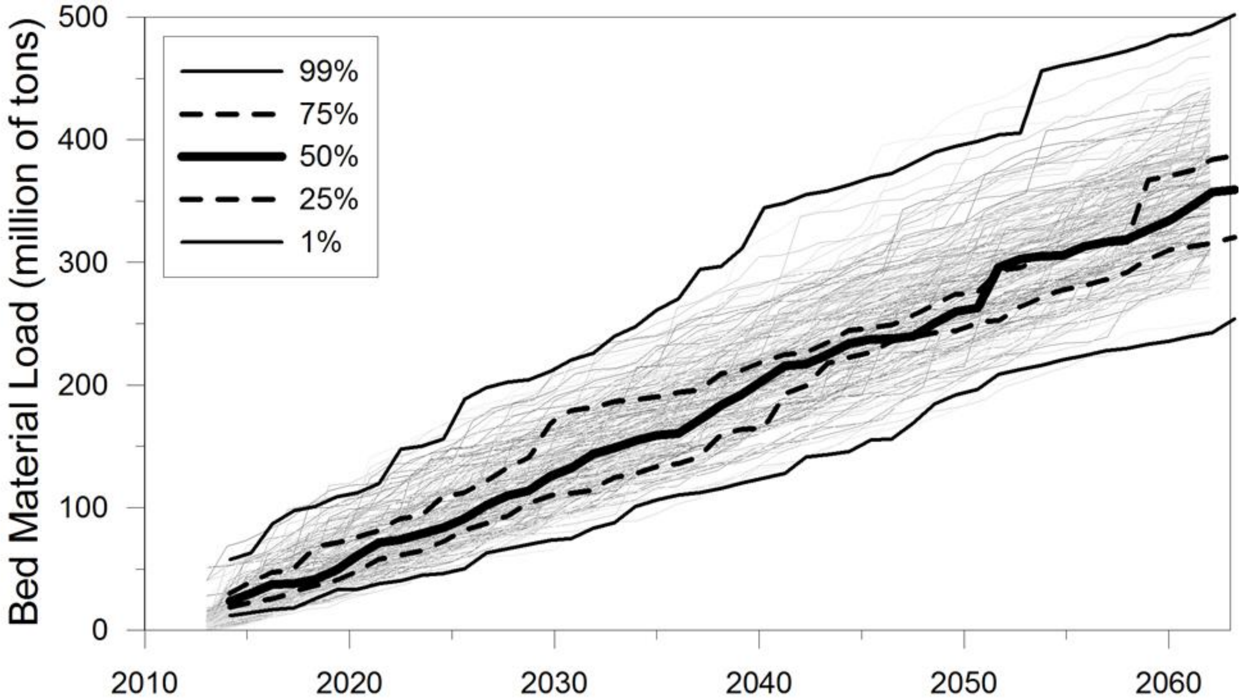


Figure 10-10. Cumulative bed material load computed from 200, 50-year flow scenarios and five percentile curves from the Missouri River at St. Joseph, Missouri (redrawn from USACE 2017b)

(e) The Hydrologic Engineering Center's Watershed Analysis Tool (HEC-WAT) includes a bootstrap hydrologic sampler that can generate stochastic, project scale (for example, 50-year) hydrologic time series, and automatically launch HEC-RAS sediment transport simulations with these hydrologic realizations to quantify the impact of potential flow variability on sediment impacts (Figure 10-11).

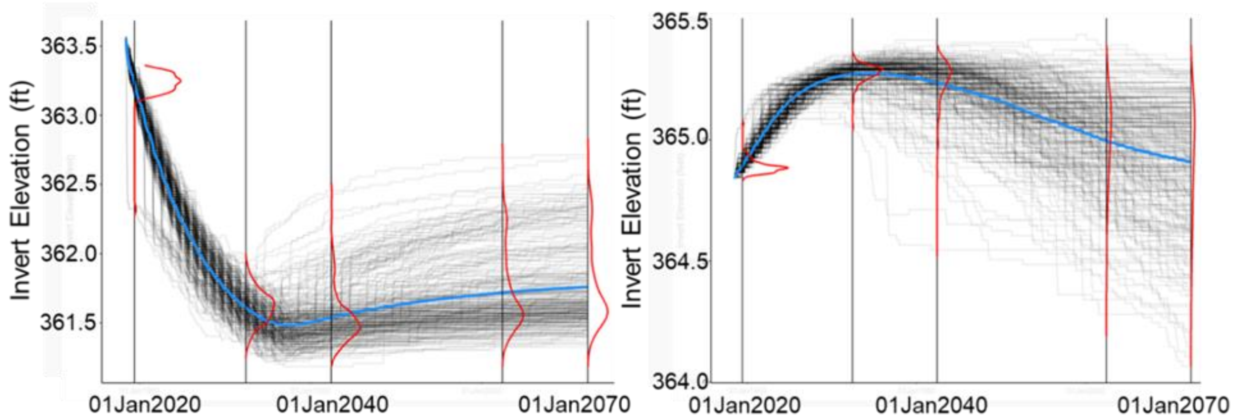


Figure 10-11. Three hundred and fifty, 50-year bed change realizations, from an HEC-RAS model of Lewis and Clark reservoir including the median bed change (blue) and progressing distribution in time (red) for two different cross sections; the realizations reflect different hydrologies, based on automated hydrologic sampling by the HEC-WAT

e. Sediment Uncertainty.

(1) There are three main categories of epistemic sediment uncertainty:

(a) Algorithmic uncertainty (the difference between a process and the generalized equation used to represent it).

(b) Measured data uncertainty (the precision and accuracy of measured values).

(c) Parametric uncertainty (natural variability, unaccounted processes, non-stationary).

(2) Uncertainty from the algorithms, measured data, and parameter sensitivity are generally higher (less certain) in sediment models than hydraulic models.

f. Algorithmic Uncertainty. Uncertainty associated with sediment transport algorithms is well documented. The equations used for sediment transport are more empirical and less physics-based than those applied to other hydraulic fields. When empirical equations are applied to the diverse sediment settings (or even in the logarithmic distribution of particle sizes) empirical equations can compute divergent results. Therefore, the empirical algorithms used to compute sediment transport have more inherent uncertainty than the hydraulic or even hydrologic equations.

(1) Dimensional Simplification. Sediment transports multidimensional, turbulent, multi-phase flow fields. Multidimensional models that account for turbulence explicitly can reduce this uncertainty over a 1D model, and fine resolution models usually have less uncertainty than coarser resolutions, but all models make flow field simplifications.

(2) Transport Function. Selection of the sediment transport equation is a critical component in sediment transport modeling. Transport functions quantify sediment flux (mass/time or volume/time) from flow properties (such as flow, shear, stream power, velocity, energy slope) and sediment properties (such as particle size, mixture properties, density), which represent the driving and resisting forces, respectively. Results vary dramatically between equations. Two carefully selected sediment transport equations, both appropriate for the system flow and sediment properties, often produce results that differ by an order of magnitude. This uncertainty can (and must) be mitigated by carefully selecting transport functions and proper calibration.

(3) Sorting, Mixing, and Armoring Approach. Sorting and armoring algorithms are usually the most complicated part of sediment transport models, generating counter-intuitive threshold effects and nonlinearities in the results. Unfortunately, the sediment models can also be very sensitive to the mixing and armoring approach selected. Ruark et al. (2011) did not find active layer thickness highly sensitive, but Gibson and Piper (2007) demonstrated that results could be very sensitive to these algorithms, particularly in eroding models. Senior USACE modelers have found that the mixing and armoring algorithm can be the most sensitive sediment modeling assumption.

(4) Bed Change Algorithm. Models translate deposition or erosion into new bathymetry, adjusting nodes or cross sections between time steps. Bed change is simpler in multidimensional models that adjust each model node independently based on local results. One-dimensional models make assumptions about how to apply deposition or erosion across a cross section, introducing uncertainty.

g. Data Uncertainty.

(1) Sediment data are rarer than hydrologic data. When sediment load and gradation data are available, they are less precise than hydrologic data, and they are often collected at low flows that do not drive the system morphologically. Therefore, in addition to natural variability and process non-stationarity that makes parameter estimation difficult (see below), the data are sometimes rare, biased, or not representative of morphologically important conditions. Data uncertainty is connected to parametric uncertainty (below), but sediment data suffer from both scatter and bias (failure of precision and accuracy):

(a) Scatter (Precision). Sediment load and gradation are more temporally episodic and spatially heterogeneous than hydrologic measurements, which decrease the probability that a sample represents transport conditions.

(b) Bias (Accuracy).

(2) There are at least three common sources of bias in sediment data (Gibson and Cai 2016). These biases can and should be accounted for when developing model input data. However, adjusting parameters to account for data bias introduces uncertainty.

(3) First, most publicly available USGS sediment data are suspended-sediment measurements, excluding bedload portions. Failing to account for the bedload portion, not only in the boundary condition mass, but also in the gradation, can lead to grossly underpredicting deposition or overpredicting erosion.

(4) Second, sediment data are disproportionally collected at the least interesting flows. Sampling sediment during flood events can be difficult and dangerous. However, sediment processes are nonlinear with flow, so morphological processes are disproportionally sensitive to rare, large flows that generally go unsampled.

(5) Finally, some sampling methods have demonstrated systematic bias. Bunte et al. (2004) demonstrated that the Haley Smith bedload sampler generally underpredicts bedload transport. Additionally, bulk bed samples in armored systems mix coarse cover layer gradations and finer subsurface gradations, confounding the numerical representation of bed gradation, which can cause a model to overpredict initial scour and overpredict long-term scour.

h. Parametric Uncertainty. After algorithm selection and data processing, the modeler manages uncertainty through parameter selection. All measurement error (scatter and bias), natural variability, and unmodeled processes are accounted for in the parameters the modeler selects. Parameters are selected and calibrated to reproduce system behavior over an extended time series, to reduce this uncertainty.

(1) Natural Variability.

(a) Sediment parameters and process vary temporally and spatially. Even if sediment sampling were completely precise and accurate, statistical sampling error, the divergence of an individual observation from the system behavior, would still add uncertainty. For example, perfect knowledge of past sediment loads and flow will not translate into a completely reliable, future flow-load relationship. Storm centering, multiple events, watershed processes, snow pack, hysteresis, and other natural variability in the system makes flow an imperfect predictor of load. Therefore, while more sediment load samples will reduce knowledge uncertainty and improve prediction, that improvement is limited by natural variability in the process.

(b) The biggest source of natural variability in sediment studies is future flow variability. Because sediment transport tends to be a non-linear function of flow, sediment impacts on USACE projects will depend on the magnitude and timing of flows during the project life. Figure 10-12 illustrates this principle by simulating deposition with a simple, non-linear, deposition model for fifty sample, 50-year hydrologic realizations. Deposition (which would affect the flow-stage relationship) is very sensitive to the hydrologic variability, and the non-linearity of the sediment processes skews the distribution, generating a “high tail” of high deposition events.

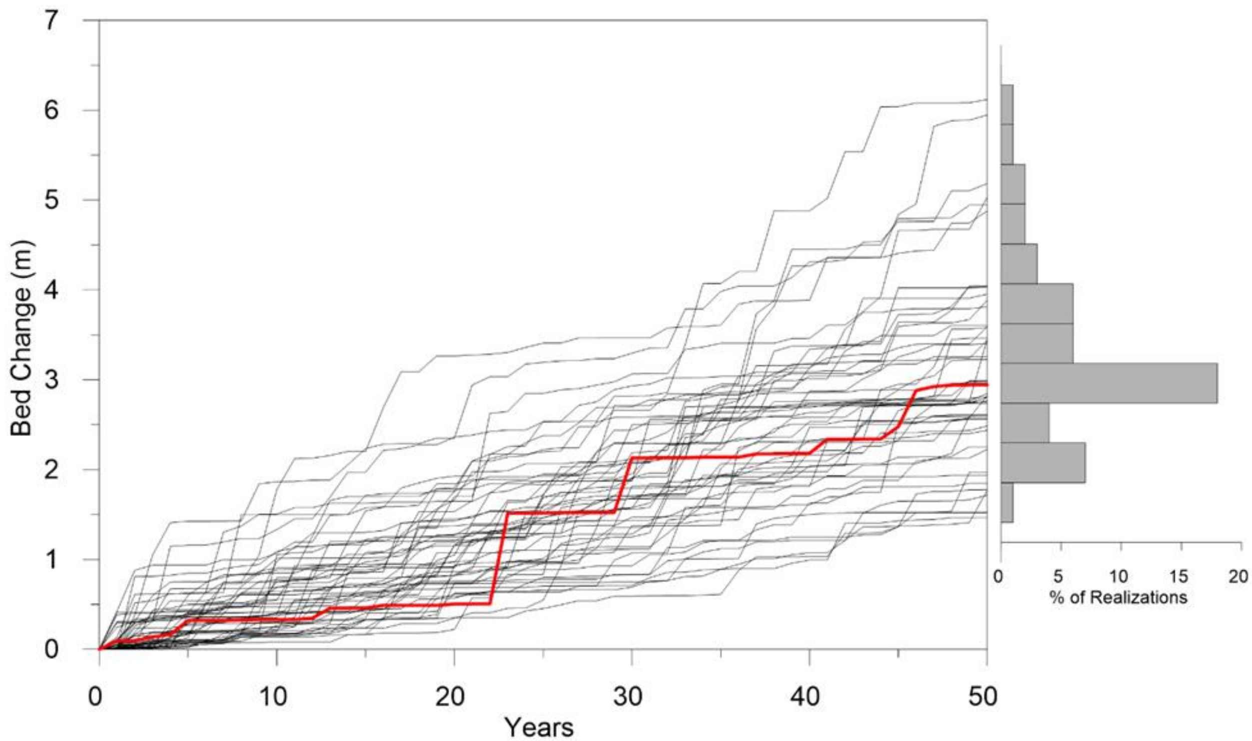


Figure 10-12. Fifty realizations of sediment response to flow variability (using a simple, nonlinear deposition model and stochastic annual hydrograph sampling); bed change is very sensitive to hydrologic variability and nonlinearity of the sediment equations skews the response, generating rare, large deposition realizations (Gibson et al., 2019)

(2) Unmodeled Processes.

(a) Sediment models simplify processes to simulate them. For example, most sediment models assume that the inflowing sediment load and gradation are a unique function of flow. The actual processes are much more complex, with seasonal effects (Gibson 2010) and hysteresis effects (that is, loads on the rising limb of the hydrograph are higher than the same flow on the falling limb) (Rubin and Topping (2001)).

(b) Measurement error, natural variability, and unmodeled processes must be accounted for in parameter selection so that simplified processes produce the same system response on long time scales. This is one of the reasons sediment models tend to be useful on project time scales. Given time, a model will smooth out the impacts of natural variability and unmodeled processes, allowing them enough time to regress to the mean. Applying a sediment model to a single event makes it vulnerable to the fluctuations of natural variability, and increases the uncertainty of the result.

(c) The uncertainty and sensitivity of each parameter should be assessed. Well-sampled parameters (those with abundant, high-quality data) and low sensitivity parameters (those that do not affect the model output) should be fixed, while highly sensitive, uncertain parameters can be adjusted as part of the model calibration process (see paragraph 9-4). The four data sources or input parameters discussed in the following sections tend to be most sensitive and most uncertain in sediment transport models.

(3) Hydraulic Parameters. Hydraulic uncertainty, including uncertainty associated with the roughness coefficients and the downstream boundary conditions affect the flow-stage relationship, model velocity, and computed shear stress. Because the sediment transport equations are extremely sensitive to these parameters, these uncertainties propagate through the sediment model and translate into sediment uncertainties. These uncertainties can be managed through the staged calibration process described in paragraph 9-4. Hydraulic uncertainty is usually best managed by calibrating these parameters during a fixed bed simulation before the mobile bed sediment transport model.

(4) Flow-Load Rating Curve.

(a) Flow-load rating curves are the most common upstream sediment boundary condition in reach scale models. Suspended-sediment load measurements, paired with flow measurements, are also the most common sediment data available. However, when flows and loads are plotted to generate a flow-load rating curve, the data cloud routinely spans 1 to 2 orders of magnitude (Figure 10-13). Some of this scatter is the result of seasonal trends, non-stationarity in the data, or event hysteresis (higher concentrations on the rising limb than the falling limb) and could be parsed. Nevertheless, it is common to estimate this relationship with a single flow-load relationship.

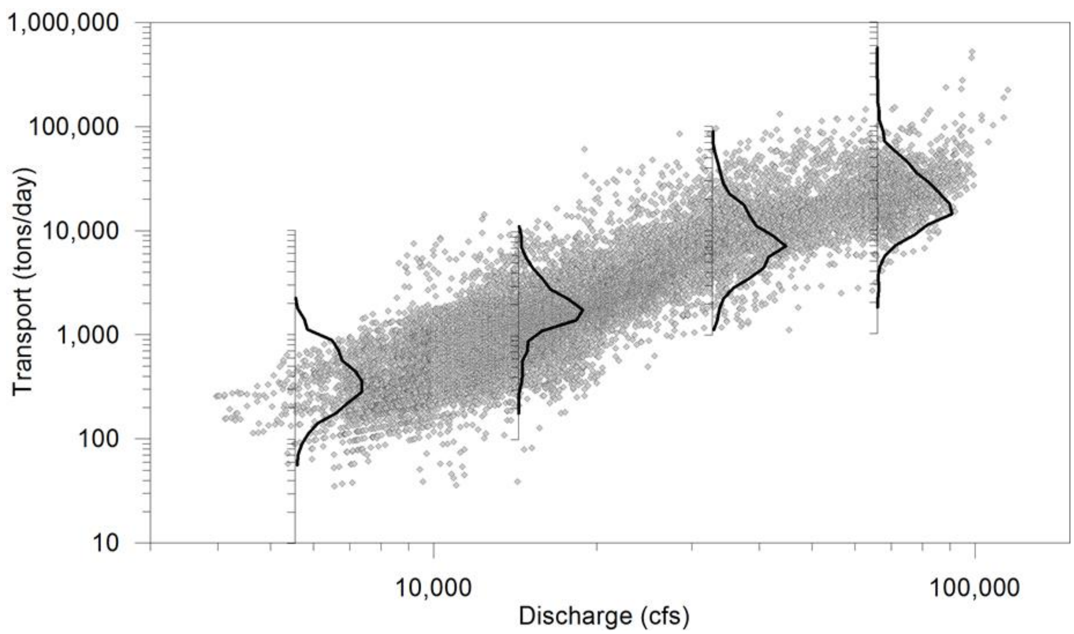


Figure 10-13. Quantifying uncertainty associated with the flow-load hydrograph

(b) In the absence of these data, modelers sometimes use an equilibrium load approach to generate a sediment boundary condition. This approach allows the sediment transport model to compute the capacity in equilibrium with the upstream cross section(s). This approach is very sensitive to the upstream channel geometry and bed gradation and generally introduces more uncertainty than the flow-load curve.

(5) Load Gradation Relationship.

(a) In multi-grain-class sediment modeling, the approach used in all modern numerical sediment models (including HEC-RAS, AdH, and CMS) in defining a load for each boundary flow is not sufficient. The modeler must also define the gradation of each inflowing load.

(b) The relationship between flow and grain size can be direct or indirect (Gibson and Cai 2016), linear or nonlinear, and even monotonic or non-monotonic (gradation fines or coarsens with increasing flow to a certain threshold and then the relationship inverts at higher flows), depending on factors such as whether competence or capacity drives the relationship and the influence of a basin sediment supply limitation (see paragraph 9-3e).

(c) The uncertainty associated with this parameter is usually amplified by data scarcity. With the exception of a few large rivers (such as Mississippi and Missouri) or particularly intensive sampling efforts (such as the Trinity River Restoration Program), most rivers have few or no suspended-sediment gradation samples. Bedload gradations are even rarer. If field measurements exist, they are usually collected at low flows requiring extrapolation to the larger flows that will drive the system, and the data are usually noisy. Of all the parameters required for sediment modeling, the load gradation relationship is the one that generally requires the most intuition and assumption on the part of the modeler. It is often a calibration parameter.

(6) Critical Shear Stress.

(a) USACE modeling approaches have not, historically, considered critical shear a model parameter. In HEC 6 and legacy multidimensional models critical shear was built into the transport functions, making it part of algorithmic uncertainty. While modelers could adjust by revisiting the flow-load relationship, they could change capacity only by selecting a different transport function (once the hydraulic parameters were fixed in the hydraulic calibration).

(b) More recent modeling outside USACE recognizes the critical shear as not only an uncertain system parameter with temporal, spatial, and process variability, but a parameter worth adjusting in response to this uncertainty (Toro-Escobar et al., 1996; Seal et al., 1995). Ruark et al. (2011) found the critical Shield's parameter to be the most sensitive parameter, accounting for nearly 90% of the first order sensitivity in an erosional model and ~25% of model uncertainty in a depositional model. In an expansive meta-analysis of 613 τ_c* computations, Buffington and Montgomery (1997) found ranges of 0.052 to 0.086 for high critical boundary Reynolds numbers and 0.030 to 0.073 for low relative roughness (typical of gravel-bed rivers), concluding that: the apparent lack of a universal τ_c* for gravel bedded rivers warrants care in choosing defendable τ_c* for particular applications (Buffington and Montgomery 1997).

(c) While adjusting the critical shear to reflect site-specific conditions can decrease parametric uncertainty, it can increase algorithmic uncertainty, departing from the generalizations of the transport function. Therefore, while contemporary models like HEC-RAS, expose τ_c* and τ_{rm}* as model parameters, it should be noted that the simulation cannot strictly claim to be using the transport function selected if critical shear is altered. Additionally, like any parameter, τ_c* should only be edited within an established range and with physical justification from system observation (the modeler must offer a process that would increase or decrease the critical shear of the modeled system over default parameters).

(7) Bed Gradations. Model results are usually very sensitive to bed gradations. Bed gradation data are much easier to collect than suspended data because they are not flow-dependent. They can be collected at low flow and do not require coincidental event timing. However, bed material samples can vary dramatically in space (Figure 10-14) and time, and are often vulnerable to sampling bias, conflating surface, and subsurface gradations.

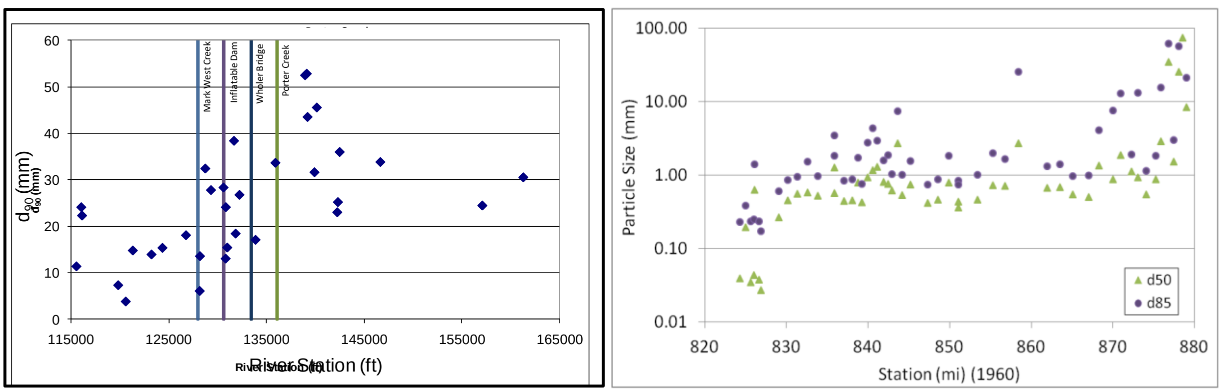


Figure 10-14. Scatter in the d_{85} of bed samples on a gravel-bed (Russian) and sand-bed (Missouri) River

<u>10-8.</u> <u>Double Counting Sediment Uncertainty</u>. Several sediment processes and parameters are connected or correlated. When evaluating sediment uncertainty, it is important to isolate uncertainty that is unique to sediment processes, and avoid double-counting uncertainties. In particular, any uncertainty quantified during the development of the hydraulic analysis of the flow-stage curve should not be added to the sediment uncertainty. Additionally, a sediment model can be calibrated by moving different parameters in opposite direction. A modeler can increase deposition by increasing the uncertain inflow load or decreasing the uncertain critical shear stress. However, these parameters are coupled. If they are calibrated, simulations that combine the maximum load and minimum critical shear are unlikely.

<u>10-9.</u> <u>Mitigating Model Uncertainty and Residual Uncertainties</u>.

a. Model Calibration. The most common approach to mitigate model uncertainty involves calibrating the model to multiple time series that represent the likely future. Calibration is covered in detail in paragraph 9-4. However, it is worth reiterating that the large and various uncertainties in a sediment model make calibration mandatory. An uncalibrated sediment model can offer some guidance on the sensitivity and variability of sediment processes, but a calibrated model is required to reduce uncertainty appreciably (Thomas and Cheng 2007). Otherwise, the noise of parametric uncertainty will overwhelm the signal of sediment impacts on flood risk.

b. Model Uncertainty. However, a calibrated model still has inherent uncertainty from at least three epistemic sources (in addition to the uncertainty from future flows).

c. Residuals. Calibrations are never perfect. Additionally, there is a modeling paradox that "the better a calibration gets, the worse it looks" (see paragraph 9-4). Modeling multiple observations requires parameterization that decreases the fit and increases the residuals for each individual time series, but increases the robustness and decreases the uncertainty of the model. The divergence of the calibration results from the observations (the residuals) represents uncertainty in a calibrated model.

d. Extrapolation. Sediment models are calibrated to particular time series. The particular flow and sediment conditions of the calibration period may not represent future due to natural variability or non-stationarity. However, because sediment equations are nonlinear, when they extrapolate outside the calibrated condition, the errors tend to grow. Model results have more uncertainty as a result of this extrapolation.

e. Equifinality. Whenever multiple parameters are adjusted to match historic data, the calibration is a non-unique solution. Multiple parameter combinations could reproduce historic behavior. This equifinality condition (see paragraph 9-4c(6)) introduces residual uncertainty into the calibration.

10-10. Sediment Risk and Uncertainty Approaches for Non-Flood Risk Management Studies. Sedimentation studies are not limited to FRM applications. Sediment analysis and modeling support a wide variety of USACE studies, analyses, or designs. Sediment analysis informs ecosystem restoration studies, navigation dredging analyses, channel mining assessments, and studies supporting many other USACE missions.

a. This chapter focuses on integrating sediment analysis into USACE's risk and uncertainty procedure for FRM (EM 1110-2-1619). However, the typical USACE FRM study focuses on uncertainty in the stage-damage function scenario, where the highest stages cause the most damage.

b. While the risk and uncertainty analysis process outlined in this chapter does not specifically apply to other analysis or design scenarios (for example, the impact of future degradation impacts on infrastructure like water intake, where damages are associated with low flow), the general principles and concepts in this chapter are recommended in USACE planning study.

10-11. Combining Stage Uncertainty.

a. Integrating Uncertainties. The previous sections detailed many sources of uncertainty in sedimentation studies. Project engineers can quantify each of these sources of uncertainty individually with sensitivity tests. However, EM 1110-2-1619 and flood damage reduction analysis software (HEC-FDA) require one distribution around the flow-stage curve that incorporates all of the uncertainty encountered in a typical USACE study from all factors that affect the water surface profile.

(1) The modeler must quantify sediment uncertainty and then combine this value with the uncertainties from other sources of stage-discharge uncertainty, including the hydraulic model, underlying terrain data, unsteady flow effects, seasonal roughness variations, and similar elements. Refer to EM 1110-2-1619 for a thorough discussion of these factors. However, duplication of processes and double-counting of uncertainty factors should be avoided.

(2) Many of the hydraulic uncertainty factors overlap with sedimentation processes. The project team should identify the factors that are independent without undue simplification that may ignore critical processes. Integrating sediment uncertainty with other uncertainties associated with the discharge-stage relationship is the primary technical challenge to integrating sediment analysis into the risk and uncertainty framework.

b. There are four main methods for combining these uncertainties:

(1) Stochastic Sampling. The most robust way to simultaneously integrate a range of parametric uncertainties is to define model parameters as probabilities rather than fixed functions, and then run a large number of simulations that randomly sample the probability distribution functions of each of the uncertain parameters. This is often called “Monte Carlo” or “Stochastic” analysis.

(a) This approach is attractive because it automatically integrates a range of model uncertainties. USACE models are actively developing stochastic capabilities and it is likely that these methods will eventually represent the state of the art (Figure 10-11).

(b) Current software limitations and run times make stochastic quantification of uncertainty impractical or impossible for most applications. Manual stochastic modeling (manually adjusting parameters for each run) can be useful to examine the effects of one to three uncertain parameters. However, the number of runs required to evaluate three or more uncertain parameters makes manual Monte Carlo impractical, and this approach will have to wait for tools to become available before it is widespread.

(2) Coincident Frequency. Coincident frequency (Faber and Gibson 2003) integrates the probabilities of two independent events. Sediment uncertainty could also be handled this way, except, instead of a deposition-duration curve of historic conditions, the deposition-duration could represent different conditions of an uncertain future sediment driver, with specified probabilities for each parameter. Coincident frequency analysis would build the uncertainty

directly into the flow-stage curve rather than the uncertainty bounds, and can only be applied for a single uncertain sediment parameter that can be easily represented in a hydraulic model. This approach is not widely applied.

(3) Arithmetic Combination.

(a) In lieu of numerically integrating uncertainties with a stochastic or Monte Carlo approach, the flow-stage uncertainty for each source of uncertainty identified can be computed independently, then combined using the following equation from EM 1110-2-1619:

$$SD_{Total} = \sqrt{SD_{X1}^2 + SD_{X2}^2 + \cdots + SD_{Xn}^2} \qquad \text{Equation 10-1}$$

where SD_{Total} is the standard deviation of the error around the flow-stage curve and each SD_X is the independent standard deviation of the error around flow-load curve for each uncertain parameter evaluated (X1, X2, … Xn).

(b) In a FRM study where sediment aggradation reduces conveyance, this equation is:

$$SD_{Total} = \sqrt{SD_n^2 + SD_{Q(t)}^2 + SD_{Q-L}^2} \qquad \text{Equation 10-2}$$

where:

SD_{Total} = the standard deviation of the stage error
SD_n = the standard deviation of the stage error associated with uncertainty in roughness parameter
$SD_{Q(t)}$ = the standard deviation of the stage error associated with uncertainty in hydrologic time series
SD_{Q-L} = the standard deviation of the stage error associated with uncertainty in sediment flow-load curve

(c) Each SD_x can be computed using the following steps:

- Run the calibrated sediment model for the project life with the best estimate of all sediment parameters. If the event is non-stationary (for example, sea level rise) run the simulation with the best estimate at each time.

- Run a static-bed hydraulic analysis for the flows of interest (for example, 50%, 10%, 4%, 2%, 1%, 0.05%, 0.02%) with the final bathymetry of the mobile bed simulation.

- Repeat the process (steps a and b) with alternate parameter estimates, higher and lower than the best estimate. Generally, testing a maximum of three to four critical sedimentation parameters will be adequate to define sediment uncertainty. Only those processes that are independent will have separate uncertainty factors. Expert opinion is an acceptable process to estimate the range of critical sediment parameters to be evaluated with the model.

- For each separate factor and flow, SD_X can be estimated from the difference in the stage between the first (best parameter estimate) run and the others.

(d) This approach can either combine all sediment uncertainty into a single $SD_{sediment}$, or evaluate the uncertainty associated with different parameters separately (such as computing separate $SD_{Q\text{-}Load}$, $SD_{\tau c*}$ and SD for the uncertainty in separate parameters). If all uncertain parameters are modified in a single run, or the results from multiple simulations are combined into a single $SD_{sediment}$, the modeler will have to use the results to estimate an overall standard deviation qualitatively. However, including the uncertainty from each model parameter separately allows the modeler to estimate standard deviations directly, but accounting for the total uncertainty of each parameter can overestimate the total uncertainty in a calibrated model.

c. Sensitivity Analysis. Finally, if there are multiple, interacting, sensitive parameters, simply adding the standard deviations or running stochastic simulations that sample all uncertain parameters might be conservative, particularly for a well-calibrated model. However, adequately defining all variables and determining a reasonable sensitivity parameter range can be challenging. In these cases, sensitivity analysis can be conducted for each uncertain parameter individually (Greimann and Klump 2000), then integrated qualitatively by expert opinion, and finally estimated using a $\pm SD_{Total}$ on the flow-stage curve from these analyses.

d. Event Sequence and Mid-Project Life Extremes.

(1) The sequence of sediment events affects the total project performance, which contributes to aleatory uncertainty (Gibson et al., 2019). For example, a 30% exceedance flood and a 2% exceedance flood in successive years may scour more total sediment if the 30% flood occurs first than if the sequence was inverted. Additionally, flow sequence of events can also affect project benefits. A large event in the first year of a project life could reduce total project benefits more than if the event happened in the 49th year, which introduces uncertainty into project performance.

(2) A simplified sediment model results illustrate this effect. A model was developed with a 50-year period of record flow data set. The sediment model was run with the best parameter estimates and then, using the process presented in the previous section, with roughness values 20% higher and lower than the best estimate.

(3) Figure 10-15 shows a plot of water surface elevations at index station computed by the best estimate, high, and low model runs. The sediment model predicts both aggradation and degradation, depending on the flow year. The historical flow order predicts maximum aggradation near the end of the 50-year simulation. However, in Figure 10-15, the best estimate parameter simulation was rerun with two different flow orders, one that front-loaded the aggradational years and back-loaded the degradational years, and another that inverted that order.

(4) Simulations illustrate that flow order affects the final, 50-year result. However, more significantly, this analysis bounds the potential variability in project benefits based on aleatory uncertainty. The re-sequenced models compute much larger maximum deposition and erosion conditions.

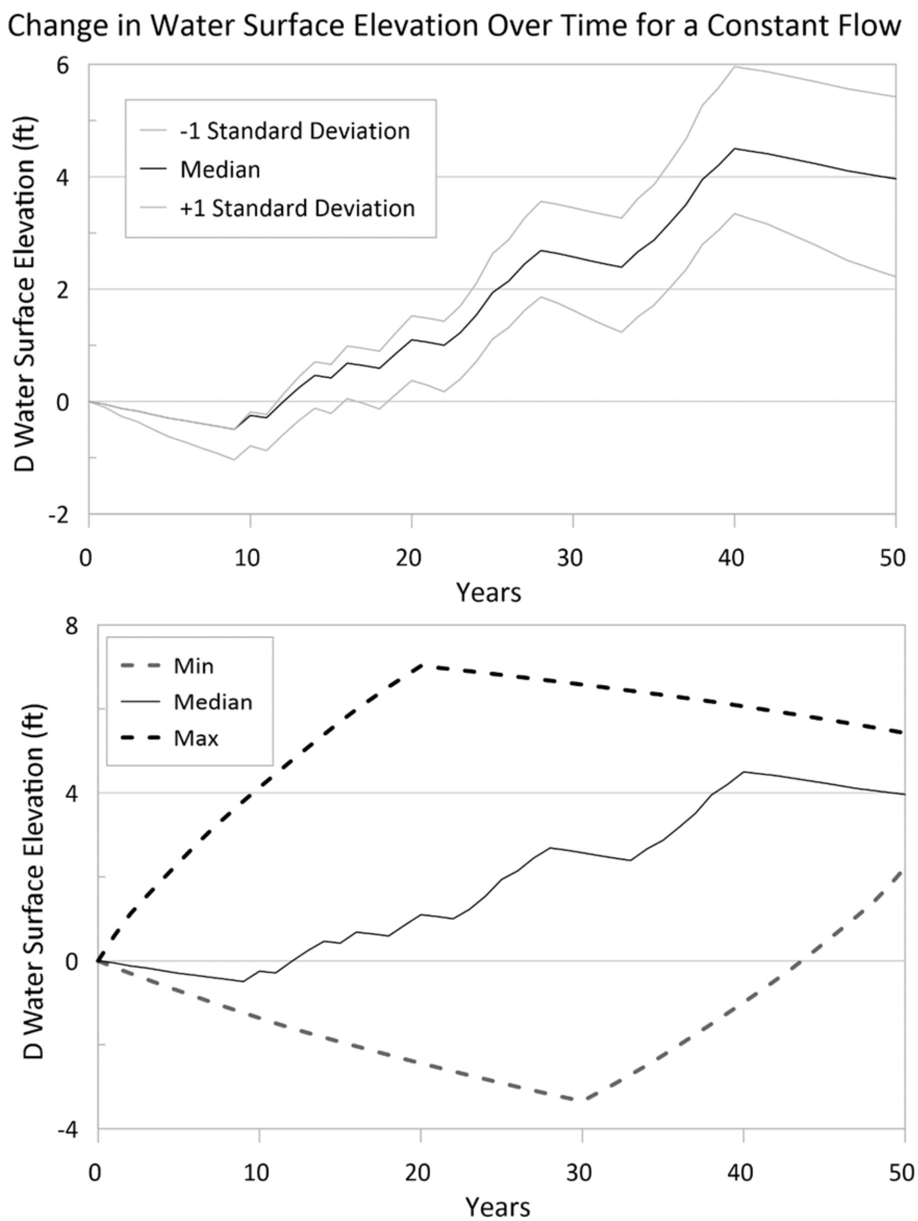


Figure 10-15. (top) Variation model water surface elevation (for the same flow) in a sediment model, due to sediment deposition and erosion; (bottom) stage variability over time, based on a sediment model that erodes and deposits, with the historic flow record and results from the same model with flows reordered to front-load the deposition and the erosion to bound the maximum and minimum expected impacts

(5) In Figure 10-15, the maximum deviation from the median condition occurs at year 50 and is approximately 1.8 feet. This value could be used with the equation presented in the arithmetic combination section to define uncertainty. Figure 10-15 illustrates complexity that arises when quantifying sediment uncertainty. Maximum uncertainty and project design elevations may not always occur simultaneously. The maximum aggradation of 7.0 feet that occurs when flows are reordered greatly exceeds the median maximum of 4.5 feet. In either case, the maximum does not occur at year 50. If numerical modeling determined results similar to Figure 10-15, additional stochastic analysis would be required to fully investigate.

10-12. Risk Registry.

a. Planning Process. The USACE planning process, currently referred to as SMART planning, requires planners to develop a detailed risk registry, a prioritized summary of project risks. The PDT must either reduce these risks by analyzing them or accept them with narrative justification. Shared Vision planning includes a risk audit approach for recognizing and qualitatively addressing low-probability, high-consequence sources of risk. These events are too unlikely or too difficult to quantify, but introduce non-zero risk, often of high consequence, and should be explicitly considered and documented. In both cases, project formulation includes qualitative analysis of low-probability, high-consequence events. Examples of these are included in this section.

b. Defining Project Risk. The risk registry is a means to define project risk and carefully consider project plan formulation and the need for detailed studies. However, it is not intended to replace necessary analysis with stated risk. Deferring critical sedimentation studies to future design phases, via the risk register, is not acceptable for USACE project evaluation. Projects with challenging sedimentation processes must be evaluated early in the study process. Accurately capturing project performance risk places paramount importance on having skilled river and sedimentation engineering team members with significant experience and understanding of sedimentation processes involved in early studies.

c. Complex Sedimentation Studies. Sediment processes will likely significantly alter project features and operation and maintenance costs. Inadequate sediment study analysis can result in catastrophic project performance. Refer to paragraph 2-2c for a detailed discussion on staged sedimentation studies that are intended to be performed within the USACE planning process framework. All of these issues should be carefully considered when preparing the risk register. The following sections highlight risk issues that, when identified in the risk register, will most likely lead to the need for conducting complex sedimentation studies that are initiated early in the USACE planning process.

d. Wildfire in the Watershed. A wildfire in the contributing watershed, particularly a mountainous watershed, can increase the sediment flow-load curve dramatically (Ryan et al., 2011; Case Study 6D in Appendix N).

e. Dam Construction, Reoperation, Failure, or Removal. Dam impacts on river sediment budgets are substantial and well documented (see Chapter 8). While sediment impacts on reservoir volume (increasing flood frequency downstream) or the delta development (increasing the flow-stage relationship upstream) can be modeled and quantified, operational uncertainties are more difficult.

(1) USACE (Gibson and Boyd 2014, 2016) and other federal agencies (Randall USBR) are considering sediment management strategies like sluicing and flushing (Morris and Fan 1997; Morris et al., 2007) to maintain reservoir capacity and deliver ecologically important sediment downstream.

(2) While sediment impacts of these options can be quantified, their probability is more difficult to assess. Final decisions about re-operating dams to pass sediment, or removing them entirely, are non-technical decisions involving political dynamics and social values, both of which are non-stationary and difficult to predict.

f. Mass Wasting Event. Large mass wasting events (Sutherland et al., 2002) are, themselves, hazards to life and property. However, they affect flood risk upstream and downstream. Mass wasting events increase the stage control of the upstream reach, temporarily increasing the flow-stage relationship. They also increase sediment load downstream, which can induce deposition and increase the stage flow relationship, and decrease the level of protection of USACE FRM projects.

g. Volcanic Activity. Mount St. Helens is the most famous and recent major eruption in the United States. The sediment impacts are well documented. The 1984 eruption mobilized 4 billion tons of material, increasing the Toutle and Cowlitz River loads dramatically, depositing between USACE levees, reducing their level of protection. USGS work (Myers et al., 2010) on rivers downstream from Mount Pinatubo in the Philippines has demonstrated the generality of these results. Sediment-related risk and uncertainties of USACE FRM alternatives on systems like the Puyallup River downstream of Mount Rainer include the difficult to quantify, but non-zero possibility of a Mount Rainer volcanic event.

h. Morphological Feedbacks between Sediment and Ice.

(1) Like sediment, ice and debris both have significant, complicated effects on the discharge-stage curve (Figure 10-2 (g)) that are difficult to predict or quantify. Sometimes sediment and ice processes influence each other, creating feedbacks that amplify the risks. On the Muskegon River, ice and sediment combine to introduce cumulative flood risk (Healy et al., 2003; Gibson 2016). Sediment deposition in a reservoir delta, exacerbated by 1 million yd^3 tons of sediment released in a 1966 dam removal, has increased the frequency and severity of ice jam flooding.

(2) Alternately, ice can create pressure flow situations that increase transport and channel capacity (Tuthill and White 2005). These complicated ice-sediment feedbacks are beyond current modeling capabilities, but are uncertainties that affect actual flood risk.

i. Listed Species Habitat. Sediment deposited in FRM channels can recruit a riparian corridor if maintenance lapses (Wong and Kondolph 2015). Emergent riparian vegetation can recruit quickly. In at least one case, this novel habitat recruited listed species, precluding future maintenance of the FRM project.

j. Policy and Operation Change. A shift in operation policy may lead to a significant change the flow regime at existing projects. Operational changes tend to occur more frequently at the end of the project's intended life and in response to changes in social values regarding water supply, flood risk, and ecosystem functionality. Project design should consider the components of the future expected sediment regime that are susceptible to policy or operation change and the possible project ramifications. For instance, an operational decision to stop or reduce normal operation and maintenance actions on a sediment detention structure could significantly alter the sediment contributed to the project.

k. Project Performance in Extreme Events. The concepts introduced in this chapter have repeatedly demonstrated how constructed projects often have broad morphologic impacts. Discussion has also repeatedly referred to the nonlinearity observed with sediment processes. Constructed projects may perform as expected for a wide range of events, while sediment processes during an extreme event in an altered system may deviate significantly from that previously experienced. Extreme flood events can cause extreme erosion or deposition (often both) within the river channel or floodplain. Examples of how river sediment processes can be significantly altered in extreme events are presented in Case Study 10A (Appendix N).

Appendix A
References

A-1. Required Publications.

Public Law

1. Public Law 93-251. USACE 1981b. Final report to Congress. "The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32." Main report, supplemented by appendices A-H in separate volumes. https://usace.contentdm.oclc.org/digital/collection/p16021coll5/id/1239/

USACE Publications

1. ECB 2014-10. USACE 2014a. "Subject: Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects." https://www.iwr.usace.army.mil/Portals/70/docs/Climate%20Change/ecb_2014_10.pdf

2. EM 1110-1-1000. USACE 2015d. "Photogrammetric and LiDAR Mapping." https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/u43544q/454D20313131302D312D31303030/

3. EM 1110-1-1002. USACE 2012f. "Survey Markers and Monumentation." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-1-1002.pdf?ver=tb31W4-X5y3Xh-ToIBELEQ%3d%3d

4. EM 1110-1-1003. USACE 2011c. "NAVSTAR Global Positioning System Surveying." https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/u43544q/454D20313131302D312D31303033/

5. EM 1110-1-1004. USACE 2002a. "Geodetic and Control Surveying." https://synectics.net/public/library/StreamResource.axd?DSN=pub&Mode=FileImage_Inline&ID=444

6. EM 1110-1-1005. USACE 2007. "Control and Topographic Surveying." https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-1-1005.pdf

7. EM 1110-2-1003. USACE 2013a. "Hydrographic Surveying." https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/u43544q/454D20313131302D322D31303033/

8. EM 1110-2-1100. USACE 2008a. "Coastal Engineering Manual." Change 2. https://www.publications.usace.army.mil/USACE-Publications/Engineer-Manuals/u43544q/636F617374616C20656E67696E656572696E67206D616E75616C/

9. EM 1110-2-1415. USACE 1993. "Hydrologic Frequency Analysis." https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-1415.pdf

10. EM 1110-2-1601. USACE 1994b. "Hydraulic Design of Flood Control Channels." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/em_1110-2-1601.pdf

11. EM 1110-2-1602. USACE 1980. "Hydraulic Design of Reservoir Outlet Works." https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-1602.pdf

12. EM 1110-2-1612. USACE 2006a. "Ice Engineering." Change 3. https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1612.pdf

13. EM 1110-2-1614. USACE 1995b. "Design of Coastal Revetments, Seawalls, and Bulkheads." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1614.pdf

14. EM 1110-2-1619. USACE 1996. "Risk Assessment for Flood Risk Management Studies." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1619.pdf?ver=U1N4ZStFXOwkxnDPvvXjzg%3d%3d

15. EM 1110-2-5025. USACE 2015b. "Dredging and Dredged Material Management." https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-5025.pdf

16. ER 1105-2-100. USACE 2000b. "Planning Guidance Notebook." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/er_1105-2-100.pdf

17. ER 1105-2-101. USACE 2017c. "Risk Assessment for Flood Risk Management Studies." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/er_1105-2-101.pdf

18. ER 1110-2-8153. USACE 1995e. "Sedimentation Investigations." https://apps.dtic.mil/sti/pdfs/ADA404084.pdf

19. Hydrologic Engineering Center (HEC). 1993a. "Assessment of Structural Flood-Control Measures on Alluvial Fans." PR-24. HEC. Davis, CA. https://www.hec.usace.army.mil/publications/ProjectReports/PR-24.pdf

20. HEC. 2016a. “Hydrologic Modeling System HEC-HMS User’s Manual, Version 5.0.” USACE. Institute for Water Resources. HEC. Davis, CA. Refer to current online documentation version: https://www.hec.usace.army.mil/software/hec-hms/documentation.aspx

21. HEC. 2016b. “HEC-RAS River Analysis System Version 5.0–Hydraulic Reference Manual.” USACE. Institute for Water Resources. HEC. Davis, CA. Refer to current online documentation version: https://www.hec.usace.army.mil/confluence/rasdocs

22. HEC. 2016c. “HEC-RAS River Analysis System Version 5.0–User’s Manual.” USACE. Institute for Water Resources. HEC. Davis, CA. Refer to current online documentation version: https://www.hec.usace.army.mil/confluence/rasdocs

23. USACE 2002b. “Regional Sediment Management: Background and Overview of Initial Implementation.” Institute for Water Resources Report 02-PS-2. https://www.iwr.usace.army.mil/Portals/70/docs/iwrreports/02ps2sed_man.pdf

24. USACE 2004a. “Advanced Streambank Protection Workshop Lecture Notes.” Engineer Research and Development Center (ERDC). Vicksburg, MS. (Online version not available.)

25. USACE 2012a. “Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations.” USACE Omaha District. Omaha, NE. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/5097

26. USACE 2014a. “Dallas Floodway Feasibility Study.” Trinity River Basin. Dallas, TX. Fort Worth District. https://www.swf.usace.army.mil/Portals/47/docs/Environmental/DF/Dallas_Floodway_Final_FR_MainReport.pdf

27. USACE 2013b. “Cochiti Baseline Study, A Data Compendium.” USACE Albuquerque District. https://www.spa.usace.army.mil/Portals/16/docs/civilworks/Cochiti%20Baseline%20Summary.pdf

28. USACE 2015a. “Adaptive Hydraulics (AdH) Version 4.5 Hydrodynamics User Manual; Sediment Transport User Manual.” ERDC Coastal and Hydraulics Laboratory (CHL). Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/39081

29. USACE 2015e. USACE Dam Safety Program website. Accessed January 2016. Periodically updated online version. http://www.usace.army.mil/Missions/CivilWorks/DamSafetyProgram.aspx

30. USACE 2017a. “Adaptive Hydraulics (AdH) Version 4.6 Documentation.” ERDC/CHL-VIII-11. Vicksburg, MS. https://hdl.handle.net/11681/39080

31. USACE 2017b. “Missouri River Bed Degradation Feasibility Study Technical Report.” USACE. Kansas City District. Kansas City, MO. https://usace.contentdm.oclc.org/digital/collection/p16021coll7/id/9323/

32. USACE 2017d. “Missouri River Stage Trends Technical Report.” Missouri River Basin Water Management Division. Omaha, Nebraska. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/5716/

Other Federal Agency and Private Sector Publications

1. American Society of Civil Engineers (ASCE). 2006. “Sedimentation Engineering, Manuals and Reports on Engineering Practice.” No. 54. Prepared by the ASCE Task Committee for the Preparation of the Manual on Sedimentation of the Sedimentation Committee of the Hydraulics Division. ASCE. Reston, VA. 2nd Edition. Online version available dated 2006; original work published 1975. https://doi.org/10.1061/9780784408230

2. ASCE. 2008. “Sedimentation Engineering, Processes, Measurements, Modeling, and Practice.” Manual No. 110. Prepared by the ASCE Task Committee to Expand and Update Manual 54 of the Sedimentation Committee of the Environmental and Water Resources Institute. Marcelo H. Garcia (Ed.). ASCE. Reston, VA. https://doi.org/10.1061/9780784408148

3. ASCE. 2011. “Sediment Dynamics upon Dam Removal.” Prepared by the Task Committee on Sediment Dynamics Post-Dam Removal of the Environmental and Water Resources Institute of ASCE. T. Papanicolaou and B. Barkdoll (Eds.). ASCE. p 175. https://ascelibrary.org/doi/book/10.1061/9780784411360

4. ASCE. 2013. “Inland Navigation Channel Training Works.” Manual No. 124. Prepared by the Task Committee on Inland Navigation of the Waterways Committee of the Coasts, Oceans, Ports, and Rivers. Thomas J. Pokrefke (Eds.). ASCE. Reston, VA. https://doi.org/10.1061/9780784412534

5. Federal Emergency Management Agency (FEMA). 1989. “Alluvial Fans, Hazards and Management.” Federal Insurance Administration. https://pw.lacounty.gov/wmd/NFIP/documents/FEMA165.pdf

6. FEMA. 2012. “Engineering Principles and Practices for Retrofitting Flood-Prone Residential Structures.” https://www.fema.gov/sites/default/files/2020-08/fema259_complete_rev.pdf

7. Federal Highway Administration (FHWA) 2012. “Evaluating Scour at Bridges.” Publication No. FHWA HIF-12-003. Hydraulic Engineering Circular No. 18. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf

8. Federal Interagency Sedimentation Project (FISP). 2013. “FISP Approval of the Use of Field-Deployed LISST-series Laser-Diffraction Analyzers for Volumetric Concentration and Particle-Size Data Memorandum (Memorandum 2013.02).” https://water.usgs.gov/fisp/docs/FISP_Tech_Memo_2013.02.pdf

9. Federal Interagency Stream Restoration Working Group (FISRWG). 1998. “Stream corridor restoration: Principles, processes, and practices.” NRCS. https://directives.sc.egov.usda.gov/34804.wba

10. National Research Council (NRC). 2004. “Adaptive Management for Water Resources Planning.” The National Academies Press. https://doi.org/10.17226/10972

11. Natural Resources Conservation Service (NRCS). 2012. “Engineering Classification of Earth Materials.” National Engineering Handbook, Part 631. U.S. Department of Agriculture. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=31847.wba

12. NRCS. 2007. “Stream Restoration Design.” National Engineering Handbook, Part 654. U.S. Department of Agriculture. https://directives.sc.egov.usda.gov/viewerFS.aspx?hid=21433

13. Simons, Li & Associates (SLA). 1982. “Engineering Analysis of Fluvial Systems.” Simons, Li & Associates. Fort Collins, CO. https://www.worldcat.org/title/engineering-analysis-of-fluvial-systems/oclc/10274509

14. U.S. Bureau of Reclamation (USBR). 2006a. “Erosion and Sedimentation Manual.” U.S. Department of the Interior, Bureau of Reclamation. Technical Service Center. Denver, CO. https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/Erosion%20and%20Sedimentation%20Manual.pdf

15. USBR. 2017. “Dam Removal Analysis Guidelines for Sediment, Advisory Committee on Water Information Subcommittee on Sedimentation.” Prepared by T.J. Randle and J. Boundtry. U.S. Bureau of Reclamation Technical Service Center. https://acwi.gov/sos/pubs/dam_removal_analysis_guidelines_for_sos_final_vote_2017_12_22_508.pdf

16. U.S. Department of Agriculture (USDA). 1966. “Procedure for Determining Rates of Land Damage, Land Depreciation, and Volume of Sediment Produced by Gully Erosion.” Technical Release 32. Soil Conservation Service. https://archive.org/details/CAT31350611

17. USDA. 1977. “Design of Open Channels.” Technical Release No. 25. Soil Conservation Service. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=18607.wba

18. USDA. 1983. "National Engineering Handbook." 2nd Edition. (Section 3: Sedimentation; Chapter 8: Sediment storage design criteria). U.S. Department of Agriculture. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17514.wba

19. USDA. 2007a. "Guidelines for Sampling Bed Material." Technical Supplement 13A to the NRCS National Engineering Handbook, Part 654. Stream Restoration Design. National Resources Conservation Service. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17835.wba

20. U.S. Fish and Wildlife Service (USFWS). 1980. "Habitat Evaluation Procedures (HEP) Ecological Services Manual 102." USFWS. Washington D.C. Habitat Evaluation Procedures (HEP). U.S. Fish and Wildlife Service. Division of Ecological Services. Google Books.

21. USGS. 2008. "Water-Data Report, 2008." 12499010 Naches River at I-82 Hwy bridge near Yakima, WA. P 2. https://waterdata.usgs.gov/nwis/inventory/?site_no=12499010

22. USGS. 2012. "Joint USGS-CUAHSI Workshop on Sediment Hydroacoustic Techniques for Rivers and Streams." March 20–22, 2012. Consortium of Universities for the Advancement of Hydrologic Science, Inc (CUAHSI). Shepherdstown, WV. https://doi.org/10.1029/2012EO260007

State and Local

1. Kansas Water Authority. 2010. "Reservoir Roadmap, Presented to the 2010 Kansas Legislature by the Kansas Water Authority, Volume I-III." Topeka, KS. https://kwo.ks.gov/docs/default-source/reservoirs/rpt_volume-i-iii_030810.pdf?sfvrsn=d2d38714_0

2. Los Angeles County. 1979. "Design Manual, Debris Dams and Basins." Los Angeles County Flood Control District. https://dpw.lacounty.gov/ldd/lib/fp/Storm%20Drain/LA%20County%20Flood%20Control%20Debris%20Dams%20and%20Basins%20Design%20Manual.pdf

3. Los Angeles County. 2006. "Sedimentation Manual." Los Angeles County Department of Public Works. 2nd Edition. https://dpw.lacounty.gov/wrd/publication/engineering/2006_sedimentation_manual/Sedimentation%20Manual-Second%20Edition.pdf

4. Maricopa County Flood Control District. 2018. "Drainage Design Manual for Maricopa County, Volume II, Hydraulics." Maricopa County, Arizona. https://apps.fcd.maricopa.gov/pub/docs/scanfcdlibrary/1006_004DrainageDesignManualforMaricopaCounty_Vol_II_Hydraulics.pdf

Research Community

1. Ables Jr., J.H., and M.B. Boyd. 1969. “Low-water weirs on Boeuf and Tensas Rivers, Bayou Macon, and Big and Colewa Creeks, Arkansas and Louisiana: Hydraulic Model Investigation.” H-69-13. USACE Waterways Experiment Station (WES). Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/bitstream/11681/13485/1/TR-H-69-13.pdf

2. Abraham, D., R. Kuhnle, and A.J. Odgaard. 2011. “Validation of Bed Load Transport Measurements with Time-Sequenced Bathymetric Data.” Journal of Hydraulic Engineering. ASCE. Vol. 137(7). pp 723–728. https://ascelibrary.org/doi/pdf/10.1061/%28ASCE%29HY.1943-7900.0000357

3. Abraham, D., N. Clifton, B. Vessels 2015. “Numerical Sedimentation Study of Shoaling on the Ohio River near Mound City, Illinois.” ERDC/CHL TR- 15-12. USACE ERDC. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/3911/

4. Ackers, P. 1972. “Sediment Transport in Channels, An Alternative Approach.” HRS Int. Report 102. Hydraulics Research Station. Wallingford, England. https://eprints.hrwallingford.com/25/4/INT102-SedimentTransportInChannels-HRS-1972.pdf

5. Ackers, P., and W.R. White 1973. “Sediment Transport: New Approach and Analysis.” ASCE Journal of the Hydraulics Division. Vol. 99(11). pp 2041–2060. https://ascelibrary.org/doi/10.1061/JYCEAJ.0003791

6. Allison M.A., B.M. Vosburg, M.T. Ramirez , and E.A. Meselhe. 2013. “Mississippi River channel response to the Bonnet Carré Spillway opening in the 2011 flood and its implications for the design and operation of river diversions.” Journal of Hydrology. Vol. 477. pp 104–118. http://dx.doi.org/10.1016/j.jhydrol.2012.11.011

7. Anderson, M.G., D.E. Walling, and P.D. Bates. 1996. “Floodplain Processes.” John Wiley & Sons. New York. https://www.wiley.com/en-us/Floodplain+Processes-p-9780471966791

8. Andrews, E.D. 1983. “Entrainment of Gravel from Naturally Sorted Riverbed Material.” Geological Society of America Bulletin 94(10). pp 1225–1531 https://pubs.geoscienceworld.org/gsa/gsabulletin/article/94/10/1225/202820/Entrainment-of-gravel-from-naturally-sorted

9. Annandale, G.W. 1987. “Reservoir Sedimentation.” Elsevier Science Publishers. Amsterdam, Netherlands. https://www.elsevier.com/books/reservoir-sedimentation/annandale/978-0-444-42729-8

10. Annandale, G.W. 1995. “Erodibility.” Journal Hydraulic Research. IAHR. Vol. 33(4). pp 471–494. https://www.tandfonline.com/doi/abs/10.1080/00221689509498656

11. Annandale, G.W. 2011. “Going full circle.” International Water Power Dam Construction. pp 30–34. http://www.waterpowermagazine.com/features/featuregoing-full-circle/

12. Annandale, G.W. 2013. “Quenching the Thirst, Sustainable Water Supply and Climate Change.” Create Space Independent Publishing Platform. Charleston, SC. https://www.hydropower.org/sediment-management-resources/publication-quenching-the-thirst-sustainable-water-supply-and-climate-change

13. Ariathurai, C.R., and K. Arulanandan. 1978. “Erosion rates of cohesive soils.” Journal of Hydraulics Division. Vol. 104(2). pp 279–282. https://doi.org/10.1061/JYCEAJ.0004937

14. Arnold, J.G., R. Srinivasan, R.S. Muttiah, and J.R. Williams. 1998. “Large-area hydrologic modeling and assessment. Part I: Model development.” Journal of the American Water Resources Association. Vol. 34(1). pp 73–89. https://onlinelibrary.wiley.com/doi/10.1111/j.1752-1688.1998.tb05961.x

15. Ayles, C.P., and M. Church. 2015. “Downstream Channel Gradation in the Regulated Peace River.” Chapter 3 in The Regulation of Peace River: A Case Study for River Management, Michael Church (Ed.). John Wiley & Sons. https://onlinelibrary.wiley.com/doi/book/10.1002/9781118906170

16. Bagnold, R.A. 1966. “An Approach to the Sediment Transport Problem from General Physics.” Geological Survey Professional Paper 422-J. https://pubs.usgs.gov/pp/0422i/report.pdf

17. Barksdale, R.D. 1991. The Aggregate Handbook.” National Stone Association. https://www.worldcat.org/title/aggregate-handbook/oclc/1024169415

18. Beasley, D.B., L.F. Huggins, and E.J. Monke. 1980. “ANSWERS: A model for watershed planning.” Transactions of the American Society of Agricultural Engineers (ASAE). Vol. 23(4). pp 938–944. https://elibrary.asabe.org/abstract.asp?aid=34692

19. Beven, K. 2005. “A manifesto for the equifinality thesis.” Journal of Hydrology. Vol. 320(1-2). pp 18–36. https://www.sciencedirect.com/science/article/pii/S002216940500332X

20. Beven, K.J. 1993. “Prophecy, reality and uncertainty in distributed hydrological modeling, Advances in Water Resources.” Vol. 16. pp 41–51. https://www.sciencedirect.com/science/article/pii/030917089390028E

21. Beverage, J.P., and J.K. Culbertson. 1964. “Hyperconcentrations of suspended sediment.” Journal of the Hydraulics Division. ASCE. Vol. 90. pp 117–128. https://ascelibrary.org/doi/abs/10.1061/JYCEAJ.0001128

22. Bicknell, B.R., J.C. Imhoff, J.L. Kittle Jr., A.S Donigian Jr., and R.C. Johanson. 1993. “Hydrologic Simulation Program – FORTRAN (HSPF): User’s manual for Release 10.” Rep. EPA/600/R-93/174. U.S. EPA Environmental Research Lab. Athens, GA. https://cfpub.epa.gov/si/si_public_record_Report.cfm?Lab=ORD&dirEntryID=32781

23. Biedenharn, D.S., M.A. Allison, C.D. Little, C.R. Thorne, and C.C. Watson. 2017. “Large-scale Geomorphic Change in the Mississippi River from St Louis, MO, to Donaldsonville, LA, as Revealed by Specific Gage Records.” Mississippi River Geomorphology & Potamology (MRG&P) Report No. 10. USACE. Mississippi Valley Division. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/4309/

24. Biedenharn, D.S., C.M. Elliott, and C.C. Watson. 1997. “The WES Stream Investigation and Streambank Stabilization Handbook.” USACE WES. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/4795

25. Biedenharn, D.S., C.C. Watson, and C.R. Thorne. 2008. “Fundamentals of Fluvial Geomorphology.” Chapter 6 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. ASCE. Online edition available dated 2013. Reston, VA. https://doi.org/10.1061/9780784408148.ch06

26. Bingner, R.L., F.D. Theurer, and Y. Yuan. 2018. “AnnAGNPS Technical Processes, Documentation version 5.5 https://www.ars.usda.gov/ARSUserFiles/60600505/AGNPS/AGNPS%20V%205.5/Technical_Documentation.pdf

27. Borah, D., E. Krug, and D. Yoder. 2008. “Watershed Sediment Yield.” Chapter 17 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. ASCE. Online edition available dated 2013. Reston, VA. https://doi.org/10.1061/9780784408148.ch17

28. Borah, D.K., R. Zia, and M. Bera. 2002. “DWSM – A dynamic watershed simulation model.” Chapter 5 in Mathematical Models of Small Watershed Hydrology and Applications. V. Singh and D. Frevert (Eds.). Water Resources Publications. Highlands Ranch, CO. pp 113–166. https://books.google.com/books/about/Mathematical_Models_of_Small_Watershed_H.html?id=Dqm8oc1KsnUC

29. Bouraoui, F., I. Braud, and T.A. Dillaha. 2002. “ANSWERS: A nonpoint-source pollution model for water, sediment, and nutrient losses.” Chapter 22 in Mathematical Models of Small Watershed Hydrology and Applications. V. Singh and D. Frevert (Eds). Water Resources Publications. Highlands Ranch, CO. pp 833–882. https://books.google.com/books/about/Mathematical_Models_of_Small_Watershed_H.html?id=Dqm8oc1KsnUC

30. Brauer, E.J. 2009. "The limitations of using specific-gage analysis to analyze the effect of navigation structures on flood heights in the Middle Mississippi River." Permanent International Association of Navigation Congress (PIANC). 2009 De Paepe-Willems Award Contest, The Netherlands. https://www.mvs-wc.usace.army.mil/arec/Documents/Physical_Effects/Brauer_(2009)_The_limitations_of_using_specific_gage_analysis_to_analyze_the_effect_of_navigation_structures_on_flood_heights_in_the_Middle_Mississippi_River.pdf

31. Briaud, J., C. Ting, H. Chen, Y. Cao, S. Han, and K. Kwak. 2001 "Erosion Functions Apparatus for Scour Rate Predictions." Journal of Geotechnical and Geoenvironmental Engineering. pp 106–113. https://ascelibrary.org/doi/10.1061/%28ASCE%291090-0241%282001%29127%3A2%28105%29

32. Briaud, J-L. 2008. "Case Histories in Soil and Rock Erosion: Woodrow Wilson Bridge, Brazos River Meander, Normandy Cliffs, and New Orleans Levees." Ralph B. Peck Award Lecture (2007). Journal of Geotechnical and Geoenvironmental Engineering. Vol. 134(10). https://ascelibrary.org/doi/10.1061/%28ASCE%291090-0241%282008%29134%3A10%281425%29

33. Brice J.C. 1984. "Planform properties of meandering rivers." In River Meandering. ASCE. pp 1–15. New York, NY. (Online version not available.) **Error! Hyperlink reference not valid.**

34. Bridge, J.S. 2003. "Rivers and Floodplains – Forms, Processes and Sedimentary Record." Blackwell Publishing. Malden, I. https://books.google.com/books?id=_mdQhM84V_EC&printsec=frontcover#v=onepage&q&f=false

35. Brier, G.W. 1950. "Verification of forecasts expressed in terms of probability." Monthly Weather Review. Vol. 78(1). pp 1–3. https://journals.ametsoc.org/view/journals/mwre/78/1/1520-0493_1950_078_0001_vofeit_2_0_co_2.xml

36. Brooks, N.H. 1963. "Calculations of Suspended Load Discharge from Velocity, and Concentration Parameters." Proceedings of the Federal Interagency Sedimentation Conference, 1963. Miscellaneous Publication No. 970. Agricultural Research Service. VSDA. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/10017/ (oclc.org)

37. Brown, C.B. 1950. "Sedimentation." Engineering Hydraulics. Proceedings of the Fourth Hydraulics Conference. H. Rouse (Ed.). Iowa Institute of Hydraulic Research. Iowa City, IA. https://www.worldcat.org/title/engineering-hydraulics-proceedings-of-the-fourth-hydraulics-conference-iowa-institute-of-hydraulic-research-june-12-15-1949/oclc/4104308

38. Brown, G., C. Trahan, J.N. Tate, and G. Savant. 2014. "SEDLIB Multiple Grain Sized Mixed Sediment Library: Technical Manual." USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/bitstream/11681/39162/17/SEDLIB_Manual-Version1.2.pdf

39. Brownlie, W.R. 1981. "Prediction of Flow Depth and Sediment Discharge in Open Channels." Report No. KH-R-43A. California Institute of Technology. W.M. Keck Laboratory of Hydraulics and Water Resources. Pasadena, CA. https://resolver.caltech.edu/CaltechKHR:KH-R-43A

40. Brune, G.M. 1953. "Trap efficiency of reservoirs." Transaction American Geophysical Union. Vol. 34(3). pp 407–418. https://agupubs.onlinelibrary.wiley.com/doi/epdf/10.1029/TR034i003p00407

41. Buffington, J.M., and D.R. Montgomery. 1997. "A Systematic Analysis of Eight Decades of Incipient Motion Studies, with Special Reference to Gravel Bedded Rivers." Water Resources Research. Vol. 33(8). pp 1993–2029. https://agupubs.onlinelibrary.wiley.com/doi/epdf/10.1029/96WR03190

42. Bunte, K., S. Abt, J. Potyondy, and S. Ryan. 2004. "Measurement of Coarse Gravel and Cobble Transport Using Portable Bedload Traps." Journal of Hydraulic Engineering. Vol. 130(9). pp 879–893. https://www.fs.usda.gov/treesearch/pubs/22266

43. Bunte, K., and S.R. Abt, 2001. "Sampling Surface and Subsurface Particle-Size Distributions in Wadable Gravel- and Cobble-Bed Streams for Analysis in Sediment Transport, Hydraulics, and Streambed Monitoring." RMRS-GTR-74. p 428. USDA Forest Service, Rocky Mountain Research Station. Fort Collins, CO. www.fs.usda.gov/biology/nsaec/assets/rmrs-gtr-74samplingsurfandsubsufpartszdist.pdf

44. Burkham, D.E., and D.R. Dawdy. 1976. "Resistance Equation for Alluvial-Channel Flow." Journal of the Hydraulics Division. ASCE. Vol. 102, No. HY 10. pp 1479–1489. https://ascelibrary.org/doi/abs/10.1061/JYCEAJ.0004632

45. Burns, M., and R. MacArthur. 1996. "Sediment deposition in Jennings Randolph Reservoir, Maryland and West Virginia." Proceedings of the 6th Federal Interagency Sedimentation Conference. Section X, Watersheds: Sediment Yield and Sediment Control, Remote Sensing and GIS Applications, Vol. 2. pp 10-16–10-21. Las Vegas, NV. https://acwi.gov/sos/pubs/6thFISC/6Fisc-V2/6Fsc2-10.PDF

46. Buttolph, A.M., C.W. Reed, N.C. Kraus, N. Ono, M. Larson, B. Camenen, H. Hanson, T. Wamsley, and A.K. Zundel. 2006. "Two-dimensional depth-averaged circulation model CMS-M2D: Version 3.0, Report 2: Sediment transport and morphology change." ERDC/CHL TR-06-9. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7613

47. Cannon, S.H., J.E. Gartner, R.C. Wilson, and J.L. Laber. 2008. "Storm rainfall conditions for floods and debris flows from recently burned areas in southwestern Colorado and southern California." Geomorphology. Vol. 96. pp 250–269. https://www.sciencedirect.com/science/article/pii/S0169555X07001778

48. Cao, Z., J. Li, G. Pender , and Q. Liu. 2014. "Whole-Process Modeling of Reservoir Turbidity Currents by a Double Layer-Averaged Model." Journal of Hydraulic Engineering. Vol. 141. https://ascelibrary.org/doi/10.1061/%28ASCE%29HY.1943-7900.0000951

49. Carr, M., and T. Dahl. 2017. "Review of Ice-Induced Scour Impacts to Navigation and Structures." ERDC SR-17-3. USACE ERDC. Hanover, NH and Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/bitstream/11681/22764/1/ERDC%20SR-17-3.pdf

50. Chanson, H. 2004. "Sabo check dams: Mountain protection systems in Japan." International Journal of River Basin Management. Vol. 2(4). pp 301–307. https://www.tandfonline.com/doi/abs/10.1080/15715124.2004.9635240

51. Chow, Ven Te. 1959. "Open Channel Hydraulics." McGraw-Hill, NY. books.google.com/books/about/Open_channel_Hydraulics.html?id=JG9_PwAACAAJ

52. Churchill, M.A. 1948. Discussion of "Analysis and use of reservoir sedimentation data." By L.C. Gottschalk, Third Session, Reservoir Sedimentation. Proceedings Federal Interagency Sedimentation Conference. Denver, CO. pp 139–40. https://acwi.gov/sos/pubs/1stFISC/1Fisc-3.pdf

53. Clark County. 1999. "Hydrologic Criteria and Drainage Design Manual." Clark County Regional Flood Control District. http://gustfront.ccrfcd.org/pdf_arch1/hcddm/current%20manual%20(complete)/hcddm.pdf

54. Colby, B.R. 1964. "Discharge of Sands and Mean Velocity Relationships in Sand-Bed Streams." Professional Paper 462-A. U.S. Geological Survey. https://doi.org/10.3133/pp462A

55. Collinson, J.D. 1978. "Alluvial Sediments, In Sedimentary Environments and Facies." H.G. Reading (Ed.), published online by Cambridge University Press, May 2009. https://doi.org/10.1017/S0016756800044113

56. Copeland, R.R. 1986. "San Lorenzo River Sedimentation Study: Numerical Model Investigation." USACE WES. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/6953/rec/2

57. Copeland, R.R. 2000. "Corte Madera Creek, Marin County, California, Modified Unit 4 Sedimentation Study." ERDC/CHL TR-00-14. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/7463

58. Copeland, R.R., L. Lombard, and R.A. Gaines. 2020. "Numerical Sedimentation Investigation, Mississippi River Cairo to Pilots Station." Mississippi River Geomorphology and Potamology (MRG&P) Program Report No. 30, USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/AD1091695.pdf

59. Copeland, R.R., D.N. McComas, C.R. Thorne, P.J. Soar, M.M. Jonas, and J.B. Fripp. 2001. "Hydraulic Design of Stream Restoration Projects." ERDC/CHL TR-01-28. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7554

60. Copeland, R.R., and W.A. Thomas. 1989. "Corte Madera Creek Sediment Study Numerical Model Investigation." TR HL-89-6. USACE WES. Vicksburg, MS. https://apps.dtic.mil/dtic/tr/fulltext/u2/a207175.pdf

61. Costa, J.E. 1984. "Physical geomorphology of debris flows." Developments and Applications of Geomorphology. pp 268–317. Springer Verlag, Berlin. https://link.springer.com/chapter/10.1007%2F978-3-642-69759-3_9

62. Costa, J.E. 1988. "Rheologic, geomorphic, and sedimentologic differentiation of water floods, hyperconcentrated flows, and debris flows." Flood Geomorphology. pp 113–122. V. Baker, R. Kochel, and P. Patton (Eds). Wiley-Interscience. NY. https://www.google.com/books/edition/Flood_Geomorphology/gl7nwAEACAAJ?hl=en

63. Costigan, K.H., M. Daniels, J.S. Perkin, and K.B. Gido. 2014. "Longitudinal variability in hydraulic geometry and substrate characteristics of a Great Plains sand-bed river." Geomorphology. Vol. 210. pp 48–58. https://www.sciencedirect.com/science/article/pii/S0169555X13006259

64. Davis, B.E. 2005. "A Guide to the Proper Selection and Use of Federally Approved Sediment and Water-Quality Samplers: Vicksburg, MS." USGS Open-File Report 2005-1087. P 20. https://pubs.usgs.gov/of/2005/1087/pdf/OFR_2005-1087.pdf

65. Davis, C.M., and J.F. Fox. 2009. "Sediment Fingerprinting: Review of the method and future improvements for allocating nonpoint source pollution." Journal of Environmental Engineering. Vol. 135(7). pp 490–504. https://ascelibrary.org/doi/full/10.1061/(ASCE)0733-9372(2009)135%3A7(490)

66. De Cesare, G., A. Schleiss, and F. Herman. 2001. "Impact of turbidity currents on reservoir sedimentation." Journal of Hydraulic Engineering. Vol. 127. pp 6–16. https://doi.org/10.1061/(ASCE)0733-9429(2001)127:1(6)

67. Demirbilek, Z., T.C. Lackey, and A.K. Zundel. 2012. "Particle Tracking Model Data Analysis Tools –Part 1: Capabilities in SMS." Dredging Operations and Environmental Research (DOER) Technical Notes Collection. ERDC TN-DOER-D15. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/8619

68. Dendy, F.E. 1974. "Sediment trap efficiency of small reservoirs." Transactions, American Society of Agricultural Engineers. Vol. 17(5). pp 898–901. https://elibrary.asabe.org/abstract.asp?aid=36994&t=1&redir=aid=36994&confalias=&redir=[volume=17&issue=5&conf=t&orgconf=t1974]&redirType=toc_journals.asp&redirType=toc_journals.asp

69. Dendy, F.E., and G.C. Bolton. 1976. "Sediment yield-runoff-drainage area relationships in the United States." Journal of Soil and Water Conservation. Vol. 31(6). pp 264–266. https://people.wou.edu/~taylors/andrews_forest/refs/dendy_bolton_1976.pdf

70. Dinis, P., and A. Castilho. 2012. "Integrating sieving and laser data to obtain bulk grain size distributions." Journal of Sedimentary Research. Vol. 82. pp 747–754. https://doi.org/10.2110/jsr.2012.62. https://pubs.geoscienceworld.org/sepm/jsedres/article-abstract/82/9/747/258324/Integrating-Sieving-and-Laser-Data-To-Obtain-Bulk?redirectedFrom=fulltext

71. Downer, C.W., and F.L. Ogden. 2006. "Gridded Surface Subsurface Hydrology Analysis (GSSHA) User's Manual." Version 1.43 for Watershed Modeling System 6.1. ERDC/CHL SR-06-1. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/1855

72. Duan, N. 1983. "Smearing estimate: A nonparametric retransformation method." Journal of the American Statistical Association. Vol. 78(383). pp 605–610. https://www.tandfonline.com/doi/abs/10.1080/01621459.1983.10478017

73. Einstein, H.A. 1950. "The Bed Load Functions for Sediment Transportation in Open Channels." Technical Bulletin 1026. USDA. Soil Conservation Service. https://ageconsearch.umn.edu/record/156389

74. Einstein, H.A., and N.L. Barbarossa. 1952. "River Channel Roughness." Transactions. ASCE. Vol. 117. pp 1121–1146. https://ascelibrary.org/doi/10.1061/TACEAT.0006666

75. El Kadi Abderrezzak, K., A.D. Moran, E. Mosselman, J. Bouchard, H. Habersack, D. Aelbrecht. 2014. "A physical, movable-bed model for nonuniform sediment transport, fluvial erosion and bank failure in rivers." Journal of Hydro-environment Research, Vol. 8(2). pp 95-114. https://doi.org/10.1016/j.jher.2013.09.004

76. Elci, Ş., P.A. Work, E.J. Hayter. 2007. "Influence of stratification and shoreline erosion on reservoir sedimentation patterns." ASCE Journal of Hydraulic Engineering. Vol. 133(3). pp 255–266. https://ascelibrary.org/doi/full/10.1061/%28ASCE%290733-9429%282007%29133%3A3%28255%29

77. Engelund, F., and E. Hansen. 1967. "A monograph on sediment transport in alluvial streams." Teknisk Forlag. Technical University of Denmark. Copenhagen, Denmark. http://resolver.tudelft.nl/uuid:81101b08-04b5-4082-9121-861949c336c9

78. Eshel, G., G.J. Levy, U. Mingelgrin, and M.J. Singer. 2004. "Critical evaluation of the use of laser diffraction for particle-size distribution analysis." Journal of Soil Science Society of America. Vol. 68. pp 736–743. https://doi.org/10.2136/sssaj2004.7360

79. Espinosa-Villegas, C.O., and J.L. Schnoor. 2009. "Comparison of long-term observed sediment trap efficiency with empirical equations for Coralville Reservoir, Iowa." ASCE Journal of Environmental Engineering. Vol. 135(7). pp 518–525. https://doi.org/10.1061/(ASCE)0733-9372(2009)135:7(518)

80. Ettema, R., and S. Daly 2004. "Sediment Transport Under Ice." ERDC/CRREL TR-04-20. Cold Regions Research and Engineering Lab (CRREL). Hanover, NH. https://apps.dtic.mil/dtic/tr/fulltext/u2/a427634.pdf

81. Evans, J.E., N.S. Levine, S.J. Roberts, J.F. Gottgens, and D.M. Newman. 2002. "Assessment using GIS and sediment routing of the proposed removal of Ballville Dam, Sandusky River, Ohio." Journal of the American Water Resources Association. Vol. 38. pp 1549–1565. https://www.researchgate.net/publication/257936603_Assessment_Using_GIS_and_Sediment_Routing_of_the_Proposed_Removal_of_Ballville_Dam_Sandusky_River_Ohio

82. Ferguson, R., T. Hoey, S. Wathen, and A. Werritty. 2009. "Field evidence for rapid downstream fining of river gravels through selective transport." Geology. Vol. 24(2). pp 179–182. https://pubs.geoscienceworld.org/gsa/geology/article/24/2/179/206489/Field-evidence-for-rapid-downstream-fining-of

83. Fischenich, C. 2001a. "Stability Thresholds for Stream Restoration Materials." Ecosystem Management and Restoration Research Program (EMRRP) Technical Notes Collection ERDC TN-EMRRP-SR-29. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/3947

84. Fischenich, C. 2001b. "Impacts of Stabilization Measures." ERDC TN-EMRRP- SR-32. EMRRP Technical Notes Collection. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/3937

85. Fischenich, J.C., and R.R. Copeland. 2001. "Environmental Considerations for Vegetation in Flood Control Projects." ERDC TR-01-16. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/8536

86. Flippin-Dudley, S.J., S.R. Abt, C.D. Bonhan, C.C. Watson, and J.C. Fischenich. 1998. "Evaluation of flow-resistance equations for vegetated channels and floodplains." Technical Report EL-98-2. USACE WES. Vicksburg, MS. https://hdl.handle.net/11681/7233

87. Floyd, I.E., S.J. Smith, S.H. Scott, and G.L. Brown. 2016. "Flocculation and settling velocity estimates for reservoir sedimentation analysis." ERDC/CHL CHETN-XIV-46. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/21569

88. Forel, F.A. 1888. “Le ravin sous-lacustre du Rhône dans le Lac Léman.” Bulletin de la Société Vaudoise des Sciences Naturelles. Vol. 23. pp 85–107. (Online version not available.)

89. Franco, J.J. 1978. “Guidelines for the design, adjustment and operation of models for the study of river sedimentation problems, Appendix C:Checklist for Model Design and Operation.” Inst. Rep. H-78-1. USACE WES. Vicksburg, MS. https://hdl.handle.net/11681/2950

90. Freeman, G.E., W.J. Rahmeyer, and R.R. Copeland. October 2000. “Determination of Resistance Due to Shrubs and Woody Vegetation.” ERDC TR-00-25. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7456

91. Frings, R.M., W. Ottevanger, and C.J. Sloff. 2010. “Downstream fining in sand-bed rivers” River Flow 2010. ISBN 978-3-939230-00-7. pp 831–838. https://izw.baw.de/e-medien/river-flow-2010/PDF/B1/B1_12.pdf

92. Frizell, K.H, J.F. Ruff, and S. Mishra. 1998. “Simplified Design Guidelines for Riprap Subjected to Overtopping Flow.” Proceedings, Association of State Dam Safety Officials Annual Meeting. Las Vegas, NV. https://www.usbr.gov/tsc/techreferences/hydraulics_lab/pubs/PAP/PAP-0790.pdf

93. Gailani J.Z., W. Lick, C.K. Zeigler, and D. Endicott. 1991. “Development and calibration of a fine-grained sediment transport model for the Buffalo River.” Journal of Great Lakes Research. Vol. 22(3). pp 765–778. https://doi.org/10.1016/S0380-1330(96)70995-4

94. Gaines, R.A., and A.M. Priestas. 2016. “Particle Size Distributions of Bed Sediments along the Mississippi River, Grafton, Illinois to Head of Passes, November 2013.” MRG&P Report No. 7. Mississippi Valley Division. USACE. Vicksburg, MS. http://hdl.handle.net/11681/7761

95. Garbrecht, J.D. 2011. “Effects of climate variations and soil conservation on sedimentation of a west-central Oklahoma reservoir.” Journal of Hydrologic Engineering. Vol. 16(11). pp 899–906. https://doi.org/10.1061/(ASCE)HE.1943-5584.0000377

96. Garcia, M.H., R.C. MacArthur, R. French, and J. Miller. 2008. “Sedimentation Hazards.” Chapter 19 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch19

97. García, R., J. Rodríguez, and J. O’Brien. 2004. “Hazard Zone Delineation for Urbanized Alluvial Fans.” Proceedings Critical Transitions in Water and Environmental Resources Management. ASCE. https://ascelibrary.org/doi/10.1061/40737%282004%2911

98. Gatto, L.W., and W.W. Doe III. 1987. "Bank conditions and erosion along selected reservoirs." Environmental Geology and Water Sciences. Vol. 9(3). pp 143–154. https://doi.org/10.1007/BF02449947

99. Gee, M. 1982 "Guidelines for the Calibration and Application of the Computer Model." HEC-6. Training Document 13. P 13. https://www.hec.usace.army.mil/publications/TrainingDocuments/TD-13.pdf

100. Gessler, J. 1971. "Chapter 7 Beginning and Ceasing of Sediment Motion." And "Chapter 8 Aggradation and Degradation." River Mechanics – Modeling of Fluvial Processes. H. W. Shen (Ed.). Vol I. Fort Collins, CO. https://www.worldcat.org/title/river-mechanics/oclc/238313

101. Gibson, S. 2011. "Simulating Deposition Driven Losses in River Bed Infiltration: A Sediment Transport Assessment and Model of the Russian River from Healdsburg to Guerneville." HEC Project Report 78. P 94. https://www.hec.usace.army.mil/publications/ProjectReports/PR-78.pdf

102. Gibson S. 2013. "Comparing Depth and Velocity Grids Computed with One- and Two-Dimensional Models at Ecohydraulic Scales." M.S. Thesis. University of California. Davis, CA. https://ucdavis.on.worldcat.org/oclc/883255232

103. Gibson, S., and P. Boyd. 2016a. "Designing Reservoir Sediment Management Alternatives with Automated Concentration Constraints in a 1D Sediment Model." Proceedings, River Flow 2016. International Symposium on River Sedimentation. pp 1448–1445. https://www.researchgate.net/publication/325603707_Designing_Reservoir_Sediment_Management_Alternatives_with_Automated_Concentration_Constraints_in_a_1D_Sediment_Model

104. Gibson, S., and P. Boyd. 2016b. "Monitoring, Measuring, and Modeling a Reservoir Flush on the Niobrara River in the Sandhills of Nebraska." Proceedings, River Flow 2016. https://www.researchgate.net/publication/311507782_Monitoring_measuring_and_modeling_a_reservoir_flush_on_the_Niobrara_River_in_the_sandhills_of_Nebraska

105. Gibson, S., and C. Cai. 2017. "Flow Dependence of Suspended Sediment Gradations." Water Resources Research. Vol. 53. Issue 11. https://doi.org/10.1002/2016WR020135

106. Gibson, S., B. Comport, and Z. Corum. 2017. "Calibrating a Sediment Transport Model through a Gravel-Sand Transition: Avoiding Equifinality Errors in HEC-RAS Models of the Puyallup and White Rivers." Proceedings, ASCE World Environmental and Water Resources Congress 2017. Hydraulics and Waterways and Water Distribution Systems Analysis. https://doi.org/10.1061/9780784480625.017

107. Gibson, S., and A. Nelson. 2016. "Modeling Differential Lateral Bed Change with a Simple Veneer Method in a One-Dimensional Sediment Transport Model." Proceedings, River Flow 2016. https://www.researchgate.net/publication/311507008_Modeling_differential_lateral_bed_change_with_a_simple_veneer_method_in_a_one-dimensional_sediment_transport_model

108. Gibson, S., A. Osorio, C. Creech, R. Amorim, M. Dircksen, T. Dahl, and M. Koohafkan. 2019. "Two pool-to-pool spacing periods on large sand-bed rivers: Mega-pools on the Madeira and Mississippi." Geomorphology. Vol. 328. pp 196–210. https://www.sciencedirect.com/science/article/pii/S0169555X18305208?via%3Dihub

109. Gibson, S., and G. Pasternack. 2015. "Selecting Between One-Dimensional and Two-Dimensional Hydrodynamic Models for Ecohydraulic Analysis." River Research and Application. Vol. 32(6). pp 1365–1381. https://doi.org/10.1002/rra.2972

110. Gibson, S., and S. Piper. 2007. "Sensitivity and Applicability of Bed Mixing Algorithms." World Environmental and Water Resources Congress – 2007. ASCE. https://doi.org/10.1061/40927(243)395

111. Gibson, S., and D. Pridal. 2015. "Negotiating Hydraulic Uncertainty in Long Term Reservoir Sediment Models: Simulating Argandab Reservoir Deposition with HEC-RAS." Sedimentation and Hydrology Joint Federal Interagency Sediment Conference (SedHyd) Proceedings. https://acwi.gov/sos/pubs/3rdJFIC/Contents/10C-Gibson.pdf

112. Gibson, S., C. Vuyovich, and M. Weidel. 2020. "Incorporating sediment non-stationarity into ice-affected flood-risk projections." River Research and Applications. Vol. 36(9). https://onlinelibrary.wiley.com/doi/epdf/10.1002/rra.3714

113. Graf, W.L. 2006. "Downstream Hydrologic and Geomorphic Effects of Large Dams on American Rivers." Geomorphology. Vol. 79. pp 336–360. https://doi.org/10.1016/j.geomorph.2006.06.022

114. Gray, J.R., and J.W. Gartner. 2010. "Overview of Selected Surrogate Technologies for High-Temporal Resolution Suspended-Sediment Monitoring." 2nd Joint Federal Interagency Sedimentation Conference (FISC). 2010. https://acwi.gov/sos/pubs/2ndJFIC/Contents/3C_Gray_surrogates_3_3_2010_paper.pdf

115. Greimann, B., Y. Lai, and J. Huang. 2008. "Two-dimensional total sediment load model equations." Journal of Hydraulic Engineering. Vol. 134(8). pp 1142–1146. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:8(1142)

116. Healy, D.F., S.J. Rheaume, and J.A. Simpson. 2003. "Environmental Effects of the Big Rapids Dam Remnant Removal." Big Rapids, MI. 2000-02. USGS Water Resources Investigations Report 03-4136. P 59. https://doi.org/10.3133/wri034136

117. Heinemann, H.G. 1981. “A new sediment trap efficiency curve for small reservoirs.” Water Resources Bulletin 17(5). pp 825–830. https://doi.org/10.1111/j.1752-1688.1981.tb01304.x

118. Heinemann, H.G., and D.L. Rausch. 1971. “Discussion of Chapter III Sediment Measurement Techniques: Reservoir Deposits.” Task Committee on Preparation of Sedimentation Manual. Committee on Sedimentation of the Hydraulics Division. Journal of the Hydraulics Division. ASCE. Vol. 97, No. HY 9. Proc. Paper 8345. pp 1555–1561. https://doi.org/10.1061/JYCEAJ.0003095

119. Holmquist-Johnson, C., D. Raff, and K. Russell. 2009. “Bureau of Reclamation Automated Modified Einstein Procedure (BORAMEP) Program for Computing Total Sediment Discharge.” Bureau of Reclamation Technical Service Center. Denver, CO. https://pubs.usgs.gov/misc/FISC_1947-2006/pdf/1st-7thFISCs-CD/8thFISC/Session%201A-3_Raff.pdf

120. Howard, A.D. 1982. “Equilibrium and time scales in geomorphology: Application to sand-bed alluvial streams.” Earth Surface Processes and Landforms. Vol. 7. pp 303–325. https://doi.org/10.1002/esp.3290070403

121. Howard, A., J. Pak, D. May, S. Gibson, C. Haring, B. Alberto, and M. Snyder. 2021. “Approaches for Assessing Riverine Scour.” ERDC/CHL TR-21-7. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/40702

122. Huang, J., R.C. Hilldale, and B.P. Greiman. 2006. “Cohesive Sediment Transport.” Chapter 4 in Erosion and Sedimentation Manual. U.S. Department of the Interior Bureau of Reclamation. P 55. https://cawaterlibrary.net/wp-content/uploads/2017/06/ErosionSedimentManual.pdf

123. Jain, S.C., R. Ettema, and I. Park. 1993. “Flow Regulation for Controlled River Ice Formation.” CRREL Report AD-A266 409. P 31. https://apps.dtic.mil/sti/pdfs/ADA266409.pdf

124. James, S.C., C.A. Jones, M.D. Grace, and J.D. Roberts. 2010. “Advances in sediment transport modelling.” Journal of Hydraulic Research. Vol. 48(6). pp 754–763. https://doi.org/10.1080/00221686.2010.515653

125. Jansson, M.B. 1988. “A Global Survey of Sediment Yield.” Geografiska Annaler. Series A, Physical Geography. Vol. 70, No. 1-2 (1988). pp 81–98. https://doi.org/10.1080/04353676.1988.11880241

126. Jepsen, R.A., J.D. Roberts, and W. Lick. 1997. “Effects of bulk density on sediment erosion rates.” Journal of Water, Air, and Soil Pollution. Vol. 99. pp 21–31. https://link.springer.com/article/10.1007/BF02406841

127. Johnson, H.K., and J.A. Zyserman. 2002. "Controlling spatial oscillations in bed level update schemes." Coastal Engineering. Vol. 46(2). pp 109–126. https://doi.org/10.1016/S0378-3839(02)00054-6

128. Julien, P.Y. 2010. "Erosion and sedimentation." Cambridge University Press. Cambridge, U.K. 2nd Edition. https://doi.org/10.1017/CBO9780511806049

129. Kantoush, S.A., and A.J. Schleiss. 2014. "Influence of geometry shape factor on trapping and flushing efficiencies. Chapter in Reservoir Sedimentation. A. Schleiss, G. de Cesare, M. Franca, and M. Pfister (Eds). 1st Edition. CRC press, London. pp 129–138. https://doi.org/10.1201/b17397

130. Kean, J.W., D.M. Staley, and S.H. Cannon. 2011. "In situ measurements of post-fire debris flows in southern California: Comparisons of the timing and magnitude of 24 debris-flow events with rainfall and soil moisture conditions." Journal Geophysical Research. Vol. 116. F04019. https://doi.org/10.1029/2011JF002005

131. Kinoshita, A.M., T.S. Hogue, and C. Napper. 2014. "Evaluating Pre- and Post-Fire Peak Discharge Predictions across Western U.S. Watersheds." Journal of the American Water Resources Association (JAWRA). pp 1–18. https://doi.org/10.1111/jawr.12226

132. Kirchner, J.W., R.C. Finkel, C. Riebe, D.E. Granger, J.L. Clayton, J.G. King, and W.F. Megahan. 2001. "Mountain erosion over 10 yr, 10 k.y., and 10 m.y. time scales." Geology. Vol. 29(7). pp 591–594. https://doi.org/10.1130/0091-7613(2001)029%3C0591:MEOYKY%3E2.0.CO;2

133. Kirsten, H.D. 1982. "A Classification System for Excavation in Natural Materials." The Civil Engineer in South Africa. pp 292–308. July. (Discussion in Vol. 25, No. 5. May 1983). https://hdl.handle.net/10520/AJA10212019_15379

134. Kondolf, G.M. 1997. "Hungry Water: Effects of Dams and Gravel Mining on River Channels." Environmental Management. Vol. 21, No. 4. pp 533–551. https://doi.org/10.1007/s002679900048

135. Kondolf, G.M., and H. Piegay. 2003. "Tools in Fluvial Geomorphology." John Wiley & Sons. West Sussex, England. https://onlinelibrary.wiley.com/doi/book/10.1002/0470868333**Error! Hyperlink reference not valid.**

136. Kostic, S., and G. Parker. 2003a. "Progradational sand–mud deltas in lakes and reservoirs. Part I: Theory and Numerical Model." Journal of Hydraulic Research. Vol. 41(2). pp 127–140. https://doi.org/10.1080/00221680309499956

137. Kostic, S., and G. Parker. 2003b. "Progradational sand–mud deltas in lakes and reservoirs. Part 2: Experiment and Numerical Simulation." Journal of Hydraulic Research. Vol. 41(2). pp 141–152. https://doi.org/10.1080/00221680309499957

138. Krone, R.B. 1962. “Flume Studies of the Transport of Sediment in Estuarial Shoaling Processes.” Final Report: Hydraulic Engineering Laboratory and Sanitary Engineering Research Laboratory. University of California. Berkeley, CA. http://www.worldcat.org/oclc/8967084

139. Krumbein, W.C. 1934. “Size Frequency Distributions of Sediments. Journal of Sedimentary Research.” Vol. 4(2). pp 65–77. https://pubs.geoscienceworld.org/sepm/jsedres/article/4/2/65/96255/Size-frequency-distributions-of-sediments

140. Lane, E.W. 1955a. “Design of stable channels.” Transactions of the ASCE. Vol. 120(1). pp 1234–1279. https://doi.org/10.1061/TACEAT.0007188

141. Lane, E.W. 1955b. “The importance of fluvial morphology in hydraulic engineering.” Proceedings ASCE. Vol. 81. Paper No. 745. (Online version not available.)

142. Lane, E.W. 1947. “Report of the Subcommittee on Sediment Terminology.” Transactions American Geophysical Union. Vol. 28, No. 6. pp 936–938. https://doi.org/10.1029/TR028i006p00936

143. Lane, E.W., and V.A. Koelzer. 1943. “Density of Sediments Deposited in Reservoirs, Report No. 9 of A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams.” Published by St. Paul United States Engineering District, Sub-Office. Hydraulic Laboratory. University of Iowa. Iowa City, IA. https://water.usgs.gov/fisp/docs/Report_9.pdf

144. Langbein, W.B., and S.A. Schumm. 1958. “Yield of Sediment in Relation to Mean Annual Precipitation.” Transactions American Geophysical Union. Vol. 39, No. 6. https://doi.org/10.1029/TR039i006p01076

145. Larsen, M., and A. Simon. 1993. “A Rainfall Intensity-Duration Threshold for Landslides in a Humid-Tropical Environment, Puerto Rico.” Geografiska Annaler. Series A, Physical Geography. Vol. 75(1-2). pp 13–23. https://doi.org/10.2307/521049

146. Laursen, E.M. 1962. “Scour at Bridge Crossings.” Transactions. ASCE. Paper No. 3294. Vol. 127. Part I. pp 166–180. https://ascelibrary.org/doi/pdf/10.1061/TACEAT.0008432

147. Leavesley, G.H., R.W. Lichty, B.M. Troutman, and L.G. Saindon. 1983. “Precipitation-runoff modeling system – User’s manual.” USGS Water resources Investigative Report 83-4238. USGS. https://doi.org/10.3133/wri834238

148. Leopold, L.B., and T. Maddock. 1953. “The hydraulic geometry of stream channels and some physiographic implications.” Geological Survey Professional Paper 252. https://doi.org/10.3133/pp252

149. Limerinos, J.T. 1970. "Determination of the Manning Coefficient from Measured Bed Roughness in Natural Channels." Geological Survey Water-Supply Paper 1989-B. USGS. https://pubs.usgs.gov/wsp/1898b/report.pdf

150. Lisle, T. 1995. "Particle size variations between bedload and bed material in natural gravel bed channels." Water Resources Research. Vol. 31(4). pp 1107–1118. https://doi.org/10.1029/94WR02526

151. Little, C., and M. Jonas. 2010. "Sediment Impact Analysis Methods (SIAM): Overview of Model Capabilities, Applications, and Limitations." Proceedings of the 2nd Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling. June 27–July 1, 2010. Las Vegas, NV. https://acwi.gov/sos/pubs/2ndJFIC/Contents/11C_Little_SIAM_overview_paper-jmbpdf.pdf

152. Loehlein, W.C. 1999. "The growing reservoir sedimentation problem in the U.S. Army Engineer District, Pittsburgh: Preparing for the 21st century." Proceedings of the 26th annual Water Resources Planning and Management Conference (WRPMD'99), June 6-9, 1999. pp 1–8. https://doi.org/10.1061/40430(1999)104

153. Lyons, J. 1996. "A Survey of Reservoir Shoreline Erosion Problems at Bureau of Reclamation Reservoirs." Proceedings of the 6th Federal Interagency Sedimentation Conference. Las Vegas, NV. Vol. 1. pp 1-1–1-6. https://acwi.gov/sos/pubs/6thFISC/6Fisc-V1/6FISC1-1.PDF

154. Mayne, C.M., D. May and D. Biedenharn. 2021. "Empirical Analysis of Effects of Dike Systems on Channel Morphology of the Lower Mississippi River." Mississippi River Geomorphology and Potamology (MRG&P) Program Report No. 36. ERDC/CHL. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/39799

155. Maynord, S.T. 1996. "Toe-Scour Estimation in Stabilized Bendways." Journal of Hydraulic Engineering Vol. 122, No. 8. pp 460–464. https://doi.org/10.1061/(ASCE)0733-9429(1996)122:8(460)

156. Meade, R.H. 1982. "Sources and sinks of river sediment in the Atlantic Drainage of the United States." The Journal of Geology. Vol. 90(3). pp 235–252. https://doi.org/10.1086/628677

157. Meade, R.H., and R.S. Parker. 1985. "Sediment in rivers of the United States." In National Water Summary 1984. USGS Water Supply Paper 2275. pp 49–60. https://doi.org/10.3133/wsp2275

158. Megahan, W.F., N.F. Day, and T.M. Bliss. 1978. "Landside occurrence in the western and central northern Rocky Mountain physiographic provinces in Idaho." Proceedings Forest Soils and Land Use. Fifth North American Forest Soils Conference. pp 126–139. Colorado State University. Fort Collins, CO. https://www.worldcat.org/title/landslide-occurrence-in-the-western-and-central-northern-rocky-mountain-physiographic-province-in-idaho/oclc/79924426

159. Melillo, J.M., T.C. Richmond, and G.W. Yohe. (Eds.). 2014. "Climate Change Impacts in the United States: The Third National Climate Assessment." U.S. Global Change Research Program. https://www.nrc.gov/docs/ML1412/ML14129A233.pdf

160. Meyer-Peter, E., and R. Muller. 1948. "Formulas for Bed Load Transport." Second Meeting of the International Association for Hydraulics Research. Stockholm, Sweden. Appendix 2. pp 39–64. https://repository.tudelft.nl/islandora/object/uuid%3A4fda9b61-be28-4703-ab06-43cdc2a21bd7

161. Miller, C.R. 1953. "Determination of the Unit Weight of Sediment for Use in Sediment Volume Computations." U.S. Bureau of Reclamation. Denver, CO. https://babel.hathitrust.org/cgi/pt?id=uiug.30112130044884&view=1up&seq=1

162. Miller, J.R., M. Lord, S. Yurkovich, G. Mackin, and L. Kolenbrander. 2005. "Historical Trends in Sedimentation Rates and Sediment Provenance." Fairfield Lake, NC. JAWRA. Vol. 41(5). pp 1053–1075. https://doi.org/10.1111/j.1752-1688.2005.tb03785.x

163. Mooney, D.M. 2006. "SIAM for Evaluating Sediment Causes and Effects." Proceedings of the 8th Federal Interagency Sedimentation Conference. pp 426–434. April 2–6, 2006. Reno, NV. https://acwi.gov/sos/pubs/8thFISC/Session%206B-2_Mooney.pdf

164. Moore, J.S., D.M. Temple, and H.A.D. Kirsten. 1994. "Headcut Advance Threshold in Earth Spillways." Bulletin of the Association of Engineering Geologists. Vol. XXXI. No. 2. pp 277–280. https://www.worldcat.org/title/4433913717

165. Morris, G.L., and J. Fan. 1998. Revised 2010. "Reservoir Sedimentation Handbook: Design and Management of Dams, Reservoirs and Watersheds for Sustainable Use." McGraw-Hill Book Co., NY. Electronic version of the original 1998 publication. http://www.reservoirsedimentation.com/

166. Morris, G.L., G. Annandale, and R. Hotchkiss. 2008. "Reservoir Sedimentation." Chapter 12 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://ascelibrary.org/doi/10.1061/9780784408148.ch12

167. Moustafa, A. 2013. "Predicting the deposition in the Aswan High Dam Reservoir using a 2-D model." Ain Shams Engineering Journal. 4. pp 143–153. https://www.sciencedirect.com/science/article/pii/S2090447912000688

168. Neill, C.R. 1968. “A Reexamination of the Beginning of Movement for Coarse Granular Bed Materials.” Report IT 68, Hydraulic Research Station. Wallingford, England. https://static.ags.aer.ca/files/document/OFR/OFR_1968_09.pdf

169. Osterkamp, W.R., P. Heilman, and L.J. Lane. 1998. “Economic considerations of a continental sediment-monitoring program.” International Journal of Sediment Research. Vol. 13(4). pp 12–24. https://pubag.nal.usda.gov/catalog/6269

170. Paaswell. R.E. 1973. “Causes and Mechanisms of Cohesive Soil Erosion: The State of the Art.” Special Report 135. Highway Research Board. USA. pp 52–74. http://onlinepubs.trb.org/Onlinepubs/sr/sr135/sr135-006.pdf

171. Paintal, A.S. 1971. “Concept of Critical Shear Stress in Loose Boundary Open Channels.” Journal of Hydraulic Research. No. 1. pp 90–113. https://doi.org/10.1080/00221687109500339

172. Paola, C., G. Parker, R. Seal, S.K. Sinha, J.B. Southard, and P.R. Wilcock. 1992. “Downstream fining by selective deposition in a laboratory flume.” Science. Vol. 258. pp 1757–1760. https://doi.org/10.1126/science.258.5089.1757

173. Parker, G. 2006. “1D Sediment Transport Morphodynamics with Applications to Rivers and Turbidity Currents.” Gary Parker’s Morphodynamic Web Page (Last updated April 13, 2006). http://hydrolab.illinois.edu/people/parkerg/default.asp

174. Partheniades, E. 1965. “Erosion and deposition of cohesive soils.” Journal of the Hydraulics Division. ASCE. Vol. 91(1). pp 105–139. https://doi.org/10.1061/JYCEAJ.0001165

175. Pemberton, E.L., and J.M. Lara. 1984. “Computing Degradation and Local Scour.” Technical Guideline for Bureau of Reclamation. Engineering and Research Center. Denver CO. https://www.usbr.gov/tsc/techreferences/reservoir/ComputingDegradtion01_1984.pdf

176. Perkey, D.W., K.A. Fall, and S.J. Smith. 2020. “Physical Factors that Influence Muddy Bed Aggregate Production Size, and Durability.” ERDC TR-20-19. USACE ERDC. Vicksburg, MS. http://dx.doi.org/10.21079/11681/38257

177. Perkey, D.W., S.J. Smith, K. Fall, G. Massey, and C. Friedrichs. 2020. “Impacts of muddy bed aggregates on sediment transport and management in the tidal James River, VA.” Journal of Waterway, Port, Coastal and Ocean Engineering. Vol. 146(5). https://doi.org/10.1061/(ASCE)WW.1943-5460.0000578

178. Perkey, D.W., S.J. Smith, and A. Priestas. 2020. “Erosion Thresholds and Rates for Sand-Mud Mixtures.” ERDC TR-20-13. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/37495

179. Philips, B.M., and D.T. Williams. “Design Considerations for Confining and Guiding Levees On Alluvial Fans.” Proceedings World Environmental and Water Resources Congress. ASCE. 2008. https://doi.org/10.1061/40976(316)203

180. Pierson, T.C. 2005a. “Distinguishing between debris flows and floods from field evidence in small watersheds.” USGS Fact Sheet 2004-3142. U.S. Department of the Interior. USGS. Reston, VA. https://doi.org/10.3133/fs20043142

181. Pierson, T.C., and J.E. Costa. 1987. “A rheologic classification of subaerial sediment-water flows.” In Debris Flows/Avalanches. J. Costa and G. Wieczorek (Eds.). Geological Society of America Reviews in Engineering Geology. Vol. 7. pp 1–12. https://doi.org/10.1130/REG7-p1

182. Pimentel, D., C. Harvey, P. Resosudarmo, K. Sinclair, D. Kurz, M. McNair, S. Crist, L. Shpritz, L. Fitton, R. Saffouri, and R. Blair. 1995. “Environmental and economic costs of soil erosion and conservation benefits.” Science. Vol. 267. pp 1117–1123. https://doi.org/10.1126/science.267.5201.1117

183. Porterfield, G. 1972. “Computation of Fluvial-Sediment Discharge.” Book 3, Chapter 3, Techniques of Water-Resources Investigations of the USGS. U.S. Government Printing Office. https://doi.org/10.3133/twri03C3

184. Pugh, C.A. 2008. “Appendix C Sediment Transport Scaling for Physical Modeling.” Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.apc

185. Randle, T.J., and J.A. Bountry. 2015. “Progress on Dam Removal Analysis Guidelines for Sediment.” Proceedings SedHyd, Interagency Sediment Conference, session 10B. http://acwi.gov/sos/pubs/3rdJFIC/Contents/10B-Randle_a.pdf

186. Raudkivi, A.J., and S.K. Tan. 1984. “Erosion of Cohesive Soils.” Journal of Hydraulic Research. Vol. 22, No. 4. pp 217–233. Online version available dated 2010. https://doi.org/10.1080/00221688409499380

187. Ravens, T. 2007. “Comparison of Two Techniques to Measure Sediment Erodibility in the Fox River, Wisconsin.” Journal of Hydraulic Engineering. Vol. 133(1). pp 111–115. https://ascelibrary.org/doi/pdf/10.1061/%28ASCE%290733-9429%282007%29133%3A1%28111%29

188. Renard, K.G., G.R. Foster, G.A. Weesies, D.K. McCool, and D.C. Yoder. 1997. “Predicting soil erosion by water: A guide to conservation planning with the Revised Universal Soil Loss Equation (RUSLE).” Agriculture Handbook No. 703. USDA Agricultural Research Service. https://www.ars.usda.gov/arsuserfiles/64080530/rusle/ah_703.pdf

189. Rice, S. 1998. “Which tributaries disrupt downstream fining along gravel bed rivers?” Geomorphology. Vol. 22. pp 39–56. https://doi.org/10.1016/S0169-555X(97)00052-4

190. Riedel, M., J. Selegean, and T. Dahl. 2010. “Sediment Budget Development for the Great Lakes Region.” Proceedings of the 2nd Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling, Session 2B. June 27–July 1, 2010. Las Vegas, NV. https://acwi.gov/sos/pubs/2ndJFIC/Contents/2B_Riedel_03_01_10_paper.pdf

191. Robinson, D.W., and R.R. Copeland. 1986. “Mill Creek Channel, Walla Walla, Washington.” Technical Report HL 86-2. USACE WES. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/6855/

192. Ruark, M.D., J.D. Niemann, B.P. Greimann, and M. Arabi. 2011. “Method for assessing impacts of parameter uncertainty in sediment transport modeling applications.” Journal of Hydraulic Engineering. Vol. 137. pp 623–636. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000343

193. Rubin, D.M., and D.J. Topping. 2001. “Quantifying the relative importance of flow regulation and grain size regulation of suspended-sediment transport (α) and tracking changes in grain size on the bed (β).” Water Resources Research. Vol. 37(1). pp 133–146. https://doi.org/10.1029/2000WR900250

194. Saint-Laurent, D., B.N. Touileb, J.P. Saucet, A. Whalen, B. Gagnon, and T. Nzakimuena. 2001. “Effects of simulated water level management on shore erosion rates. Case study: Baskatong Reservoir, Quebec.” Canadian Journal of Civil Engineering. Vol. 28. pp 482–495. http://dx.doi.org/10.1139/l01-018

195. Sánchez, A., and W. Wu. 2011. “A nonequilibrium sediment transport model for coastal inlets and navigation channels.” Journal of Coastal Research. Special Issue 59. pp 39–48. http://dx.doi.org/10.2112/SI59-005.1

196. Sánchez, A., W. Wu, T. Beck, H. Li, J.D. Rosati, Z. Demirbilek, and M. Brown. 2011. “Validation of the coastal modeling system. Report IV, CMS-Flow sediment transport and morphology change.” ERDC/CHL-TR-11-10. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7658

197. Sánchez, A., W. Wu, H. Li, M. Brown, C. Reed, J. Rosati, and Z. Demirbilek. 2014. “Coastal Modeling System: Mathematical formulations and numerical methods.” ERDC/CHL TR-14-2. USACE ERDC. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/3570/

198. Savant, G., R.C. Berger, T.O. McAlpin, and C. Trahan. 2014. “Three-dimensional Shallow-Water Adaptive Hydraulics (AdH-SW3): Hydrodynamic verification and validation.” USACE ERDC/CHL TR-14-7. Vicksburg, MS. http://hdl.handle.net/11681/8602

199. Schumm, S.A. 1977. “The Fluvial System.” John Wiley & Sons. New York, NY. https://doi.org/10.1002/esp.3290040121

200. Schumm, S.A. 1979. “Geomorphic thresholds: the concept and its applications.” Institute of British Geographers. Vol. 4. pp 485–515. https://doi.org/10.2307/622211 **Error! Hyperlink reference not valid.**

201. Schumm, S.A., M.D. Harvey, and C.C. Watson. 1984. “Incised channels, morphology, dynamics, and control.” Water Resources Publications. Littleton, CO. https://www.wrpllc.com/books/ic.html

202. Scott, S. 2012. “Evaluation of Uncertainties in Conowingo Reservoir Sediment Transport Modeling.” Lower Susquehanna River Watershed Assessment. Final Report Prepared for Baltimore District. https://dnr.maryland.gov/waters/bay/Documents/LSRWA/SteveScott_ModelingUncertaintyReport_Final_101612.pdf

203. Sharp, J.A., R.E. Heath, H.E. Park, and T.O. McAlpin. 2016. “Scale Factor Study for 1:30 Local Scour Model.” ERDC/CHL CHETN-VII-15. USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/AD1013557.pdf

204. Sharp, J., C.D. Little, G.L. Brown, T.C. Pratt, R.H. Heath, L.C. Hubbard, F. Pinkard, S.K. Martin, N.D. Clifton, D.W. Perkey, and N.B. Ganesh. 2013. “West Bay Sediment Diversion Effects.” ERDC/CHL TR-13-15. USACE ERDC. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/3646/

205. Shelley, J., D. Abraham, and T. McAlpin. 2013. “Removing Systemic Bias in Bed-Load Transport Measurements in Large Sand-Bed Rivers.” Journal of Hydraulic Engineering. Vol. 139. pp 1107–1111. http://dx.doi.org/10.1061/(ASCE)HY.1943-7900.0000760

206. Shelley, J., and S. Gibson. 2015. “Modeling bed degradation of a large, sand-bed river with in-channel mining with HEC-RAS.” SedHyd Proceedings, Federal Interagency Sediment Conference. https://acwi.gov/sos/pubs/3rdJFIC/Contents/1A-Shelley.pdf

207. Shelley, J. 2017. “Reservoir Sedimentation Problems: In-reservoir problems.” Presentation at Reservoir Sediment Management Workshop for Planners, Regulators, and Managers. August 15–17, 2017. Lakewood, CO. Regional Sediment Management. https://rsm.usace.army.mil/techtransfer/FY17/ReservoirWorkshop-Aug2017/pdfs/2_02_ReservoirProblems-LostBenefitsFromSedimentation.pdf

208. Shields, A. 1936. “Anwendung der ahnlichkeits-mechanik und der turblenzforschung auf die geschiebebewegung.” Mitt. Preuss. Versuchsanst. Wasser. Schiffsbau. Vol. 26. pp 1–26. http://resolver.tudelft.nl/uuid:61a19716-a994-4942-9906-f680eb9952d6

209. Shields, F.D., R.R. Copeland, P.C. Klingeman, M.W. Doyle, and A. Simon. 2008. “Stream Restoration.” Chapter 9 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch09

210. Simon, A.S. 2006. “Evaluation of the Importance of Channel Processes in CEAP-Watershed Suspended Sediment Yields.” Proceedings of the 8th Federal Interagency Sedimentation Conference. Session 10-C, pp 815–822. April 2006. Reno, NV. https://acwi.gov/sos/pubs/8thFISC/Session%2010C-3_Simon.pdf

211. Simons, D.B., and E.V. Richardson. 1962. “The Effect of Bed Roughness on Depth-Discharge Relations in Alluvial Channels.” USGS Water-Supply Paper 1498-E. https://doi.org/10.3133/wsp1498E

212. Singh, V.P. 1995. “MIKE SHE.” Chapter 23 in Computer Models of Watershed Hydrology. Water Resources Publications. Highlands Ranch, CO. pp 809–846. https://www.wrpllc.com/books/cmwhn.html

213. Strand, R.I., and E.L. Pemberton. 1987. “Reservoir sedimentation. Design of Small Dams.” U.S. Bureau of Reclamation. Denver, CO. pp 529–564. 3rd Edition. https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf

214. Sundborg, A., and M.B. Jansson. 1992. “Present and Future Conditions of Reservoir Sedimentation.” Chapter 14 in Sedimentological Studies on the Cachí Reservoir, Costa Rica. pp 158–164. UNGI Report No. 81. Department of Physical Geography. Uppsala University. Sweden. https://www.researchgate.net/publication/318967363_14_Present_and_future_conditions_of_reservoir_sedimentation

215. Swanson, F.J., and C.T. Dyrness. 1975. “Impacts of clear-cutting and road construction on soil erosion by landslides in the western Cascade Range, Oregon.” Journal of Geology. Vol. 3. No 7. pp 393–397. https://andrewsforest.oregonstate.edu/sites/default/files/lter/pubs/pdf/pub491.pdf

216. Tate, J.N., R.C. Berger, and R.L. Stockstill. 2006. “Refinement Indicator for Mesh Adaption in Shallow-Water Modeling.” ASCE Journal of Hydraulic Engineering. Vol. 132(8). pp 854–857. https://doi.org/10.1061/(ASCE)0733-9429(2006)132:8(854)

217. Tatum, F.E. 1963. “A New Method of Estimating Debris Storage Requirements for Debris Basins.” U.S. Army Engineer District. Los Angeles, CA. (Online version not available.)

218. Taylor B.D., and V.A. Vanoni. 1971. “Temperature Effects in Alluvial Streams.” W.M. Keck Laboratory of Hydraulics and Water Resources. California Institute of Technology. “ Report No. KH-R-27. California Institute of Technology, Pasadena CA. https://authors.library.caltech.edu/26013/1/KH_R_27.pdf

219. Temple, D., and J. Moore. 1997. "Headcut advance prediction for earth spillways." Transactions of the ASAE. Vol. 40, No. 3. pp 557–562. https://doi.org/10.1002/esp.3290080619

220. Thomas, W.A. 1982 "Mathematical Modeling of Sediment Movement." In Gravel Bed Rivers. R.D. Hey, J.C. Bathurst, and C.R. Thorne (Eds.). pp 487–508. https://doi.org/10.1002/esp.3290080619

221. Thomas, W.A. 2007. "Sediment Model Calibration." HEC-RAS Sediment Transport Prospect Class Notes. HEC. Davis, CA. (Online version not available.)

222. Thomas, W.A. 2002. "Sedimentation in Stream Networks (HEC-6T): User Manual." p 338. http://www.mbh2o.com/wp-content/uploads/2015/05/HEC6T_UserManuel.pdf

223. Thomas, W.A., and H. Chang. 2008. "Computationally Modeling of Sedimentation Processes." Chapter 14 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch14

224. Toffaleti, F.B. 1968. "A Procedure for Computation of the Total River Sand Discharge and Detailed Distribution Bed to Surface." Committee on Channel Stabilization. USACE. U.S. Army. http://hdl.handle.net/11681/1822

225. Toniolo H., G. Parker, and V. Voller. 2007. "Role of ponded turbidity currents in reservoir trap efficiency." Journal of Hydraulic Engineering. Vol. 133. pp 579–595. https://doi.org/10.1061/(ASCE)0733-9429(2007)133:6(579)

226. Trimble, S.W., and W.P. Carey. 1992. "A comparison of the Brune and Churchill methods for computing sediment yields applied to a reservoir system." Subitzky, S. (Ed.) in "Selected Papers in the Hydrologic Sciences." USGS Water Supply Paper 2340. pp 195–202. https://pubs.usgs.gov/wsp/wsp2340/

227. Turowski, J.M., D. Rickenmann, and S.J. Dadson. 2010. "The Partitioning of the Total Sediment Load of a River into Suspended Load and Bedload: A Review of Empirical Data." Sedimentology. Vol. 57. pp 1126–1146. https://doi.org/10.1111/j.1365-3091.2009.01140.x

228. Van Rijn, L.C. 1984a. "Sediment Transport, Part I: Bed Load Transport." Journal of Hydraulic Engineering. ASCE. Vol. 110. pp 1431– 1455. https://doi.org/10.1061/(ASCE)0733-9429(1984)110:10(1431)

229. Van Rijn, L.C. 1984b. "Sediment Transport, Part II: Suspended Load Transport." Journal of Hydraulic Engineering. ASCE. Vol. 110. pp 1613–1641. https://doi.org/10.1061/(ASCE)0733-9429(1984)110:11(1613)

230. Van Rijn, L.C., D.J.R. Walstra, B.T. Grasmeijer, J. Sutherland, S. Pan, and J.P. Sierra. 2003. "The predictability of cross-shore bed evolution of sandy beaches at the time scale of storms and seasons using process-based profile models." Coastal Engineering. Vol. 47. pp 295–327. https://doi.org/10.1016/S0378-3839(02)00120-5

231. Van Rijn, L.C., D.J.R. Walstra, and M.V. van Ormondt. 2007. "Unified view of sediment transport by currents and waves. IV: Application of morphodynamic model." Journal of Hydraulic Engineering. Vol. 133(7). pp 776–793. https://doi.org/10.1061/(ASCE)0733-9429(2007)133:7(776)

232. Vanoni, V.A. 2006. "Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54." ASCE Task Committee for the Preparation of the Manual on Sedimentation of the Sedimentation Committee of the Hydraulics Division. 2nd Edition. Online version available dated 2006; original work published 1975. https://doi.org/10.1061/9780784408230

233. Verstraeten, G., and J. Poesen. 2000. "Estimating trap efficiency of small reservoirs and ponds: Methods and implications for the assessment of sediment yield." Progress in Physical Geography. Vol. 24(2). pp 219–251. https://doi.org/10.1177%2F030913330002400204

234. Walling D.E., and D. Fang. 2003. "Recent trends in the suspended sediment loads of the world's rivers." Global and Planetary Change. Vol. 39. pp 111–126. https://doi.org/10.1016/S0921-8181(03)00020-1

235. Walling, D.E., and P.W. Moorhead. 1989. "The particle size characteristics of fluvial suspended sediment: an overview." Hydrobiologia. Vol. 176/177. pp 125–149. https://link.springer.com/article/10.1007/BF00026549

236. Wannanen, A.O., D.D. Harris, and R.C. Williams. 1970. "Floods of December 1964 and January 1965 in the Far Western States." Part 2: Streamflow and Sediment Data, USGS Water-Supply Paper 1866B. p 861. USGS. Reston, VA. https://doi.org/10.3133/wsp1866B

237. Warner, J.C., C.R. Sherwood, R.P. Signell, C.K. Harris, and H.G. Arango. 2008. "Development of a three-dimensional, regional, coupled, wave, current and sediment transport model." Computers and Geosciences. Vol. 34. pp 1284–1306. https://doi.org/10.1016/j.cageo.2008.02.012

238. Watson, C.C., D.S. Biedenharn, and C.S. Thorne. 2013a. "Analysis of the impacts of dikes on flood stages in the Middle Mississippi River." Journal of Hydraulic Engineering. Vol. 139(10). pp 1071–1078. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000786

239. Watson, C.C., R.R. Holmes Jr., and D.S. Biedenharn. 2013b. "Mississippi river streamflow measurement techniques at St. Louis, Missouri." Journal of Hydraulic Engineering. Vol. 139(10). pp 1062–1070. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000752

240. Webb, R.H., P.G. Griffiths, and D.R. Hartley. 2001. "Techniques for Estimating Sediment Yield of Ungaged Tributaries on the Southern Colorado Plateau." Proceedings of the 7th Federal Interagency Sedimentation Conference, Volume I Sediment and Flow Modeling. pp I-24–I-31. March 2001. Reno, NV. https://acwi.gov/sos/pubs/7thFISC/7Fisc-V1/7FISC1-1.pdf

241. Wen, B., A. Aydin, and N.S. Duzgoren-Aydin. 2002. "A Comparative Study of Particle Size Analyses by Sieve-Hydrometer and Laser Diffraction Methods." Geotechnical Testing Journal. Vol. 25, No. 4. Paper ID GTJ200210036_254. https://www.astm.org/gtj11289j.html

242. White, S. 2005. "Sediment Yield Prediction and Modelling, Hydrological Processes." Vol. 19. pp 3053–3057. https://onlinelibrary.wiley.com/doi/pdf/10.1002/hyp.6003

243. Wibowo, J.L., D.E. Yule, and E. Villanueva. 2005. "Earth and Rock Surface Spillway Erosion Risk Assessment." Presented at Alaska Rocks 2005, the 40th U.S. Symposium on Rock Mechanics (USRMS), Anchorage, AK. https://onepetro.org/ARMAUSRMS/proceedings-abstract/ARMA05/All-ARMA05/ARMA-05-813/117818

244. Wilcock, P.R., and J.C. Crowe. 2003. "Surface-based transport model for mixed-size sediment." Journal of Hydraulic Engineering. Vol. 129. pp 120–128. https://doi.org/10.1061/(ASCE)0733-9429(2003)129:2(120)

245. Williams, G.P. 1989. "Sediment concentration versus water discharge during single hydrologic events in rivers." Journal of Hydrology. Vol. 111. pp 89–106. https://doi.org/10.1016/0022-1694(89)90254-0

246. Williams, J.R. 1995. "The EPIC Model." Chapter 25 in Computer Models of Watershed Hydrology. Singh, V.P., Ed. Water Resources Publications. Highlands Ranch, CO. pp 909–1000. https://www.google.com/books/edition/Computer_Models_of_Watershed_Hydrology/DXFRLwEACAAJ?hl=en

247. Williams, J., M.A. Arsenaultm, B.J. Buckowski, J.A. Reid, J.A. Flocks, M.A. Kulp, S. Penland, and C.J. Jenkins. 2006. "Surficial sediment character of the Louisiana offshore continental shelf region: A GIS Compilation." USGS Open-File Report 2006–1195. https://doi.org/10.3133/ofr20061195

248. Winterwerp, J.C., W.G. van Kesteren, B. van Prooijen, and W. Jacobs. 2012. "A Conceptual framework for shear flow-induced erosion of soft cohesive sediment beds." Journal of Geophysical Research. Vol. 117. 2012JC008072. https://doi.org/10.1029/2012JC008072

249. Wischmeier, H., and D.D. Smith. 1978. "Predicting rainfall erosion losses, A Guide to Conservation Planning." Agriculture Handbook No. 537. USDA Science and Education Administration. P 69. (Online version not available)

250. Wolman, M.G., and J.P. Miller. 1960. "Magnitude and frequency of forces in geomorphic processes." Journal of Geology. Vol. 68. pp 54–74. https://doi.org/10.1086/626637

251. Wong, M. 2003. "Does the bedload equation of Meyer-Peter and Muller fit its own data?" Proceedings, 30th Congress, International Association of Hydraulic Research Thessaloniki. J.F.K. Computation Volume. pp 73–80. https://www.iahr.org/library/infor?pid=25777

252. Wong, M., and G. Parker. 2006. "Reanalysis and Correction of Bed Load Relation of Meyer-Peter and Muller Using Their Own Database." Journal of Hydraulic Engineering. Vol. 132(11). pp 1159–1168. https://doi.org/10.1061/(ASCE)0733-9429(2006)132:11(1159)

253. Woolhiser, D.A., R.E. Smith, and D.C. Goodrich. 1990. "KINEROS, A Kinematic Runoff and Erosion Model: Documentation and user manual." ARS-77. USDA Agricultural Research Service. Fort Collins, CO. (Online version not available)

254. Wright, W.E. 1990. "Geologic History of Minnesota Rivers." Minnesota Geological Survey. Educational Series 7. University of Minnesota. St. Paul, MN. https://www.swwdmn.org/wp-content/uploads/2016/03/Geologic-History-of-MN-Rivers.pdf

255. Wu, W., S.S.Y. Wang, and Y. Jia. 2000. "Nonuniform sediment transport in alluvial rivers." Journal of Hydraulic Research. Vol. 38(6). pp 427–434. https://doi.org/10.1080/00221680009498296

256. Wu, W., C. Perera, J. Smith, and A. Sanchez. 2018. "Critical shear stress for erosion of sand and mud mixtures." Journal of Hydraulic Research, 56(1), 96-110. https://doi.org/10.1080/00221686.2017.1300195

257. Wulandari, D.A., D. Legono, and S. Darsono. 2015. "Evaluation of deposition pattern of Wonogiri reservoir sedimentation." International Journal of Civil and Environmental Engineering. Vol. 15, No. 02. pp 15–20. https://doc-pak.undip.ac.id/id/eprint/9430/4/Paper%20C2.pdf

258. Xia, Q., and J. Liu. 2003. "Sediment Management at Naodehai Reservoir." ASCE World Water and Environmental Resources Congress. Philadelphia, PA. https://doi.org/10.1061/40685(2003)54

259. Yang, C.T. 1973. "Incipient Motion and Sediment Transport." Journal of the Hydraulics Division. ASCE. Vol. 99, No. HY 10. pp 1679–1704. https://doi.org/10.1061/JYCEAJ.0003766

260. Yang, C.T. 1984. “Unit Stream Power Equation for Gravel.” Journal of the Hydraulics Division. ASCE. Vol. 110, No. 12. pp 1783–1797. https://ascelibrary.org/doi/pdf/10.1061/%28ASCE%290733-9429%281984%29110%3A12%281783%29

261. Young, R.A., C.A. Onstad, D.D. Bosch, and W.P. Anderson. 1987. “AGNPS: A nonpoint source pollution model for evaluating agricultural watersheds.” Journal of Soil and Water Conservation. Vol. 44(2). pp 168–173. https://www.jswconline.org/content/44/2/168

A-2. Recommended Publications.

USACE Publications

1. EC 1105-2-412. USACE 2011a. “Assuring Quality of Planning Models.” https://planning.erdc.dren.mil/toolbox/library/ECs/EC_1105-2-412_2011Mar.pdf

2. ECB 2018-14. USACE 2018a. “Subject: Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects.” https://www.wbdg.org/FFC/ARMYCOE/COEECB/ecb_2018_14_rev_1.pdf

3. EM 1110-2-1418. USACE 1994a. “Channel Stability Assessment for Flood Control Projects.” https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-1418.pdf

4. EM 1110-2-1906. USACE 1970. “Laboratory Soils Testing.” Change 2. August 20, 1986. https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-1906.pdf

5. EM 1110-2-5026. USACE 1987. “Beneficial Uses of Dredged Material.” https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-5026.pdf

6. HEC. 1988. “Review of the U.S. Army Corps of Engineers Involvement with Alluvial Fan Flooding Problems.” TP 124. HEC. Davis, CA. https://www.hec.usace.army.mil/publications/TechnicalPapers/TP-124.pdf

7. HEC. 2017. “Hydrologic Modeling System HEC-HMS Applications Guide.” USACE Institute for Water Resources. HEC. Davis, CA. https://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Applications_Guide_June2017.pdf

8. USACE 1976. “Missouri River, Temperature Effects in the Transition from Dunes to Plane Bed.” MRD Sediment Series No. 14. Omaha District. https://erdclibrary.on.worldcat.org/oclc/3836779

9. USACE 1977. “Missouri River, Temperature Effects on Stage-Discharge Relationship in a Missouri River Reach Near Omaha.” MRD Sediment Series No. 15. Omaha District. https://erdclibrary.on.worldcat.org/oclc/33293223

10. USACE 1981c. “Final Report to Congress, Streambank Erosion Control and Demonstration Project (Section 32). Appendix F, Yazoo River Basin Demonstration Projects.” https://usace.contentdm.oclc.org/digital/collection/p16021coll5/id/1234

11. USACE 1984. “Analysis of Channel Degradation and Bank Erosion in the Lower Kansas River.” Missouri River Division Sediment Series Number 35. USACE Kansas City District. Kansas City, MO. Prepared by Simons, Li, and Associates. Fort Collins, CO. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/7867/rec/1

12. USACE 2000a. “Los Angeles District Method for Prediction of Debris Yield.” Los Angeles District Corps of Engineers, Updated. Los Angeles, CA. https://www.hec.usace.army.mil/confluence/hmsdocs/hmsguides/files/86908188/89621118/1/1646347766266/Gatwood_2000_Los+Angeles+District+Debris+Method.pdf

13. USACE 2001. “Hydraulic Design of Stream Restoration Projects.” TR 1-28. https://hdl.handle.net/11681/7554

14. USACE 2009. “Missouri River Bed Degradation Reconnaissance Study Section 905(b) (Water Resources Development Act of 1986) Analysis.” USACE Kansas City District. Kansas City, MO. https://usace.contentdm.oclc.org/utils/getfile/collection/p16021coll7/id/9315

15. USACE 2010. “Lower Yuba River Gravel Augmentation Project, Yuba and Nevada Counties, California.” Final Environmental Impact Assessment. https://pasternack.ucdavis.edu/application/files/5113/7581/7498/EDR_gravelinject_EA_20101117.pdf

16. USACE 2015g. “Lower Susquehanna River Watershed Assessment, Maryland and Pennsylvania.” USACE Baltimore District with Cooperating Partner Maryland Department of the Environment. https://dnr.maryland.gov/waters/bay/documents/lsrwa/reports/lsrwafinalmain20160307.pdf

17. USACE 2015h. “Lower Naches River Sediment Study, Naches, Washington.” USACE Seattle District. Prepared for Yakima County Under Planning Assistance Program to the States (PAS).

18. Corum, Z., E. Cheng, S. Gibson, and T. Dahl. 2022. “Optimization of Reach-Scale Gravel Nourishment on the Green River below Howard Hanson Dam, King County, Washington.” ERDC-CHL TR 22-6. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/43887

Other Federal Agency and Private Sector Publications

1. American Society for Testing and Materials (ASTM). "Standard Guide for Core Sampling Submerged, Unconsolidated Sediments." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 4823-95. https://doi.org/10.1520/D4823-95R19

2. ASTM. "Standard Guide for Sampling Fluvial Sediment in Motion." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 4411-03. https://doi.org/10.1520/D4411-03R19

3. ASTM. "Standard Guide for Selection of Methods of Particle Size Analysis of Fluvial Sediments (Manual Methods)." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 4822-88. https://doi.org/10.1520/D4822-88R19

4. ASTM. "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 2487. https://www.astm.org/cgi-bin/resolver.cgi?D2487

5. ASTM. "Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures." ASTM International Standard D 7012-14. https://doi.org/10.1520/D7012-14E01

6. ASTM. "Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 7928. https://doi.org/10.1520/D7928-21E01

7. ASTM. "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 2166. https://www.astm.org/cgi-bin/resolver.cgi?D2166D2166M

8. ASTM. "Standard Test Methods for Determining Sediment Concentration in Water Samples." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 3977-97. https://doi.org/10.1520/D3977-97R19

9. ASTM. "Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis." Annual Book of Standards. Section 11, Water and Environmental Technology. Vol. 11.02. Designation: D 6913-17. https://doi.org/10.1520/D6913_D6913M-17

10. Federal Emergency Management Agency (FEMA). 2002. “Guidelines and Specifications for Flood Hazard Mapping Partners.” Appendix G: Guidance for Alluvial Fan Flooding Analyses and Mapping. https://www.fema.gov/sites/default/files/nepa/Guidelines_and_Specifications_for_Flood_Hazard_Mapping_Partners_Appendix_G-Guidance_for_Alluvial_Fan_Flooding_Analyses_and_Mapping__Feb_2002_.pdf

11. FEMA. 2016. “Guidelines for Flood Risk Analysis and Mapping, Alluvial Fans.” https://www.fema.gov/sites/default/files/2020-02/Alluvial_Fans_Guidance_Nov_2016.pdf

12. FHWA. 2009. “Bridge scour and stream instability countermeasures experience, selection, and design guidance.” Publication No. FHWA NHI-09–112. Hydraulic Engineering Circular No. 23. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf

13. FISP. 2000. “Report OO, Pipet and X-Ray Grain Size Analyzers: Comparison of Methods and Basic Data.” https://water.usgs.gov/fisp/docs/Report_OO_Pipet_and_X-Ray_Grain_Size_Analyzers_000706.pdf

14. FISP. 2009. “FISP Technical Committee Approval of the Use of Bedload Traps in Wadeable Coarse-Bedded Streams.” Technical Committee Memorandum. https://water.usgs.gov/fisp/docs/FISP_Tech_Memo_2009-1_Bedload_Traps.pdf

15. FISRWG. 1998. “Stream Corridor Restoration: Principles, Processes, and Practices.” FISRWG. 15 Federal agencies of the U.S. Government. https://nepis.epa.gov/Exe/ZyPURL.cgi?Dockey=50000Y7R.TXT

16. Interagency Committee on Water Resources, Subcommittee on Sedimentation. 1957. “A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams: Report No. 12. Some Fundamentals of Particle Size Analysis.” St. Anthony Falls Hydraulic Laboratory. Minneapolis, MN. https://water.usgs.gov/fisp/docs/Report_12.pdf

17. NRC. 1996. “Alluvial Fan Flooding.” National Academy Press. https://doi.org/10.17226/5364

18. NRCS. 2016. “Hydrologic Analyses of Post-Wildfire Conditions.” Title 210. Hydrology Technical Note No. 4. USDA. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=39877.wba

19. National Cooperative Highway Research Program (NCHRP). 2004. “Environmentally Sensitive Channel and Bank Protection Measures.” NCHRP Project 24-19. NCHRP. CRP-CD-58. https://doi.org/10.17226/13556

20. USBR. 1987. “Reservoir sedimentation. Design of Small Dams.” U.S. Bureau of Reclamation. Denver, CO. pp 529–564. 3rd Edition. https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf

21. USBR. 2006b. “Hydrology, Hydraulics, and Sediment Studies for the Matilija Dam Ecosystem Restoration Project, Ventura, CA.” U.S. Department of the Interior, Bureau of Reclamation Technical Service Center. Denver, CO. http://www.matilijadam.org/documents/Matilija%20H&H%20Design%20v2.pdf

22. USBR. 2015. “Bank Stabilization Design Guidelines.” Report No. SRH-2015-25. Bureau of Reclamation Technical Service Center, Sedimentation and River Hydraulics Group. Denver, CO. https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/A-BankStab-final6-25-2015.pdf

23. USDA. 1997. “Earth Spillway Erosion Model.” Chapter 51 in Part 628 Dams, National Engineering Handbook. Revised online version dated 2014. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=35547.wba

24. USDA. 2001. “Field Procedures Guide for the Headcut Erodibility Index.” Chapter 52 in Part 628 Dams, National Engineering Handbook. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=36147.wba

25. USDA. 2007b. “National Engineering Handbook, Part 654, Stream Restoration Design.” Natural Resources Conservation Service. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17807.wba

26. U.S. Environmental Protection Agency (USEPA)/USACE. 2007. “Identifying, planning, and financing beneficial use projects using dredged material: Beneficial use planning manual.” EPA 842-B-07-001. https://www.epa.gov/sites/production/files/2015-08/documents/identifying_planning_and_financing_beneficial_use_projects.pdf

27. USGS. 2005. “Distinguishing between Debris Flows and Floods from Field Evidence in Small Watersheds.” USGS. Fact Sheet 2004-3142. https://doi.org/10.3133/fs20043142

28. Water Resource Council. 1983. “Economic and Environmental Principles and Guidelines for Water Related Land Resources Implementation Studies.” https://planning.erdc.dren.mil/toolbox/library/Guidance/Principles_Guidelines.pdf

State and Local

1. Kansas Water Office. 2014. “Final Programmatic Environmental Impact Statement Prepared for the Removal and Disposal of Sediment and Restoration of Water Storage at John Redmond Reservoir, Kansas.” Kansas Water Office. Topeka, KS. www.swt.usace.army.mil/Portals/41/docs/library/john_redmond/fpeis_remove_dispose_sediment_jrr_ks.pdf

2. Orange County Water District. 2015. “Engineer’s Report for Prado Basin Sediment Management Demonstration Project.” Prepared by C. Olsen. www.ocwd.com/wp-content/uploads/wic13cpradosmdpengineersreportjanuary2015.pdf

3. Ventura County. 2016. “Matilija Dam Removal, Sediment Transport, and Robles Diversion Mitigation Project, Dam Removal Concepts Evaluation Report.” Prepared by AECOM and Stillwater Sciences for Ventura County Watershed Protection District. http://www.matilijadam.org/documents/2016-03%20Matilija%20Task%201.3%20Concepts%20Evaluation%20Report.pdf

Research Community

1. Abraham, D., J. Shelley, et al. 2016. “Verification of Bedload Transport Formula at the Sub-Cross-Section Scale.” Proceedings of the International Conference on Fluvial Hydraulics. July 11–14, 2016. River Flow. St. Louis, MO. CRC Press/Balkema. https://www.worldcat.org/title/river-flow-2016-proceedings-of-the-international-conference-on-fluvial-hydralics-river-flow-2016-st-louis-usa-11-14-july-2016/oclc/1003204237

2. Abt, S.R., S.M. Scurlock, C.I. Thornton, A.L. Cox, and N.J. Holste. 2016. “Bendway Weir Riprap Sizing Criteria.” ASCE Journal of Hydraulics. Vol. 142(12). https://ascelibrary.org/doi/full/10.1061/%28ASCE%29HY.1943-7900.0001209

3. Allen, H.H., and J.R. Leech. 1997. “Bioengineering for Streambank Erosion Control, Report 1, Guidelines.” EL-97-8. USACE. WES. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/9983

4. Allmendinger, N.E., J.E. Pizzuto, G.E. Moglen, M. Lewicki. 2007. “A sediment budget for an urbanizing watershed, 1951–1996, Montgomery County, Maryland, U.S.A.” Journal of the American Water Resources Association. Vol. 43(6). pp 1483–1498. https://onlinelibrary.wiley.com/doi/full/10.1111/j.1752-1688.2007.00122.x

5. Andrews, E.D., and G. Parker. 1987. “Formation of a Coarse Surface Layer as the Response to Gravel Mobility.” Sediment Transport in Gravel-bed Rivers. John Wiley & Sons. Chichester, Great Britain. pp 269–325. https://onlinelibrary.wiley.com/doi/abs/10.1002/esp.3290140809

6. Annandale, G.W., G.L. Morris, P. Karki. 2016. “Extending the Life of Reservoirs: Sustainable Sediment Management for Dams and Run-of-River Hydropower.” Directions in Development. D.C. World Bank. License: Creative Commons Attribution CC BY 3.0 IGO. https://elibrary.worldbank.org/doi/abs/10.1596/978-1-4648-0838-8

7. Baskaran, M., C. Miller, A. Kumar, E. Andersen, J. Hui, J.P. Selegean, C. Creech, and J. Barkach. 2015. “Sediment accumulation rates and sediment dynamics using five different methods in a well-constrained impoundment: Case study from Union Lake, MI.” Journal of Great Lakes Research, Vol 41(2). https://www.sciencedirect.com/science/article/pii/S0380133015000611

8. Bellmore J.R., J.J. Duda, L.S. Craig, S.L. Greene, C.E. Torgersen, M.J. Collins, and K. Vittum. 2017. "Status and trends of dam removal research in the United States." WIREs Water 2016. https://doi.org/10.1002/wat2.1164

9. Beuselinck, L., G. Govers, J. Poesen, G. Degraer, and L. Froyen. 1998. "Grain−size analysis by laser diffractometry: Comparison with the sieve−pipette method." Catena. Vol. 32(3−4). pp 193−208. https://doi.org/10.1016/S0341-8162(98)00051-4

10. Biedenharn, D.S., N.K. Raphelt, and C.C. Watson. 2000. "Demonstration Erosion Control Project Monitoring Program, Fiscal Year 1995 Report." ERDC/CHL TR-00-1. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/26535

11. Borrowman, T.D., E.R. Smith, J.Z. Gailani, and L. Caviness. 2006. "Erodibility Study of Passaic River Sediments Using USACE SEDflume." ERDC/CHL TR-06-7. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/8498

12. Brown, G.L., J.V. Letter, R.E. Heath, R. McAdory, L.L. Wehmeyer, and B.L. Gunkel. 2013. "A Simplified Analytic Investigation of the Riverside Effects of Sediment Diversions." ERDC/CHL CHETN-VII-13. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/2024

13. Brown, M.E., and N.C. Kraus. 2007. "Tips for developing bathymetry grids for Coastal Modeling System Applications." ERDC/CHL CHETN-IV-69. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/1978

14. Brownlie, W.R. 1983. "Flow Depth in Sand-Bed Channels." Journal of Hydraulic Engineering. ASCE. Vol. 109, No. 7. pp 959–990. https://doi.org/10.1061/(ASCE)0733-9429(1983)109:7(959)

15. Bruner, G., and S. Gibson. 2005. "Sediment Transport Modeling in HEC-RAS." Proceedings World Water and Environmental Resources Congress 2005. May 15–19, 2005. ASCE. Anchorage, AK. https://doi.org/10.1061/40792(173)442

16. Brunner, G.W. 2016. "HEC-RAS River Analysis System: Hydraulic Reference Manual, Version 5.0." Technical Report CPD-69. USACE Institute for Water Resources HEC. Davis, CA. https://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS%205.0%20Reference%20Manual.pdf

17. Bunte, K., and S.R. Abt. 2009. "Transport Relationships between Bedload Traps and a 3-inch Helley-Smith Sampler in Coarse Gravel-Bed Streams and Development of Adjustment Functions." p 138. Report submitted to the Federal Interagency Sedimentation Project. Engineering Research Center. Colorado State University. Fort Collins, CO. https://water.usgs.gov/fisp/docs/091221_Bunte_and_Abt_Trap_HS_comparison_and_adjustment_functions_final.pdf

18. Buurman, P., T. Pape, F. Reijneveld, and E. Gelder. 2001. "Laser-diffraction and pipette-method grain sizing of Dutch sediments: Correlations for fine fractions of marine, fluvial, and loess samples." Netherlands Journal of Geosciences. Vol. 80(2). Published online by Cambridge University Press 2016. https://doi.org/10.1017/S0016774600022319

19. Childers, D., 1999. "Field comparisons of six pressure difference bedload samplers in high-energy flow." USGS Water-Resources Investigations Report 92-4068. https://pubs.usgs.gov/wri/1992/4068/report.pdf

20. Daniels, M.D., and M.H. McCusker. 2010. "Operator bias characterizing stream substrates using Wolman pebble counts with a standard measurement template." Geomorphology. Vol. 113. pp 194–198. https://doi.org/10.1016/j.geomorph.2009.09.038

21. Davies, M., N. MacDonald, Z. Demirbilek, J. Smith, A. Zundel, and R. Jones. 2005. "Particle Tracking Model (PTM) in the SMS: II. Overview of features and capabilities." DOER Technical Note Collection. ERDC-TN-DOER-D5. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/8725

22. Davinroy, R.D., R.A. Gaines, D.C. Gordon, L.L. Hopkins, D.P. Barretta, J.L. Brown, M.T. Rodgers, E. Brauer, A.N. Cox, T. Lauth, and B.J. Krischel. 2011. "Hydraulic Sediment Response Modeling, Replication Accuracy to the River." Technical Paper M53. USACE St. Louis and Memphis Districts. https://www.mvs-wc.usace.army.mil/arec/Documents/Publications/M53_Hydraulic_Sediment_Response_Modeling_Replication_Accuracy.pdf

23. Davis, R.B. 1990. "The Scope of Quaternary Paleolimnology." In Paleolimnology and the Reconstruction of Ancient Environments. R.B. Davis (Ed.). Kluwer Academic Publishers. Boston, MA. pp 1–24. https://doi.org/10.1007/978-94-009-2655-4_1

24. Dean, R.G., J.T. Wells, J. Fernando, and P. Goodwin. 2012. "River Diversions: Principles, Processes, Challenges and Opportunities, A Guidance Document." https://www.mvd.usace.army.mil/Portals/52/docs/SandT/LCA/Dean%20and%20Wells%20Final%20Guidance%20Document.pdf

25. Dodge, R.A. 1983. "Model Similitude – Extended for Active Sediment Transport PAP-450." USBR Water Resources Research Laboratory. Denver, CO. https://www.usbr.gov/tsc/techreferences/hydraulics_lab/pubs/PAP/PAP-0450.pdf

26. Dorough, W.G., and J.R. Ried. 1991. "Shoreline Erosion Process, Lake Sakakaea." Section 15, pp 15.1–15.8. 5th Federal Interagency Sedimentation Conference. Las Vegas, NV. https://acwi.gov/sos/pubs/5thFISC/5Fisc-V2/5Fsc2-15.PDF

27. Dunne, T., and L.B. Leopold. 1978. "Water in environmental planning." W.H. Freeman Co. San Francisco, CA. https://www.worldcat.org/title/611295026

28. Edwards, T.K., and G.D. Glysson. 1988. "Field Methods for Measurement of Fluvial Sediment." USGS Open-File Report 86-531. Book 3, Chapter C2. P 89. Reston, VA. https://pubs.usgs.gov/of/1986/0531/report.pdf

29. Edwards, T.K., and G.D. Glysson. 1999. "Field Methods for Measurement of Fluvial Sediment." Book 3, Chapter C2 in Techniques of Water-Resources Investigation of the USGS. https://pubs.usgs.gov/twri/twri3-c2/

30. Einstein, H.A., and N. Chien. 1953. "Transport of Sediment Mixtures with Large Ranges of Grain Sizes." Missouri River Division Sediment Series No. 2. U.S. Army Engineer Division, Missouri River. Omaha, NE. https://usace.contentdm.oclc.org/digital/api/collection/p266001coll1/id/7591/download

31. Ettema, R., R. Arndt, P. Roberts, and T. Wahl. 2000a. "Hydraulic Modeling, Concepts and Practice, Manuals and Reports on Engineering Practice No. 97." ASCE. Reston, VA. https://ascelibrary.org/doi/book/10.1061/9780784404157

32. Ettema, R., F. Braileanu, and M. Muste. 2000b. "Methods for Estimating Sediment Transport in Ice-Covered Channels." Journal of Cold Regions Engineering. Vol. 14(3). pp 130–144. https://ascelibrary.org/doi/abs/10.1061/%28ASCE%290887-381X%282000%2914%3A3%28130%29

33. Ettema, R. 2008. "Ice Effects on Sediment Transport in Rivers." Chapter 13 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch13

34. Etzler, F.M., and M.S. Sanderson. 1995. "Particle Size Analysis: A comparative study of various methods." Particle and Particle Systems Characterization. Vol. 12. pp 217–224. https://onlinelibrary.wiley.com/doi/10.1002/ppsc.19950120503

35. Ferrick, M.G., and N.D. Mulherin. 1989. "Framework for control of dynamic ice breakup by river regulation." CRREL Report 89-12. USACE CRREL. Hanover, NH. https://erdc-library.erdc.dren.mil/jspui/bitstream/11681/9058/1/CRREL-89-12.pdf

36. Fischenich, C., and C. Little. 2007. "Sediment Sampling and Analysis for Stream Restoration Projects." ERDC-TN-EMRRP-SR-39. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/3972

37. Foltz, R.B., P.R. Robichaud, and H. Rhee. 2009. "A synthesis of post-fire road treatments for BAER teams: methods, treatment effectiveness, and decision-making tools for rehabilitation." General Technical Report RMRS-GTR-228. USDA Forest Service, Rocky Mountain Research Station. Fort Collins, CO. https://www.fs.usda.gov/treesearch/pubs/32967

38. Gao, P., X.M. Mu, F. Wang, and R. Li. 2011. "Changes in streamflow and sediment discharge and the response to human activities in the middle reaches of the Yellow River." Hydrology and Earth Systems Sciences. Vol. 15. pp 1–10. https://doi.org/10.5194/hess-15-1-2011

39. Garcia, M.H., et al. 2008a. "Sedimentation Engineering, Processes, Measurements, Modeling, and Practice." Prepared by the ASCE Task Committee to Expand and Update Manual 54 of the Sedimentation Committee of the Environmental and Water Resources Institute. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148

40. Garcia, M.H. 2008b. "Sediment Transport and Morphodynamics." Chapter 2 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://ascelibrary.org/doi/10.1061/9780784408148.ch02

41. Gibson, S., and P. Boyd. 2014. "Modeling Long Term Alternatives for Sustainable Sediment Management Using Operational Sediment Transport Rules." In: Reservoir Sedimentation. pp 229–236. CRC Press. https://www.taylorfrancis.com/chapters/edit/10.1201/b17397-31/modeling-long-term-alternatives-sustainable-sediment-management-using-operational-sediment-transport-rules-gibson-boyd?context=ubx&refId=0204c0fa-a001-4cd9-8880-fab9ac7e3138

42. Gibson, S., W. Lehman, M. Koohafkan, and D. Pridal. 2019. "Using HEC-WAT and HEC-RAS Sediment to Evaluate the Effect of Hydrologic Uncertainty on Bed Evolution." Proceedings of the Federal Interagency Sedimentation and Hydrologic Modeling Conference. Vol 2, Hydraulic and Sediment Transport Modeling. June 24–28, 2019. Reno, NV. https://www.sedhyd.org/past/SEDHYD_Proceedings_2019_Volume2r.pdf

43. Gomez, B., R.L. Naff, and D.W. Hubbell. 2006. "Temporal variation in bedload transport rates associated with the migration of bedforms." Earth Surface Processes and Landforms." Vol. 14. pp 135–156. https://doi.org/10.1002/esp.3290140205

44. Goode, J.R., C. Luce, and J.M. Buffington. 2010. "Enhanced Sediment Delivery to large basins in a Changing Climate: Implications for management and habitat." https://doi.org/10.1016/j.geomorph.2011.06.021

45. Gray, J.R., and J.M. Simões. 2008. "Estimating sediment discharge." Appendix D in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. pp 1067–1088. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.apd

46. Greimann, B. 2015. Analysis of Sediment Loads and Reservoir Sedimentation Volume, Lecture 3. USAID workshop on reservoir sedimentation, Vang Vieng, Laos. November 2015. (Online edition not available.)

47. Greimann, B., and C. Klumpp. 2000. "Numerical Modeling of Sediment Migration During Dam Removal." Joint Conference on Water Resource Engineering and Water Resources Planning and Management 2000. July 30–August 2, 2000. Minneapolis, MN. https://ascelibrary.org/doi/10.1061/40517%282000%29259

48. Grover, N.C., and C.S. Howard. 1938. "The passage of turbid water through Lake Mead." Transactions of ASCE. Vol. 103. pp 720–790. https://ascelibrary.org/doi/10.1061/TACEAT.0004942

49. Gucinski, H., M.J. Furniss, R.R. Ziemer, and M.H. Brookes. 2001. "Forest roads: A synthesis of scientific information." General Technical Report PNWGTR-509. p 103. USDA. Forest Service, Pacific Northwest Research Station. Portland, OR. https://www.fs.usda.gov/research/treesearch/7769

50. Guy, H.P. 1969. "Laboratory Theory and Methods for Sediment Analysis." USGS Techniques of Water-Resources Investigations. Book 5, Chapter C1. https://doi.org/10.3133/twri05C1

51. Hall, B.R., and T.L. Fagerburg. 1991. "Field Measurement of Hydrodynamics and Sediment Transport of the Mississippi River at Old River." Hydraulic Engineering Proceedings, National Conference of the Hydraulics Division. pp 847–852. ASCE. https://www.worldcat.org/title/23942043 or https://cedb.asce.org/CEDBsearch/record.jsp?dockey=0072732

52. Hanson G.J. 1990. "Surface erodibility of earthen channels at high stresses. Part II—developing an in-situ testing device." Transactions of the American Society of Agricultural Engineers. Vol. 33(1). pp 132–137. https://doi.org/10.13031/2013.31306

53. Hanson G.J. 1991. "Development of a jet index to characterize erosion resistance of soils in earthen spillways." Transactions of the American Society of Agricultural Engineers. Vol. 36(5). pp 2015–2352. https://doi.org/10.13031/2013.31831

54. Hanson G.J, and K.R. Cook. 1997. "Development of excess shear stress parameters for circular jet testing." Presented at the 1997 Annual ASAE International Meeting, Minneapolis MN, Aug 10-14, 1997. ASAE Paper No. 97–2227. St. Joseph, MI. https://www.bae.ksu.edu/watershed/other/jetworkshop/Hanson%20and%20Cook%201997.pdf

55. Hanson, G.J., and A. Simon. 2001. "Erodibility of cohesive streambeds in the loess area of the midwestern USA." Hydrological Processes. Vol. 15(1). pp 23–38. https://doi.org/10.1002/hyp.149

56. Haring, C.H., and D.W. Biedenharn. 2021. "Channel Assessment Tools for Rapid Watershed Assessment." ERDC/CHL CHETN-VII-24. USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/AD1128590.pdf

57. Haring, C.H., C.H. Theiling, and M.P. Dougherty. 2020. "Rapid Watershed Assessment Planning Tools Based on High-Resolution Terrain Analysis." ERDC/CHL CHETN-VII-22. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/40379

58. Hutter, K., B. Svendsen, and D. Rickenmann. 1994. "Debris flow modeling: A review." Continuum Mechanics and Thermodynamics 8. pp 1–35. https://link.springer.com/article/10.1007/BF01175749

59. Jansen, R.B. 1983. "Dams and Public Safety." U.S. Bureau of Reclamation. Denver, CO. https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/AZ1130.pdf

60. Jowett, I. 1984. "Sedimentation in New Zealand Hydroelectric Schemes." Water International. Vol. 9(4). https://doi.org/10.1080/02508068408686533

61. Juracek, K.E., and D.P. Mau. 2002. "Sediment deposition and occurrence of selected nutrients and other chemical constituents in bottom sediment, Tuttle Creek Lake, Northeast Kansas, 1962–1999." USGS Water Resources Investigation Report 2002-4048. P 73. https://pubs.er.usgs.gov/publication/wri024048

62. Juracek, K.E., and A.C. Ziegler. 2007. "Estimation of sediment sources using selected chemical tracers in the Perry Lake and Lake Wabaunsee Basins, Northeast Kansas." USGS. Scientific Investigations Report 2007-5020. P 53. https://doi.org/10.3133/sir20075020

63. Khanal, A., K.R. Klavaon, G.A. Fox, and E.R. Daly. 2015. "Comparison of Linear and Nonlinear Models for Cohesive Sediment Detachment: Rill Erosion, Hole Erosion Test, and Streambank Erosion Studies." Journal of Hydraulic Engineering. Vol. 142. No 9. https://doi.org/10.1061/(ASCE)HY.1943-7900.0001147

64. Knott, J.M., G.D. Glysson, B.A. Malo. And L.J. Schroeder. 1993. "Quality assurance plan for the collection and processing of sediment data by the U.S. Geological Survey, Water Resources Division." USGS Open-File Report 92-499. https://doi.org/10.3133/ofr92499

65. Kondolf, G.M., Y. Gao, G.W. Annandale, G.L. Morris, E. Jiang, R. Hotchkiss, P. Carling, B. Wu, J. Zhang, C. Peteuil, H.W. Wang, C. Yongtao, K. Fu, Q. Guo, T. Sumi, Z. Wang, Z. Wei, C. Wu, and C.T. Yang. 2014. "Sustainable sediment management in reservoirs and regulated rivers: experiences from five continents." Earth's Future. Vol. 2. AGU Publications. https://doi.org/10.1002/2013EF000184

66. Kondolf, G.M., and W.V.G. Matthews. 1991. "Management of coarse sediment in regulated rivers of California." Report No. 80. University of California Water Resources Center. Riverside, CA. https://escholarship.org/uc/item/5rw9k19j

67. Kuhnle, R.A. 1989. "Bed-Surface Size Changes in Gravel-Bed Channel." Journal of Hydraulic Engineering. Vol. 115, No. 6. pp 731–743. https://doi.org/10.1061/(ASCE)0733-9429(1989)115:6(731)

68. Landers, M.N., T.D. Straub, M.S. Wood, and M.M. Domanski. 2016. "Sediment acoustic index method for computing continuous suspended-sediment concentrations." USGS Techniques and Methods. Book 3, Chapter 5. p 63. http://dx.doi.org/10.3133/tm3C5

69. Leech, J.R., and D.S. Biedenharn. 2012. "Sediment Management at the Watershed Level." ERDC/CHL CHETN-VII-12. USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/ADA571746.pdf

70. Letter, J.V., C.F. Pinkard Jr., and N.A. Raphelt. 2008. "River diversions and shoaling." ERDC/CHL CHETN-VII-9. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/2036

71. Lu, N., G. Ristow, W. Likos. 2000. "The accuracy of hydrometer analysis for fine-grained particles." Geotechnical Testing Journal. Vol. 23, No. 4. https://doi.org/10.1520/GTJ11069J

72. Lyons, J., and L. Lest. 1996. "Theodore Roosevelt Reservoir 1995 Sedimentation survey." U.S. Bureau of Reclamation, Sedimentation and River Hydraulics Group Water Resources Services Technical Service Center. p 71. Denver, CO. https://www.usbr.gov/tsc/techreferences/reservoir/Theodore%20Roosevelt%20Reservoir%201995%20Sedimentation%20Survey.pdf

73. MacDonald, N.J., M.H. Davies, A.K. Zundel, J.D. Howlett, T.C. Lackey, Z. Demirbilek, and J.Z. Gailani. 2006. "PTM, Report 1: Model theory, implementation, and example applications." Coastal and Hydraulics Laboratory Technical Report. ERDC/CHL TR-06-20. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7695

74. Mahmood, K. 1980. "Verification of sediment transport functions, Missouri River." MRD Sediment Series No. 19. USACE, Missouri River Division. Omaha, NE. https://erdclibrary.on.worldcat.org/oclc/9212449

75. Martin, M., D. Biedenharn, C. Little, K. Smith, and C. Waterson. 2010. "Analysis of Three Delta Headwaters Project Streams Using the SIAM Model." Proceedings of the 2nd Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling. June 27–July 1, 2010. Las Vegas, NV. https://acwi.gov/sos/pubs/2ndJFIC/Contents/11C_Martin_3_1_10_paper-jmbpdf.pdf

76. McCully, D.W., and G.J. Mech. 1993. "Saylorville Spillway: Emergency to Service Spillway." Proceedings Geotechnical Practice in Dam Rehabilitation. ASCE. https://www.worldcat.org/title/27811417

77. McGee, R.G., R.F. Ballard, and D.D. Caulfield. 1995. "A Technique to Assess the Characteristics of Bottom and Sub-bottom Marine Sediments." Technical Report DRP-95-3. USACE WES. Vicksburg, MS. https://apps.dtic.mil/sti/citations/ADA295105

78. McNeil, J., C. Taylor, and W. Lick. 1996. "Measurements of erosion of undisturbed bottom sediments with depth." Journal of Hydraulic Engineering. ASCE. Vol. 122(6). pp 316–324. https://ascelibrary.org/doi/pdf/10.1061/(ASCE)0733-9429(1996)122%3A6(316)

79. Mehta, A. J., and W.H. McAnally. "Fine-Grained Sediment Transport." Chapter 4 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch04

80. Miller, C. 2001. "Use of Global Positioning and Geographic Information Systems in the Huntington District Reservoir Sedimentation Investigative Program." Proceedings of the 7th Federal Interagency Sedimentation Conference. March 25–29, 2001. Vol. 2, Section IX Reservoirs. pp IX-1–IX-8. Reno, NV. https://acwi.gov/sos/pubs/7thFISC/7Fisc-V2/7FISC2-9.PDF

81. Miller, M.E., L.H. MacDonald, P.R. Robichaud, and W.J. Elliot. 2011. "Predicting post-fire hillslope erosion in forest lands of the western United States." International Journal of Wildland Fire 20. pp 982–999. https://doi.org/10.1071/WF09142

82. Montgomery, D.R., and J.M. Buffington. 1993. "Channel classification, prediction of channel response, and assessment of channel condition." Washington State Department of Natural Resources Report TFW-SH10-93-002. Olympia, WA. https://www.dnr.wa.gov/publications/fp_tfw_sh10_93_002.pdf

83. Moody, J.A., and D.A. Martin, 2001. "Post-Fire, Rainfall Intensity-Peak Discharge Relations for Three Mountainous Watersheds in the Western USA." Hydrological Processes. Vol. 15. pp 2981–2993. https://doi.org/10.1002/hyp.386

84. Morris, G.L. 2016. "Sedimentation Dynamics of Reservoirs." Presented at Conowingo Infill Influence on Chesapeake Bay. Chesapeake Research Consortium Workshop. Annapolis MD. http://www.chesapeake.org/stac/presentations/249_Morris%20Conowingo.pdf

85. Nairn, R.J., and J.P. Selegean. 2014. "Sediment management at St. Joseph River and Harbor, Michigan, USA, 1837–Present." Proceedings, PIANC 33rd World Congress Conference. San Francisco, CA. https://www.proceedings.com/25524.html

86. Nakato, T. 1990. "Tests of selected sediment-transport formulas." Journal of Hydraulic Engineering, ASCE. Vol. 116, No. 3. https://doi.org/10.1061/(ASCE)0733-9429(1990)116:3(362)

87. Newbury, R.W. 2013. "Designing fish-passable riffles as gradient controls in Canadian streams." Canadian Water Resources Journal. Vol. 38(3). pp 232–250. https://doi.org/10.1080/07011784.2013.803742

88. Nittrouer, J.A., M.A. Allison, and R. Campanella. 2008. "Bedform transport rates for the lowermost Mississippi River." Journal of Geophysical Research. Vol. 113. https://doi.org/10.1029/2007JF000795

89. O'Brien, J.S., and P.Y. Julien. 1985. "Physical properties and mechanics of hyperconcentrated sediment flows." Proceedings, Specialty Conference on Delineation of Landslides, Flash Flood, and Debris Flow Hazards in Utah, June 14-15, 1984. D. Bowles (Ed.). pp 260–279. Utah Water Research Laboratory. Utah State University. Logan, UT. https://www.engr.colostate.edu/~pierre/ce_old/Projects/Paperspdf/O%27Brien-Julien%20UtahPDF.pdf

90. Pierson, T.C. 2005. "Hyperconcentrated flow – transitional process between water flow and debris flow." Chapter 8 in Debris-flow Hazards and Related Phenomena. Springer Praxis Publishing. UK. DOI: 10.1007/3-540-27129-5_8. https://link.springer.com/chapter/10.1007/3-540-27129-5_8 **Error! Hyperlink reference not valid.**

91. Piest, R.F., and A.J. Bowie. 1975. "Gully and Streambank Erosion." Proceedings of the 29th Annual Meeting. Soil Conservation Society of America. https://www.ars.usda.gov/ARSUserFiles/50701000/cswq-t6428-piest.pdf

92. Pinson, A., B. Baker, P. Boyd, R. Grandpre, K.D. White, and M. Jonas. 2016. "USACE Reservoir Sedimentation in the Context of Climate Change." Civil Works Technical Report. CWTS 2016-05. USACE. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/5202

93. Pitlick, J., Y. Cui, and P. Wilcock. 2009. "Manual for Computing Bed Load Transport Using BAGS (Bedload Assessment for Gravel-bed Streams) Software." Rocky Mountain Research Station, General Technical Report. RMRS-GTR-223. p 50. USDA Forest Service. https://doi.org/10.2737/RMRS-GTR-223

94. Podolak, C.J.P., and M.W. Doyle. 2015. "Reservoir Sedimentation and Storage Capacity in the United States: Management Needs for the 21st Century." ASCE Journal of Hydrologic Engineering. Vol. 141(4). p 5. Reston, VA. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000999

95. Raphelt, N.K., and M.P. Alexander. 2001. "Multidimensional models used in rivers and streams." ERDC/CHL CHETN-VII-2, USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/2026

96. Raphelt, K.N., S. Tingle, R.E. Heath, and Y. Chu. 2001. "Development of methodology for predicting fate of dredged material placed on riverbanks." ERDC TN-DOER-N14. USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/ADA398396.pdf

97. Richardson, E.V., D.B. Simons, and P.Y. Julien. 1990. “Highways in the River Environment Participant Book.” FHWA-HI-90-016. Federal Highway Administration, U.S. Department of Transportation. https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf

98. Richardson, J.R., and E.V. Richardson. 2008. “Bridge Scour Evaluation.” Chapter 10 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch10

99. Roberts, J., and R. Jepsen. 2001. “Separation of bedload and suspended load with modified high shear stress flume.” SAND2001–2162. Sandia National Laboratories. Albuquerque, NM. https://doi.org/10.2172/789249

100. Rosgen, D.L. 1996. “Applied river morphology.” Wildland Hydrology. Pagosa Springs, CO. https://wildlandhydrology.com/books/?id=32&course=Applied+River+Morphology

101. Rosgen, D.L. 2001. “A Practical Method of Computing Streambank Erosion Rate.” Proceedings of the 7th Federal Interagency Sedimentation Conference. March 25–29, 2001. Section II, Stream Restoration, Vol. 1, pp II-9–II-15. Reno, NV. https://acwi.gov/sos/pubs/7thFISC/7Fisc-V1/7FISC1-2.pdf

102. Ryan, S.E., K.A. Dwire, and M.K. Dixon. 2011. “Impacts of wildfire on runoff and sediment loads at Little Granite Creek, western Wyoming.” Geomorphology. Vol. 129. pp 113–130. http://dx.doi.org/10.1016/j.geomorph.2011.01.017

103. Schumm, S.A., and M.D. Harvey. 2008. “Engineering Geomorphology.” Chapter 18 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://doi.org/10.1061/9780784408148.ch18

104. Service, R.F. 2011. “Will Busting Dams boost Salmon?” Science. Vol. 332. pp 888–892. https://doi.org/10.1126/science.334.6058.888

105. Shelley, J., M. Boyer, J. Granet, and A. Williams. 2016. “Environmental Benefits of Restoring Sediment Continuity to the Kansas River.” ERDC/CHL CHETN-XIV-50, USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/AD1011945.pdf

106. Simon, A. 1989. “A model of channel response in disturbed alluvial channels.” Earth Surface Processes and Landforms. Vol. 14. pp 11–26. https://doi.org/10.1002/esp.3290140103

107. Simon, A., and C.R. Hupp. 1986. “Channel Evolution in Modified Tennessee Channels.” Proceedings of the 4th Federal Interagency Sedimentation Conference. Section 5, Channel Adjustments, Vol. 2. pp 5.71–5.82. https://acwi.gov/sos/pubs/4thFISC/4Fisc-V2/4Fisc2-5.PDF

108. Simon, A., M. Rinaldi, and G. Hadish. 1996. “Channel evolution in the loess area of the midwestern United States.” Proceeding of the 6th Federal Interagency Sedimentation Conference. Vol. 1, Section III Fluvial: Channel Evolution and Channel Stabilization, pp 3-86–3-93. Las Vegas, NV. https://acwi.gov/sos/pubs/6thFISC/6Fisc-V1/6Fisc1-3.PDF

109. Simons, D.B., and F. Sentürk. 1992. “Sediment Transport Technology: Water and Sediment Dynamics.” Water Resources Publications, ISBN 0-918334-66-7. https://www.wrpllc.com/books/stt.html

110. Smith, H.G., and D. Dragovitch. 2009. “Interpreting sediment delivery processes using suspended sediment-discharge hysteresis patterns from nested upland catchments, south-eastern Australia.” Hydrological Processes. Vol. 23. pp 2415–2426. http://dx.doi.org/10.1002/hyp.7357

111. Smith, S.J., and C.T. Friedrichs. 2011. “Size and settling velocities of cohesive flocs and suspended sediment aggregates in a trailing suction hopper dredge plume.” Proceedings of the 9th International Conference on Nearshore and Estuarine Cohesive Sediment Transport Processes. Continental Shelf Research. Vol. 31(10) Supplement. pp S50–S63. Elsevier. Amsterdam, Netherlands. http://dx.doi.org/10.1016/j.csr.2010.04.002

112. Soar, P.J., and C.R. Thorne. 2001. “Channel Restoration Design for Meandering Rivers.” ERDC/CHL CR-01-1. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/1800

113. Spasojevic, M., and F.M. Holly. 1994. “Three-dimensional numerical simulation of mobile-bed hydrodynamics.” Contract Report HL-94-2. USACE WES. Vicksburg, MS. http://hdl.handle.net/11681/6159

114. Spasojevic, M., and F.M. Holly. 2008. “Two- and Three-Dimensional Numerical Simulation of Mobile-Bed Hydrodynamics and Sedimentation.” Chapter 15 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://ascelibrary.org/doi/10.1061/9780784408148.ch15

115. Stevens, H.H., and C.T. Yang. 1989. “Summary and use of selected fluvial sediment-discharge formulas.” USGS. Water Resources Investigations Report 89-4026. https://doi.org/10.3133/wri894026

116. Sutherland, D.G., M.H. Ball, S.J. Hilton, and T.E. Lisle. 2002. “Evolution of a landslide-induced sediment wave in the Navarro River, California.” Geologic Society of America Bulletin 114(8). https://doi.org/10.1130/0016-7606(2002)114%3C1036:EOALIS%3E2.0.CO;2

117. Tananaev, N.I. 2015. “Hysteresis effects of suspended sediment transport in relation to geomorphic conditions and dominant sediment sources in medium and large rivers of the Russian Arctic.” Hydrology Research. Vol. 46(2). pp 232–243. http://dx.doi.org/10.2166/nh.2013.199

118. Thomas, W.A., R.R. Copeland, and D.N. McComas. 2002. “SAM – Hydraulic Design Package for Channels.” USACE ERDC. Vicksburg, MS. http://www.ayresprojectinfo.com/Web_SAMwin/Manual/Chapter-1.pdf

119. Thorne, C.R. 1993. “Guidelines for the Use of Stream Reconnaissance Record Sheets in the Field.” Contract Report HL-93-2. USACE WES. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/6180

120. Thorne, C.R. 1998. “Stream reconnaissance handbook.” John Wiley & Sons. Chichester, U.K. https://www.worldcat.org/title/757297629

121. Tomer, M.D., and K.E. Schilling. 2009. “A simple Approach to Distinguish Land-use and Climate-change Effects on Watershed Hydrology.” Journal of Hydrology. Vol. 376. pp 24–33. https://doi.org/10.1016/j.jhydrol.2009.07.029

122. Toro-Escobar, C.E., G. Parker, and C. Paola. 1996. “Transfer Function for the Deposition of Poorly Sorted Gravel in Response to Streambed Aggradation.” Journal of Hydraulic Research. Vol. 34(1). pp 35–53. https://doi.org/10.1080/00221689609498763

123. Trimble, S.W. 1997. “Contribution of stream channel erosion to sediment yield from an urbanizing watershed.” Science. 278(5342). pp 1442–1444. https://doi.org/10.1126/science.278.5342.1442

124. Tuthill, A.M. 1999. “Flow Control to Manage River Ice.” CRREL Special Report 99-8. p 25. https://apps.dtic.mil/sti/pdfs/ADA366402.pdf

125. Tuthill, A.M., and K.D. White. 2005. “Ice Engineering: Methods to Evaluate Potential for Ice Impacts on Sediment Stability.” ERDC/CRREL TN 05-1. USACE CRREL. Hanover, NH. http://hdl.handle.net/11681/2598

126. Tuthill, A.M., J.L. Wuebben, S.F. Daly, and K.D. White. 1996. “Probability Distributions for Peak Stages on Rivers Affected by Ice Jams.” ASCE. Journal of Cold Regions Engineering. Vol. 10(1). pp 36–57. https://doi.org/10.1061/(ASCE)0887-381X(1996)10:1(36)

127. Ward, D., R. Starner, and B. Vaage. 2016. “Effects of Turbidity on Predation Vulnerability of Juvenile Humpback Chub to Rainbow Trout and Brown Trout.” Journal of Fish and Wildlife Management. Vol. 7(1). pp 205–212. https://doi.org/10.3996/102015-JFWM-101

128. White, K. 1999. “Hydraulic and Physical Properties Affecting Ice Jams.” CRREL Report 99-11. P 30. https://apps.dtic.mil/sti/pdfs/ADA375289.pdf

129. White, K.D., and J.N. Moore. 2002. "Impacts of Dam Removal on Riverine Ice Regime." Journal of Cold Regions Engineering. Vol. 16(1). pp 2–16. https://doi.org/10.1061/(ASCE)0887-381X(2002)16:1(2)

130. Wilcox, P.R., and B.W. McArdell. 1993. "Surface-Based Fractional Transport Rates: Mobilization Thresholds and Partial Transport of a Sand-Gravel Sediment." Water Resources Research. Vol. 29, No. 4. pp 1297–1312. https://doi.org/10.1029/92WR02748

131. Williams, D.T., and P.Y. Julien. 1989. "Examination of Stage-Discharge Relationships of Compound/Composite Channels." Proceedings of the International Conference on Channel Flow and Catchment Runoff. B.C. Yen (Ed.). May 22–26, 1989. pp 478–488. University of Virginia. Charlottesville, VA. https://books.google.com/books/about/Proceedings_of_the_International_Confere.html?id=3H1jSwAACAAJ

132. Williams G.P., and M.G. Wolman. 1984. "Downstream Effects of Dams on Alluvial Rivers." Professional Paper 1286. U.S. Department of the Interior. USGS. https://pubs.usgs.gov/pp/1286/report.pdf

133. Wilson, C.G., A.N. Papanicolaou, K.D. Denn, and B. Abban. 2014. "Quantifying Sediment Sources to the Suspended Load of an Agricultural Stream Using Radioisotopes." World Environmental and Water Resources Congress 2014. Water Without Borders. https://doi.org/10.1061/9780784413548.125

134. Woelfle-Erskine, C., A.C. Wilcox, and J.N. Moore. 2012. "Combining historical and process perspectives to infer ranges of geomorphic variability and inform river restoration in a wandering gravel-bed river." Earth Surface Processes and Landforms. Vol. 37. pp 1302–1312. https://doi.org/10.1002/esp.3276

135. Yoo, K., R. Amundson, A.M. Meimsath, and W.E. Dietrich. 2005. "Process-based model linking pocket gopher (Thomomys bottae) activity to sediment transport and soil thickness." Geology. Vol. 33(11). pp 917–920. https://pubs.geoscienceworld.org/gsa/geology/article/33/11/917/29575/Process-based-model-linking-pocket-gopher-Thomomys

136. Yuzyk, T.R. 1986. "Bed material sampling in gravel-bed streams, sediment survey section, environment Canada." IWD-HQ-WRB-SS-86-8. Ottawa, Ontario. (Online version not available.)

A-3. Related Publications.

Public Law

1. Public Law 93-241. Waterways Experiment Station (WES) 1981. Final Report to Congress. Main Report: "The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32." USACE WES. Vicksburg. MS. https://usace.contentdm.oclc.org/digital/collection/p16021coll5/id/1236/

2. Public Law 84-99. Emergency Response to Natural Disasters. https://www.usace.army.mil/Missions/Emergency-Operations/pl-84-99/

USACE Publications

1. DA Pam 25-403. "Guide to Recordkeeping in the Army." https://armypubs.army.mil/epubs/DR_pubs/DR_a/ARN38438-PAM_25-403-002-WEB-3.pdf

2. EM 1110-2-1204. USACE 1989a. "Environmental Engineering for Coastal Shore Protection." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1204.pdf

3. EM 1110-2-1205. USACE 1989b. "Environmental Engineering for Flood Control Channels." https://www.publications.usace.army.mil/portals/76/publications/engineermanuals/em_1110-2-1205.pdf

4. EM 1110-2-1419. USACE 1995d. "Hydrologic Engineering Requirements for Flood Damage Reduction Studies." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1419.pdf?ver=2013-09-04-070800-500

5. EM 1110-2-1607. USACE 1991b. "Tidal Hydraulics." www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1607.pdf

6. EM 1110-2-4000. USACE 1995f. "Sedimentation Investigations of Rivers and Reservoirs." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-4000.pdf?ver=RLatBSC9b1xYJqckcOHkQQ%3d%3d

7. ER 1110-2-1405. USACE 1982a. "Hydraulic Design for Local Flood Protection Projects." https://www.publications.usace.army.mil/portals/76/publications/engineerregulations/er_1110-2-1405.pdf

8. ER 1110-2-4001. USACE 1981a. "Engineering and Design, Notes on Sedimentation Activities." https://www.publications.usace.army.mil/Portals/76/Publications/EngineerRegulations/ER_1110-2-4001.pdf

9. ETL 1110-2-120. USACE 1971a. "Additional Guidance for Riprap Channel Protection." (Online version not available.)

10. HEC. 1992. "Guidelines for the Calibration and Application of Computer Program HEC-6." Training Document No. 13. HEC. Davis, CA. https://www.hec.usace.army.mil/publications/TrainingDocuments/TD-13.pdf

11. HEC. 1993b. "HEC 6 Scour and Deposition in Rivers and Reservoirs: User Manual." HEC. Davis, CA. https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HEC-6_UsersManual_(CPD-6).pdf

12. HEC. 2016d. "HEC-SSP Statistical Software Package, User's Manual." USACE Institute for Water Resources. HEC. Davis, CA. https://www.hec.usace.army.mil/software/hec-ssp/documentation/HEC-SSP_21_Release_Notes.pdf

13. USACE 1971b. "Relocation of Niobrara, NE, and Acquisition of Niobrara State Park, Gavins Point Dam, Lewis and Clark Lake, Omaha District (June 1970)." Design Memorandum MG-91. (Online version not available.)

14. USACE 1998. "HEC-1 Flood Hydrograph Package, User's Manual." USACE HEC. Davis, CA. https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HEC-1_UsersManual_(CPD-1a).pdf

15. USACE 1982b. "Report on Standardization of Riprap Gradations." (Revised). USACE Lower Mississippi Valley Division. Vicksburg, MS. (Online version not available.)

16. USACE 1988a. "Muddy Creek Grade Control Structure, Muddy Creek, Mississippi and Tennessee." Technical Report HL-88-11. USACE WES. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/bitstream/11681/13535/1/TR-HL-88-11.pdf

17. USACE 1988b. "Stable Riprap Size for Open Channel Flows." HL-88-4. Stephen Maynord (Ed.). USACE WES. Vicksburg, MS. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/6852/

18. USACE 1990a. "HEC-2 Water Surface Profiles, User's Manual." USACE HEC. Davis, CA. https://www.hec.usace.army.mil/publications/ComputerProgramDocumentation/HEC-2_UsersManual_(CPD-2a).pdf

19. USACE 1990b. “Phase I Sediment Engineering Investigation of the Caliente Creek Drainage Basin.” PR-13. USACE HEC. Davis, CA. https://www.hec.usace.army.mil/publications/ProjectReports/PR-13.pdf

20. USACE 1990c. “Yatesville Lake: Big Sandy River Basin Blaine Creek, Kentucky Foundation Report.” Vol. 1. West Virginia. USACE Huntington District. https://apps.dtic.mil/sti/pdfs/ADA240111.pdf

21. USACE 1991a. “Soil Conservation Service Low Drop Structure Model Study.” Technical Report HL-91-13. USACE WES. Vicksburg, MS. https://hdl.handle.net/11681/13383

22. USACE 1995a. “Application of Methods and Models for Prediction of Land Surface Erosion and Yield.” Training Document No. 36 (TD-36). USACE HEC. Davis, CA. https://apps.dtic.mil/sti/pdfs/ADA294130.pdf

23. USACE 2003a. “F-1 Channel and Debris Basin and F-2 Channel and Debris Basin, Las Vegas Wash and Tributaries (Tropicana and Flamingo Washes).” Design Documentation Report. Los Angeles District. (Online version not available.)

24. USACE 2003b. “Upper Mississippi River System Flow Frequency Study – Hydrology and Hydraulics, Appendix F Missouri River.” USACE Omaha District. Omaha, NE. https://www.mvr.usace.army.mil/Portals/48/docs/FRM/UpperMissFlowFreq/App.%20F%20Omaha%20Dist.%20Hydrology_Hydraulics%20Report.pdf

25. USACE 2004b. “Matilija Dam Ecosystem Restoration Feasibility Study Final EIS.” USACE Los Angeles District. https://www.matilijadam.org/reports/finaleis/Final%20EIS-EIR%2009-04.pdf

26. USACE 2008b. “Yatesville Lake: Dam Outlet Works and Spillway.” Periodic Inspection Report No. 8. West Virginia. USACE Huntington District. https://erdclibrary.on.worldcat.org/oclc/613344602

27. USACE 2011b. “Missouri River Commercial Dredging.” Final Environmental Impact Statement. Prepared by Cardno Entrix. USACE Kansas City District. Kansas City, MO. https://usace.contentdm.oclc.org/digital/collection/p16021coll7/id/8163/

28. USACE 2012b. “Final Environmental Assessment, Sediment Retention Structure (SRS) Spillway Raise Project, Mount St. Helens Sediment Management for Flood Risk Reduction.” USACE Portland District. https://usace.contentdm.oclc.org/digital/collection/p16021coll7/id/8454/

29. USACE 2012c. “Guidelines for Solving Two-Dimensional Shallow Water Problems with the Adaptive Hydraulics Modeling System.” USACE ERDC. Vicksburg, MS. (Online version not available.)

30. USACE 2012d. "Missouri River Stage Trends." Technical Report. Missouri River Basin Water Management Division. Omaha, NE. https://usace.contentdm.oclc.org/digital/collection/p266001coll1/id/5714/

31. USACE 2012e. "Sediment Management at the Watershed Level." ERDC/CHL CHETN-VII-12. USACE ERDC. Vicksburg MS.. https://apps.dtic.mil/sti/pdfs/ADA571746.pdf

32. USACE 2015c. "American River Watershed, Common Features General Reevaluation Report, Final Environmental Impact Statement, Environmental Impact Report." Sacramento District. https://hdl.handle.net/11681/39339

33. USACE 2015f. "Missouri River Bed and Water Surface Changes between 2009 and 2014 as They Relate to Renewal of Commercial Dredging Permits Under Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act (Section 404/10 Permits)." Memorandum for CENWK-OR-R. USACE Kansas City District. (Online version not available.)

34. USACE 2019. "Best Practices in Dam and Levee Safety Risk Analysis." Chapter D-1 in Erosion of Rock and Soil. Version 4.1. Joint publication by U.S. Department of the Interior Bureau of Reclamation and USACE. https://www.usbr.gov/damsafety/risk/BestPractices/Chapters/D1-ErosionOfRockAndSoil.pdf

35. USACE-NWW. 2014. "Lower Snake River Programmatic Sediment Management Plan, Final Environmental Impact Statement." Appendix F – Hydrology and Hydraulics. Lower Granite Reservoir Sedimentation Analysis and Lewiston Levee Flood Risk Analysis. Walla Walla, WA. https://usace.contentdm.oclc.org/digital/collection/p16021coll7/id/9789

Other Federal Agency and Private Sector Publications

1. American Association of State Highway Transportation Officials (AASHTO). 1998. "Standard Specifications for Transportation Materials and Methods of Sampling and Testing." Part 1 – Specifications. Reference AASHTO Method M 145 (Refer to AASHTO online store for current version). https://store.transportation.org/Item/PublicationDetail?ID=457

2. FHWA 2014. "Hydraulic Toolbox Desktop Reference Guide." Version 4.2. https://www.fhwa.dot.gov/engineering/hydraulics/software/toolbox404.cfm

3. FISP. U.S. SAH-97 Analyzer (Gravelometer). http://water.usgs.gov/fisp/products/4112002.html

4. NCHRP. 2010. "Estimation of Scour Depth at Bridge Abutments." NCHRP Project 24-20. Transportation Research Board. National Academy of Science. (R. Ettema, T. Nakato, and M. Muste). https://onlinepubs.trb.org/onlinepubs/nchrp/docs/NCHRP24-20_FR.pdf

5. NRC. 1982. “Selecting a methodology for delineating mudslide hazard areas for the National Flood Insurance Program.” Committee on Methodologies for Predicting Mudflow Areas. National Academy Press. https://www.nap.edu/catalog/19560/selecting-a-methodology-for-delineating-mudslide-hazard-areas-for-the-national-flood-insurance-program

6. NRCS. 2010. “2010 Natural Resources Inventory.” Summary Report. USDA. https://www.eesi.org/files/stelprdb1167354.pdf

7. National Weather Service (NWS). 1977. “Probable Maximum Precipitation Estimates.” Colorado River and Great Basin Drainages. Hydrometeorological Report No. 49. NWS. Silver Spring, MD. https://www.weather.gov/media/owp/hdsc_documents/PMP/HMR49.pdf

8. Office of Water Quality. 2001. “Collection and Use of Total Suspended Solids Data.” Technical Memorandum 2001.03. https://water.usgs.gov/water-resources/memos/memo.php?id=723

9. Pacific Southwest Interagency Committee (PSIAC). 1968. “Factors Affecting Sediment Yield in the Pacific Southwest Area and Selection and Evaluation of Measures for Reduction of Erosion and Sediment Yield.” Report No. HY 12. ASCE Water Management Subcommittee of ASCE. P 13. https://books.google.com/books/about/Report_of_the_Water_Management_Subcommit.html?id=_nasHAAACAAJ

10. USBR. 2011. “West-Wide Climate Risk Assessments: Bias-Corrected and Spatially Downscaled Surface Water Projections.” Technical Memorandum No. 86-68210-2011-01. U.S. Department of the Interior. Bureau of Reclamation Technical Services Center. Denver, CO. https://www.usbr.gov/watersmart/docs/west-wide-climate-risk-assessments.pdf

11. USBR. 2013. “Downscaled CMIP3 and CMIP5 Climate and Hydrology Projections: Release of Downscaled CMIP5 Climate Projections, Comparison with Preceding Information, and Summary of User Needs.” P 47. U.S. Department of the Interior. Bureau of Reclamation Technical Services Center. Denver, CO. https://gdo-dcp.ucllnl.org/downscaled_cmip_projections/dcpInterface.html

12. USDA. 1951. “Soil Survey Manual.” USDA Agricultural Handbook No. 18. U.S. Government Printing Office. Online version dated 2017. https://www.nrcs.usda.gov/resources/guides-and-instructions/soil-survey-manual

13. USDA. 1976. “Hydraulic Design of Riprap Gradient Control Structure.” Technical Release No. 59 and Amendment 1, Soil Conservation Service. U.S. Department of Agriculture Soil Conservation Service Engineering Division. https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=18633.wba

14. USDA 1993. “Repairing Flood Damaged Fields.” Technical Notes. Agronomy #22. USDA Soil Conservation Service. https://efotg.sc.egov.usda.gov/references/public/IA/AgronomyTECHnote22.pdf

15. USDA. 1996. “SITES: Water resource site analysis computer program user’s guide.” NRCS, USDA. https://www.ars.usda.gov/research/publications/publication/?seqNo115=72810

16. USDA-ARS. 2010. “SWAT format climate data for the U.S. January 1, 1950–December 31, 2010.” http://www.ars.usda.gov/Research/docs.htm?docid=19388

17. U.S. Global Change Research Program. 2014. “National Climate Assessment.” http://nca2014.globalchange.gov/

18. USGS. 1993. “Nationwide summary of U.S. Geological Survey regional regression equations for estimating magnitude and frequency of floods for ungaged sites.” Water-Resources Investigations Report 94-4002. https://doi.org/10.3133/wri944002

19. USGS. 1988. “Flood-carrying capacities and changes in channels of the lower Puyallup, White, and Carbon Rivers in western Washington.” Water-Resources Investigations Report 87-4129. P 43. https://pubs.usgs.gov/wri/1987/4129/report.pdf

20. USGS. 2001. “Natural Hazards on Alluvial Fans: The Venezuela Debris Flow and Flash Flood Disaster.” USGS Fact Sheet FS 103 01. https://pubs.usgs.gov/fs/fs-0103-01/fs-0103-01.pdf

21. USGS. 2002. “Water-Quality Field Methods Phaseout of US D-77 and Frame-type Samplers.” Office of Water Quality Technical Memorandum 2002.09. https://water.usgs.gov/water-resources/memos/memo.php?id=2237

22. USGS. 2010. “Channel-conveyance capacity, channel change, and sediment transport in the lower Puyallup, White, and Carbon Rivers, western Washington.” USGS Scientific Investigations Report 2010–5240. P 104. https://doi.org/10.3133/sir20105240

23. USGS. 2012. “Geomorphic analysis of the river response to sedimentation downstream of Mount Rainier, Washington.” USGS Open-File Report 2012-1242. P 134. https://doi.org/10.3133/ofr20121242

24. USGS. 2018. “Suspended Sediment Transport by the Green-Duwamish River to the Lower Duwamish Waterway.” 2013–2017. Seattle, WA. https://doi.org/10.3133/ofr20181029

State and Local

1. Colorado Department of Transportation. 2001. “Calculation of Bridge Scour Using the Erodibility Index Method.” Report No. CDOT-DTD-R-2000-9. (G. Annandale and S. Smith). https://expresslanes.codot.gov/programs/research/pdfs/2000/bridgepierscour.pdf

Research Community

1. Abraham, D., T. Pratt, M. Ramos-Villanueva, N. Ganesh, D. May, W. Butler, T. McAlpin, K. Jones, J. Shelley, and D. Pridal D. 2017. "Sediment and hydraulic measurements with computed bed load on the Missouri River, Sioux City to Hermann, 2014." ERDC-CHL TR 17-8. USACE ERDC. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/AD1037455.pdf

2. Abraham, D., J. Shelley, T. McAlpin, D. May, and J. Adenihun. 2016. "Verification of Bedload Transport Formula at the Sub-Cross-Section Scale." Proceedings of the International Conference on Fluvial Hydraulics (River Flow 2016). July 11–14, 2016. St. Louis, MO. CRC Press/Balkema. https://www.worldcat.org/title/river-flow-2016-proceedings-of-the-international-conference-on-fluvial-hydralics-river-flow-2016-st-louis-usa-11-14-july-2016/oclc/1003204237

3. Abramovich, R., M. Molnau, and C. Craine. 1998. "Climates of Idaho." University of Idaho. Cooperative Extension System. College of Agriculture Bulletin 784. https://digital.lib.uidaho.edu/utils/getfile/collection/ui_ep/id/27117.pdf?_ga=2.187910699.2029620236.1680797878-664333347.1680797877

4. Andrews, E.D. 1980. "Effective and bankfull discharge of streams in the Yampa basin, western Wyoming." Journal of Hydrology. Vol. 46. pp 311–330. https://pubs.er.usgs.gov/publication/70012250

5. Annandale, G.W. 1985. "Forecasting sediment distribution in reservoirs during unstable conditions." Journal of Hydrology. Vol. 80. pp 361–370. https://www.sciencedirect.com/science/article/pii/0022169485901283

6. Annandale, G.W. 2006. "Scour Technology – Mechanics and Engineering Practice." McGraw-Hill Civil Engineering Series. 1st Edition. P 430. https://www.worldcat.org/title/scour-technology-mechanics-and-engineering-practice/oclc/62282516

7. Bagnold, R.A. 1954. "Experiments on a gravity-free dispersion of large solid spheres in a Newtonian fluid under shear." Proceedings of the Royal Society (London). Series A. Vol. 225. Issue 1160. pp 49–63. https://royalsocietypublishing.org/doi/10.1098/rspa.1954.0186

8. Barton, N., R. Lien, and J. Lunde. 1974. "Engineering classification of rock masses for the design of tunnel support." Rock Mechanics and Rock Engineering. Vol. 6(4). pp 189–236. https://doi.org/10.1007/BF01239496

9. Basson, G.R., and A. Rooseboom. 1999. "Dealing with Reservoir Sedimentation – Dredging." Water Research Commission, South Africa. http://www.wrc.org.za/wp-content/uploads/mdocs/TT-110-99.pdf

10. Benson, M.A., and D.M. Thomas. 1966. "A definition of dominant discharge." Bulletin of the International Association of Scientific Hydrology. Vol. XI. pp 76–80. https://doi.org/10.1080/02626666609493460

11. Biedenharn, D.S., and R.R. Copeland. 2000. "Effective Discharge Calculation." ERDC/CHL CHETN-VIII-4. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/2042

12. Biedenharn, D.S., R.R. Copeland, C.R. Thorne, P.J. Soar, R.D. Hey, and C.C. Watson. 2000. "Effective Discharge Calculation: A Practical Guide." ERDC\CHL TR-00-15. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7464

13. Biedenharn, D.S., and L.C. Hubbard. 2001. "Design Considerations for Siting Grade Control Structures." ERDC/CHL CHETN-VII-3. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/2027

14. Biedenharn, D.S., and C.R. Thorne. 1994. "Magnitude-frequency analysis of sediment transport in the Lower Mississippi River." Regulated Rivers: Research and Management. Vol. 9. pp 237–251. https://doi.org/10.1002/rrr.3450090405

15. Blaisdell, F.W., L.A. Clayton, and G.G. Hebaus. 1981. "Ultimate dimension of local scour." Journal of the Hydraulics Division. ASCE 107(HY 3). pp 327–337. https://doi.org/10.1061/JYCEAJ.0005630

16. Bollaert, E., 2002. "Transient Water Pressures in Joints and Formation Rock Scour due to High-Velocity Jet Impact." Communication 13. Laboratoire de Constructions Hydrauliques Ecole Polytechnique Federale de Lausanne. Switzerland. https://infoscience.epfl.ch/record/116162?ln=en

17. Borland, W.M., and C.R. Miller. 1958. "Distribution of sediment in large reservoirs." Journal of the Hydraulic Engineering Division. ASCE. Vol. 84(2). pp 1–18. https://ascelibrary.org/doi/10.1061/TACEAT.0007776

18. Bracmort, K.S., M. Arabi, J.R. Frankenberger, B.A. Engle, and J.G. Arnold. 2006. "Modeling long-term water quality impact of structural BMPs." Transactions of the ASABE. Vol. 49(2). pp 367–374. https://elibrary.asabe.org/abstract.asp?aid=20411

19. Bray, D.I. 1972. "Generalized regime-type analysis of Alberta rivers." Ph.D. Dissertation. Department of Civil Engineering. University of Alberta Press. Edmonton, Alberta. https://doi.org/10.7939/R34F1MN8S

20. Brochu, R., and R. Laprise. 2007. "Surface water and energy budgets over the Mississippi and Columbia River basins as simulated by two generations of the Canadian regional climate model." Atmosphere-Ocean. Vol. 45:1. pp 19–25. https://www.tandfonline.com/doi/pdf/10.3137/ao.v450102?needAccess=true

21. Brown, A.V., M.M. Lyttle, and K.B. Brown. 1998. "Impacts of gravel mining on gravel bed streams." Transaction of the American Fisheries Society. Vol. 127. pp 979–994. https://doi.org/10.1577/1548-8659(1998)127<0979:IOGMOG>2.0.CO;2

22. Brown, C.B. 1943. "Discussion of Sedimentation in Reservoirs." By J. Witzig. Proceedings of the ASCE. Vol. 69. pp 1493–1500. (Online version not available.)

23. Brummer, C.J., and D.R. Montgomery. 2003. "Downstream coarsening in headwater channels." Water Resources Research. Vol. 39(10). P 1294. https://agupubs.onlinelibrary.wiley.com/doi/10.1029/2003WR001981

24. Bunte, K., S.R. Abt, J.P. Potyondy, and K.W. Swingle. 2008. "A Comparison of Coarse Bedload Transport Measured with Bedload Traps and Helley-Smith Samplers." Geodinamica Acta. Vol. 21(1-2). pp 53–66. https://www.tandfonline.com/doi/abs/10.3166/ga.21.53-66

25. Bunte, K., S.R. Abt, J.P. Potyondy, and K.W. Swingle. 2009. "Comparison of Three Pebble Count Protocols (EMAP, PIBO, and SFT) in Two Mountain Gravel-Bed Streams." JAWRA. Vol. 45(5). pp 1209–1227. https://onlinelibrary.wiley.com/doi/full/10.1111/j.1752-1688.2009.00355.x

26. Bunte, K., K. Swingle, and S.R. Abt. 2007. "Guidelines for using bedload traps in coarse-bedded mountain streams: construction, installation, operation, and sample processing." General Technical Report RMRS-GTR-191. P 91. USDA Forest Service. Rocky Mountain Research Station. Fort Collins, CO. https://www.fs.usda.gov/rm/pubs/rmrs_gtr191.pdf

27. Burrell, G.N. 1951. "Constant-Factor Method Aids Computation of Reservoir Sediment." Civil Engineering. Vol. 21. pp 51–52. (Online version not available.)

28. Buscombe, D. 2013. "Transferable wavelet method for grain-size distribution from images of sediment surfaces and thin sections, and other natural granular patterns." Sedimentology. Vol. 60(7). pp 1709–1732. https://onlinelibrary.wiley.com/doi/abs/10.1111/sed.12049

29. Buscombe, D., and D.M. Rubin. 2012. "Advances in the simulation and automated measurement of well-sorted granular material: 1. Simulation." Journal of Geophysical Research. Earth Surface. Vol. 117(F2). https://agupubs.onlinelibrary.wiley.com/doi/full/10.1029/2011JF001974

30. Carling, P.A. 1988. "The concept of dominant discharge applied to two gravel-bed streams in relation to channel stability thresholds." Earth Surface Processes and Landforms. Vol. 13. pp 355–367. https://onlinelibrary.wiley.com/doi/abs/10.1002/esp.3290130407

31. Chang, H.H. 1988. "Fluvial Processes in River Engineering." John Wiley & Sons. New York and other cities. Citing S. Fortier and F.C. Scobey's "Permissible canal velocities." (1926). Transactions of the ASCE. Vol. 89. pp 940–984. https://www.worldcat.org/title/fluvial-processes-in-river-engineering/oclc/16405050

32. Cheng, N-S. 1997. “Simplified Settling Velocity Formula for Sediment Particle.” Journal of Hydrologic Engineering. Vol. 123. https://doi.org/10.1061/(ASCE)0733-9429(1997)123:2(149)

33. Cohn, T.A., L.L. DeLong, E.J. Gilroy, R.M. Hirsch, and D.K. Wells. 1989. “Estimating constituent loads.” Water Resources Research. Vol. 25(5). pp 937–942. https://agupubs.onlinelibrary.wiley.com/doi/abs/10.1029/WR025i005p00937

34. Collins, B.D., and T. Dunne. 1989. “Gravel Transport, Gravel Harvesting, and Channel-bed Degradation in Rivers Draining the Southern Olympic Mountains, Washington, U.S.A.” Environmental Geology and Water Sciences. Vol. 13, Issue 3. pp 213–224. https://link.springer.com/article/10.1007/BF01665371

35. Copeland, R.R. 1995. “Sediment impact assessment.” Carlsbad, NM. Miscellaneous Paper HL-95-7. USACE WES. Vicksburg, MS. https://hdl.handle.net/11681/10805

36. Copeland, R.R., D.C. McVan, and S.E. Stonestreet. 2000. “Sedimentation study and flume investigation, Mission Creek, Santa Barbara; California and Corte Madera Creek, Marin County, California.” ERDC/CHL-00-05. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7469

37. Dawdy, D.R., and V.A. Vanoni. 1986. “Modeling Alluvial Channels.” Water Resources Research. Vol. 22, No. 9. pp 71s–81s. https://doi.org/10.1029/WR022i09Sp0071S

38. Deere, D.U., and D.W. Deere. 1989. “Rock Quality Designation (RQD) after twenty years.” CR GL-89-1. USACE WES. Vicksburg, MS. https://hdl.handle.net/11681/6143

39. Delworth, T.L. et al. 2006. “GFDL’s CM2 global coupled climate models part 1: formulation and simulation characteristics.” Journal of Climate. Vol. 19. pp 643–674. https://journals.ametsoc.org/view/journals/clim/19/5/jcli3629.1.xml

40. Eakin, H.M (ref. C.B. Brown). 1936. “Silting of Reservoirs.” USDA Technical Bulletin 524. P 168, revised 1939. https://ageconsearch.umn.edu/bitstream/164938/2/tb524.pdf **Error! Hyperlink reference not valid.**

41. Einstein, H.A., and N. Chien. 1954. “Similarity of Distorted River Models with Movable Beds.” Transactions of ASCE. https://ascelibrary.org/doi/10.1061/TACEAT.0007300

42. Elliot, W.J. 2013. “Erosion Processes and Prediction with WEPP Technology in Forests in the Northwestern U.S.” https://www.fs.usda.gov/research/treesearch/43831

43. Emmett, W.W. 1975. “The channels and waters of the upper Salmon River, Idaho.” Professional Paper 870A. USGS. https://pubs.usgs.gov/pp/0870a/report.pdf

44. Ferguson, R.I. 1986. "River loads underestimated by rating curves." Water Resources Research. Vol. 22(1). pp 74–76. https://agupubs.onlinelibrary.wiley.com/doi/epdf/10.1029/WR022i001p00074

45. Fischenich, J.C., H.H. Allen, and D.N. McComas. 2001. "Plant Material Acquisition, Layout, and Handling for Flood Control Projects." ERDC TR-01-14. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/8538

46. Forsythe, P. 1985. "Performance of a grade control structure system during extreme floods." American Society of Agricultural Engineers Winter Meeting. Chicago, IL. (Online version paper summarized in annotated bibliography by F. Neilson, T. Waller, and K. Kennedy, USACE Miscellaneous Paper HL-91-4) https://apps.dtic.mil/dtic/tr/fulltext/u2/a241356.pdf

47. Geiger, A.F. 1963. "Developing sediment storage requirements for upstream retarding reservoirs." Proceedings of the Federal Interagency Sediment Conference. USDA-ARS Miscellaneous Publication 970. USDA. pp 881–885. https://acwi.gov/sos/pubs/2ndFISC/2Fisc-4.PDF

48. Gesch, D., G. Evans, J. Mauck, J. Hutchinson, and W.J. Carswell Jr. 2009. "The National Map—Elevation." USGS Fact Sheet 2009-3053. P 4. https://pubs.usgs.gov/fs/2009/3053/

49. Gibson, C.E. 2006. "A Northern Rocky Mountain polygon fire history: Accuracy, limitations, strengths, applications, and recommended protocol of digital fire perimeter data." M.S. Thesis. University of Idaho. https://www.researchgate.net/publication/241801487_a_northern_rocky_mountainpolygon_fire_history_accuracy_limitations_strengths_applications_and_recommended_protocol_of_digital_fire_perimeter_data

50. Gibson, C.E., and P. Morgan. 2009. "Atlas of digital polygon fire extents for Idaho and western Montana (1889–2003)." [Online] USDA. Forest Service, Rocky Mountain Research Station. Fort Collins, CO. https://doi.org/10.2737/RDS-2009-0006-2

51. Gibson, S., C. Nygaard, and P. Sclafani. 2010. "Mobile Bed Modeling of the Cowlitz River Using HEC-RAS: Assessing Flood Risk and Impact Due to System Sediment." Proceedings of the 2nd Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling. June 27–July 1, 2010. Las Vegas, NV. https://acwi.gov/sos/pubs/2ndJFIC/Contents/6C_Gibson_Nygaard_Modeling_of_Cowlitz_paper.pdf

52. Gibson, S., J. Shelley, and C. Cai. 2016. "Downstream Coarsening on the Missouri River." Proceedings from the International Conference On Fluvial Hydraulics. River Flow. https://www.researchgate.net/publication/311507977_Downstream_coarsening_on_the_Missouri_river

53. Gibson, S., A. Simon, E. Langendoen, N. Bankhead, and J. Shelley. 2015. "A Physically-Based Channel-Modeling Framework Integrating HEC-RAS Sediment Transport Capabilities and the USDA-ARS Bank Stability and Toe-Erosion Model (BSTEM)." Federal Interagency Sediment Conference. SedHyd Proceedings. https://acwi.gov/sos/pubs/3rdJFIC/Contents/1A-Gibson.pdf

54. Goitom, T.G., and M.E. Zeller. 1989. "Design procedures for soil-cement grade control structures." Proceedings of the National Conference on Hydraulic Engineering. ASCE. New Orleans, LA. https://cedb.asce.org/CEDBsearch/record.jsp?dockey=0062687

55. Goodrich, D.C., H.E. Canfield, I.S. Burns, D.J. Semmens, S.N. Miller, M. Hernandez, L.R. Levick, D.P. Guertin, and W.G. Kepner. 2005. "Rapid Post-Fire Hydrologic Watershed Assessment using the AGWA GIS-based Hydrologic Modeling Tool." Managing Watersheds for Human and Natural Impacts: Engineering, Ecological, and Economic Challenges. ASCE Conference Proceedings. Williamsburg, VA. https://ascelibrary.org/doi/10.1061/40763%28178%2944

56. Google Maps. http://www.googlemaps.com

57. Gray, D.H., and R.B. Sotir. 1996. "Biotechnical and soil bioengineering: a practical guide for erosion control." John Wiley & Sons. New York. https://www.wiley.com/en-us/Biotechnical+and+Soil+Bioengineering+Slope+Stabilization%3A+A+Practical+Guide+for+Erosion+Control-p-9780471049784

58. Gray, J., D. Glysson, and T. Edwards. 2008. "Suspended Sediment Samplers and Sampling Methods." Chapter 5 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. pp 320–339. https://doi.org/10.1061/9780784408148.ch05

59. Gray, J.R., J.B. Laronne, and J.D.G. Marr. 2010. "Bedload surrogate monitoring technologies." USGS Scientific Investigations Report 2010–5091. https://doi.org/10.3133/sir20105091

60. Grigg, N., C. McCarthy, B. Lawrence, and D. Ockerman. 2012. "Review of the Regulation of the Missouri River Mainstem Reservoir System During the Flood of 2011." Technical Review Panel, Northwest Division. USACE. Omaha, NE. https://www.nwd-mr.usace.army.mil/rcc/MRFTF/docs/MRIndependentReviewPanel.pdf

61. Guy, H.P., and V.W. Norman. 1982. "Field Methods for Measurement of Fluvial Sediment." Book 3, Chapter C2 "Applications of Hydraulics" of USGS Techniques of Water-Resources Investigations. https://pubs.usgs.gov/twri/twri3-c2/

62. Haan, C.T., B.J. Barfield, and J.C. Hayes. 1994. "Design hydrology and sedimentology for small catchments." Academic Press. San Diego, CA. https://www.sciencedirect.com/book/9780123123404/design-hydrology-and-sedimentology-for-small-catchments

63. Hanson, G. J., and K.R. Cook. 2004. "Apparatus, test procedures and analytical methods to measure soil erodibility in situ." Applied Engineering in Agriculture. Vol. 20(4). pp 455–462. https://doi.org/10.13031/2013.16492

64. Heath, R.E., G.L. Brown, C.D. Little, T.C. Pratt, J.J. Ratcliff, D.D. Abraham, D. Perkey, N.B. Ganesh, K. Martin, and D.P. May. 2015. "Old River Control Complex sedimentation investigation." ERDC/CHL TR-15-8. USACE ERDC. Vicksburg, MS. https://hdl.handle.net/11681/7729

65. Heinemann, H.G. 1984. "Reservoir Trap Efficiency" from Erosion and Sediment Yield: Some Methods of Measurement and Modelling. R.F. Hadley and D.E. Walling (Eds.). GeoBooks. Norwich, England. https://www.ars.usda.gov/ARSUserFiles/50701000/cswq-t5318-rausch.pdf

66. Heinemann, H.G., and V.I. Dvorak. 1965. "Improved Volumetric Survey and Computation Procedures for Small Reservoirs." Proceedings from the Federal Inter-Agency Sedimentation Conference (1963). pp 845–856. Prepared by the USDA Agricultural Research Service. https://acwi.gov/sos/pubs/2ndFISC/2Fisc-4.PDF

67. Hendrickson, J.S., and F.R. Haase. 1994. "Hydrodynamic Conditions in the Black River Delta/Lake Onalaska Area, Pool 7, Upper Mississippi River 1980-81 and 1991-92." USACE St. Paul District. https://apps.dtic.mil/sti/citations/ADA301631

68. Hey, R.D., and C.R. Thome. 1986. "Stable channels with mobile gravel beds." Journal of Hydraulic Engineering. Vol. 112(8). pp 671–689. https://doi.org/10.1061/(ASCE)0733-9429(1986)112:8(671)

69. Hey, R.D., and C.R. Thorne. 1983. "Accuracy of surface samples from gravel-bed material." Journal of Hydraulic Engineering. Vol. 109. pp 842–851. https://doi.org/10.1061/(ASCE)0733-9429(1983)109:6(842)

70. Hite, J.W., and G.A. Pickering. 1982. "South Fork Tillatoba Creek drop structures, Mississippi; Hydraulic model investigation." Technical Report HL-82-22. USACE WES. Vicksburg, MS. https://apps.dtic.mil/sti/citations/ADA121274

71. Hobbs, B.L. 1969. "Forecasting Distribution of Sediment Deposits in Large Reservoirs." Appendix I. ETL 1110-2-64. Department of the Army, Office of the Chief of Engineers. (Online version not available.)

72. Hotchkiss, R.H., S.F. Jorgensen, M.C. Stone, T.A. Fontaine. 2000. "Regulated River Modeling for Climate Change Impact Assessment: The Missouri River." Journal of the American Water Resources Association. Vol. 36(2). pp 375–386. https://doi.org/10.1111/j.1752-1688.2000.tb04275.x

73. Hupp, C.R., and E.E. Morris. 1990. "A Dendrogeomorphic Approach to Measurement of Sedimentation in a Forested Wetland, Black Swamp, Arkansas." Wetlands. Vol. 10, No. 1. http://dx.doi.org/10.1007/BF03160827

74. Inglis, C.C. 1947. "Meanders and their bearing on river training." Institution of Civil Engineers. Maritime and Waterways. Paper No. 7. https://doi.org/10.1680/idivp.1947.13075

75. Iowa Climate Change Advisory Council. 2010. "Climate Change Impacts on Iowa 2010." Report to the Governor and General Assembly. http://publications.iowa.gov/id/eprint/14167

76. Kellerhals, R. 1967. "Stable channels with gravel-paved beds." Journal of the Waterways and Harbours Division. ASCE Proceedings. Vol. 93. pp 63–84. https://ascelibrary.org/doi/10.1061/JWHEAU.0000482

77. Keulegan, G.H. 1938. "Laws of turbulent flow in open channels." Research Paper. https://nvlpubs.nist.gov/nistpubs/jres/21/jresv21n6p707_A1b.pdf

78. King County. 2017. "Lower White River 2009–2016 In-Channel Sediment Monitoring." Prepared by Terry Butler, Fred Lott, and Chris Brummer. King County Department of Natural Resources and Parks. Water and Land Resources Division. Seattle, WA. https://your.kingcounty.gov/dnrp/library/water-and-land/flooding/white_river/PRB/11%20lower-white-r-sediment-monitoring-tech-memo-2017.pdf

79. Kinoshita, A.M., T.S. Hogue, and C. Napper. 2013. "A Guide for Pre- and Postfire Modeling and Application In the Western United States." JAWRA. Vol. 50(6). https://doi.org/10.1111/jawr.12226

80. Knighton, A.D. 1984. "Fluvial forms and processes." Arnold. London, England. https://doi.org/10.1002/esp.3290110319

81. Kondolf, G.M. 1994. "Geomorphic and environmental effects of instream gravel mining." Landscape Urban Planning. Issue 28. pp 225–243. https://doi.org/10.1016/0169-2046(94)90010-8

82. Kondolf, G.M., T.E. Lisle, and G.M. Wolman. 2003. "Bed sediment measurement." Tools in Fluvial Geomorphology. pp 347–395. John Wiley & Sons. Chichester, England. https://doi.org/10.1002/0470868333.ch13

83. Kouwen, N., R.M. Li, and D.B. Simons. 1980. “A stability criteria for vegetated Waterways.” Proceedings from the International Symposium on Urban Storm Runoff. July 28–31, 1980. pp 203–210. University of Kentucky. Lexington, KY. https://books.google.com/books/about/International_Symposium_on_Urban_Storm_R.html?id=2KK91Pftio0C

84. Krumbein, W.C. 1941. “Measurement and geological significance of shape and roundness of sedimentary particles.” Journal of Sedimentary Research. Vol. 11, No. 2. pp 64–72. https://pubs.geoscienceworld.org/sepm/jsedres/article/11/2/64/94958/Measurement-and-geological-significance-of-shape

85. Langland, M.J., J.D. Blomquist, D.L. Moyer, K.E. Hyer, and J.G. Chanat. 2013. “Total nutrient and sediment loads, trends, yields, and nontidal water-quality indicators for selected nontidal stations.” Chesapeake Bay Watershed, 1985–2011. USGS Open-File Report 2013-1052. p 51. https://doi.org/10.3133/ofr20131052

86. Lara, J.M. 1962. “Revision of the Procedure to Compute Sediment Distribution in Large Reservoirs.” U.S. Bureau of Reclamation. Denver, CO. https://www.worldcat.org/title/revision-of-the-procedure-to-compute-sediment-distribution-in-large-reservoirs/oclc/6972775

87. Lara, J.M., and E.L. Pemberton. 1963. “Initial Unit Weight of Deposited Sediments.” Proceedings of the Federal Interagency Sedimentation Conference. USDA Agricultural Research Service (USDA-ARS) Miscellaneous. Publication No. 970. pp 818–845. https://acwi.gov/sos/pubs/2ndFISC/2Fisc-4.PDF

88. Laursen, E.M. 1958. “The Total Sediment Load of Streams.” Journal of the Hydraulics Division. ASCE. Vol. 84, No. HY 1. pp 1530-1–1530-36. https://doi.org/10.1061/JYCEAJ.0000158

89. Leopold, L.B., and M.G. Wolman. 1957. “River channel patterns – braided, meandering and straight.” Professional Paper 282B. USGS. https://doi.org/10.3133/pp282B

90. Leopold, L.B., M.G. Wolman, and J.P. Miller. 1964. “Fluvial processes in geomorphology.” W.H. Freeman & Company. San Francisco, CA. https://pubs.er.usgs.gov/publication/70185663

91. Leopold, L.B. 1994. “A view of the river.” Harvard University Press. Cambridge, UK. https://www.worldcat.org/title/view-of-the-river/oclc/28889034

92. Littell, J.S., and R. Gwozdz. 2011. “Climatic water balance and regional fire years in the Pacific Northwest, USA: linking regional climate and fire at landscape scales.” In: McKenzie, C.M., and D. Falk (Eds), The Landscape Ecology of Fire. Ecological Studies. Vol. 213. pp 117–139. Springer Science+Business Media. https://doi.org/10.1007/978-94-007-0301-8_5

93. Littell J.S., D. McKenzie, D.L. Peterson, and A.L. Westerling. 2009. “Climate and wildfire area burned in western U.S. ecoprovinces (1916–2003).” Ecological Applications. Vol. 19. pp 1003–1021. https://doi.org/10.1890/07-1183.1

94. Lyons, J.K., M.J. Pucherelli, and R.C. Clark. 1992. “Sediment transport and channel characteristics of a sand-bed portion of the Green River below Flaming Gorge Dam, Utah USA.” Regulated Rivers: Research and Management. Vol. 7. pp 219–232. https://doi.org/10.1002/rrr.3450070302

95. MacArthur, R.C., D. Hamilton, and D.M. Gee. 1995. “Application of methods and models of prediction of land surface erosion and yield.” Training Document No. 36. USACE-HEC. p 132. https://apps.dtic.mil/sti/pdfs/ADA294130.pdf

96. Mackin, J.H. 1948. “Concept of the graded river.” Geologic Society of America. Bulletin 59. pp 463–512. https://doi.org/10.1130/0016-7606(1948)59[463:COTGR]2.0.CO;2

97. Madden, E.B. 1993. “Modified Laursen Method for Estimating Bed Material Sediment Load.” Contract Report HL-93-3. Flood Control Channels Research Program. USACE WES. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/6179

98. Major, J.M., and T.C. Pierson. 1992. “Debris flow rheology: experimental analysis of fine-grained slurries.” Water Resources Research. Vol. 28(3). pp 841–857. https://doi.org/10.1029/91WR02834

99. Manning, R. 1891. “On the flow of water in open channels.” Transactions. Institution of Civil Engineers of Ireland. Vol. 20. pp 161–207. Dublin. Supplement Vol. 24. pp 179–207. 1895. http://sources.nli.ie/Record/PS_UR_067903

100. Maurer, E.P., and H.G. Hidalgo. 2008. “Utility of Daily vs. Monthly Large-Scale Climate Data: An Intercomparison of Two Statistical Downscaling Methods.” Hydrology & Earth System Sciences. Vol. 12(2). pp 551–563. https://doi.org/10.5194/hess-12-551-2008

101. Maurer, E.P., A.W. Wood, J.C. Adam, D.P. Lettenmaier, and B. Nijssen. 2002. “A Long-Term Hydrologically Based Data Set of Land Surface Fluxes and States for the Conterminous United States.” Journal of Climate. Vol. 15(22). pp 3237–3251. https://doi.org/10.1175/1520-0442(2002)015%3C3237:ALTHBD%3E2.0.CO;2

102. Maynord. S.T. 1987. “Stable Riprap Size for Open Channel Flows.” Ph.D. dissertation. Colorado State University. Fort Collins, CO. https://erdc-library.erdc.dren.mil/jspui/handle/11681/13288

103. McComas, D.N., and R.R. Copeland. 1996. “Sediment Impact Assessment for Navigation Channel Maintenance, Alabama River, Alabama, and Apalachicola River, Florida.” Miscellaneous Paper HL-96-1. USACE WES. Vicksburg, MS. https://apps.dtic.mil/sti/pdfs/ADA311211.pdf

104. Mearns, L.O., et al. 2007. (Updated 2011). “The North American Regional Climate Change Assessment Program Dataset.” National Center for Atmospheric Research Earth System Grid data portal. Boulder, CO. Data downloaded 21 August 2012. https://www.earthsystemgrid.org/project/narccap.html

105. Miller, J.F., R.H. Frederick, and R.J. Tracey. 1973. “Precipitation-frequency atlas of the western United States.” NOAA Atlas 2. Vol. 7, Nevada. NWS. Silver Spring, MD. https://repository.library.noaa.gov/view/noaa/28188

106. Miller, J.D., and H.D. Safford. 2012. “Trends in Wildfire Severity: 1984 to 2010 in the Sierra Nevada, Modoc Plateau, and Southern Cascades, California, USA.” Fire Ecology. Vol. 8. pp 41–57. https://doi.org/10.4996/fireecology.0803041

107. Mohammadzadeh-Habili, J., and M. Heidarpour. 2010. “New empirical method for prediction of sediment distribution in reservoirs.” Journal of Hydrologic Engineering. Vol. 15, No. 10. pp 813–821. https://doi.org/10.1061/(ASCE)HE.1943-5584.0000259

108. Morgan, P., E.K. Heyerdahl, and C.E. Gibson. 2008. “Multi-season Climate Synchronized Forest Fires Throughout the 20th Century, Northern Rockies, USA.” Ecology. Vol. 89(3). pp 717–728. https://www.fs.usda.gov/treesearch/pubs/30689

109. Moriasi, D.N., J.G. Arnold, M.W. Van Liew, R.L. Bingner, R.D. Harmel, and T.L. Veith. 2007. “Model Evaluation Guidelines for Systematic Quantification of Accuracy in Watershed Simulations.” Transactions of the ASABE. Vol. 50(3). pp 885−900. https://doi.org/10.13031/2013.23153

110. Morris, G.L., T.A. Dahl, M. Ramos-Villanueva, J.R. Leech, and M.M. Jonas. 2023. “Sustainable Sediment Management at Corps of Engineers Reservoirs.” ERDC/CHL TR-23-2. USACE ERDC. Vicksburg, MS. http://dx.doi.org/10.21079/11681/46470

111. Moustafa, A., and A. Moussa. 2013. “Predicting the deposition in the Aswan High Dam Reservoir using a 2-D model.” Ain Shams Engineering Journal, Vol 4(2). Creative Commons Attribution license (CC BY-NC-ND 3.0). https://doi.org/10.1016/j.asej.2012.08.004.

112. Nakicenovic, N. and R. Swart (Eds.). 2000. “Special report on emissions scenarios.” A special report of Working Group III of the Intergovernmental Panel on Climate Change. Cambridge University Press. Cambridge, U.K. and New York, NY. http://pure.iiasa.ac.at/id/eprint/6101/

113. Neitsch, S., J. Arnold, J.R. Kiniry, and J. Williams. 2011. “Soil and water assessment tool theoretical documentation version 2009.” Texas Water Resource Institute. https://hdl.handle.net/1969.1/128050

114. Nelson, J.M., R.R. McDonald, P.J. Kinzel, and Y. Shimizu. 2012. “Using computational modeling of river flow with remotely sensed data to Infer Channel Bathymetry.” ISBN 978-0-415-62129-8. River Flow. CRC Press. Taylor & Francis Group. London, England. https://pubs.er.usgs.gov/publication/70189363

115. Nelson, M.E., and P.C. Benedict. 1951. “Measurement and Analysis of Suspended Sediment Loads in Streams.” Transactions. ASCE. Vol. 116. Paper No. 2450. pp 891–918. https://doi.org/10.1061/TACEAT.0006548

116. Nixon, M. 1959. “A study of bankfull discharges of rivers in England and Wales.” Proceedings of the Institution of Civil Engineers. Vol. 12. pp 157–175. https://doi.org/10.1680/iicep.1959.12123

117. Nolan, K.M., T.E. Lisle, and H.M. Kelsy. 1987. “Bankfull discharge and sediment transport in northwestern California.” Proceedings of the Corvallis Symposium. Erosion and Sedimentation in the Pacific Rim. International Association of Hydrological Sciences. Publication No. 65. https://www.fs.usda.gov/treesearch/pubs/7887

118. Norman, J.N. 1975. “Design of stable channels with flexible linings.” Hydraulic Engineering Circular 15. U.S. Department of Transportation. Federal Highway Administration. https://ntrl.ntis.gov/NTRL/dashboard/searchResults/titleDetail/PB86184835.xhtml

119. Parisien, M.A., S. Snetsinger, J.A. Greenberg, C.R. Nelson, T. Schoennagel, S.Z. Dobrowski, and M.A. Moritz. 2012. “Spatial variability in wildfire probability across the western United States.” International Journal of Wildland Fire (early online publication). https://doi.org/10.1071/WF11044

120. Parker, G. 1990. “Surface-based bedload transport relation for gravel rivers.” Journal of Hydraulic Research. Vol. 28(4). pp 417–436. https://doi.org/10.1080/00221689009499058

121. Parker, G., Y. Fukushima, and H.M. Pantin. 1986. “Self-accelerating turbidity currents.” Journal of Fluid Mechanics. Vol. 171. pp 145–181. https://doi.org/10.1017/S0022112086001404

122. Pemberton, E.L. 1972. “Einstein’s Bed Load Function Applied to Channel Design and Degradation.” Chapter 16 in Sediment Symposium to Honor Professor H.A. Einstein. H.W. Shen (Ed.). https://books.google.com/books/about/Sedimentation_Symposium_to_Honor_Profess.html?id=bSZAAAAAIAAJ

123. Perlea, V., D. Mathews, and F. Walberg. 2010. “Modeling Erosion of an Unlined Spillway Chute Cut in Rock.” International Conference on Scour and Erosion. ASCE. https://doi.org/10.1061/41147(392)51

124. Pickup, G., and R.F. Warner. 1976. “Effects of hydrologic regime on magnitude and frequency of dominant discharge.” Journal of Hydrology. Vol. 29. pp 51–75. https://doi.org/10.1016/0022-1694(76)90005-6

125. Pizzuto, J.E. 2008. “Streambank Erosion and River Width Adjustment.” Chapter 7 in Sedimentation Engineering, Processes, Measurements, Modeling, and Practice. ASCE Manuals and Reports on Engineering Practice No. 110. Online edition available dated 2013. ASCE. Reston, VA. https://books.google.com/books/about/Sedimentation_Engineering.html?id=1AsypwBUa_wC

126. Pryor, S.C., D. Scavia, C. Downer, M. Gaden, L. Iverson, R. Nordstrom, J. Patz, and G.P. Robertson. 2014. “Chapter 18: Midwest. Climate Change Impacts in the United States: The Third National Climate Assessment.” U.S. Global Change Research Program. pp 418–440. https://www.fs.usda.gov/research/treesearch/47989

127. Rahmanian, M.R., and M.A. Banihashemi. 2012. “Characterization of Sedimentation Pattern in Reservoirs.” Canadian Journal of Civil Engineering. Vol. 39. pp 951–956. https://doi.org/10.1139/l2012-079

128. Ran, D.C., Q.H. Luo, B. Liu, and H. Wang. 2004. “Effect of soil-retaining dams on flood and sediment reduction in middle reaches of Yellow River.” Journal of Hydraulic Engineering. Vol. 5. pp 7–12. https://www.researchgate.net/publication/285528643_Effect_of_soil-retaining_dams_on_flood_and_sediment_reduction_in_middle_reaches_of_Yellow_River

129. Raupach, M.R., G. Marland, P. Ciais, C. Lequere, J.G. Canadell, G. Klepper and C.B. Field. 2007. “Global and regional drivers of accelerating CO2 emissions.” PNAS. Vol. 104, No 24. pp 10288–10293. https://doi.org/10.1073/pnas.0700609104

130. Renard, K.G., G.R Foster, G.A. Weesies, and J.P. Porter. 1991. “RUSLE: Revised universal soil loss equation.” Journal of Soil and Water Conservation. Vol. 46(1). pp 30–33. https://www.jswconline.org/content/46/1/30

131. RMJOC. 2011. “Climate and Hydrology Datasets for Use in the River Management Joint Operating Committee (RMJOC) Agencies’ Longer-Term Planning Studies Part IV – Summary.” River Management Joint Operating Committee. Online. https://www.usbr.gov/pn/climate/planning/reports/index.html

132. Robichaud, P.R., J.L. Beyers, and D.G. Neary. 2000. “Evaluating the Effectiveness of Postfire Rehabilitation Treatments.” USDA RMRS-GTR-63. https://www.fs.usda.gov/research/treesearch/23617

133. Samaranayake, V.A. 2009. “The statistical review of three papers on specific gage analysis.” Report to USACE St. Louis District. https://www.mvs-wc.usace.army.mil/arec/Documents/Physical_Effects/The_Statistical_Review_of_Three_Paperson_Specific_Gage_Analysis.pdf

134. Schumm, S.A. 1960. “The shape of alluvial channels in relation to sediment type.” USGS Professional Paper 352-B. pp 17–30. U.S. Government Printing Office. https://pubs.usgs.gov/pp/0352b/report.pdf

135. Schumm, S.A., and H.R. Khan. 1972. “Experimental Study of Channel Patterns.” Geological Society of American Bulletin No. 83. https://doi.org/10.1130/0016-7606(1972)83[1755:ESOCP]2.0.CO;2

136. Seal, R., C. Paola, G. Parker, J.B. Southard, and P.R. Wilcock. 1997. “Experiments on downstream fining of gravel: I. Narrow-channel runs.” Journal of Hydraulic Engineering. Vol. 123. pp 874–884. http://dx.doi.org/10.1061/(ASCE)0733-9429(1997)123:10(874)

137. Shelley, J., and P. Bailey. 2017. “The Cross-section Viewer: A Tool for Automating Geomorphic Analysis Using Riverine Cross Section Data.” ERDC/TN RSM-18-3. USACE ERDC. Vicksburg, MS. http://dx.doi.org/10.21079/11681/26284

138. Shen H.W. 1971. “Fluvial Geomorphology: Historical Perspective and Fluvial Geomorphology: Channel Adjustment and River Metamorphosis.” River Mechanics. Water Resources Publications. Littleton, CO. https://www.worldcat.org/title/river-mechanics/oclc/238313

139. Shen, H.W., and J.Y. Lu. 1983. “Development and Prediction of Bed Armoring.” Journal of Hydraulic Engineering. ASCE. Vol. 109, No. 4. pp 611–629. https://doi.org/10.1061/(ASCE)0733-9429(1983)109:4(611)

140. Shulitis, S. 1935. “The Schoklitsch bed-load formula.” Engineering London, England. June 21, pp 644–646 and June 28, p 687. (Online version not available.)

141. Simons, D.B. 1957. “Theory and design of stable channels in alluvial materials.” Ph.D. Dissertation. Colorado State University. Fort Collins, CO. http://hdl.handle.net/10217/68215

142. Simons, D.B., and M.L. Albertson. 1963. “Uniform water conveyance channels in alluvial material.” Transactions. ASCE. Vol. 128. pp 65–167. https://doi.org/10.1061/TACEAT.0008728

143. Simons, D.B., and E.V. Richardson. 1961. “Forms of bed roughness in alluvial channels.” Journal of the Hydraulics Division. ASCE. Vol. 87(3). pp 87–105. https://doi.org/10.1061/JYCEAJ.0000612

144. Simons, W.D., and M.I. Rorabaugh. 1971. “Hydrology of Hungry Horse Reservoir, Northwestern Montana.” USGS Professional Paper 682, U.S. Government Printing Office. https://doi.org/10.3133/pp682

145. Strickler, A. 1923. “Some contributions to the problem of velocity formula and roughness coefficient for rivers, canals, and closed conduits.” Mitteilungen des eidenossischen Amtes fur Wasserwirtschaft. No. 16. Bern, Switzerland. (Online version not available.)

146. Syvitski, J. 1991. “Principles, methods, and application of particle size analysis.” Cambridge University Press. Cambridge, England. Online version dated January 2010. https://doi.org/10.1017/CBO9780511626142

147. Temple, D.M. 1980. “Tractive force design of vegetated channels.” Transactions of the ASCE. Vol. 23. pp 884–890. https://elibrary.asabe.org/abstract.asp?aid=34681

148. Udden, J.A. 1914. “Mechanical composition of clastic sediments.” GSA Bulletin. Vol. 25, No. 1. pp 655–744. (1914). https://doi.org/10.1130/GSAB-25-655

149. Vanoni, V.A. 1984. “Fifty Years of Sedimentation.” Journal of Hydraulic Engineering. Vol. 110(8). ASCE. https://doi.org/10.1061/(ASCE)0733-9429(1984)110:8(1021)

150. Villanueva, E., and J.L. Wibowo. 2008. “Risk Assessment of Rock Surface Spillway Erosion Using Parametric Studies.” ERDC/CHL TR-08-22. USACE ERDC. Vicksburg, MS. https://erdc-library.erdc.dren.mil/jspui/handle/11681/10416

151. Watson, C.C. 2010. “Specific Gage Analyses of Stage Trends on the Middle Mississippi River.” Proceedings of the 2nd Joint Federal Interagency Conference on Sedimentation and Hydrologic Modeling. June 27–July 1, 2010. Las Vegas, NV. https://acwi.gov/sos/pubs/2ndJFIC/Contents/7D_Watson_Final.pdf

152. Webb, B.W., and D.E. Walling. 1982. “The magnitude and frequency characteristics of fluvial transport in the Devon drainage basin and some geomorphological implications.” Catena. Vol. 9. pp 9–23. https://doi.org/10.1016/S0341-8162(82)80002-7

153. Weidner, K.L. 2012. “Evaluation of the jet test method for determining the erosional properties of cohesive soils: a numerical approach.” M.S. Thesis. Virginia Polytechnic Institute and State University. Blacksburg, VA. (Online version not available.)

154. Wentworth, C.K. 1922. “A scale of grade and class terms for clastic sediments.” Journal of Geology. Vol. 30. pp 377–392. https://www.jstor.org/stable/30063207?seq=1#metadata_info_tab_contents

155. Wieczorek, G.F., S. Ellen, W.W. Lips, S.H. Cannon, and D.N. Short. 1983. “Potential for debris flow and debris flood along the Wasatch Front between Salt Lake City and Willard, Utah, and measures for their mitigation.” USGS Open File Report 83-635. Menlo Park, CA. https://doi.org/10.3133/ofr83635

156. Williams, G.P. 1978. “Bankfull discharge of rivers.” Water Resource. Research. Vol. 14(6). pp 1141–1154. https://agupubs.onlinelibrary.wiley.com/doi/pdf/10.1029/WR014i006p01141

157. Williams, G.P., and D.L. Rosgen. 1989. “Measured total sediment loads (suspended loads and bedloads) for 93 United States streams.” USGS. Open File Report 89-67. https://pubs.usgs.gov/of/1989/0067/report.pdf

158. Williams, J.R. 1975. “Sediment-yield prediction with universal equation using runoff energy factor.” In the “Present and prospective technology for predicting sediment yield and sources.” Proceedings of the sediment yield workshop. November 28–30, 1972. USDA Sedimentation Lab. ARS-S-40. Oxford, MS. (Online version not available.)

159. Wolman, M.G. 1954. “A method for sampling coarse river-bed material.” Transactions American Geophysical Union. Vol. 35(6). pp 951–956. https://doi.org/10.1029/TR035i006p00951

160. Wolman, M.G. 1955. “The natural channel of Brandywine Creek, Pennsylvania, U.S.” Geological Survey Professional Paper. Vol. 271. https://pubs.usgs.gov/pp/0271/report.pdf

161. Wolman, M.G., and R. Gerson. 1978. “Relative scale of time and effectiveness.” Earth Surface Processes and Landforms. Vol. 3. pp 189–208. https://doi.org/10.1002/esp.3290030207

162. Woodyer, K.D. 1968. “Bankfull frequency in rivers.” Journal of Hydrology. Vol. 6. pp 114–142. https://doi.org/10.1016/0022-1694(68)90155-8

163. Worrall, F., T.P. Burt, N.J.K. Howden, and G.R. Hancock. 2014. “Variation in suspended sediment yield across the UK – A failure of the concept and interpretation of the sediment delivery ratio.” Journal of Hydrology. Vol. 519, Part B. pp 1985–1996. https://doi.org/10.1016/j.jhydrol.2014.09.066

164. Wu, W. 2004. “Depth-averaged two-dimensional numerical modeling of unsteady flow and nonuniform sediment transport in open channels.” Journal of Hydraulic Engineering. Vol. 130(1). pp 1013–1024. https://doi.org/10.1061/(ASCE)0733-9429(2004)130:10(1013)

165. Wu, W., and S.S. Wang. 2006. “Formulas for sediment porosity and settling velocity.” Journal of Hydraulic Engineering. ASCE. Vol. 132, No. 8. pp 858–862. https://doi.org/10.1061/(ASCE)0733-9429(2006)132:8(858)

166. Yang, C.T. 1977. “The Movement of Sediment in Rivers.” Geophysical Survey. Vol. 3. pp 39–68. https://link.springer.com/article/10.1007/BF01449182

A-4. Unit Conversion Table.

Table A-1
Unit Conversion Factors

To Convert From	Multiple By[1]	To Obtain
Length		
inches (in)	25.4	millimeters (mm)
feet (ft)	0.3048	meters (m)
yards (yds)	0.9144	meters (m)
mile (mi)	1609	kilometers (km)
Area		
square inches (in^2)	645.16	square millimeters
square feet (ft^2)	0.09290304	square meters
square yards (yds^2)	0.83612736	square meters
acre (ac)	0.4046856	hectares
square miles (mi^2)	2.58999	square kilometers
Volume		
cubic inches (in^3)	16.387064	cubic centimeters (cm^3)
cubic feet (ft^3)	0.02831685	cubic meters (m^3)
cubic yards (yd^3)	0.764555	cubic meters (m^3)
gallon (gal)	3.78541	liters (l)
acre-foot (ac-ft)	1233.5	cubic meters (m^3)
Mass		
pounds (lbs)	0.4535924	kilograms (kg)
ton (short tons, 2000 pounds)	0.9071848	metric tons
Force		
pounds force (lbf)	4.448222	newtons (N)
pounds per square foot (psf)	47.88026	Pascals (pa)
Velocity		
feet per second (ft/sec)	0.3048	meters per second (m/sec)
Discharge		
cubic feet per second (cfs)	0.0283168	cubic meters per second (cms)
cubic feet per second (cfs)	1.98347	acre-foot per day (ac-ft/day)
Temperature		
degree Fahrenheit (°F)	(°F – 32) x 5/9	degree Centigrade (°C)
Density		
pounds per cubic foot (pcf)	16.01846	kilograms per cubic meter (kg/m^3)
Power		
horsepower (hp)	745.7	watts (W)

[1] To reverse conversion of the from/to obtain units shown, divide by the conversion factor.

Appendix B
Staged Sediment Studies Guidelines

B-1. Introduction.

a. Guidelines for the different stages of sediment analysis are available from multiple sources. The Sediment Studies Work Plan (SSWP) is the overall linking document that defines the planned studies throughout the entire design and evaluation process for a study. The three stages recommended in staged sediment studies include: the sediment impact assessment, the detailed sedimentation study, and the feature design sedimentation study. These stages are guidelines and may not be sufficient for all studies. The engineer is responsible for supplementing these stages as needed to ensure adequate project evaluation.

b. Risk and uncertainty principles should be applied within the staged study approach. A risk-based approach to water resources planning captures and quantifies the extent of risk and uncertainty in the various planning and design components of an investment project. Within USACE, risk-based analysis must be used to compare plans in terms of the likelihood and variability of their physical performance, economic success, and residual risks. Scope during the various study analysis stages should be designed to ensure risk analysis compatibility and to provide adequate input to economic analysis for typical USACE projects.

B-2. Incorporation with the Sediment Studies Work Plan.

a. The purpose of the SSWP is to adequately scope the development of a comprehensive work plan to identify and evaluate sediment analysis needs and problems for USACE projects, and to track progress throughout the investigation. Developing the SSWP requires critical thinking to align the staged sediment study level of detail, including analysis method and data collection, with the project planning study phases. Significant hurdles to developing the SSWP are accurately defining overall study scope, the geographical scale of the affected environment, necessary results accuracy, and the anticipated alternatives.

b. Within the USACE study process, it is critical to accurately define costs for data collection, analysis, design, initial project construction, and operation and maintenance. Costs from sedimentation impacts to the project during formulation, construction, operation, and maintenance should be included in the initial study effort and included in cost estimates. Early in project formulation, adequate data to develop cost estimates with the necessary accuracy can be challenging and should be addressed through a reasonable assessment of risk to project performance and the impacts to the SSWP. Refer to Chapter 2 for a detailed discussion of the SSWP.

c. Incorporation of the sediment study stages with the USACE planning process is a critical element for successful USACE project design, implementation, and operation. The USACE study planning process has evolved considerably. Assigning an equivalent planning phase study title with sediment studies is not possible, as current practices include overlap with staged sediment study tasks. While USACE planning study nomenclature may change, linking sediment study needs with current planning study definition will facilitate providing the needed study products. The three sediment study stages applied to USACE river sedimentation studies, with the typical planning phase for reference, are:

(1) Sediment Impact Assessment (Reconnaissance Phase). The sediment impact assessment identifies the magnitude of potential sediment problems that could be induced by the project. This study can usually be accomplished using a geomorphic assessment with a sediment budget approach.

(2) Detailed Sediment Investigation (Feasibility or General Reevaluation). A detailed investigation incorporates the principle of sediment continuity through the project reach and is conducted using 1D numerical sedimentation models such as the HEC-RAS or HEC-6T, or 2D models such as HEC-RAS 2D with sediment or Adaptive Hydraulics (AdH). Problems related to aggradation and/or degradation are quantified during the detailed sediment investigation and corrective measures are identified.

(3) Feature Design Sedimentation Study (Preconstruction Engineering and Design, Design Documentation Report, also General Reevaluation in some cases). The feature design study is focused on sedimentation problems associated with project structures. These are 3D problems and may be evaluated using multidimensional numerical models and/or movable-bed physical models.

B-3. Stage 1 Sediment Impact Assessment.

a. The sediment impact assessment includes a geomorphic assessment of the existing channel stability in the project reach and in the upstream and downstream reaches. Sediment budget calculations may be facilitated using programs such as SAM-Win (Thomas et al., 2002) or sediment impact analysis method (SIAM) within HEC-RAS (HEC 2016b). If sediment problems are determined to be negligible, the sediment impact assessment can be the final stage in the sediment investigation process. On the other hand, if sediment problems are found to be significant enough that they could affect the economic viability of the project, then a detailed sediment study is required during the early phases of the planning process.

b. When this assessment indicates that sediment problems will substantially affect project design and project economics or will result in significant environmental impacts, then additional data should be collected and investigations performed in Stage 1. This guideline is implemented to meet the WRDA requirement that project cost-sharing design costs do not escalate after the cost-sharing agreement has been signed. The sediment impact assessment should include a plan for data collection and analysis to be completed during subsequent staged studies. The subsequent stages will normally provide increasing levels of information for decision-makers as project formulation progresses.

c. Purpose. It is important to assess a proposed project's effect on river system stability in the early stages of the planning process, where several alternatives are typically considered. The primary purpose at this stage of planning is to determine feasible engineering solutions, economical project justification, and environmentally acceptable projects.

d. The purpose of the sediment impact assessment is to identify the magnitude of possible sediment problems associated with proposed engineering projects as opposed to specific quantitative results. This section provides a brief overview of several types of sediment impact assessments, along with their rigor and level of uncertainty. Sediment impacts are generally determined using a sediment budget approach where sediment transport and sediment yields for existing and project conditions are compared.

e. Recommendations are made during this stage for appropriate sediment studies at the next level of planning study. An assessment that identifies "no sediment problems" is as important as one identifying problems. The sediment impact assessment might be the only sediment investigation required if the present reach is stable; and the proposed improvements are minor in nature and do not significantly alter the existing sediment, hydraulic, or hydrologic variables.

f. Scope. The sediment impact assessment is conducted to assess the effect that a full range of natural flows will have on aggradation or degradation within the project reach.

(1) Sediment impact assessments can range widely in effort and output. These assessments can be accomplished using visual or qualitative techniques for relatively simple projects or by using a numerical model that incorporates solution of the sediment continuity equation for more complex projects.

(2) The sediment impact assessment report conveys to reviewing authorities the amount of effort expended to date in investigating sedimentation problems; the amount and type of field data available for the assessment; the anticipated impact of sedimentation on project performance and maintenance; and the anticipated impact of the project on stream system morphology. The report should discuss, at a minimum, the reservoir or river sedimentation problems identified in this EM and estuarine problems identified in EM 1110-2-1607. In addition, any problems unique to this site are to be included.

g. Typical Tasks. A detailed explanation of tasks for a typical sediment impact assessment are arranged in a sequence of steps in the following section. Not all steps are necessary and site-specific needs should be considered when developing study scope. Table B-1 provides a summary task list for use as a reference and checklist.

Table B-1
Steps in a Typical Sediment Impact Assessment

Sequence of Steps	Primary Components
Geomorphic Assessment with Field Reconnaissance	Data collection, field reconnaissance, assess the ongoing sediment processes.
Analytical Bed Stability Analysis	Velocity and shear stress-based, flow when bed motion occurs, evaluate armor layer.
Sediment Budget Approach (SAM, HEC-RAS)	Transport capacity vs. sediment supply, flow duration sediment discharge curve (Biedenharn et al., 2000).
Hydraulic Parameters Existing Conditions Using Stable Channel Tool (SAM, HEC-RAS)	Typical section(s), determine stage-discharge, depth-velocity, depth-slope, depth-bed shear stress curve.
Sediment Transport Existing Condition	Separate into bed material load and wash load, select transport function(s) for site, compute for flow range.
Plot Soil Borings	Review variation in bed material data spatially, identify potential problem areas.
Develop Design Features for Project	Channel geometry (cross section, longitudinal slope, planform).
Hydraulic Parameters for Project Conditions	Compare to existing, revise proposed if needed to provide reasonable agreement in critical hydraulic parameters.
Preliminary Screening for Sedimentation Problems	Check for threshold using critical shear or allowable velocity. Revise project geometry, project structures, or include channel lining if necessary.
Project Impact on Sedimentation	Check sediment budget for existing and project condition (project sediment yield from flow duration sediment discharge rating curve compared to incoming load), also compare to downstream channel transport capacity.
Design Flow Analysis	Repeat sedimentation impact evaluation using the design flow hydrograph.
Local Scour	Compare to similar projects (geometry, bed material, hydraulic parameters).
Bank Erosion and Failure	Use available guidance, also BSTEM in HEC-RAS.
Estimate Long-Term Maintenance	From sediment budget, local and general aggradation/deposition. Features to limit bridge scour and structures, bank protection needed, channel lining and repair.
Conclusion Evaluation	Evaluation of project impact on system stability. Estimate type and locations of design features. Sedimentation magnitude is considered to determine recommendations for need for more detailed future study.

Additional considerations: (1) complex stream geometry may require analysis by reach; (2) carefully consider selection of sediment transport function; (3) collect necessary field data including bed material samples at multiple times if feasible; and (4) study sequence starts upstream, when multiple reaches exist, compare each for areas of scour and deposition.

h. The steps listed in Table B-1 are further detailed as:

(1) Geomorphic Assessment with Field Reconnaissance. The first step in the sediment impact assessment is the geomorphic assessment. The geomorphic assessment includes data collection, field reconnaissance, and determination regarding the existing channel's stability over the most recent 20 to 30 years. The dominate sedimentation processes at work in the system should also be determined during the geomorphic assessment. Thoroughly document results of the field reconnaissance and evaluation in a geomorphic assessment report.

(2) Analytical Analysis. In most cases, the geomorphic assessment provides only qualitative determinations of stability trends. Observations and hydraulic geometry relations may be used to identify possible stability problems, but analytical methods are required to determine the magnitude of a stability problem. An analytical stability analysis requires calculation of hydraulic parameters, such as velocity and shear stress for the range of natural discharges. The hydraulic resistance of the channel boundary is determined from field observations and measurements. Sufficient field sampling of the streambed should be conducted to determine the spatial variability and gradation of the bed material. Sediment inflow is estimated from measured data or by calculation.

(3) Bed Stability. After hydraulic parameters have been calculated for a range of discharges, it is important to determine the discharge at which the streambed begins to move or becomes alluvial. This can be accomplished using the threshold criteria described in EM 1110-2-1418. This step is especially important in a channel with an armor layer. Sediment transport capacity dramatically increases when the armor layer is destroyed, and the coarse material becomes thoroughly mixed with the substrate material. Stability of vegetated or gravel banks can be determined using allowable velocity methods or shear stress methods. Just because a streambed is mobile does not mean that it is unstable, but it does indicate that a higher level of analysis is required to determine stability.

(4) Sediment Budget Approach. Channel stability is ultimately determined by the channel transport capacity, and by incoming and exiting sediment load. If sediment transport capacity is less than sediment supply, the channel will aggrade. On the other hand, if the capacity is more than the supply and the bed is alluvial, the channel will degrade. A determination of the potential for aggradation or degradation in a channel reach requires an assessment of the sediment budget. The sediment budget compares the quantity of sediment transported into the reach with the sediment transport capacity of the reach. This is accomplished using the magnitude and frequency of all sediment-transporting flows.

(5) Stable Channel Calculations. The SAM Hydraulic Design Package was developed to facilitate calculations required for a sediment budget analysis. This functionality has also been added to the stable channel design function within HEC-RAS. The analysis method used in the software is based on uniform flow assumptions using "typical" or "representative" cross sections. More details on application of the stable channel design process are available in Chapter 10 of this manual, the HEC-RAS User's Manual (HEC 2016b), the SAM Hydraulic Design Package for Channels User's Guide (Thomas et al., 2002), and EM 1110-2-1601.

(a) Hydraulics Computations. Normal depth and composite hydraulic parameters are calculated for complex channels with variable roughness. Hydraulic model roughness can be assigned using multiple methods, including Manning's roughness coefficient, relative roughness height, grain size, or grass type. Models also contain methods for determining stable channel dimensions based on sediment continuity principles.

(b) Sediment Transport. Sediment transport equations are used to calculate the bed material sediment discharge. The calculations are made by particle size class.

(c) Sediment Yield. Calculate sediment yield using the flow duration sediment discharge curve method (Biedenharn et al., 2000). This method consists of integrating the sediment discharge rating curve and a flow duration curve, or flood hydrograph, to calculate sediment yield on an average annual basis or for a flood event.

(6) Hydraulic Parameters for Existing Conditions. Hydraulic parameters can be determined using normal depth assumptions or by a more rigorous backwater analysis.

(a) Normal Depth.

- For the normal depth approach, representative cross sections need to be identified in the project reach, the upstream supply reach, and the downstream receiving reach.

- Reliability of the normal depth calculation is directly related to the reliability of the input data. Good engineering judgment is required in the selection of a representative cross section. The cross section should be located in a uniform reach where flow is essentially parallel to the bankline (no reverse flow or eddies). This typically occurs at a crossing or riffle.

- Determination of the average energy slope can be difficult. Thalweg slopes and low flow water-surface slopes may not be representative of the energy slope at morphologically significant flows. Slope estimates should be made over a significant length of the stream (a minimum of a meander wavelength or twenty channel widths). However, reach average slopes may not be representative of smaller segments within the reach.

- Hydraulic roughness must be estimated based on field observations and measurements. Depending on the complexity and length of the project, more than one representative cross section may be required for the project reach.

- Ideally, at least one of these cross sections would be at a standard discharge range so that hydraulic parameters can be obtained from field measurements.

- Measured water velocities, discharges, and water surface elevations may be used to confirm assigned roughness coefficients and energy slopes. If that source is not available, the measurements made during the field reconnaissance may be used to support the hydraulic calculations. In either case, the following graphs for the project, upstream supply, and downstream receiving reaches are suggested: a stage-discharge relationship, a depth-velocity relationship, a depth-slope relationship, a depth-bed shear stress relationship, and a depth-percent of total flow in the channel relationship. All these curves can be readily developed using the stable channel analysis method in HEC-RAS and SAM-Win.

(b) Bed Roughness.

- Use a "bed roughness predictor" to tie the hydraulics to the bed sediment samples taken during the field reconnaissance trip. The Brownlie (1983) equations are suggested for sand-bed rivers, and the Limerinos (1970) equation is recommended for gravel-bed rivers.

- Composite this n value with other cross-section roughness values.

- Plot a graph of channel velocity vs. hydraulic radius for the range of water discharges through the project design flood discharge.

(c) Flow Distribution Between Channel and Overbanks.

- Plot the channel velocity for the full range of water discharges. Such a plot should show those velocities increasing with depth. If they decrease with increasing depth, either justify that trend or correct the n-values between the main channel and overbanks before proceeding.

- Use the channel velocity from the bed roughness predictor to aid in calibrating the distribution between channel and overbanks.

(d) Sensitivity to Geometry.

- If channel characteristics are so varied that one curve is not representative of the project reach, use a water surface profile computer program, such as HEC-RAS, to calculate the hydraulic parameters.

- Evaluate a minimum of two conditions: one with the best estimate of n-values from the field assessment and analysis; and one using the predicted bed roughness n-values for the channel bed portion of the cross section.

- Calculate length-weighted, reach-averaged hydraulic parameters for the sediment budget analysis.

(7) Sediment Transport for the Existing Conditions.

(a) Measured Data.

- If measured data are available, separate the total sediment discharge into bed material load and wash load components.

- Compare the measured bed material rating curve to calculated curves using several sediment transport functions, selecting the one that best reproduces measured data.

(b) No Measured Data.

- Without sediment data, select two sediment transport formulas and calculate a sediment transport relationship for the full range of water discharges on the stage-discharge relationship. That will provide bed material discharge curves for existing conditions.

- If the curves are drastically different, apply a third transport function and select the most consistent one. Guidance for selecting a sediment transport function is available in the SAM.aid module, the HEC-RAS Hydraulic Reference Manual (HEC 2016b), and Chapter 14 of ASCE Manual No. 110 (Thomas and Chang 2008).

- Implementing the selection guidance relies on using site data for bed gradation and calculated hydraulic parameters, including velocity, depth, slope, width, and median grain size as screening parameters. These screening parameters are compared to a library of field data compiled from the literature. Transport functions that were developed with screening parameters similar to the project conditions should be selected.

(8) Plotting Soil Borings. It is very useful to plot the channel boring logs on a channel profile. This allows quick identification of potential problem areas and spatial variation. If detailed channel material information is available over a large reach length, consider establishing a stable channel design channel grade such that the channel will be embedded in erosion-resistant material rather than exceeding cut into soils that are easily eroded.

(9) Develop Design Features for the Proposed Project. Project features should include a geomorphic analysis to develop a sustainable planform.

(10) If the proposed design cross section is not similar to the regime cross section, sediment problems usually require extensive maintenance to keep the project in operation. This concept is valid for flood risk management, restoration, and navigation channels.

(11) Hydraulic Parameters for Project Conditions. Hydraulic parameters can be determined using normal depth assumptions or by a more rigorous hydraulic model analysis. Calculate and plot the same variables as presented above for the existing channel.

(12) Preliminary Screening for Sedimentation Problems. Determine if the design channel is a threshold channel or an alluvial channel using critical shear stress or allowable velocity methods. If a threshold channel is desired, the velocities and shear stresses in the improved channel should not exceed the maximum allowable velocity or critical shear stress for the stream material. If the shear stress or channel velocity is too high for the threshold channel, either redesign the channel cross section, include a channel lining, or add design features such as drop structures to flatten the slope. Project condition velocities for low flows should not be so low that deposition will be induced beyond that which occurs under existing conditions.

(13) Sediment Transport for Project Conditions. Using the same transport formula used to calculate sediment transport for existing conditions, calculate a sediment discharge for the full

range discharges on the stage-discharge relationship. Plot the calculated discharges on the graph with existing conditions in the project reach, the upstream supply reach, and the downstream receiving reach.

(14) Impact of Sedimentation on the Performance of the Proposed Project. In the sediment budget approach, project performance is evaluated by comparing trap efficiencies with and without the project. If the representative reach approach indicates significant sedimentation problems, a more comprehensive numerical modeling approach is required.

(a) General Aggradation or Degradation. The project TE is calculated by subtracting the sediment yield of the bed material sediment load in the project reach from that for the upstream supply reach. If the result is positive, aggradation is indicated. If the result is negative, degradation is indicated. When degradation is indicated the bed sediment should be checked for resistance to erosion. The downstream receiving reach should be compared to the project reach to evaluate downstream bed stability.

(b) Calculate Sediment Yield for Existing Conditions, Ys(in). Use the flow duration sediment discharge rating curve method (Biedenharn et al., 2000) to calculate the average annual sediment yield for the existing channel, the upstream supply reach, and the downstream receiving reach. Separate that total into the bed material load component and the wash load component. The result is average annual yield of bed material sediment. Confirm that result with sediment yields determined by the other methods and reconcile differences before proceeding.

(c) Calculate Sediment Yield for Project Conditions, Ys(out). Use the flow duration sediment discharge rating curve method and make a bed material sediment yield calculation for project conditions.

(d) Calculate the Sediment TE. Calculate the sediment TE by subtracting the sediment yield for project conditions from the sediment yield for existing conditions and supply reach conditions. If that result is positive, deposition is indicated. Using simple geometries and available specific weights, calculate how much time will pass before deposition is sufficiently deep to affect project performance. If the sediment budget produces a negative difference, erosion is indicated. Choose design features accordingly.

$$TE = \frac{100\,[Y_{s(in)} - Y_{s(out)}]}{Y_{s(in)}} \qquad \text{Equation B-1}$$

(15) Design Flow Analysis. Repeat the sediment budget calculation for the design flow hydrograph.

(16) Local Scour. At this level of study, the approach for estimating local scour potential at bridges and hydraulic structures is to compare this project with similar projects.

(17) Bank Erosion and Failure. Bank erosion and failure methodologies are outlined in EM 1110-2-1418. Additionally, HEC-RAS includes the USDA-ARS BSTEM tool and can compute interactions between channel incision, toe erosion, and bank failure (HEC 2016b).

(18) Estimate Long-Term Maintenance. This refers to both local and general scour and deposition in the project reach. The approach for estimating maintenance for general deposition is to use the sediment budget analysis. At this stage of the planning process, estimating the required treatments to arrest local scour at bridges, hydraulic structures, and bank protection sites is accomplished by comparisons with similar existing projects.

(19) Conclusion Evaluation. Conclude whether the improvements will or will not cause the reach to be unstable. The type and probable locations of design features should be estimated. If the magnitude of sedimentation problems is important to basic formulation decisions, further study should be recommended. However, if the results of this impact assessment can be changed substantially, such as by a factor of 2, without changing the basic project stability and performance, it will probably be acceptable to proceed with formulation, initiate a data collection program, and refine the sedimentation investigation in a detailed sedimentation study.

(20) Considerations for Performing a Sediment Impact Assessment.

(a) Normal Depth Approach. Hydraulic characteristics can always be determined from flow line computations, but that is not always necessary.

(b) Complex Geometry. The study area may be so irregular that the assessment must be adapted to reaches rather than having one for the entire project. Do whatever is necessary to arrive at defendable results.

(c) Sediment Transport. Suitable sediment transport equations are listed in Vanoni (1975, 2006, 2008), SAM-Win (Thomas et al., 2002), and HEC-RAS (USACE 2016b).

(d) Sediment Data. Appropriate data necessary for the chosen equations should have been gathered during the field reconnaissance. Ideally, bed samples should be taken at several different times to ensure that a representative bed sample has been obtained. One set is better than none.

(e) Study Sequence. The first area to study is the upstream end of the project reach. When multiple reaches exist in the project, potential areas of scour and deposition are identified by comparing the transport capacity of the interest reach to the transport capacity of the next upstream reach.

B-4. Stage 2 Detailed Sedimentation Study.

a. Purpose. The purpose of the detailed sedimentation study is to further address problems reported in the sediment impact assessment, recommend corrective measures, and assess the effectiveness of these measures. A detailed study will be required if the sediment impact assessment predicts a sedimentation problem; if a similar, existing project is experiencing sedimentation problems; or if significant adverse environmental impacts are predicted.

b. Scope. The scope and depth of study in this stage is controlled by the level of technical details required to resolve the problems. The Detailed Sedimentation Study identifies the location and type of project features that will be required to achieve the project purpose with the minimum amount of maintenance.

(1) The primary criteria are “What is required for the project to function without major sedimentation problems?” and “How will those features affect the stream system?” Typically, the detailed sedimentation study includes analysis using a numerical sedimentation model.

(2) The sedimentation model should provide for sediment continuity by particle size class. Several proven models are available and have been used extensively. Examples are HEC-RAS, HEC-6T, SED-2D, and AdH.

(3) The differences between this level and that presented in the Sediment Impact Assessment are in the breadth and depth of the computations and the amount of data that is available. In addition, flow hydrographs should be used instead of a few selected discharges, and the period of simulation should span from a single event to the life of the project.

(4) Sensitivity runs should be made to test the response of the project to uncertainties in sediment yield, water runoff, or downstream controls. Sensitivity results will provide a better basis for developing conclusions.

c. Typical Tasks. The typical general task list for a detailed sediment study has been developed from multiple USACE sediment studies. Further amplification on these tasks can be found in Chapter 14 of ASCE Manual 110 (Thomas and Chang 2008), USACE-HEC (1992), and Chapter 9 of this manual. This task list is not a recipe, but it is suggested as a means to organize a model study. Exceptions and additions to this organization should be expected. Table B-2 provides a summary task list for use as a reference and check list.

Table B-2
Typical Tasks for a Detailed Sediment Study

Tasks	Primary Components
Field Reconnaissance	Another field recon to review the site, observe possible changes in conditions, and collect supplemental field data.
Assemble Data	Assemble additional modeling data, review model user manuals for needed input.
Develop Geometric Data	Construct geometry for multiple time periods needed for numerical model, check results for a 2-year flow.
Develop Hydrology Data	Collect flow input at all model boundaries, determine downstream boundary. Data required for entire modeling period, also consider design flow hydrograph, future conditions, and need for synthetic events.
Develop Sediment Data	Sediment inflow rating curve by particle size class each boundary, also bed material gradations, possibly sediment diversion ratios.
Model Development and Analysis (Construction and Calibration)	Time-consuming process to develop model coefficients, sediment transport function, etc. Start with fixed-bed hydraulic modeling, single discharge movable bed, quasi-unsteady/unsteady movable bed.
Split-Record Testing	Demonstrate model ability to simulate different record than used in development, compare to observed.
Review Model Proposed Use	Review model use after completing testing; computational analysis vs. calibrated model (refer to Chapter 9 for definition). Is it capable of study needs?
Model Base (no-action)	Model computations for base (no-action) condition using projected future (hydrograph and sediment inflow).
Model Project Alternatives	Model computations with project geometry and same inputs as base condition analysis.
Sensitivity Test – Risk and Uncertainty	Model sensitivity test to evaluate model inputs and response. Evaluate boundary conditions and adjust parameters used in model development for both base and alternatives.

d. The tasks listed in Table B-2 are further detailed as:

(1) Field Reconnaissance. Another field investigation is recommended to visually verify current conditions and the validity of collected data. Estimates of channel and overbank hydraulic roughness should be made. Reaches where abrupt changes in channel geometry occur should be identified. Requests for additional or missing data should be prepared. Before

conducting the field reconnaissance, consider all elements on the task list, including data collection to develop a list of tasks for the field trip. Thoroughly document results of the reconnaissance in a trip report.

(2) Assemble Data. Additional available data should be added to data already collected for the sediment impact assessment. These data include: maps, cross sections, suspended-sediment measurements, bedload data, bed material gradations, soil types, sediment yield estimates, hydrographs, water temperature, observed water surface profiles, reservoir operations, and construction activities. Consult the user's manual for the selected numerical model (such as HEC-6T, HEC-RAS, SED-2D, and AdH) for additional information regarding necessary input data.

(3) Develop Geometric Data. Cross-section data should be developed and incorporated into the numerical model. A steady-state discharge equivalent to a 2-year frequency peak flow should be run through the model to identify and correct trouble spots and data gaps. Note that multiple data sets separated by a significant time period are generally required to allow reasonable model development and accuracy.

(4) Develop Hydrology Data. The numerical model requires hydrologic data at all model boundaries. A downstream stage-discharge rating curve or a historical stage hydrograph is required at the downstream end of the model. The numerical model simulation will also require a historical long-term hydrograph that can be used to calibrate historical aggradation, degradation, and roughness coefficients. A design flood hydrograph is needed to evaluate project performance during a flood. It will be necessary to estimate a future-conditions, long-term hydrograph that can be used in a long-term simulation to predict future maintenance requirements and project performance over the project life. Future-conditions hydrographs need to be developed for both the with-project and without-project conditions.

(5) Develop Sediment Data. Using available data, a sediment inflow rating curve for each particle size class must be developed for each sediment boundary. If the model reach includes flow distributaries, then sediment diversion ratios will need to be developed for each size class at each distributary outflow. Bed material gradations need to be determined for each cross section in the model. Typically, bed material gradations are assigned on a reach basis, determined by available data. The numerical model interpolates gradations for the cross sections between cross sections with assigned bed material gradations.

(6) Calibrate/Circumstantiate the Model. This is typically the most time-consuming part of the numerical model investigation. Calibration, or depending on usage, the process known as circumstantiation (Thomas and Chang 2008), is the development of representative hydraulic roughness coefficients, a sediment transport function, and model parameters that allow the model to calculate values that agree with prototype data. Model calibration does not imply the use of physically unrealistic coefficients to force a poorly conceived mode to exactly match prototype behavior.

(7) Fixed Bed Tests. The first phase of model calibration is conducted with a range of steady-state discharges, selected to cover the entire flow range. Fixed beds are used for model calibration to observe river stages. Assigned roughness coefficients must be within a reasonable range as determined from observations during the field reconnaissance. Observed velocity and flow data should also be evaluated for acceptable model results. Use of measured velocity distribution data within the cross section can be helpful to verify model performance.

(8) Model Stability. Perform a review of model results for large changes in velocity, cross-sectional area, energy gradient, and similar indications of model stability issues. Numerical instabilities can occur at cross sections where the 1D assumption is invalid, where too much conveyance is being allowed in overbank areas that act primarily as storage areas in the prototype, or at cross sections where the model cross sections are not representative for the reach. Stability issues and errors in a poorly performing model that cannot reproduce observed water surface elevations and velocities will be greatly exaggerated when sediment transport is added.

(9) Single-Discharge, Movable-Bed Tests. Model performance should be evaluated for a steady-state, long-term, channel-forming discharge. This test is conducted to adjust initial bed material gradations at individual cross sections to reach average conditions. Cross-section changes over time should tend to stabilize with a channel-forming discharge. Model numerical oscillations are an indicator of stability issues due to factors such as too long a computation time step or when a reach length is too short. Model cross-section spacing can be critical in a sediment transport model. It is common to remove cross sections from a fixed-bed HEC-RAS model to increase cross-section spacing and to remove model instabilities.

(10) Quasi-Unsteady or Unsteady, Movable-Bed Tests. Model performance is evaluated with the movable bed and a historical hydrograph. Model input parameters are adjusted in this phase of calibration to match historical river behavior. This may include aggradation or degradation trends, shifts in specific gage curves, matching downstream sediment measurements, or changes in bed material gradation. In stable rivers, the model can be calibrated to a no-change condition. Adjustment parameters include cross-section geometry, movable-bed limits, hydraulic roughness, initial bed gradations, and sediment inflow. The selection of a sediment transport function and armoring algorithm are key calibration factors. Parameters with the most uncertainty should be adjusted first.

(11) Split-Record Testing. Split-record model testing is a demonstration of the calibrated model's ability to simulate prototype behavior for a time record different from that used in calibration. The split-record testing study is conducted using different boundary conditions with the same calibration parameters. The most common technique is to extend the historical simulation beyond the calibration time period to another time period where adequate data exist. Split-record testing may be achieved by comparing calculated and observed water surface profiles, accumulated bed volume changes, sediment yield, and bed material changes. Predictions from a calibrated and split-record-tested numerical sediment model provide higher confidence prediction results.

(12) Run the Base (No-Action) Test. The base, or no-action, simulation should be run using projected future conditions. Future conditions should be used for both the simulation hydrograph and sediment inflow concentrations.

(13) Run the Plan (Project and Alternative) Tests. The plan and alternative project evaluations should use the same boundary conditions as the base test. For each plan, changes in geometry and any changes in sediment or hydrology associated with the plan should be incorporated into the base test model. Calculated results from the base test and the plan tests should be compared to determine the sediment impact of each alternative. Impacts on stream morphology, the environment, and project maintenance must be determined. Long-term simulations using a period of record are often employed to test the river response to the project.

(14) Perform a Risk and Uncertainty Sensitivity Test. The sensitivity test is conducted to establish the limits of uncertainty in the model results and provide input to the risk and uncertainty analysis. The sensitivity tests should evaluate the effect of changing both the boundary conditions and the adjustment parameters used in the calibration. Not all of these variables will have a significant effect on either the base test or the plan test.

B-5. <u>Stage 3 Feature Design Sedimentation Study</u>. A feature design sedimentation study is focused on sedimentation problems associated with project structures and site-specific features. These are 3D problems and may be evaluated using multidimensional numerical models and/or movable-bed physical models.

a. Purpose.

(1) The purpose of the feature design sedimentation study is to develop and present a detailed plan to protect the project against failure from sedimentation and to establish special operational procedures, as necessary. This type of study is an extension of the Detailed Sedimentation Study to test the final design of the project and relocation features. It is usually conducted at a specific location on a stream where extensive data are available. It includes all of the original data plus all data collected since the Detailed Sedimentation Study was completed.

(2) Example applications are the depth of both local and general scour at bridges; the head loss and potential local scour at weirs and drop structures; the potential deposition in expansions and at inflow points; the performance of debris basins in the design; the stability of the channel invert against erosion; the ability of the approach structure to eliminate head cuts upstream from the project; the local erosion at the approach structure; and the changes in tailwater as the result of changes in the exit channel.

b. Scope. The final stage, the scope and depth of study is the most rigorous of the three stages. The study scope is controlled by the level of technical details required to resolve the problems.

c. Typical Tasks. The task list for a feature design sediment study is usually site-specific. A typical task overview is provided in Table B-3.

Table B-3
Typical Tasks for Feature Design Sedimentation Study

Tasks	Primary Components
Field Reconnaissance	Another field reconnaissance to review the site, observe possible changes in conditions, collect supplemental field data.
Extension of Detailed Study Model (One-Dimension Model)	Additional model elevations with the 1D model used in the Detailed Sediment Study to evaluate site-specific issues such as scour adjacent to levees and critical infrastructure, effects of dredging operations, vegetation maintenance requirements, etc.
Multidimensional Model Study	Detailed evaluation of local scour and protection requirements at critical project features (dikes, drop structures, bridges, and weirs). These are complex models that will require intensive data collection and modeling effort. Refer to the Chapter 9 for further details.

d. Typical tasks listed in Table B-3 are further described as:

(1) Field Reconnaissance. An additional field investigation is necessary to further review the site, visually verify current conditions and compare to previous, evaluate significant areas of change, evaluate data previously collected, and conduct supplemental field data collection if necessary.

(2) Extension of 1D Model Study. The 1D model developed in the Detailed Sediment Study may be used to evaluate some design features related to general scour and long-term effects. Examples of 1D model application in the feature design study stage include evaluation of general scour depth adjacent to project levees or banks, frequency of required dredging operations, and long-term maintenance requirements for vegetation in the channel. Detailed examination of these types of issues will require model expansion with supplemental data collected from the previous modeling stage.

(3) Multidimensional Model Studies. Determination of issues, such as local scour depths and protection requirements at project features, such as dikes, drop structures, low weirs, and bridges, usually requires a multidimensional model study that may be paired with a physical model. Experience with existing structures in a similar hydraulic and sediment environment is also beneficial. It is important to understand the dominant mode of sediment transport in application of multidimensional models. Advection and diffusion equations typically drive numerical sedimentation models so that simulation of suspended load is good. However, multidimensional sediment models do not replicate all the physical processes involved in local scour around structures. Check computer-generated results with empirical scour equations developed from field and laboratory measurements.

(4) Physical Models. Typically, physical models do a good job of predicting the sedimentation impacts related to bedload transport and the stability of riprap. However, due to scale effects, simulating suspended-sediment transport in a physical model is more difficult, requiring a very large model and/or a low-density sediment material.

B-6. Conclusions. The study results from each stage of the investigation should be clearly summarized with as strong as possible conclusions. At the conclusion of all phases, USACE project performance with respect to sedimentation processes should be clearly defined. Conclusions of "results are approximate because of funding or data limitations," or "more study is recommended," are not suitable. The intent of the staged sediment studies, and the overall SSWP, is to have identified these limitations and resolved them during the various study stages to achieve reliable conclusions.

Appendix C
Example Scopes of Work

C-1. Appendix Content. This appendix contains three example scopes of work assembled from various documents pertaining to different aspects of sedimentation analysis. The purpose of the scopes is to provide examples of content used in other applications.

a. These scopes are not intended to illustrate application of general concepts. Scope of work development for an actual USACE project will require additional scope content that is specific to meet project objectives.

b. The example scopes are condensed from the original version with some portions removed for guidance document purposes.

c. Scopes consist of a Geomorphic Stability Assessment (paragraph C-2), an Aggradation Assessment (paragraph C-3), and a Missouri River RAS sediment analysis (paragraph C-4).

C-2. Example Scope of Work for Geomorphic Stability Assessment. This example scope of work for a geomorphic stability assessment project was adapted from content originally contained in Copeland et al. (2001). Portions of this scope have been removed for use with this guidance document. A final scope should conform to USACE standards and clearly state scope items, government-furnished information, and all contractor deliverables.

a. Preliminary Stream Restoration Assessment of Upper Studebaker River Watershed and Project Reach. Upper Studebaker River Section 206 Aquatic Ecosystem Restoration.

(1) Location. Upper Studebaker River, South Lake, California.

(2) Project Description. USACE and the City of South Lake are undertaking an aquatic ecosystem restoration project on the Upper Studebaker River (USR) as authorized by section 206 of the Water Resources Development Act of 1996. Anthropogenic activities in the USR watershed such as logging, grazing, and commercial and residential development have impaired the natural functioning of its ecosystem. These activities have also contributed to the ecosystem degradation of the USR's terminus, South Lake. The Upper Studebaker River Section 206 Aquatic Ecosystem Restoration Project seeks to remedy anthropogenic impacts to the USR watershed and South Lake aquatic ecosystems by implementing measures to restore the channel, riparian, and wetland habitats.

(3) Study Area, Project Reach, and Sub-Reaches.

(a) The study area for the USR 206 geomorphic assessment is the entire USR watershed. The watershed of the Upper Studebaker River covers more than 50 square miles. The upper end of the drainage basin begins in the mountains at about elevation 3,048 m (10,000 feet). From its headwaters, the USR flows westerly about 1.60934 km (1 mile), and then northerly about 8.04672 km (5 miles) through a steep, narrow canyon. On leaving the canyon, the stream flows through a gently sloping valley that ends in South Lake.

(b) The upper basin is characterized by steeply rising, heavily timbered mountain slopes that terminate in large granitic outcrops at their crests. The upper basin is relatively pristine and has experienced minimal anthropogenic impacts. The lower basin has been impacted by logging, agriculture, and urban development. Figure C-1 displays the USR 206 study area location.

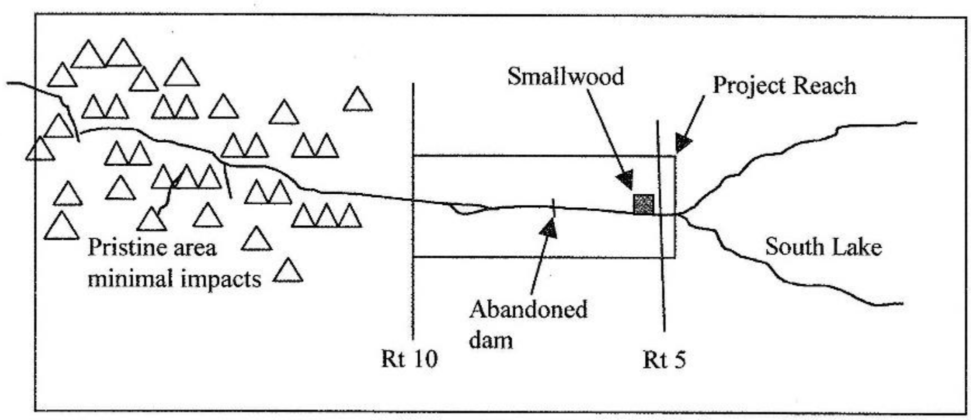


Figure C-1. Study area and project reach (Copeland et al., 2001)

(c) The project reach for the USR 206 project is located in the lower basin. It is defined as the USR channel as well as the current and historic floodplain between South Lake and the Highway 10 bridge crossing upstream and includes the community of Smallwood. The lower portion of the project reach has been channelized.

(d) The USR 206 project reach has been subdivided into hydraulically relevant sub-reaches. Table C-1 lists definitions of the hydraulic sub-reaches.

Table C-1
USR 206 Project Hydraulic Sub-reaches

Hydraulic Sub-Reach	Start Station	End Station	Boundary Descriptions (D/S ° U/S)
1	0+00	6+00	South Lake Blvd. to Route 5 bridge crossing.
2	6+00	25+00	Route 5 bridge crossing to upstream town limits of Smallwood. Channelized reach.
3	25+00	90+00	Upstream town limits of Smallwood to abandoned dam. Agricultural overbank.
4	90+00	160+00	Abandoned dam to downstream main channel/cutoff channel confluence. Agricultural overbank.
5	160+00	200+00L	Downstream main channel/cutoff channel confluence to upstream main channel/cutoff channel confluence. Left descending channel. Gravel mining in reach.
6	160+00	200+00R	Downstream main channel/cutoff channel confluence to upstream main channel/cutoff channel confluence. Right descending channel. Gravel mining in reach.
7	200+00	250+00	Upstream main channel/cutoff channel confluence to Route 10 Bridge. Logging in overbank area.

b. Description of Services Required.

(1) General.

(a) The geomorphic processes of sediment generation and fluvial transport are fundamental and determining factors in the condition of the USR watershed's aquatic habitats. Therefore, a geomorphic assessment (GA) is to be performed in support of the USR 206 project to characterize how geomorphic conditions within the USR watershed influence its ecosystem.

(b) Habitats of primary interest to the restoration effort are aquatic channel, riparian, and wetland habitats. Parameters of primary importance to the quality of aquatic habitat are diversity and stability. In this context, stability is relative to channel conditions of dynamic equilibrium, that is, the balancing of sediment inflow and sediment transport capacity by relatively modest adjustments in channel dimensions. Therefore, particular emphasis must be placed on characterizing how geomorphic conditions within the USR watershed impact the ecology and relative stability of morphometric channel and floodplain conditions within the project reach.

(c) The geomorphic processes of sediment generation and fluvial transport within the USR watershed are also relevant to the issue of South Lake's diminishing clarity, the most visible and noteworthy symptom of ecosystem degradation. Therefore, another goal of the GA is to characterize how geomorphic conditions within the USR watershed influence South Lake's diminishing clarity.

(d) The GA will consist of a reconnaissance-level analysis of the entire USR watershed and a more detailed level analysis of the project reach. Watershed analysis must focus on characterizing sediment sources and contributing land use practices, particularly where sediment generation rates appear to be inordinately high. Project reach analysis must focus on assessing the channel's ecology, geomorphic stability, and behavior.

(2) Review of Available Material. Familiarization of the study area will be achieved through discussions with pertinent individuals and agencies and review of prior reports of the study area, including Federal Emergency Management Agency (FEMA) studies, hydrologic models, and aquatic surveys.

(3) Field Investigations. Generally, field investigations must be performed as necessary to characterize geomorphic conditions in the USR watershed and support a geomorphic stability assessment of the project reach. It is expected that the field efforts will be conducted with an engineer who is familiar with hydraulics, sedimentation, and geomorphology as well as a biologist familiar with the ecological area. The specific field investigations required are described in the following sections.

(4) Watershed Assessment. Available material must be researched, compiled, and reviewed. Field investigations will be conducted as necessary to perform the following tasks. It is expected that field examinations and verifications will be necessary to complete these tasks:

(a) Define the relevant geologic characteristics of the USR watershed and the USR 206 project reach. Acquire and review available topographic data, including U.S. Geological Survey (USGS) topo maps, surveyed topography, aerial photos, etc. Identify general subsurface, soil types, cover conditions, and relevant properties within the project reach.

(b) Prepare a summary and a timeline of the history of land use activities and associated geomorphic conditions in the USR watershed. Include all identifiable events of geomorphic significance.

(c) Identify USR sediment sources and contributing land use practices. Note where sediment generation rates appear to be inordinately high. Provide a general characterization of sediment sources based on their relative contribution to the project reach's bedload, suspended load, and wash load supply.

(d) Identify dominant geomorphic processes within the USR watershed. Assess whether each process is natural or anthropogenic.

(e) Identify any apparent geologic and/or anthropogenic structural controls on the geomorphic conditions of the watershed.

(f) Identify aquatic species that have been impacted in the study reach.

(g) Assess point and nonpoint source water quality impacts on the watershed with particular emphasis on the limitations that it may place on target aquatic species.

(h) Contact the state fish and wildlife agencies about their stocking program and locate stocking sites on the maps (frequency, species, etc.). Determine if the state has historic records of declining fisheries on any particular streams and/or reaches and locate these on a map. Assess records and studies to determine if specific blockages to fish passage have adversely impacted fisheries.

(5) Project Reach Assessment. Available material must be researched, compiled, and reviewed. Field investigations will be conducted as necessary to perform the following tasks. It is expected that an experienced field crew will need to walk the entire reach.

(a) Acquire and review available topographic data including USGS topo maps, surveyed topography, aerial photos, etc. Identify current and historic channel types, locations, planform characteristics (to include sinuosity, wavelength, and meander belt width). Prepare a figure displaying historic channel locations.

(b) Compare current and historic topographic (planform and vertical) data. Identify indications of historic channel behavior and current channel condition. When feasible, identify the effects of relevant anthropogenic activities on channel morphology. Estimate historic lateral migration rates. Prepare a figure displaying locations.

(c) Identify current geomorphic sub-reaches. Prepare a table displaying the beginning and ending stations of each sub-reach. Table C-1 may be used as a reference. Subsequent tasks will reference geomorphic sub-reaches identified, where appropriate.

(d) Assess point and nonpoint source water quality impacts to the reach. Collect water quality samples from high flow and low flow conditions from each of the sub-reaches. Prepare a figure displaying locations.

(e) Characterize the longitudinal location of the project reach relative to the watershed in terms of its sediment transport/generation behavior (zone of erosion, transportation, or deposition). Prepare a figure displaying locations.

(f) From field measurements, identify average channel morphometry for each sub-reach to include bankfull channel dimensions (defined as minimum width to depth ratio), overbank characteristics, and base flow channel dimensions.

(g) Identify active and remnant floodplain surfaces (terraces). Prepare a figure displaying locations.

(h) Characterize the bed material and bed forms. For each sub-reach, sample a representative reach for bed material load calculations and one representative area for low-flow habitat conditions. Prepare a figure or table identifying sample locations and characteristic hydraulic conditions relative to project stationing. Perform standard laboratory-size distribution (sieve) analysis. Prepare a standard plot(s) of bed sample gradation curves and a table(s) of bed sample grain size data. Provide a general characterization of the sources of project reach bed material.

(i) Provide a general characterization the ecology of each sub-reach. The inventory of aquatic habitat will use existing data on benthos and finfish sampling, as well as a rapid bioassessment of physical instream habitat to indicate current habitat conditions.

(j) Characterize the bank material and stratigraphy for each of the geomorphic sub-reaches. For each sub-reach, collect representative bank material samples. Perform standard laboratory-size distribution (sieve) or hydrometer analysis, as appropriate. Identify bank material, soil types, and properties. Provide a general characterization of the relative cohesiveness and erodibility of the bank materials. Identify bank erosion and failure mechanisms. Characterize the existing bank vegetation.

(k) Identify significant sediment sources and sinks within the study reach. Assess sediment impacts from sources in the upper basin above the study reach.

(l) Observe tributary, distributary, and relict channels in the project reach, and identify indications of channel behavior and geomorphic conditions.

(m) Observe anthropogenic features, including bridge abutments and piers, grade control structures, low-flow crossings, and bank protection. Identify impacts of features and indications of channel behavior and geomorphic conditions. Identify significant geomorphic controls in the project reach.

(n) Acquire and review USGS gaging station records, including surveyed cross sections and rating tables. Perform specific gage analysis and identify indications of channel behavior and geomorphic conditions.

(o) Characterize the current geomorphic stability of the sub-reaches in the project reach channel, whether incising or aggrading. Identify the severity and extent of any existing vertical or horizontal instabilities via a qualitative index.

(p) Characterize the grade conditions of the channel, whether incised, aggraded, or at-grade. Apply appropriate channel evolution models to identify current channel stage, subsequent stages of evolution, and the evolved stable channel form. Qualitatively estimate the time scale of channel recovery. Characterize the impacts of anthropogenic features on channel morphology and stability.

(q) Identify potential problem areas in the project reach. Characterize the potential for significant increase and/or decrease in the project reach sediment supply. Characterize the sensitivity of channel form to such variations, including expected channel form response, and the magnitude and time scale of expected adjustments. Characterize the impacts of anthropogenic features on expected channel behavior.

(r) Characterize the relative uncertainty of the assessment performed. Identify any additional analyses required to develop a reasonably certain GA. Typically, this effort would be performed in a subsequent phase. However, significant issues may need to be addressed immediately and require an overall study scope and schedule adjustment.

(6) Restoration Recommendations. Based on the GA performed, recommend restoration measures and appropriate analysis and design methodologies. Specific items to be addressed are described in the following sections.

(a) Recommend measures to restore geomorphic stability and ecological health to the project reach. Include an assessment of the appropriateness of bank protection, grade protection, instream habitat enhancement, channel realignment, wetland creation, and fish blockage removal.

(b) Recommend measures to reduce impacts to water quality from the project reach area. Include an assessment of the appropriateness of bank protection, grade protection, channel realignment, and riparian modifications.

(c) Document each proposed measure. The information will include an estimate of the potential project size, the general project type, a sketch of the site, the impact to and proximity of utilities, and an assessment of construction area and access. Provide a general estimate of costs.

(d) Address specifically the appropriateness and feasibility of possible projects. Particularly address impacts to existing floodplains. Address social and biologic controls that may limit possible projects.

(7) Meeting Attendance. There will be four required meetings to review study progress. These will be at the initiation of work, completion of field investigation activities, midpoint of the study process, and end of the study to present findings.

c. Quality Control and Quality Assurance. Quality control (QC) of the technical products produced under this scope of work will consist of development and execution of a Quality Control Plan (QCP), independent technical review (ITR), and Quality Control Certification (QCC). The experience and background of personnel selected for the field assessments will be reviewed to ensure that qualified personnel conduct the study. Initial team assignments should reflect technical team capability. Use of USACE technical experts should be considered when necessary. Products must be reviewed for compliance with standard engineering and professional practices, adequacy of the scope of the associated document, appropriateness of data used, consistency, accuracy, comprehensiveness, and reasonableness of results.

C-3. Example Scope of Work for Aggradation Assessment. This example scope of work was developed for an aggradation assessment at Garrison Dam performed by the Omaha District. The study purpose was to evaluate the potential for real estate impacts to occur in the future due to headwaters aggradation. Portions of this scope have been removed for use with this guidance document. A final scope should conform to USACE standards and clearly state scope items, government furnished information, and all contractor deliverables.

a. Introduction. The closure of Garrison Dam on the Missouri River at river mile 1,389.9 occurred in April 1953 to form Lake Sakakawea. Significant delta deposits have formed in the headwaters of the lake since closure. Engineering analysis is required to perform an evaluation of the impact of aggradation on water levels. Analysis will be performed for the pre-dam condition, the current condition, and a future aggradation condition. The purpose of the analysis is to provide a basis for future operational decisions and the evaluation of real estate claims related to aggradation and rising ground and surface water stages. As shown in Figure C-2, the analysis will focus on the Missouri River in the Williston, North Dakota area and significant tributaries including the Little Muddy and Yellowstone Rivers.

Figure C-2. Study area

b. Existing Data. Existing data available for the study includes:

(1) Previous studies (separate list provided, all documents will be sent to contractor along with scope for use by contractor in proposal preparation).

(2) Range line surveys (new and historical from 1956 and 1988) with state-plane coordinates. New surveys are currently being collected with data provided to the contractor in December 2012.

(3) Available historical groundwater information.

(4) Historical 1988 area-capacity data for Lake Sakakawea.

(5) Best available DEM for the area. This data is large and will be furnished on a portable hard drive. Note that DEM data is proprietary. The data furnished by the Government is for the sole use of the contractor on this project only. The contractor cannot copy it, provide it, or use it for any other projects/purposes without purchasing the data/license.

(6) Historical area-capacity Microsoft Excel table with sediment volume depletion and depletion rates per segment at elevation 1,860 (from 1953,1959,1969,1979, and 1988).

(7) Spreadsheet of available gage data for a portion of the gages (will require updating).

(8) *Note*: The contractor is responsible for acquiring all engineering analysis computer programs.

c. Coordinate System. All data and models will use the following coordinate system, Horizontal Datum: NAD 83, UTM 14 U.S. feet; Vertical Datum: NGVD 1,929 feet. Study results that require comparison of elevations, plan, etc., may require conversion of data to the current coordinate system, as not all source data will be in the specified project coordinate system. The coordinate system will be clearly stated in all report documents with the vertical datum referenced on all plots.

d. Description of Work. Engineering analysis tasks consist of the following:

(1) An overview of scope tasks is as follows:

(a) Project management coordination, quality control plan, etc.

(b) Review previous reports and available material; verify study area for each stream.

(c) Conduct data collection of topographic, flow-frequency, hydraulics, and sediment information at required locations.

(d) Run area-capacity assessments on the Missouri River and tributaries.

(e) Assess sediment loading from sources in the river reach between Culbertson and Garrison Dam and compare to area-capacity results.

(f) Assess future aggradation trends and develop information for input to the HEC-RAS model for the 50-year future condition.

(g) Construct steady-state, fixed-bed HEC-RAS models for the identified streams. Models will be constructed for the pre-dam, current condition, and future condition scenarios.

(h) Evaluate groundwater concerns with respect to well fields, septic tanks, domestic wells, and impact to crop lands.

(i) Fifty percent submittal.

(j) Compute water surface profiles for the 2-, 5-, 7-, 10-, 50-, 100-, and 500-year events.

(k) Prepare flood outline plates illustrating the 7- and 100-year flood outlines for all three conditions (pre-, current, and the 50-year future aggradation).

(l) Prepare engineering report.

(m) Prepare real estate block-out line.

(n) Prepare GIS database of model outputs, flood outlines, and collected data for all river reaches in the study.

(o) Complete final submission.

(2) Project Management. Project management tasks for the overall project includes attending a kick-off teleconference meeting, overall project coordination, preparing the overall quality control plan, and other minor tasks.

(3) Previous Report Reviews, Material Availability Analysis, and Study Area Verification. Familiarization of the study area will be achieved through the review of prior reports of the study area, including hydrologic models and survey information. The Omaha District will provide these studies in pdf format. Applicable previous studies include, but are not limited to:

(a) Missouri River Main Stem Reservoirs Hydrologic Statistics, RCC Technical Report (Updated).

(b) Missouri River Stage Trends, RCC Technical Report A-10, (Updated).

(c) Flood Hazard Report, Fort Peck Dam to Garrison Dam, 1986.

(d) Lake Sakakawea Headwaters Aggradation Study, 1962–1989, September 1990.

(e) Missouri River, Buford-Trenton Irrigation District, North Dakota, Reconnaissance Report, 1993, Volume 1 and Volume 2, published in 1998. Volume 2 includes sedimentation analysis, groundwater analysis, hydraulic analysis, etc.

(f) Bismarck Ground Water Study, 1983.

(g) Impacts of Siltation of the Missouri River in the State of North Dakota, Summary Report, 2009.

(4) Study Area Reviews. For the identified streams, the upstream extent will be defined by the existing survey data and the additional surveys collected as part of this effort. The contractor will review the available survey data to verify the extent of upstream cross sections required on each tributary. Extents are estimated as: Missouri River: 1,960 river miles (RM) 1,512.18 (Range 1,600.7, upstream of Tobacco Garden Creek) to RM 1,590.2 (Missouri River from Lake Sakakawea to upstream of the Yellowstone River). Tributaries included are the Yellowstone River (RM 0.0 to RM 19.6) and Little Muddy Creek (RM 0.0 to RM 8.4). No site visit or field surveys are proposed within the scope.

(5) Data Collection (Topographic). Omaha District public domain digital terrain models or other topographic data sources are not available for the site. The Omaha District will provide the contractor the best available source, a 3-meter DEM from a proprietary source, for floodplain mapping. Use of the proprietary data for purposes other than this study by the contractor is not permissible. The best available orthophotos will be acquired by the contractor from the data gateway center or similar location. The contractor is responsible for any necessary coordination system conversion. The data gateway center is available at http://datagateway.nrcs.usda.gov/.

(6) Flow-Frequency Information Analysis. Flow-frequency information for the Missouri River and tributaries will be determined by the contractor for locations shown in Table C-2. Some interpretation is likely required to develop flow distribution for the Missouri River model reach. The procedure is as follows:

(a) Missouri River: Combine the daily flows from the Missouri River at Culbertson, Montana, and Yellowstone River at Sydney, Montana, for the period of record. Add in the Little Missouri River record for the downstream reach. Perform statistical analysis on the annual peaks. Review existing reports and compare results. Compare to the Garrison estimated daily inflow record developed by the COE and previous studies. Develop the distributed flow frequency for the model reach on the Missouri River. *Note*: This method does not follow standard practice and ignores possible impacts from regulation.

(b) Tributaries: Annual discharge frequency relationships will be derived at the USGS stream gages listed in the table below. Frequency analysis will be accomplished using the methodology recommended in Water Resources Council Bulletin 17B with the best available gage station information at the Little Muddy River. The contractor will employ suitable procedures as outlined in Bulletin 17B.

Table C-2
Gages for Flow-Frequency Development

Gage Name	USGS Gage #	Period of Record
Little Muddy River below Cow Creek Near Williston, North Dakota	06331000	1954–2012
Missouri River, Culbertson, Montana	06185500	1941–2012
Yellowstone River, Sydney, Montana	06329500	1910–2012

(7) Hydraulic Data Reviews. The available sediment range data, the new survey data, and relevant data from previous studies should be assembled and reviewed. *Note*: New surveys of the Garrison aggradation sediment range lines are being collected now and will be furnished to the contractor upon completion, estimated to occur in December 2012. New field data collection is not part of this task.

(8) Sediment Data Reviews. Available sediment data from the previous studies should be assembled and reviewed. The available data should be used to assess previous estimates of stage trends in the study reach. Primary data sources will include the study "Impacts of Siltation of the Missouri River in the State of North Dakota" (USACE 2009) and the "Lake Sakakawea Headwaters Aggradation Study, 1962–1989" (USACE 1990).

(9) Extend Area-Capacity Assessment. Routine practice by the Omaha District is to perform the area-capacity analysis for Lake Sakakawea to the top of the dam elevation. Range line surveys indicate that sedimentation has occurred at elevations above that value. The contractor will extend the area capacity analysis above that level using the available range line surveys and the range line location. Currently the Garrison area-capacity tables are calculated to an elevation of 1,860.0 feet (NGVD 1929). The assessment should extend capacity to elevation 1,870.0 feet (NGVD 1929) using the Omaha District Windows™-based programs (OUP version 2012) which will be provided to the contractor. The assessment will be performed using both the 1988 and 2012 survey cross sections.

(10) Sediment Loading Assessment. The contractor will develop a sediment load assessment for Missouri River at Culbertson, Montana, and the area between Culbertson and the downstream study limit, 1,960 river miles 1,512.18 (Range line 1,600.7, upstream of Tobacco Garden Creek). The assessment will identify sediment sources and develop an estimate of contribution during the period measured by the area-capacity analysis. The contractor will evaluate accuracy of the sediment loading by comparing the results with the volume of material indicated by the area-capacity analysis. This includes evaluating results of the previous task and the historic excel table with sediment volume depletion and depletion rates per segment at elevation 1,860 (from 1953, 1959, 1969, 1979, and 1988).

(11) Future Aggradation Trends Assessment. Using historical and current survey data, the aggradation trend to the current geometry will be determined. The future aggradation trend assessment will consider the results of the area-capacity, sediment loading, and previous reports.

This trend will be projected into the future for 50 years from the most recent surveys (2012). The assessment will provide a mean, minimum, and maximum aggradation prediction for the 50-year prediction.

(12) Construction of HEC-RAS Models. A fixed-bed HEC-RAS model will be assembled from survey data for each identified stream for the pre-dam, current, and future conditions. The pre-dam and current condition models will be calibrated to available data from the respective time period. HEC-RAS model construction will follow accepted standards and guideline as stated in the HEC-RAS User's Manual and the HEC-RAS Applications Guide. Spacing of many sediment range sections exceeds two miles. Topographic data beyond the cross-section extent consists only of the best available DEM data. The contractor should add additional interpolated sections merged with overbank data as necessary to develop a detailed water surface profile. Use aerial photos to provide guidance for channel width and geometry when locating new sections.

(13) Existing Condition Model and Calibration. The constructed model should be calibrated to existing condition (post 2011 flood). The calibrated model will be based on observed trends at USGS and USACE gage stations. Gages available for calibration in the study area are shown below in Table C-3:

Table C-3
Calibration Gages

	Location	USGS No.	River Mile	Remarks
Missouri River Gages Culbertson, Montana, to Lake Sakakawea	Culbertson	6185500	1,620.76	USGS (stage/flow)
	Nohly	6185600	1,595.7	COE name M-4
	Nohly	6185650	1,587.7	COE name M-5
	Buford	6329640	1,577.5	COE name M-5A
	Buford	6329650	1,573.1	COE name M-6
	Trenton	6329660	1,566.7	COE name M-7_Historic, Discontinued
	Trenton	6329680	1,557.2	COE name M-8_Historic, Discontinued
	Williston	6330000	1,552.7	USGS (stage only)
	Williston	6330110	1,546.2	COE name M-9
	Williston	6331600	1,540.7	COE name M-10_Historic, Discontinued
Yellowstone River Gages Sidney, Montana, to Missouri River Confluence	Sidney	6329500	–	USGS (stage/flow)
	Fairview	6329590	–	COE name #3
	Cartwright	6329610	–	COE name #2
	Near Buford	6329620	–	COE name #1
Little Muddy River Gage	Williston	6331000	–	USGS (stage/flow)

(14) Pre-Dam Condition Computations. Geometry for the pre-dam condition model will be determined from the available historic range lines. The pre-dam condition for all models will be assumed to be from the 1956 dataset. The pre-dam condition model will have cross sections at all locations used by the current conditions model. At locations where data is missing for the pre-dam condition, the contractor will derive an approximate section elevation using trends from adjacent sections. Roughness adjustments will be minimal between the models and considered only when documented land use changes occurred.

(15) Construction of Future Aggradation Conditions. The future condition HEC-RAS model will be constructed from the existing condition model by applying the results of the aggradation trend assessment.

(a) Using 2011 surveys, the 50-year future condition will be assigned the year 2061.

(b) Three predictive future conditions will be assessed for the future condition. These will be the current trend aggradation future conditions, a 50% decrease in the aggradation rate, and 50% rate increase.

(c) All HEC-RAS models should maintain cross section geo-referencing.

(d) Two geometries will be constructed with different effective flow area encroachments. One condition will assume encroachment at selected locations (to be determined during model development and calibration) with no overtopping or effective flow landward of the encroachment station to create a confined condition. The second condition will be a normal unconfined condition.

(e) Analysis results will be presented to evaluate the effects of variable encroachment on future deposition locations. Report text is required to clearly describe the two modeling conditions. Notations on plots, profiles, and other similar type products is also required to avoid confusion.

(f) *Note*: The HEC-RAS modeling effort will not include any development of typical FEMA associated products or floodway preparation, the focus of this study is the future aggradation.

(16) Evaluation of Groundwater Concerns. Aggradation impacts also affect groundwater levels. Groundwater impacts have been roughly assessed in previous studies. The contractor will evaluate the previous studies and the limited additional information available.

(a) The groundwater evaluation will identify concerns with respect to well fields, septic tanks, domestic wells, and impact to crop lands.

(b) Potential groundwater impacts will be assessed based on the aggradation condition with variable lake levels. Findings from this analysis will be used to prepare a groundwater impact outline.

(c) The Omaha District will provide available information relating to groundwater issues addressed in previous studies. Two previous studies will also be provided that illustrate examples of the appropriate level of detail for the groundwater evaluation.

(17) Fifty Percent Review Submittal. Submit the preliminary results of work to date (aggradation assessment, RAS models, flow-frequency results, groundwater assessment) for review via teleconference. A brief summary for each effort will be required to determine progress. USACE will provide comments at this time to be addressed during the remainder of the effort. Annotated responses from the contractor should be included in final report. The review will be conducted via teleconference.

(18) Computation of Water Surface Profiles. Water surface profiles for selected streams (Missouri River, Yellowstone River, and Little Muddy River) will be computed for the 2-, 5-, 7-, 10-, 50-, 100-, 500-year profiles. Profiles will be computed for the three geometry conditions

(pre-dam, current, and future). The future condition profile should include both with and without conditions for the levee at Williston. For frequent events without levee overtopping, only a single profile is needed. Profiles will be summarized in both tabular and graphical format.

(19) Preparation of Flood Outlines. Flood outlines should be prepared for the 7- and 100-year event for all three conditions (pre-dam, current, and future). Groundwater impact areas should also be identified. The groundwater information should be presented on separate maps at legible scales for all streams. Flood outlines will be prepared using the cross-section data information and the best available DEM topographic information. Flood outlines will be prepared for all streams (Missouri River, Yellowstone River, and Little Muddy River). Flood outlines will be prepared showing both future levee conditions. For easy comparison purposes, a single plot showing both outlines is preferred. Areas without discernible difference may occur and should be labeled with text.

(20) Preparation of Real Estate Block-Out Line. The contractor will establish the boundaries of the flowage easements by blocking out the 7- and 100-year flood outlines for all three conditions (pre-dam, current, and future) consistent with sound real estate practices incorporating minor 2.5-acre sectional subdivisions based on the Public Land Survey System (PLSS). This information will be provided in a GIS format. For use with the real estate block-out, the contractor will select a single 100-year future condition profile of either levee overtopping or non-overtopping, based on which condition results in the greater areal extent. The selected condition should be noted on the plot and in the report text.

(21) Development of GIS Database. The contractor will prepare a GIS database to include, at a minimum, available aerial imagery, topographic mapping, GIS shape files of NRCS land use data, USGS gage data, model geometry data, flood boundaries for 7- and 100-year return probability events, real estate block out lines, and other data used or developed as part of this effort. The contractor will provide a GIS project in ArcGIS v. 9.3+ format conforming to the USACE GIS standard. USACE will provide this standard to the contractor. Metadata will be provided to explain each GIS coverage conforming to the Federal Geospatial Data Committee's "Content Standard for Digital Geospatial Metadata" (GSDGM), v. 2.

(22) Development of Written Report and Draft Report Review Conference. The contractor will perform all computations as described in the scope and will document the design, analysis, and computations in a report. The contractor report will address all engineering components and analysis tasks such that a single report contains a complete description of all performed tasks. All prepared reports will conform to the guidance in the engineering analysis report format section. The contractor will provide the draft report and all associated electronic files for review. The Government will provide comments within three weeks after receiving the draft report. The draft report review conference will be held via teleconference to discuss review comments.

(23) Final Report Delivery. The contractor will incorporate all comments and provide a final report.

C-4. Example Scope of Work, Missouri River RAS Sediment Analysis. This is an example scope of work for Missouri River sediment analysis with HEC-RAS. Tasks are for the development of an HEC-RAS sediment model to assess alternatives and compare to a base condition model. *Note*: Significant portions of this scope have been removed for use with this guidance document. A final scope should conform to USACE standards and clearly state scope items, government furnished information, and all contractor deliverables.

a. Introduction. The existing Missouri River unsteady HEC-RAS model provides a base model for use with planning studies which could be used to simulate and analyze broad scale watershed alternatives. Sediment modeling is proposed to assess current conditions and evaluate future aggradation/degradation changes. Future conditions that include aggradation and degradation within the reservoir reaches and navigation channel would likely result in significant change to the river stage-flow relationship. This scope discusses the method selected to evaluate aggradation/degradation changes from the current condition, development of a future condition, and the ability to provide future condition model outputs for impact analyses.

(1) Background. Historically, the Missouri River was a free-flowing, highly dynamic, multi-channel river, consisting of highly variable flows, high turbidity conditions, with many channel sandbars throughout the river channel and floodplain, which provided a diversity of habitat and food resources for many terrestrial and aquatic organisms.

(a) Since the late 1800s, the Missouri River has been modified by reservoirs, bank stabilization, construction of the navigation channel, and many other water resources development projects that have affected basin sediment yield and sediment transport within the mainstem Missouri River and tributaries.

(b) Navigation channel and bank stabilization structures have altered the historic multi-channeled, highly variable river system into a predominantly deep and swift single channel in the lower river downstream of Sioux City.

(c) These actions have resulted in noticeable aggradation and degradation trends on the mainstem Missouri River. For instance, degradation downstream of Gavins Point Dam has been observed in the range of 10 to 12 feet since dam closure in 1955.

(d) During the extreme 2011 event, many areas on the Missouri River experienced single-event degradation and aggradation in the range of 2 to 4 feet.

(e) In summary, aggradation and degradation trends are documented, recent, and known to be ongoing.

(2) Study Modeling Extents. USACE has developed five separate HEC-RAS models for discrete reaches of the Missouri River between the mainstem dams and downstream. This scope pertains only to RAS modeling for the Gavins to Rulo reach within the Omaha District. Figure C-3 illustrates the locations of the individual HEC-RAS models.

Figure C-3. Location of HEC-RAS modeled reaches

(3) Modeling Method Selection. Two approaches were considered for projecting future aggradation/degradation within the modeling reaches consisting of HEC-RAS with sediment modeling for the study reach and using historic trends at gage stations (ft/yr water surface change) extrapolated to the entire study reach. Sediment modeling was selected to develop a technically sound product that follows modeling standard practice. The RAS model will be able to evaluate how flow and geometry changes affect sediment transport capacity and identify differences between alternatives. The final product suite of models provides a tool that can be used in the future to address study issues and plan revisions.

(4) HEC-RAS with Sediment. HEC-RAS with sediment was selected as the preferred model to provide a technically sound product that will follow modeling standard practice to develop estimates of the distribution of future aggradation and degradation. HEC-RAS model results can be used to project future water surface elevations.

(5) Historic Modeling. A historic HEC-RAS model will be developed for each study reach and calibrated to the historic conditions. Historic surveys are available from the 1970s or 1980s to create the historic condition model. Sediment parameters including sediment load inflows, grain size, and transport functions will be selected to produce a reasonably accurate model of geometry changes from the historic to current condition. The same parameters will then be

applied to the current condition model to evaluate how alternative geometry and flow changes affect sediment transport capacity.

b. Scope Overview.

(1) An overview of the primary sediment modeling tasks are as follows:

(a) Create a historic condition HEC-RAS model and calibrate the model for the historic period.

(b) Add parameters and simulate with HEC-RAS sediment from the historic to current period. Calibrate the sediment parameters to reproduce bed and water surface elevation changes.

(c) Import the sediment parameters to the previously created current condition (2012) model.

(d) Simulate the future flow record.

(e) Review results and revise modeling parameters as needed.

(f) Create report documentation discussing RAS modeling and analysis results.

(2) Models and Historic Data Assembly. Assemble the 2012 calibration condition HEC-RAS model, current and historic hydrographic survey data, and other available models within each model reach. Interim survey data will be assembled for use with assessing sediment model performance. Models and historical data are summarized in Table C-4.

Table C-4
Summary of Historical Data and Model Information

Model/RM Range	Historic to Current Survey Data to Assemble	Approx. No. Cross Sections[2]	Other HEC-RAS and HEC-2 Historic to Assemble
Gavins to Rulo 811.1 to 498.0	1975[1]	1451	Flow frequency model with 1994/1995 hydrographic data
	1994/1995	–	–
	2007	100	LCLSMS Model, 2007 geometry (to Sioux City, Iowa)
	2011	100	LCLSMS Model, 2013 geometry with BSTEM (to SUX)
	2013	–	–

[1] Estimated date, actual date is unknown. Survey is only available in HEC-2 model format.
[2] Number of sections within each model will be reduced for sediment modeling, approximate spacing 2 to 4 miles.

(3) Creation of Historic Condition HEC-RAS Model. Create a HEC-RAS working model for the historic condition. Calibrate the historic model to available gage data and water surface profiles. Specific tasks include:

(a) Collect historic period hydrographic survey sections. Convert to station-elevation and import to HEC-RAS in cross section format. Revise to 88 vertical datum.

(b) Compare reach lengths to the 2012 current condition model and ratio reach lengths if necessary to have similar distance at comparable section locations, using river mile as the basis for comparison.

(c) Merge with best available floodplain topography (use current if nothing better is available electronically).

(d) Set bank stations to near top of bank using 10-year profile.

(e) Set ineffective and encroachment stations to provide reasonable top width. Verify transition from narrow to broad channel in the reservoir delta. Use the 2012 calibration model as a guide.

(f) Assign channel and overbank roughness values. Use the values in Table C-5 as an initial estimate:

Table C-5
Initial Roughness Value Estimate

Location	**Manning's n Value**
Channel	0.025
Floodplain: Trees and Vegetation	0.065
Floodplain: Farmland	0.045
Floodplain: Marsh	0.055
Floodplain: Urban	0.085

(g) Simulate a range of flow profiles (minimum flow of 5,000 cfs, 2-, 10-, 50-, 100-year). Review model outputs and perform initial debug of model geometry for any high-velocity areas, large changes in water surface elevation, top width, flow area, velocity, etc.

(h) Calibrate the steady-flow model to the historic condition using water surface profile and gage data (corrected to the 88 vertical datum). Verify with any measured velocity data that is available. Multiple repetitions of model debugging and revisions may be needed in this step.

(4) Quasi-Unsteady Flow Model Conversion. Sediment modeling will be performed using a quasi-unsteady flow model with daily flows for the historic (1975) to 2012 period. Simulation will require the following tasks:

(a) Create the quasi-unsteady flow version of the historic condition model. Run the model using a constant daily flow of 20,000 to 30,000 cfs to debug the model. Compare profiles to the steady flow model at the same flow to verify conversion. Use current condition modeling parameters as the initial values for the HTAB, computational tolerances, and computational time step as an initial guide when setting up the quasi-unsteady flow model.

(b) The upstream boundary will be Gavins Point Dam releases. Extend the model downstream for a sufficient length (approximately 50 river miles) to allow the use of a normal depth boundary.

(c) Assemble daily flow data for all input sources for the modeling period (historic to 2012). Data is required for the mainstem upstream release, all available mainstem gage station locations (stage and flow), and tributaries. All flow data for use with the historic period quasi-unsteady flow simulation will be observed data and will not use any flows that are corrected for basin water development or depletions due to consumptive use.

(d) Simulate the historic period with the assembled flow data. Review gage station calibration. Adjust calibration parameters (ineffective flow, encroachments, roughness) as necessary. This step will also require use of ungaged inflow for the historic period. Ungaged flows were previously developed with the unsteady flow RAS model and are available for use.

(5) Reuse of Daily Flows. All modeling efforts will use the previously developed daily flows for the Missouri River and all tributary inflows that spans from 1930 to 2012. Data is available in dss format.

(6) Simulation of Bed Change and Calibration of Sediment Model for Historic Period. Use HEC-RAS with sediment to simulate from the historic period geometry to current condition. Comparison between the model simulated and actual current condition geometry will demonstrate the model ability to generate aggradation/degradation with reasonable accuracy. The historic model period varies by location but is generally from the 1970s to 1990s to current (2012). Sediment modeling within HEC-RAS uses sediment computations to determine bed change. Bank erosion will be specified as an input to the model using previously determined BSTEM results.

(7) Simplifying Assumptions. The RAS model will utilize only the sand fraction of the sediment load (material greater than 0.0625 mm). Silts and clays, which act as wash load and remain suspended in the water column, will not be included in the sediment model.

(8) Assembly of the Sediment Model Input Data.

(a) The model starts at the upstream Gavins Point Dam, assume no sediment input from the reservoir release.

(b) Assemble Missouri River bed gradation from historic bed samples. Review data to ensure that the bed material includes larger gradations to reflect armoring in the immediate reach downstream of the reservoir. Plot D_{10}, D_{50}, and D_{90} by river mile to identify any data outliers.

(c) For tributary sediment input, assemble sediment load information using best available USGS gage data and previous studies. Plot all sediment load curves and material sizes (D_{10}, D_{50}, D_{90}) to compare input data in Excel. Also compare sediment load/square mile of drainage area for each inflow point.

(d) Due to the influence of bed forms on river stage, calibrate all models to the warm season (mid-April through mid-October), which is of primary interest.

(e) Within the navigation channel reach of the Gavins to Rulo model, temperature is known to affect bed forms. Do not adjust roughness for the bed form temperature shift.

(f) Set mobile bed limits using the bank stations.

(g) Select Toffaleti, Laursen, Copeland, or Yang as the transport function (initial that will be verified in calibration).

(h) Select Copeland (Exner 7) or Exner 5 as the sorting method (initial that will be verified in calibration).

(i) Select Report 12 or Ruby fall velocity method (initial that will be verified in calibration).

(9) Sediment Model Startup. Perform a stability check of the assembled sediment model and initial sediment parameter review for low, medium, and high flows.

(a) Create 30-day constant flow files of low (10k), medium (near bankfull; for example, 70,000 for below Gavins), and high flows (above bank in the non-degradation reach; for example, 100,000 for below Gavins).

(b) Simulate sediment model for each flow condition, start with the medium flow condition for initial evaluation and debugging.

(c) Review performance at each flow, revise model levee stations, encroachments, ineffective flows, and/or cross-section spacing to achieve reasonable sediment response.

(d) Check bed level change and debug problem areas.

(e) Confirm that model sediment inputs are reasonable by comparing to gage station sediment load rating curves.

(f) Perform initial adjustment of sediment input parameters to achieve 30-day model stability.

(10) Refinement of Sediment Model for Selected Initial Periods.

(a) Simulate the flow period from March 1 through December 15, 2006.

(b) Review performance at each flow, revise model levee stations, encroachments, and/or ineffective flows to achieve reasonable sediment response.

(c) Check bed level change and debug problem areas.

(d) Confirm that model sediment inputs are reasonable by comparing to suspended gage records at gage stations (when available) or sediment load rating curves.

(e) Model results should be fairly stable for this period throughout with minimal degradation/aggradation.

(f) Repeat process for another annual flow period selected to provide a low- and high-flow range (for example, 2009 and 2010 for downstream of Gavins).

(11) Final Calibration and Simulation of Historic Period.

(a) Simulate the flow period for the difference between surveys as shown in Table C-4 (for example, Gavins reach is 1975 thru 2012). Use daily unsteady flow with tributary inflow and ungaged as previously developed.

(b) The initial assumption will be that roughness is consistent during the modeling period and no adjustment is required.

(c) Review and refine model sediment inputs. Compare model results to available measured data during the historic period at gage locations (such as Omaha).

(d) Compare model-computed volume change to actual on a reach basis. Utilize historic hydrographic interim surveys when available.

(e) Compare model-computed sediment concentration to gage stations for periods when available.

(f) Conduct sensitivity analyses to determine the limit of uncertainty in the model results.

(g) Use estimated project sediment inputs for channel revision projects (such as construction of habitat modification projects that include dredge discharge and river process widening).

(12) Tabulation of Sediment Model Calibration Parameters. The primary product from the development and calibration of the historic sediment model are the sediment modeling parameters. These parameters will be tabulated and reviewed for use with the 2012 condition sediment model.

(13) Update of the RAS Current Condition Model. Using the bed change model created for the historic condition, update the 2012 RAS model sediment.

(a) Within the current condition model, review and revise bank stations, levee points, and ineffective flow areas as necessary to generally correspond with the historic condition model methodology (the task is to limit the variation in sediment transport due to variance in model geometry).

(b) Compute steady-condition water surface profiles for a low, medium, and high flow. Compare to 2012 observed data water surface profiles and gage data. Revise calibration if necessary.

(c) Create a 2012 sediment data input file that corresponds with the historic model calibrated sediment data input file.

(d) Debug 2012 sediment model by running the model 30-day constant flow files of low (10k), medium (near bankfull; for example, 70,000 for below Gavins), and high flows (above bank in the non-degradation reach; for example, 100,000 for below Gavins). Review results and modify as needed.

(e) Simulate the historic period with the new sediment model. While this will be starting with an initial model much different than at the start of the historic period, it should provide an indication of model resiliency. Compare model bed change and water surface elevation change (ft/yr and total) to that determined with the historic condition model. Review computations in areas with significant change from historic.

(f) Perform minor sediment parameter adjustments in both the historic and 2012 models if needed to achieve reasonable results and model stability. If at all practical, both the historic and 2012 models should use the same parameters.

(g) The completion of these steps provides a current condition sediment model that can be used to evaluate future aggradation/degradation.

(14) Creation of Alternative Condition Geometry Models. Revise the current condition model geometry for the various alternative conditions as summarized in Table C-6.

Table C-6
Geometry Summary

Geometry	**Target Habitat**	**Reference Flow (cfs)**
No Action (Alt 1)	20 acre/mile	August 50% exceedance
BiOp (Alt 2)	30 acre/mile + floodplain connectivity	Summer low, median August, and spring pulse
IRC (Alt 3–6)	260 acre/year	Median June

(15) Sediment Analysis for Alternative Conditions and Evaluate Results.

(a) For each alternative geometry, simulate the flow record with the RAS alternative geometries.

(b) Compute a future condition alternative condition design profile from the base sediment model. Simulate steady flow at 30,000, 70,000, and 100,000 cfs with the final geometry from the sediment analysis.

(c) Compare bed profile and volume change between the base condition and alternatives.

(d) Evaluate the model predicted bed changes at the flow period and develop an average rate of bed change (ft/yr).

(e) Also compute the spatial distribution of bed and volume change using break points derived from results.

(16) Drainage Area Accounting. Tasks should be oriented to focus on the major drainage area tributaries using the drainage area accounting (Table C-7) that provides an accumulation of drainage area for each inflow point.

(17) Documentation. Prepare documentation consisting of a main report with methodology overview and separate appendixes to illustrate detailed calibration and individual alternative results. Within the main report, provide detailed discussion on difference between bed change for each alternative. Also provide a qualitative discussion of possible implications on damages and costs.

c. Remaining Scope Tasks. The remaining portion of the scope (including quality control, government-furnished items, deliverables, schedule, meetings, etc.) were removed for guidance document purposes.

Table C-7
Drainage Area Accounting, Gavins Point Dam to Rulo, Nebraska

Stream	Station Name/ Location	Station Number	Missouri River Mile	Bank (Left/Right)	Missouri River Drainage Area (mi^2)	USGS Flow Record (daily data)	Tributary Drainage Area (mi^2)	Cumulative Drainage Area (mi^2)	RM Weighted Unassigned Area (mi^2)	Cumulative with Unassigned Drainage Area (mi^2)	Percent Unassigned Drainage Areas	Contributing Drainage Area (mi^2)
Missouri River	**Gavins Point Dam at Yankton, SD**	**06467500**	811.1	- - -	279,500	10/1/1930 -->		279,500		279,500		- - -
James River	Yankton, SD	06478513	797.7	Left		10/1/1981 -->	20,947	300,447	318	300,765		18,891
Bow Creek	St. James, NE	06478518	- - -	- - -		10/1/1978 --> 9/30/1993	304	- - -	- - -	- - -		- - -
	Ungaged	- - -	- - -	- - -		- - -	158	- - -	- - -	- - -		- - -
	Mouth	- - -	787.6	Right		- - -	462	300,909	239	301,466		- - -
Lime Creek	Ungaged	- - -	776.3	Right		- - -	31	300,940	268	301,764		- - -
Vermillion River	Vermillion, SD	06479010	- - -	- - -		10/1/1983 -->	2,253	- - -	- - -	- - -		1,775
	Ungaged	- - -	- - -	- - -		- - -	344	- - -	- - -	- - -		- - -
	Mouth	- - -	771.9	Left		- - -	2,597	303,536	104	304,465		- - -
Aowa Creek	Ungaged	- - -	745.2	Right		- - -	222	303,758	633	305,319		- - -
Elk Creek	Ungaged	- - -	737.3	Right		- - -	131	303,888	187	305,637		- - -
Big Sioux River	Akron, IA	06485500	- - -	- - -		10/1/1928 -->	7,879	- - -	- - -	- - -		6,996
	Ungaged	- - -	- - -	- - -		- - -	965	- - -	- - -	- - -		- - -
	Mouth	- - -	734.0	Left		- - -	8,844	312,733	78	314,560		- - -
Unassigned	Ungaged	- - -	- - -	- - -		- - -	1,867		40	314,600	**5.3%**	
Missouri River	**Sioux City, IA**	**06486000**	732.3	- - -	314,600	10/1/1928 -->		314,600		314,600		- - -
Perry Creek	Sioux City, IA	06600000	732.1	Left		10/1/1945 -->	65	314,665	1.2	314,666		- - -
Floyd River	James, IA	06600500	- - -	- - -		12/8/1934 -->	886	- - -	- - -	- - -		- - -
	Ungaged	- - -	- - -	- - -		- - -	30	- - -	- - -	- - -		- - -
	Mouth	- - -	731.3	Left		- - -	916	315,581	4.7	315,587		- - -
Pigeon Creek	Ungaged	- - -	720.3	Right		- - -	72	315,653	65.2	315,724		- - -
Omaha Creek	Homer, NE	06601000	- - -	- - -		10/1/1945 -->	174	- - -	- - -	- - -		- - -
	Ungaged	- - -	- - -	- - -		- - -	13	- - -	- - -	- - -		- - -
	Mouth	- - -	719.9	Right		- - -	187	315,840	2.4	315,913		- - -
Horsehead Creek	Ungaged	- - -	703.6	Right		- - -	13	315,853	96.6	316,023		- - -
Blackbird Creek	Macy, NE	6601100	697.6	Right		10/1/1978 --> 10/6/1980	102	315,955	35.6	316,161		- - -
Unassigned	Ungaged	- - -	- - -	- - -		- - -	245		39.1	316,200	**15.3%**	
Missouri River	**Decatur, NE**	**06601200**	691.0	- - -	316,200	10/1/1987 -->		316,200		316,200		- - -
Elm Creek	Ungaged	- - -	690.7	Right		- - -	32	316,232	2	316,234		- - -
Monona Harrison Ditch	Turin, IA	06602400	- - -	- - -		5/7/1942 -->	900	- - -	- - -	- - -		- - -
	Ungaged	- - -	- - -	- - -		- - -	66	- - -	- - -	- - -		- - -
	Mouth	- - -	670.0	Left		- - -	966	317,199	106	317,306		- - -
Little Sioux River	Turin, IA	06607500	669.2	Left		5/7/1942 -->	3,526	320,725	4	320,836		- - -
Tekamah Creek	Tekamah, NE	6608000	664.9	Right		7/1/1949 --> 9/30/1981	23	320,748	22	320,881		- - -
Soldier River	Pisgah, IA	06608500	664.0	Left		3/5/1940 -->	407	321,155	5	321,293		- - -
Old Soldier River Ditch	Ungaged	- - -	649.3	Left		- - -	100	321,255	75	321,468		- - -
Fish Creek	Ungaged	- - -	647.9	Right		- - -	124	321,379	7	321,599		- - -
Boyer River	Logan, IA	06609500	635.2	Left		5/24/1918 -->	871	322,250	65	322,535		- - -
Pigeon Creek	Ungaged	- - -	622.0	Left		- - -	166	322,416	68	322,769		- - -
Unassigned	Ungaged	- - -	- - -	- - -			384		31	322,800	**5.8%**	
Missouri River	**Omaha, NE**	**06610000**	615.9	- - -	322,800	9/1/1928 -->		322,800		322,800		- - -
Mosquito Creek	Ungaged	- - -	605.8	Left		- - -	238	323,038	148.2	323,186		- - -
Big Papillion Creek	Fort Crook, NE	06610795	596.6	Right		- - -	384	323,422	135.0	323,705		- - -
Platte River	Ashland-Louis.	06805500	594.8	Right		6/1/1953 -->	85,370	408,792	26.4	409,102		71,000
Watkins Ditch	Ungaged	- - -	587.5	Left		- - -	185	408,977	107.1	409,394		- - -
Weeping Water Creek	Union, NE	06806500	568.7	Right		3/1/1950 -->	241	409,218	275.8	409,911		- - -
Unassigned	Ungaged	- - -	- - -	- - -			782		89.5	410,000	**0.9%**	
Missouri River	**Nebraska City, NE**	**06807000**	562.6	- - -	410,000	8/11/1929 -->		410,000		410,000		- - -
Nishnabotna River	Hamburg, IA	06810000	542.1	Left		3/1/1922 -->	2,806	412,806	181.8	412,988		- - -
Little Nemaha River	Auburn, NE	06811500	- - -	- - -		9/1/1949 -->	792	- - -	- - -	- - -		- - -
	Ungaged	- - -	- - -	- - -		- - -	101	- - -	- - -	- - -		- - -
	Mouth	- - -	527.8	Right		- - -	893	413,699	126.8	414,007		- - -
Rock Creek	Ungaged	- - -	522.2	Left		- - -	109	413,807	49.6	414,165		- - -
Tarkio River	Fairfax, MO	06813000	507.6	Left		3/8/1922 -->	520	414,327	129.4	414,815		- - -
Unassigned	Ungaged	- - -	- - -	- - -		- - -	573		85.1	414,900	**11.7%**	
Missouri River	**Rulo, NE**	**06813500**	498.0	- - -	414,900	10/1/1949 -->				414,900		- - -

Appendix D
River and Reservoir Site Visit Checklists

D-1. Introduction.

a. A checklist of tasks and observations should be made during the field reconnaissance that pertain specifically to reservoir sedimentation processes and associated impacts in the river. Topics contained in the checklist are general and will require additional research and investigation from applicable sources.

b. The site visit checklists provide basic guidance for preparation, questions, basic sediment sampling, trip report, and field tables. The checklists are intended as a supplement to available technical guidance to provide guidance for common issues encountered.

c. All reservoirs have significant geomorphic and hydraulic impacts on the local river reaches. In that sense, they are connected as a system. However, separate checklists have been developed for the footprint of the reservoir and the river reaches. In most cases, both checklists will be used.

d. The checklists are shown in Figures D-1(a–d). These sheets should be used during the site visit to ensure that all the necessary observations are documented.

D-2. Study Objectives. Before a site visit, a thorough assessment of study objectives should be performed. Detailed studies will usually include the development of a data collection plan that is based on study phase and objectives and considers future needs and study evolution. Site observations and data collection tasks will vary substantially depending on study objectives, analysis tasks, and study phase. An assessment of project risk and performance consequences is also a component of the data collection plan. A trip report at the conclusion of the site visit to record observations and photos is a critical component.

D-3. Typical Site Visit Focus Areas. The outcome of the site visit typically addresses these primary areas:

a. Bed and bank erosion controls.

b. Bed and bank sediment description.

c. Top of bank indicators.

d. Bank stability.

e. Vegetation characteristics.

f. Planform and channel geometry.

g. Hydrologic indicators.

h. Environmental features.

i. Sediment sinks and sources.

j. Erosive areas of concern.

k. Existing structures.

l. Floodplain/riparian characteristics.

D-4. Pre-Field Visit Tasks.

a. Perform a series of tasks before the site visit to develop a background understanding of the area, sediment processes that have been active in the past, and existing or potential future problem areas.

b. Typical tasks could be grouped into three sections:

(1) Literature Review.

(a) Consult the governing USACE Safety Plan and review applicable activity hazard analysis.

(b) Collect topographic maps of area from multiple time periods, if available.

(c) Research available reports. Typical sources include USACE, flood insurance studies, USGS, journal articles, etc.

(d) Evaluate historic and current aerial photographs.

(e) Before the site visit, identify areas of concern related to study objectives, historic problems in the study area, and an assessment of likely future problems.

(2) Data Gathering and Review.

(a) Check for available hydraulic models.

(b) Use Google Earth or similar web-based image server to learn about the location, access, and current conditions. Look for adjacent or surrounding hydraulic controls or infrastructure (upstream dams, bridges, diversions, bedrock outcrops, alluvial fans from tributaries, etc.). If possible, use historic photos in Google Earth to look for any visible changes in the river bed/banks, sandbars, land use, vegetation, etc.

(c) Evaluate recent flow history, river, and reservoir stage data. This information can be useful in the field when assessing current stability signs.

(d) Examine multiple surveys for trends.

(e) In reservoirs, examine the historical area-capacity relationship and evaluate trends.

(3) Visit Planning.

(a) Prepare all required travel documents, make travel arrangements, and coordinate with fellow team members regarding site visit purpose and tasks.

(b) Prepare a list of items to take on the field visit, including a camera, digital voice recorder, mobile phone, pen or pencil, paper to take notes, and applicable public communication brochures. Include seasonal items such as bug spray, sunscreen, etc. Include appropriate field clothing and footwear.

(c) Note boundary conditions including stream inflow rate, reservoir pool elevation, etc. Compare to normal levels.

(d) Examine data from the nearest stream gages and determine the flow at the time of the visit. Are you visiting on a high- or low-flow day?

(e) Compile the available sediment samples and plot the data to look for peak discharge correlation with sediment concentrations. This information can indicate whether the basin is flashy or base flow driven.

(f) After the data gathering effort, what additional data do you want to collect during a visit? Consider the size of the project and identify the best and safest methods to fill data gaps. Would a handheld velocity meter be useful, or is the channel too deep? If collecting sediment samples, will the analysis be in-field or returned to a sediment lab? Would a rod and level or a laser rangefinder be beneficial?

(g) Assemble the Reservoir and River Site Visit Checklists (see following figures).

D-5. Site Visit Questions and Checklist for Reservoirs.

a. Site observations and data collection should follow the tasks previously identified in the data collection plan. Attempt to answer the following questions while on the site visit:

(1) Ask the local sponsor or operations personnel (if applicable) about the history of the reservoir.

(2) Check on magnitude and frequency of past flood events. What was the reservoir pool elevation? Were there visible impacts to the reservoir?

(3) Are there existing levees or bank stabilization in the reservoir?

(4) Are there areas where wind wave erosion has occurred?

(5) Have sedimentation impacts due to delta formation been observed or measured?

(6) Is sedimentation affecting reservoir access (marinas, boat ramps, etc.)?

(7) Is sedimentation affecting water quality and/or water supply? Are municipal water intakes in the reservoir experiencing problems related to sedimentation?

(8) What is the most recent sedimentation survey on the reservoir?

(9) Are any reservoir management techniques being used to manage sediment (flushing, sluicing, bypass)?

(10) Is there a sediment management plan in place?

(11) Has any reservoir sediment management modeling been done?

b. Typical reservoir observations include:

(1) Representative photos of both normal and identified problem areas.

(2) Take pictures of any bridges crossing the reservoir. Determine rough estimates of bridge pier widths, bridge length, and bridge width.

(3) Bed and bank material slope within the study area.

(4) Record all identified headcuts and knickpoints on the tributaries entering the reservoir.

(5) Obtain representative samples of the bed and bank material in both primary and any tributary deltas.

(6) Note condition of banks, whether stable or caving, and the type of material found in the reservoir banks, particularly any lenses.

(7) Note presence of reservoir bank protection measures, their size, why they were placed.

(8) Record variation in observed conditions by spatial location.

(9) Record drift accumulations, debris.

(10) Record the extent and distribution of vegetation on delta deposits.

(11) Note changes in bed gradation and take representative samples for the sediment study.

(12) Note any processes removing sediment (mining, dredging, etc.).

(13) Note tributary entry points, the amount of flow, turbidity of flow, condition of the tributary.

(14) Note any flow diversion points.

(15) Note natural grade controls such as rock outcrops.

(16) Note gage locations, type of gage.

(17) Note existing similar projects on same or adjacent streams (for future comparison).

(18) Note overbank conditions, possible sloughing due to erosion at the normal water level.

(19) Take velocity measurements at several locations.

(20) Talk with local stakeholders to identify problem areas, get a time estimate of problems.

(21) Obtain a list of local contact information for items such as city/county engineer, flood history, economics, etc.

(22) Obtain information on all significant structures on maps/photos for future reference. Perform field measurements of drainage structures. Note any obstructions or anomalies regarding entrance and exit conditions.

(23) View all gage sites and collect photos. Note any concerns with the gage station location with reference to structures, roads, etc. Note any staff gage readings during the site visit.

D-6. Site Visit Questions and Checklist for Rivers. Site observations and data collection should follow the tasks previously identified in the data collection plan. Attempt to answer the following questions while on the site visit:

a. Check with local sponsor on magnitude and frequency of past flood events. Any records in newspapers, photos, discharge measurements? Where and when does flooding occur? Does debris aggravate the flooding? Does flooding occur in the winter from ice jams? Were dollar damages estimated from the flooding? Does a stage-discharge gage exist in the study area?

b. Take pictures of bridges and channel upstream and downstream from the bridge. Rough estimates of bridge pier widths, bridge length, and bridge width. Does the city or county have bridge plans?

c. Are there existing levees or channel improvements? Does the city or county have plans?

d. Has the stream been stable? Has it deepened or widened?

e. Obtain a list of local contact information for items such as city/county engineer, flood history, economics, etc.

f. Obtain information on all significant structures on maps/photos for future reference. Perform field measurements of drainage structures. Note any obstructions or anomalies regarding entrance and exit conditions.

g. View all gage sites and collect photos. Note any concerns with the gage station location with reference to structures, roads, etc. Note any staff gage readings during the site visit.

D-7. Typical River Observations. Typical river observations include:

a. Representative photos of both normal and identified problem areas.

b. Bed and bank material slope within the study area.

c. Stream general slope and any break points. Record all identified headcuts and knickpoints in both the primary stream and any tributaries within the study area.

d. Representative samples of the bed and bank material in both the main stream and tributaries with significant sediment input.

e. Notes on condition of banks, whether stable or caving.

f. Records of variation in observed conditions by spatial location.

g. Notes on any processes removing sediment (mining, dredging, etc.).

h. Notes on tributary entry points, the amount of flow, turbidity of flow, condition of the tributary.

i. Notes on any flow diversion points.

j. Notes on natural grade controls such as rock outcrops.

k. Notes on presence of protection measures, their size, why they were placed.

l. Notes on gage locations, type of gage.

m. Notes on structural feature locations and observe bank and bed conditions in the vicinity of the structures.

n. Notes on overbank conditions; areas of scour or deposition. If deposition exists, obtain samples, measure depth, and note extent on map. Are there high water signs on trees or debris in the floodplain indicating previous events?

o. Velocity measurements at several locations.

p. Conversations with local stakeholders to identify problem areas, get an estimate of time of problem.

q. Inquiries into local land use history; when urbanized, cleared, etc.

D-8. Collection of Physical Sediment Data. It is useful to collect basic sediment data in the river or reservoir being examined. While the data collection described here would not be sufficient for any empirical or numerical analysis, it can serve to inform on the system characteristics at the reconnaissance level.

a. Bed Material Data. Chapter 4 and Appendix E of this document provides information for the collection of bed material data.

b. Suspended-Sediment Data.

(1) Chapter 4 of this document provides information regarding suspended sediment samplers.

(2) If laboratory resources are available after the site visit, it may be valuable to collect suspended-sediment samples in flowing water above and/or below the reservoir.

(3) In the river reaches, individual suspended-sediment samples can be collected easily in shallow rivers (less than 5-foot deep) using a DH-81 Suspended-Sediment Sampler. The samples collected will then need to be analyzed at a laboratory. While collecting sediment data by this method is not comprehensive, it can be very valuable when there is little or no historical data available.

(4) The DH-81 sampler and others are available through Federal Interagency Sedimentation Program (FISP) (http://water.usgs.gov/fisp/). The FISP website also includes links to the operator's manuals for all samplers. For example, the DH-81 manual is located at http://water.usgs.gov/fisp/docs/Instructions_US_DH-81_010612.pdf.

(5) In most cases, suspended sediment in reservoirs is very fine and has to be sampled using appropriate water quality sampling techniques, or boat access is required to use samplers (Chapter 4).

c. Hydraulics Data.

(1) Use a hand-held velocity meter to estimate channel velocity.

(2) Estimate width and average depth of channel, if possible.

D-9. Trip Report. After a site visit, a trip report should be prepared. The report should include field survey notes and applicable photos with footnotes to describe photo location and significance. Topics for the trip report include the following:

a. Purpose of the trip.

b. Pictures labeled with descriptions.

c. Date and time of trip.

d. List of people attending any meetings held.

e. Contact information.

f. Summary of observations.

g. Reservoir and River Site Visit checklists.

h. Summary of any data collected.

D-10. Post-Field Reconnaissance Activities. Once the field reconnaissance is completed, the engineer should have a good idea of the existing problems, possible future impacts, and potential responses to proposed project improvements, and of the parameters that are the most sensitive to change. The engineer should also be able to outline a plan of study. The complexity of the study and quality of the results will likely depend on the availability of historic and contemporary data, along with the study objectives. Based on the data available and additional field observation, the engineer should be able to ascertain the following:

a. Assess the Present Stream Stability. On a stable reach, there should be little or no evidence of significant overbank deposition or recent bank erosion. For instance, the presence of mature trees established on a presently stable bank indicates that the bank has been in that position for a relatively long period. Other systems such as ephemeral channels, including arroyos, are, by definition, nonequilibrium systems. Systems such as alluvial fans are highly event driven that can change significantly. Vertical stability indicators, indicating either degradation or aggradation, are often more difficult to determine in the field. Stability indicator lists are available in multiple references (USACE 2001; NRCS 2007).

(1) Characteristic instability signs include actively caving banks, large amounts of drift material, trees undermined and leaning toward the channel, exposed bridge piers or other infrastructure, buried features or significant overbank deposition, and similar warnings.

(2) Assess the adequacy of present structural features.

(3) Assess the adequacy of past channel improvements and/or alignment changes.

(4) Depending on the availability of historic data, the engineer may be able to ascertain the following:

(a) Long-term stability trends.

(b) Stream response to land use changes.

(c) Stream response to past improvements.

(d) Depending on the availability of historic and contemporary hydraulic, hydrologic, topographic, and sediment data, the engineer should be able to evaluate, either qualitatively or quantitatively:

- Future long-term stability with and without the proposed improvement.

- Future maintenance requirements with and without the project.

- Design alternatives that address the interaction of sedimentation and all other project considerations to arrive at the “best” design.

b. Figures D-1(a–d) provide reservoir and river site visit checklists (adapted from Copeland et al., 2001, Appendix D).

Reservoir Site Visit Worksheet		
Stream:	Watershed ID:	Dam Name:
Date:	Coordinates of Reach:	Dam Owner:
Team:	Drainage Area:	HUC ID:
Landmarks:	Dam NID:	

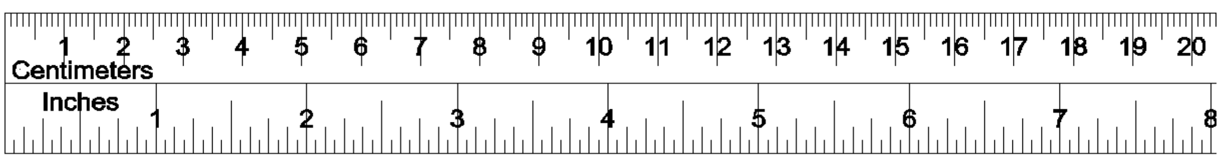
Site/Project Description and Current Conditions:

Reservoir Observations:

	Yes	No	Comments
1) Representative photos of both normal and identified problem areas.			
2) Take pictures of any bridges crossing the reservoir. Determine rough estimates of bridge pier widths, bridge length, and bridge width.			
3) Bed and bank material slope within the study area.			
4) Record all identified headcuts and knick points on the tributaries entering the reservoir.			
5) Obtain representative samples of the bed and bank material in both primary and any tributary deltas			
6) Note condition of banks, whether stable or caving, and the type of material found in the reservoir banks, particularly any lenses.			
7) Note presence of reservoir bank protection measures, their size, why they were placed.			
8) Record variation in observed conditions by location			
9) Record drift accumulations, debris.			
10) Record the extent and distribution of vegetation on delta deposits.			
11) Note changes in bed gradation and take representative samples for the sediment study.			
12) Note any processes removing sediment (mining, dredging, etc)			
13) Note tributary entry points, the amount of flow, turbidity of flow, condition of the tributary.			
14) Note any flow diversion points.			
15) Note natural grade controls such as rock outcrops.			
16) Note gage locations, type of gage.			
17) Note existing similar projects on same or adjacent streams (for future comparison)			
18) Note overbank conditions, possible sloughing due to erosion at the normal water level.			
19) Take velocity measurements at several locations			
20) Talk with local stakeholders to identify problem areas, get an estimate of time of problem			

Figure D-1a. Reservoir Site Visit Worksheet

Visual Reservoir Bed Material Samples				Boulder	Cobble	Gravel	Sand	Silt		
Sample #	Latitude	Longitude	Bedrock	(10-160 in)	(2.5-10 in)	(.08-2.5 in)	(.0625-2 mm)	(.004-.0625 mm)	Clay	Organics



Ruler correct to scale if printed on 8 1/2 x 11 page

Figure D-1b. Visual reservoir bed material samples list

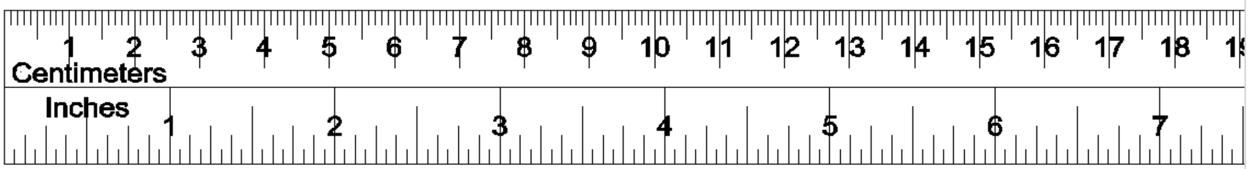
River Site Visit Worksheet			
Stream:	Watershed ID:	HUC ID:	
Date:	Coordinates of Reach:		
Team:	Drainage Area:		
Landmarks:	Dam NID:		

Site/Project Description and Current Conditions:

River Observations:			
	Yes	No	Comments
1) Representative photos of both normal and identified problem areas			
2) Bend and bank matieral slope within study area			
3) Stream gneral slope and break points, headcuts, knicks			
4) Obtain representative samples of the bed and bank material - record in section below			
5) Condition of banks, stable or caving			
6) Record variation in observed conditions by spatial location			
7) Note any process that is removing sediment (mining, dredging, excavation)			
8) Note tributary entry points, the amount of flow, turbidity, and condition			
9) Note any flow diversion points or distributaries			
10) Note natural grade controls such as bedrock or rock outcrops			
11) Note presenence of protection measures, their size, and determine why they were placed			
12) Note gage locations, type of gage			
13) Note structural feature loactions and observe bed and bank conditions in the vicinity of structures			
14) Note overbank scour and deposition locations			
15) Take velocity measurements at several locations, record in section below			
16) Talk with local stakeholders to identify problem areas, get a timeline of problems			
17) Inquire as to local land history, when urbanized, cleared, etc.			

Figure D-1c. River Site Visit Worksheet

Visual River Bed Material Samples			Bedrock	Boulder (10-160 in)	Cobble (2.5-10 in)	Gravel (.08-2.5 in)	Sand (.0625-2 mm)	Silt (.004-.0625 mm)	Clay	**Velocity (fps)**
Sample #	**Latitude**	**Longitude**								



Ruler correct to scale if printed on 8 1/2 x 11 page

Figure D-1d. Visual river bed material samples list

Appendix E
Guidelines for Sampling Bed Material

E-1. Reference. These bed material sampling guidelines for a stability assessment project are adapted from Appendix D in Copeland et al. (2001) with revisions and additional material.

E-2. Purpose of Bed Material Sampling.

a. The objective of a bed material sampling program may be to determine a representative bed gradation for a particular reach of a stream, or it may be to determine the variability and diversity of the sediment bed.

b. Deposited sediment is sampled to provide information on the individual sediment particles, the sediment mixture, and the bulk sediment deposit. Particle characteristics include grain size, shape, specific gravity, lithology, and mineralogy. The quantity and type of contaminates attached to particles are frequently of interest. Data that describes the distribution of the various particle sizes and of specific contaminates are frequently required.

c. Knowledge of streambed characteristics is necessary when considering the transport mechanics of sediment both into and out of reservoirs.

d. For example, reservoir sedimentation studies may include objectives related to the source, transport, and fate of pollutants; fish habitat; resource management; morphological trends; and/or river engineering works.

e. Fish habitat studies may be concerned with the suitability of the streambed for spawning. Sampling for this type of study should be relatively extensive, identifying lateral, longitudinal, and temporal variations in the surface layer over a wide area of the stream. Contaminates typically attach to cohesive sediment and therefore are distributed over a wide area, especially in areas where flow velocity is low.

f. Resource management studies are frequently concerned with the need for or feasibility of sand and gravel mining. Core or substrate sampling that identifies vertical variation of the streambed is essential for this type of study. Morphologic and engineering studies are typically concerned with changes in the character of the river over time. These studies frequently require knowledge of the grain size distribution of both the bed surface material and subsurface material for sediment transport calculations, critical shear stress determinations, determining potential for particle sorting and armoring, and for determining hydraulic roughness.

g. Bed material sampling is also frequently conducted to make sediment transport calculations by identifying a representative bed material gradation and any lateral, longitudinal, vertical, and/or temporal variation in bed material composition. Lateral variations in bed material gradation can be much more significant than longitudinal variations. In sand-bed streams, the sample is typically taken from the upper 5 cm of the bed surface. In gravel-bed streams with coarse surface layers, samples of both the surface and subsurface layers are required. Ideally, bed material samples should be taken at different times during the year to account for seasonal variations.

h. Experienced investigators are required for successful field data collection. Although sampling guidelines are relatively simple, field application for a given project is seldom without site-specific challenges.

i. Table E-1 provides guidance relative to where a bed material sample might be taken as a function of the type of geomorphologic or engineering analysis to be conducted.

Table E-1
Bed Material Sampling Sites (revised from Copeland et al., 2001)

Purpose of Analysis	Sample location
To estimate the maximum permissible velocity in a threshold stream	Riffle
To estimate the minimum permissible velocity in a threshold stream	Areas of local deposition
To estimate sediment yield for an alluvial stream	Crossing or middle bar
To quantify general physical habitat substrate condition	Bars, riffles, and pools

E-3. Bed Material Characteristics.

a. Characteristics of the sediment deposit itself include: stratigraphy, density, and compaction. For some of these purposes a sample can be disturbed, others require undisturbed sampling. Different samplers and sampling procedures are appropriate for different environments. Water depth and velocity and bed material size are the most important factors used to identify appropriate samplers and sampling procedures.

b. When the sediment particles are noncohesive, mechanical forces dominate the behavior of the sediment in water. The three most important properties that govern the hydrodynamics of noncohesive sediments are particle size, shape, and specific gravity. A discussion of these properties is found in Chapter 3.

c. The boundary between cohesive and noncohesive sediments is not clearly defined. However, that cohesion increases with decreasing particle size for the same type of material. Clays are much more cohesive than silts. Electrochemical forces dominate cohesive sediment behavior. The three most common minerals that have electrochemical forces binding particles are illite, kaolinate, and montmorillonite. The dispersed particle fall velocity, flocculated fall velocity of the suspension, clay and non-clay mineralogy, organic content, and cation exchange capacity characterize cohesive sediment. The fluid is characterized by the concentration of important cations, anions, salt, pH, and temperature. More detailed information is presented in EM 1110-2-1607.

E-4. Sampling Procedures. General Considerations. Several factors influence both sampling site selection and sampling procedure.

a. The most significant factor is the data necessary to meet the objectives of the study at hand. Data needs should be clearly defined before the sampling program is planned.

b. Another factor to consider is field conditions. Will the bed of the stream be wet or dry? Is the site accessible by road, boat, trail, or only by helicopter? Field conditions determine both the practicality and type of sampling equipment to be used in the sampling program.

c. Another factor that influences the type of sampling equipment and the appropriate sampling procedure is the character of the streambed itself. Sand-bed streams typically have a more uniform bed gradation and therefore require a smaller volume sample than gravel-bed streams. Typically, equipment appropriate for sampling sand-bed streams is inappropriate for gravel-bed streams. Thus, it is necessary to know the general streambed characteristics before the sampling program is established.

d. Available resources are often a limiting factor when establishing a bed sampling program. Equipment, workforce, and funds are frequently limited; therefore, priorities must be established.

e. Selection of equipment and methods is an integral step in determining sampling procedures. Refer to Chapter 4 of this manual for a detailed discussion of sampling procedures and measurement equipment.

E-5. Site Selection for Representative Sampling. There is no single rule for locating representative sampling sites or reaches. General guidance is as follows:

a. Carefully select sampling locations and avoid anomalies that would bias either the calculated sediment discharge or the calculated bed stability. The location must be representative of the hydraulic and sedimentation processes that occur in that reach of the river.

b. The site should be morphologically stable (constant slope and width upstream and downstream).

c. Select a site that reflects reach-averaged river conditions. Avoid sites with tributary inflow in the proximity of the site as it may interfere with the homogeneity of the section by supplying sediment for deposition. The site should not be located adjacent to a zone of active bank erosion as the material deposited in the channel near the area may not be representative of the reach.

d. Although bridges provide good access, bridge crossings are typically not appropriate sampling sites because they are located at natural river constrictions or because their abutments and piers create constrictions and local scour. Dead water areas behind sand bars or other obstructions should be avoided as these are not representative of average flow conditions.

E-6. Sand-bed Streams.

a. Sand-bed streams are characterized by a relatively homogeneous bed material gradation. Vertical and temporal variability is typically insignificant in stable streams. Longitudinal variability typically occurs over distances of many kilometers. However, lateral variability, especially in bends, can be significant.

b. In sand-bed rivers, sampling of material is most frequently carried out under low flow conditions. The equipment and methodology depend on the river depth and velocity. The task can be accomplished in flowing streams either by wading or from a boat, or in ephemeral and intermittent streams in the dry. Vertical variations in the bed material are usually insignificant in flowing water and samples are collected from the surface. However, in standing water or on dry beds, a layer of fine material deposited on the recession of a flood hydrograph is sometimes found on the surface.

c. Figure E-1 illustrates typical bed material gradation patterns on a point bar. Note that although the typical grain sizes found on the bar surface form a pattern from coarse to fine, there is no one location that always captures the precise distribution that will represent the entire range of sedimentation processes.

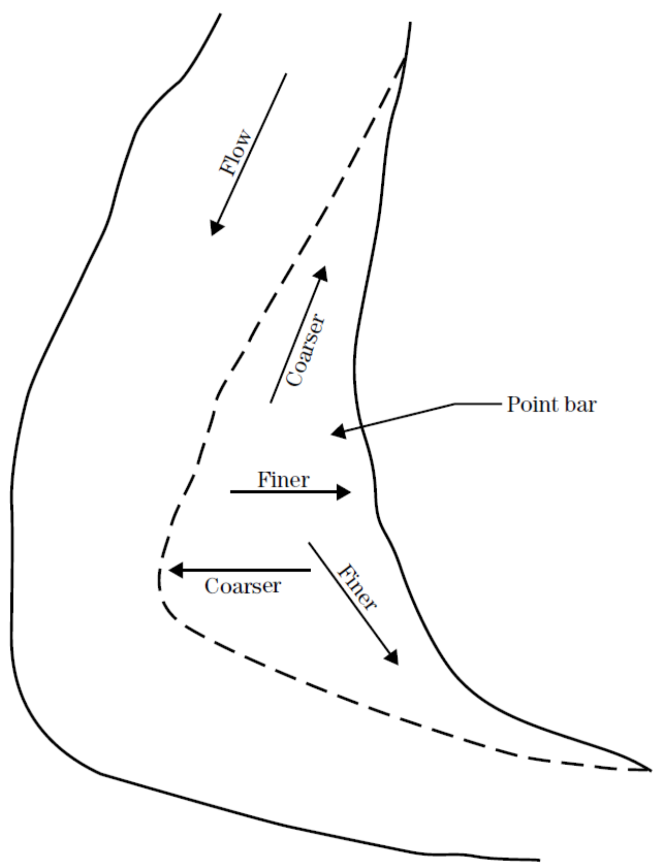


Figure E-1. Gradation pattern on a point bar (Copeland et al., 2001)

d. Einstein (1950) recommended using only the coarsest 90% of the sampled bed gradation for computations of bed material load. He reasoned that the finest 10% of sediment on the bed was either material trapped in the interstices of the deposit or a lag deposit from the recession of the hydrograph and should not be included in bed material load computations.

e. Representative bed material sampling in sand-bed streams may be accomplished by either a cross-section approach or a reach approach.

(1) Cross-Section Approach.

(a) This approach requires the selection of a representative cross section. Employing the cross-section approach requires selecting a site and time for sampling where and when the bed characteristics are typical. This method requires considerable experience, and unanimous opinion about where and when the typical condition occurs cannot be expected even among experienced river scientists. Frequently, judgment is influenced by the type of streams the sampler has experienced and by the intended use of the data.

(b) In streams with relatively uniform depths, between three and five samples should be taken across the section to account for lateral variations. In streams with variable depths, more samples are required.

(c) Bed material sampling at crossings where flow distribution is typically more uniform reduces the lateral variation in the samples but will not reflect the bed material variation due to depth and velocity change. At low flow, crossings may develop a surface layer gradation that reflects sediment transport conditions at the lower discharge, which may be coarser or finer than the bed gradation at bankfull discharge. Crossings are typically submerged, and more elaborate sampling equipment is required than at exposed bars where a shovel is frequently sufficient. However, samples collected on a point or alternate bar may exhibit considerable variation.

(2) Reach Approach. An alternative to the cross-section approach is the reach approach. Employing the reach approach where samples from several systematically selected cross sections are averaged to obtain a representative sample may eliminate some uncertainty associated with the cross-section approach.

(a) A reach is defined as a portion of the stream with similar morphology (identified by its homogeneity). Generally, five cross sections are laid out in the homogeneous reach.

(b) If there is a gage in the reach, locating the center cross section near the gage is preferred.

(c) If the stream reach is straight, the spacing should be approximately two to five stream widths, and if the reach is meandering, the spacing should occur within one meander length, as shown in Figure E-2.

(d) The same criteria used in the cross-section approach to determine the number of verticals.

(e) The reach approach is especially applicable to rivers with meanders of different wavelengths and amplitudes.

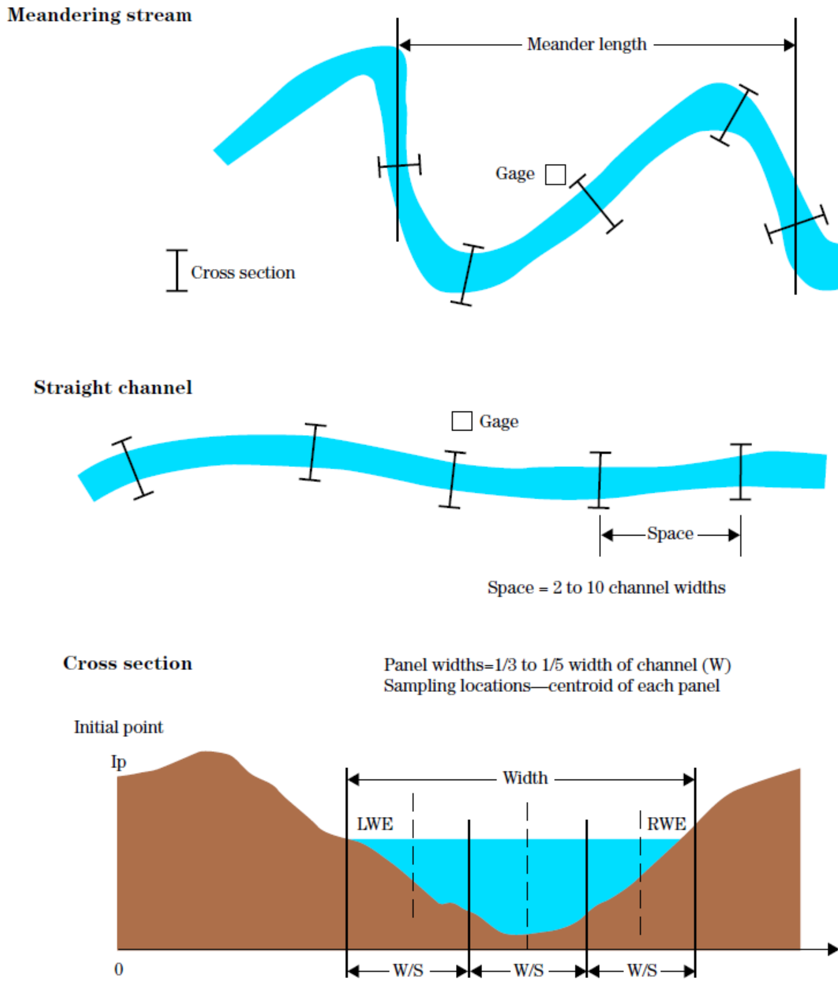


Figure E-2. Bed sampling locations for sand-bed streams (USDA 2007a; NRCS 2007)
Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

E-7. Gravel-Bed Streams.

a. Coarse beds (gravel, cobble, and boulder) are characterized by significant vertical, spatial, and temporal bed material variability. The most distinctive characteristic is a coarse surface layer that may form in both the low-flow channel and on bars. Frequently, the low-flow channels of coarse bed streams are armored with large cobbles and boulders, while bars consist primarily of sand and gravel.

b. Coarse bed streams are principally distinguished from sand-bed streams by their particle size distributions. Gravel-bed streams have a mean particle size in the range of 2 to 64 mm, and cobble-bed streams in the range of 64 to 256 mm. Gravel and cobble beds usually contain some sand, typically less than 10% in mountainous areas, and maximally, up to about 50% (Bunte and Abt 2001).

c. Since the spatial variability in most coarse bed streams is high, it is very difficult to perform representative sampling. River bars are frequently chosen as sampling sites and specific bar types have been determined to be more representative than others. Figure E-3 shows a bar type hierarchy established to aid site selection (Bray 1972; Yuzyk 1986).

(1) Mid-channel and diagonal bars are selected as most ideal sites because they are exposed to the highest velocities, which transport the largest materials.

(2) Point bars are not as ideal because velocities are highly variable, decreasing toward the inside bank.

(3) Channel side or lateral bars are least desirable because they exist in zones of low velocities due to boundary and bank effects.

(4) In small streams with no bars and a pool-riffle sequence, the riffles may be sampled to characterize bed material size. However, the riffle can be expected to be much coarser at low flow when sediment transport is typically negligible than at bankfull flow when sediment transport is active.

(5) Based on the assumption that the coarsest materials in the bed exert the predominant effect on channel behavior and flow resistance, some recommend that samples be collected at the upstream end of bars (Bray 1972; Church and Kellerhals 1978; Yuzyk 1986). Sediments at this location are indicative of the sediments in the main channel, readily identifiable, and generally exposed. The upstream end of bars usually consists of the coarsest material in the channel and not the average size in the reach because the upstream end of the bars is the location most frequently exposed to the highest stream velocities.

(6) In coarse bed streams, particle size analysis is typically based on the intermediate axis. The particle is described by approximating with three mutually perpendicular particle axes: the longest (a-axis), the intermediate (b-axis), and the shortest (c-axis).

d. In coarse bed streams, it is necessary to determine the characteristics of both the surface and subsurface bed layers.

(1) Bulk sampling is usually employed to characterize the subsurface layer.

(2) Both bulk and areal sampling are employed to characterize the surface layer. Bulk surface sampling, which involves collecting a predetermined volume of sample that includes all material, is preferred if it is possible to identify and collect only the surface layer material. This is difficult when the surface layer has a wide range of size classes.

(3) Bulk surface sampling provides information about the finer grain sizes trapped in the interstices of the surface layer. The armor layer should be sampled to a maximum depth of about

one to two times the diameter of the maximum size class in the bed. This information is useful for permeability studies including fish habitat and also for sediment transport studies.

(4) Areal surface sampling is used to characterize the coarse surface layer and is used to determine hydraulic roughness, critical shear stresses, armoring, and sediment transport. A common methodology for areal sampling is a pebble count (Wolman 1954) where individual particles are collected at random by hand and the intermediate axis is measured. Refer to paragraph E-8 for further details.

e. Determination of stream bank characteristics is often important. The bank material can help define the stability of the channel section and may be responsible for a significant percentage of the total sediment load. When sampling the bank, look for features such as layering and lensing. Also note evidence of piping and seepage and related features. Several bulk samples are often required at a given location to assess the soil type and gradation of each of the bank strata.

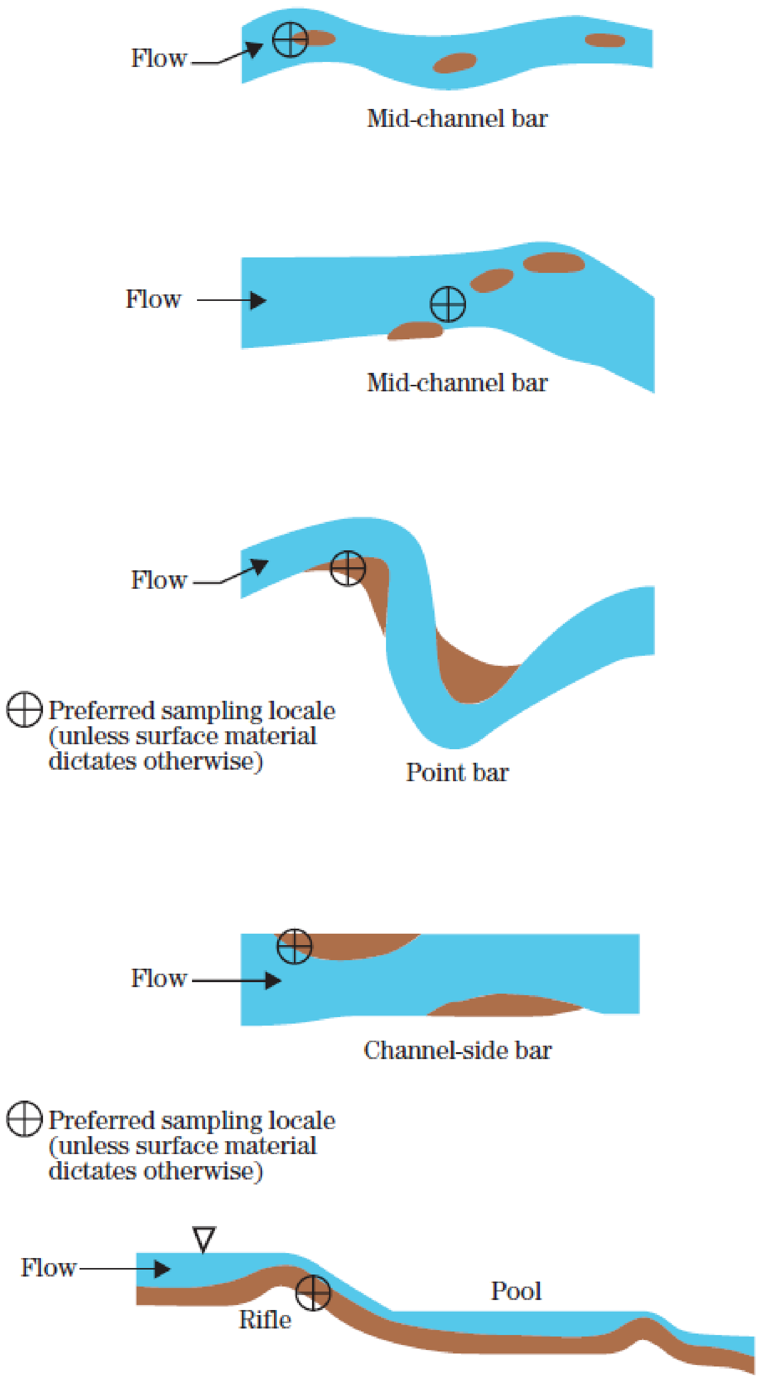


Figure E-3. Coarse bed stream sampling hierarchy (USDA 2007a; NRCS 2007)
Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

E-8. Field Sampling Procedures.

a. Field sampling procedures include multiple steps that vary with material and stream characteristics. Procedures are described as follows:

(1) Develop a sampling plan that includes the creation of detailed records of all data collected, a data storage method, and necessary documentation. The sampling plan is a component of the Sediment Studies Work Plan (SSWP) described in Chapter 2 of this manual.

(2) Select and mark out the required cross sections and the sampling locations. Use as many of the previously outlined site selection criteria as possible. The fixed permanent initial point should be on the left bank (looking downstream). Establish the control (horizontal and vertical) and reference all points. Installing monuments for repetitive sampling maybe beneficial for project objectives.

(3) Sketch the site on data forms and reference the control points. If the streambed contains a mixture of sand and gravel deposits, then map areas and record deposits of different size material. Develop a sampling strategy that will sample each zone.

(4) Collect a photographic record of the reach, controls, cross sections, sample locations (if possible), bed material (use a scale for reference), and bank conditions.

(5) Select an appropriate sampler for the task (based on depth, velocity, and sample requirements. The FISP (http://water.usgs.gov/fisp/) maintains a summary of common bed material samplers. The FISP website also includes links to the operator's manuals for all samplers. Chapter 4 of this manual also provides additional information on sampling equipment. Once selected, ensure that qualified persons are collecting the samples and verify that the sampler is operational.

(6) Use of global positioning system (GPS) equipment is preferred to accurately locate the sample site. Cross-section surveys at the sample site can be helpful. Comparison to reach-wide cross sections can illuminate sample location geometry variation from the remainder of the reach. If survey equipment is not available, consider locating the sample site at existing survey locations if previously described sampling location criteria is met.

b. Surface Sampling.

(1) Sand Bed (Bulk). Select the sampling location following guidance previously stated. Locate the section sampling point for lateral variation via distance from section end point or GPS. Approach the sampling verticals from the downstream side to prevent disturbing the bed at the sampling section. In deep streams, using a boat and some type of positioning system (tag line in narrow streams, electronic distance measurement (EDM) in wide streams), hold the boat steady over the sampling location. Obtain a sample of about 250 g at each chosen location using the selected sampler. The upper few inches may provide a representative sample in some sand-bed streams. Coring may be necessary for USACE studies with bed vertical variation. Coarsening may occur that is an important factor for numerical modeling studies.

(2) Gravel Bed (Bulk). To obtain a surface bulk sample, carefully remove and collect all sediment in the surface layer to a thickness of the intermediate axis, often referred to as the b-axis, of the largest particle in the area. Care should be taken to ensure that fine sediment is not washed out of the sample.

(3) Gravel Bed (Surface Areal). To obtain a surface areal sample in a coarse bed stream, several techniques may be employed. These include random walks, setting up square or linear grids, and removing all the surface particles within a specified area.

(a) The gradation curve developed from these data is based on the number of particles in each size class, not their weights or projected surface areas. Conversions are given in Chapter 5 of ASCE Manual No. 110 (Gray et al., 2008). Sources of error are also described in detail.

(b) Studies have shown that particles smaller than 8 mm are typically missed with areal sampling, especially if the bed surface is submerged, and thus the pebble count may be biased toward the larger sizes. This problem can be overcome by truncating pebble count samples at 8 mm and using a bulk surface sample to define the percentage and distribution of material finer than 8 mm. Daniels and McCusker (2010) provide a discussion on bias when performing pebble counts.

(c) Grid sampling methods include laying out a linear tape and selecting the pebble at a designated interval and laying out a preconstructed rectangular grid and selecting the pebble at grid point intersections. Additional information on different field methods and comparison of results is available (Bunte et al., 2009).

- Collecting the entire surface layer within a specified area generally requires a specialized sampler.

- The process may be aided by spray painting the surface if the bed is dry, although this technique is rather tedious.

- Regardless of the approach chosen, the measuring process may be streamlined in the field by using a gravelometer to measure the sieve diameter of each particle immediately after the particle is selected.

(d) The random walk method devised by Wolman (1954) can easily be employed on a dry bed or in wadeable flow, and with more difficulty by divers. Guidelines are presented below, refer to Bunte and Abt (2001) and Bunte et al., (2009) for additional details.

- The spacing of the sampling points must be at least two times the diameter of the largest particle in the sampling area. This reduces the influence of nearby particles.

- The random walk chooses particles while proceeding laterally across the stream or longitudinally along a point bar.

- Use 100 sample points (Wolman 1954; Hey and Thorne 1983; Yuzyk 1986) for ease of data reduction at each sampling site. To be very precise or to accurately measure small percentiles, the number of sampled particles should be increased.

- To obtain a sample, a team member paces along a selected path, collecting a pebble with each step. The first pebble touched is selected, with eyes closed or averted.

- Measure the b-axis (Figure E-4) and record in data book. The practical application to field measurement of individual particles is not precise. Differences in visual interpretation and variations in measuring techniques are common. The analyses of particle sizes and particle shape parameters are based on the length of three mutually perpendicular particle axes: the longest (a-axis), the intermediate (b-axis), and the shortest (c-axis) axis (Bunte and Abt 2001). For embedded pebbles or those that are too large to move, measure the shortest axis visible.

- Take another step across the stream and repeat the previous steps until you reach the opposite side. Establish a new transect and begin the process over again.

- If the stream reach is relatively narrow (<2 m), modify the method by walking upstream in a zig-zag pattern instead of perpendicular to flow.

- Collect enough measurements (minimum of 100, as previously recommended) to accurately quantify bed material distribution in the study area. This method generally produces a sample biased toward coarse size classes.

(4) Subsurface Bulk Sample, Coarse Bed. If the surface layer has not already been removed, scrape away the surface layer of coarse material to the thickness of the intermediate axis of the largest particle in the area. Bunte and Abt (2001) present a method developed by Church to determine sample volume to ensure that the weight of the largest particle is no more than 0.1% (D_{max}<32 mm), 1% (D_{max}<128 mm), or 5% (D_{max}>128 mm) of the total sample weight. Refinement of the sample volume percentage may be beneficial according to specific project material size and objectives. Refer to Copeland et al. (2001) and Bunte and Abt (2001) for further details. The required sample mass is a function of the largest particle in the subsurface and can be determined from Figure E-5.

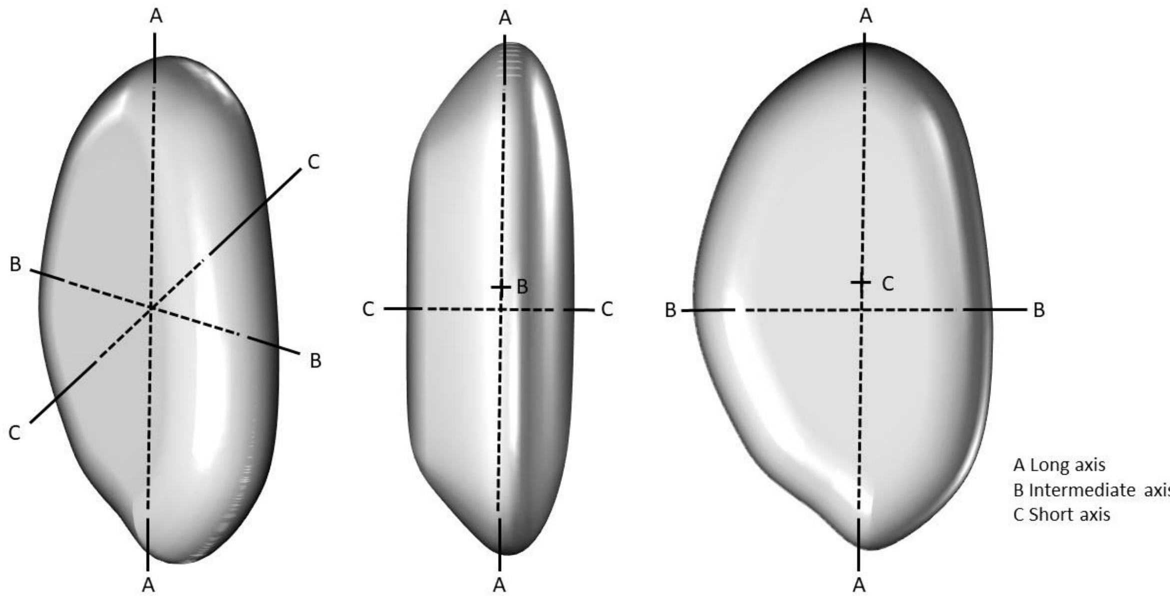


Figure E-4. Three perpendicular axes

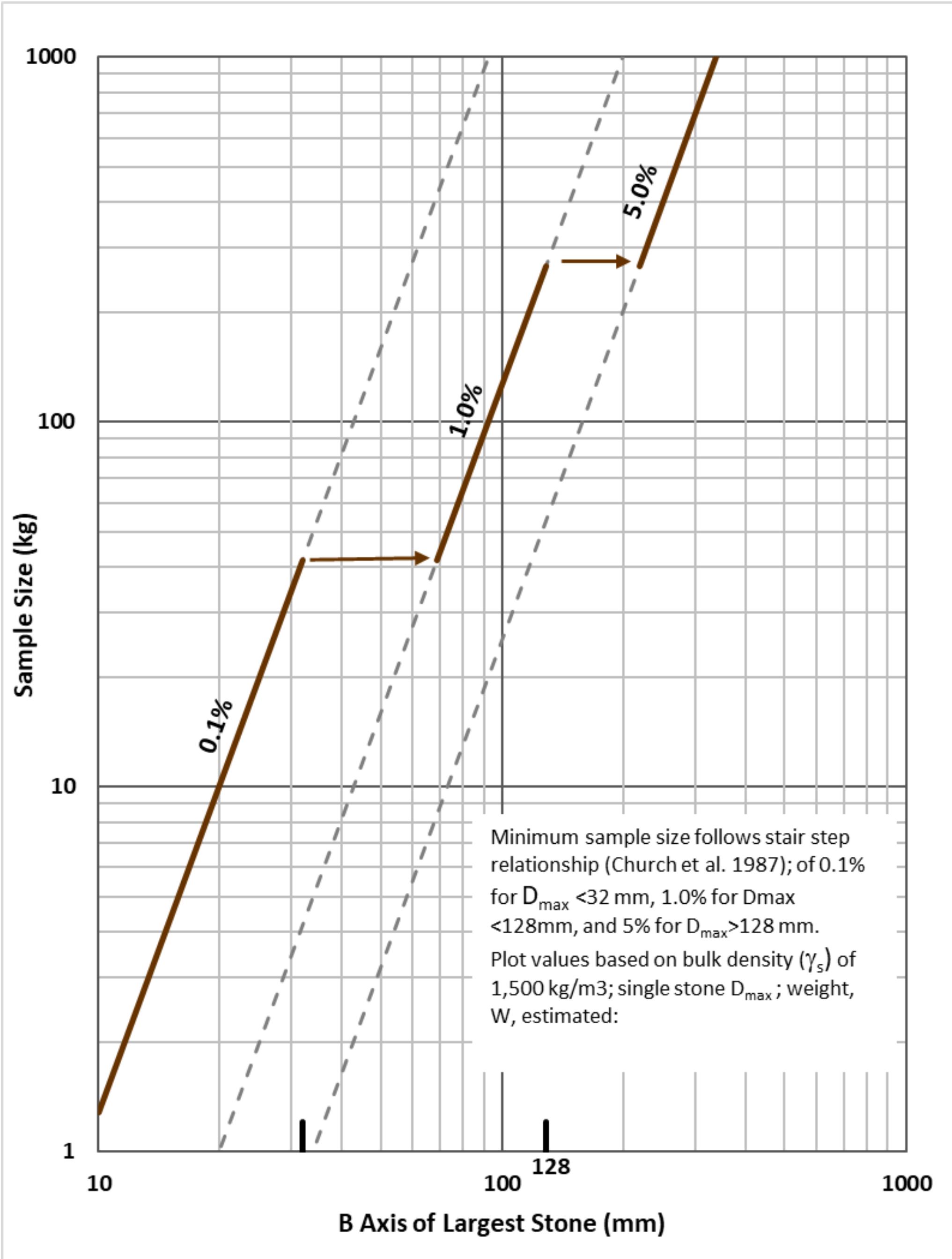


Figure E-5. Bulk sampling standards for gravel and cobble streams (redrawn from Copeland et al., 2001)

c. Field Sieving. Field sieving can be an efficient alternative to transporting large bulk samples to a testing lab. Simple methods to create a weight station include using a tripod with a scale suspended for weighing pails of material or by using small light weight electronic scales. Typical steps, which will vary with equipment used, are as follows:

(1) Assemble field equipment (varies with methods), typically consisting of pails, spades, template, labels, field note forms, sturdy plastic bags, and tarpaulins. Assemble the field sieve sets and insert the correct sieve sizes.

(2) Obtain tare weights for the pails. Shovel material into pails, weigh, and record.

(3) Pour material into top of the field sieves (typically 8-, 16-, 32-, 64-, 128-mm sieves). Rock and shake the sieve set until material has moved to its retained size sieve.

(4) Weigh material retained on each sieve and on the pan. Record in field notes.

(5) If desired for characterization, save the 10 largest particles and measure the three perpendicular axes.

(6) If desired for lab hydrometer testing, retain up to 10 kg of the material from the pan. Transfer the retained sample to a clean heavy-gage plastic bag.

(7) Repeat the process until the required mass has been sieved.

(8) Complete field forms and attach a label and sediment field note form for each retained sample. Field forms should specify relevant collection data including the stream, date, time, crew personnel, sampler type, sample number, flow conditions, river mile/station, cross section, vertical location, bed form, and sample depth (when in water).

Appendix F
Qualitative Analysis of General River Response to Change

F-1. Introduction. Sufficient hydraulic and sediment data to perform a quantitative analysis may be unavailable for a particular study or project. However, this does not preclude a sediment analysis. The analysis must, by necessity, be qualitative in nature. This requires an understanding of fluvial processes per Leopold et al. (1964), Schumm (1977), and Simons and Sentürk (1992) as well as more recent references Copeland et al. (2001), Biedenharn et al. (2008).

F-2. General Relationships.

a. Gilbert (1914) established the general relationships between bedload transport and slope, discharge, median grain size and width/depth ratio. In his flume studies, sediment transport was treated as the dependent variable as he varied the other variables in such a fashion as to isolate the effect of each variable on transport. Gilbert determined that power equations could be fit to relationships between bedload transport and slope, discharge, and grain size when a critical or "competent" value was included in the relationship. Gilbert's relationships are shown in the following equations:

$$Q_s = a_1(S - S_c)^{b1} \qquad \text{Equation F-1}$$

$$Q_s = a_2(Q - Qc)^{b2} \qquad \text{Equation F-2}$$

$$Q_s = a_3\left(\frac{1}{d_{50}} - \frac{1}{d_{50C}}\right)^{b3} \qquad \text{Equation F-3}$$

where:

Q_s = bedload transport (gm/sec)
S = water surface slope (percent)
Q = water discharge (cfs)
d_{50} = median diameter (ft)
subscript c = competent (critical) value
a and b = constants related to a specific experiment

b. Gilbert's coefficient, a, and exponent b, varied for each experiment. This result confirmed that the bedload transport is a function of more than just one variable. Gilbert proposed a combined equation using the variables of slope, discharge, and mean grain diameter.

$$C = a_4(S - S_c)^{b1}(Q - Qc)^{b2}\left(\frac{1}{d_{50}} - \frac{1}{d_{50C}}\right)^{b3} \qquad \text{Equation F-4}$$

c. The coefficient and exponents in this equation also varied depending on the experiment.

d. Whereas Gilbert found the relationships between bedload transport and slope, discharge, and median grain size to be "harmonious," the relationship between bedload transport and width/depth ratio was found to be “discordant.” Instead of advancing by a continuous law from zero to infinity, transport first rose to a finite maximum and then returned to zero. Later investigators identified this condition as the minimum stream power or maximum sediment transport case. Gilbert found bedload transport to be less sensitive to the width/depth ratio than to the other three variables.

e. Although Gilbert published the experimentally determined ranges for his exponents and coefficients, his data were based on flume tests and his equation was never widely used in sedimentation studies. However, his data are among the most frequently referenced by future researchers.

f. Lane (1955) simplified Gilbert’s approach by introducing a proportional relationship rather than an equation. Lane proposed that river system response could be predicted by evaluating the relative change in dependent variables when an independent variable changed. Lane considered sediment transport as the bed material load (includes particles that move as suspended load or bedload and periodically exchange with the bed, rather than the bedload (see Chapter 3, paragraph 3-4b).

$$Q \cdot S \propto Q_s \cdot d_{50} \qquad \text{Equation F-5}$$

where:

Q = water discharge
Q_s = bed material discharge
S = channel slope
d_{50} = median diameter of bed material

g. Simons (Simons and Sentürk 1992) expanded the Lane relationship for predicting system response to include channel dimensions.

$$Q_s \propto \frac{[(\gamma_w \cdot D \cdot S) \cdot W \cdot U]}{\frac{d_{50}}{C_f}} = \frac{[\gamma_w \cdot Q \cdot S]}{\frac{d_{50}}{C_f}} \qquad \text{Equation F-6}$$

where:

C_f = concentration of fine material load
D = depth of flow
d_{50} = median fall diameter of bed material
γ_w = specific weight of water
Q = water discharge
Q_s = sediment discharge
S = channel slope
U = average velocity
W = channel width

h. If the specific weight of water, γ_w, is assumed to be constant and the concentration of fine material C_f is incorporated in the fall diameter, then the Lane relationship (Equation F-7) can be expressed in the identical form except that the median fall diameter of bed material, d_{50}, which includes the effect of temperature on transport, has been substituted for the physical median bed diameter d_{50} used by Lane.

i. Leopold (1994) argued that the Gilbert, Lane, and Simons relationships do not adequately account for the dominant processes that affect adjustment in alluvial rivers. Leopold states that "in fact, slope adjusts but little to a change in the amount of introduced sediment load. The adjustment takes place principally among other hydraulic factors: width, depth, velocity, bedforms, channel pattern, and pool-riffle sequence." Consequently, Leopold places more emphasis on hydraulic geometry relationships for determining how an alluvial river will adjust to changes in independent variables such as sediment inflow, discharge, or base level change.

j. Hydraulic geometry theory is based on the concept that a river system tends to develop in a way that produces an approximate equilibrium between the channel dimensions and the inflowing water and sediment loads (Leopold and Maddock 1953). Hydraulic geometry relationships typically correlate an independent driving variable, such as discharge or drainage area to a dependent variable, such as width, depth, slope, or velocity. However, because stable channel dimensions are not dependent on a single independent variable, specific hydraulic geometry relationships apply only to cases where the design stream is physiographically similar.

k. Hydraulic geometry relationships have been developed for many cases. Guidance can be found in USACE (1994a), Copeland et al. (2001), Leopold and Maddock (1953), Emmett (1975), Charlton et al. (1978), Bray (1982), and Hey and Thorne (1986).

F-3. Application of Qualitative Analysis.

a. To evaluate natural or imposed changes to a river system with the Lane or Simons relationships, the engineer must remember that the proportionality must remain balanced. For example, if median grain diameter and water discharge are assumed constant and a decrease in slope is proposed for a reach of stream, the relationships indicate that the sediment discharge must also decrease. These relationships provide no information regarding the magnitude of any of the variables, or even the variable that, on one side of the proportionality, will change. These relationships are useful only for estimating trends. Knowledge of fluvial geomorphic processes is required to determine the dependent variables that are most likely to change.

b. Simons and Sentürk (1992) offer several good examples of the application of qualitative analysis. Two of these are characterized below.

F-4. Main Channel Degradation.

a. Figure F-1 shows the effect that a degradation, or a drop in the base level on a main channel, has on a tributary stream. In this case, the base level in the main channel has been lowered by a natural change or by constructed projects. Under the new condition, the local tributary gradient is significantly increased.

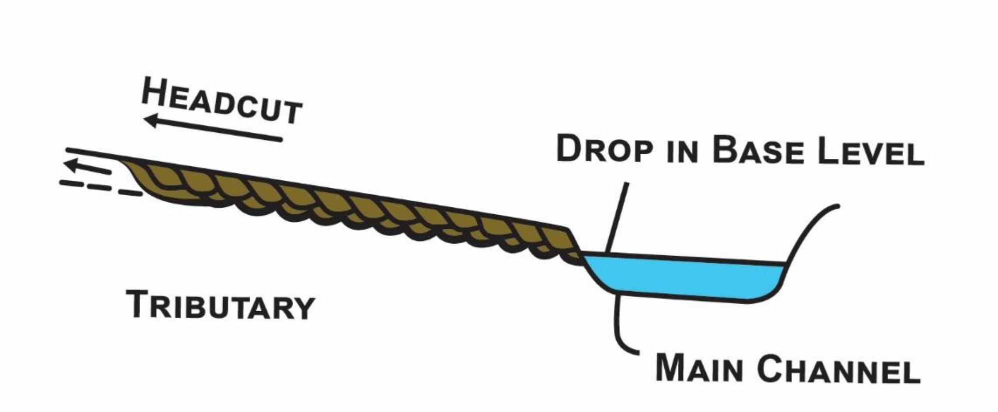


Figure F-1. Main channel degradation effect on tributary

b. Applying the Lane relationship to the tributary stream shows that the increase in tributary slope must be balanced by an increase in sediment transport, Q_s, if the discharge and median bed material diameter (d_{50}) are unchanged, the revised relationship is:

$$Q^0 \cdot S^+ \propto Q_S^+ \cdot d_{50}^0 \qquad \text{Equation F-7}$$

c. Therefore, the new slope could induce headcutting and flow velocity increase in the tributary stream, resulting in bank instability and increased sediment transport from the tributary, an overload of sediment in the main stream, and major changes in the geomorphic characteristics of the stream system. However, the relationship does not provide any information about the magnitude of the increased sediment load, the relative distribution of bed and bank erosion, or the potential for bed armoring, which could counteract the increase in sediment discharge. Table F-1 lists a summary of effects.

Table F-1
Summary Impact of Drop in Main Channel

Local Effects	**Upstream Effects**	**Downstream Effects**
Headcutting General scour Local scour Bank instability High velocities	Increased velocity Increased transport of bed material Unstable channel Possible change in planform of river	Increased transport to main channel Aggradation in main channel Increased flood stage in main channel Possible change in planform of river

F-5. Effects of In-Channel Structures.

a. Qualitative analysis can be used to analyze the response of reaches on two major tributaries a considerable distance upstream of their confluence in which a diversion structure is constructed (Figure F-2).

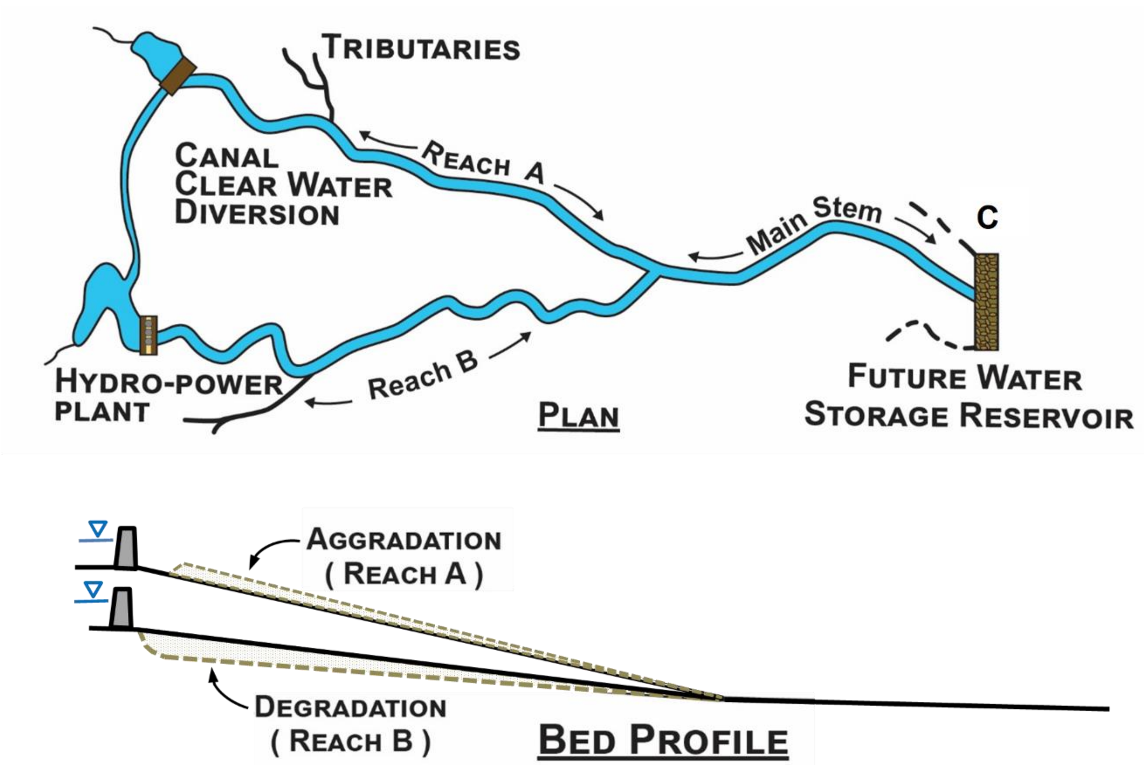


Figure F-2. Response on tributaries to clear water diversion structure and downstream storage

b. Upstream of Reach A, a diversion structure is built to divert essentially clear water to the adjacent tributary on which Reach B is located. Upstream of Reach B, the clear water diverted from the other channel plus water from the tributary is released through a hydropower plant. Eventually, a large storage reservoir will be constructed downstream of the tributary confluence on the main stem at Point C. By altering the normal river flows, these structures initiate several complex responses in Reaches A and B as well as on the main stem.

c. Applying the Lane relationship, it can be seen that clear water diversion alters the balance in both Reach A and B. Initially, there may be a lowering of the Reach A channel bed downstream of the diversion structure due to deposition upstream of the diversion dam and the initial release of essentially clear water for a relatively short period of time until the sediment storage requirement of the diversion reservoir is satisfied. After this initial response, Reach A may aggrade due to the excess sediment left in that tributary when clear water is diverted, the revised relationship:

$Q^{-} \cdot S^{+} \propto Q_s^0 \cdot d_{50}^0$ Equation F-8

d. Reach B is likely to degrade due to the increased discharge and essentially clear water release. However, it is possible that the degradation in the Reach B channel may induce headcutting on tributaries of Reach B that results in an increased sediment supply to Reach B. This response may offset, to some degree, additional degradation downstream. For the estimated degradation condition, the revised relationship:

$Q^{+} \cdot S^{-} \propto Q_s^0 \cdot d_{50}^0$ Equation F-9

e. Upstream of Reaches A and B, both the diversion dam and the hydropower plant create a backwater pool. As the water and sediment enter the pool, most of the coarse sediments deposit and create a delta at the head of the pool, which slowly advances downstream. The sediment deposition induces aggradation in the channel upstream. Depending on pool level raise, this aggradation may extend many miles upstream and produce significant changes in river geometry and increase flood stages. The decrease in slope S must be accompanied by a decrease in transport capacity Q_s, the revised relationship:

$Q^{0} \cdot S^{-} \propto Q_s^{-} \cdot d_{50}^0$ Equation F-10

f. Downstream of Reaches A and B, the combined changes will likely have a significant effect on the main stem. The flow diversion may deplete or alter the normal flow pattern Q. Upstream, Reach A aggradation and Reach B degradation will likely alter both the incoming sediment supply and material size. The final condition on the main stem is difficult to predict using qualitative analysis. It is likely that channel instabilities and significant effects on flood stage will occur.

g. Complex changes in a river system that affect multiple components over a varying time scale are common. If the water storage reservoir at location C is constructed in the future, additional response will occur. A complete analysis of such a system must consider the effect of each response both individually and collectively in a more detailed sedimentation study. Qualitative analysis in complex systems such as this example can be limited by estimating the proportional change between variables due to the non-linear relationship between flow, slope, sediment load, and material size. Final condition is the combined interaction of all these factors and must be analyzed using detailed sediment process modeling. Process changes may occur gradually over a long time period. Table F-2 lists a summary of impacts.

Table F-2
Clear Water Diversion Impact of Change on Stream System

Local Effects	Upstream Effects	Downstream Effects
Reach A may be subjected to channel aggradation due to excess sediment left in the channel after diversion of clear water; also potential for aggradation in tributaries caused by base level raise. Reach B may be subjected to degradation due to increased discharge in the channel (this effect is compounded since the additional flow is clear water); also tributary degradation caused by base level lowering. If future water storage reservoir were constructed at C, it could induce aggradation.	Upstream of Reach A and B, aggradation, and possible change of river form. Channel instabilities. Significant effects on flood stage.	Main stem response is complex due to multiple changes, likely channel instabilities and significant effects on flood stage. Construction of reservoir C could induce aggradation in both Reach A and B channels and in the tributaries, possible degradation in the downstream main stem.

Appendix G
Sediment Range Overview

G-1. Sediment Range Introduction. This appendix provides information on the sediment range program that was used at many USACE reservoirs. Many USACE offices have migrated to a revised methodology. However, historic information should be retained and is especially important when transitioning to new methods for surveying and storage volume capacity computations.

a. A "sediment range" refers to a fixed line across a reservoir, a stream channel, or flood plain along which elevations are measured. Ranges are classified according to the purposes they will serve and the field conditions affecting survey methods. Most previously constructed USACE reservoir projects included the establishment of aggradation ranges, located within the reservoir pool and upstream deposition zone, and degradation ranges that were located downstream of the project.

b. The operational objectives of establishing a network of sediment ranges upstream of the reservoir, located in the aggradation zone, typically included determination of actual storage depletion, reallocation of storage, revising reservoir regulation rules, and modification of facilities. Sediment ranges located downstream of the dam, located within the degradation zone, had the objectives of monitoring channel stability and changes in water surface elevation. Sediment sampling was often conducted at sediment range locations to provide a spatial reference for monitoring changes in bed material size. Temporal change can be viewed by plotting repetitive sediment range surveys.

c. Although the use of sediment ranges has curtailed somewhat with the advancement of survey technology, sediment ranges still perform a vital function at many USACE projects. Sediment ranges provide a relatively low-cost method to monitor delta advancement into the pool and reservoir bank erosion, a rapid method to demonstrate reservoir geometry change with time, and an established survey line to evaluate topographic mapping survey data accuracy. Sediment ranges are likely the least costly alternative to monitor sedimentation processes on large reservoirs, such as those on the Missouri River, which have a large total reservoir surface area at normal pool.

G-2. Classification of Sediment Ranges. Sediment range classification is illustrated in Table G-1 and described in the following paragraphs.

a. Classification According to Scope of Study.

(1) A "detailed study" range is one included in a general network established to measure sedimentation effects on a comprehensive basis.

(2) An "index" range differs from a "study" range in that it is usually more isolated from any correlated study network and established for the purpose of obtaining qualitative information.

b. Classification According to Technical Objectives.

(1) To facilitate the discussion of governing criteria and administrative details, ranges are identified by categories outlined below to meet technical objectives.

(2) Category A: Ranges crossing the main body and principal arms of a reservoir. They are required as a basis for determining storage capacity depletions resulting from sediment deposition, with a degree of accuracy commensurate with general engineering needs. Example criteria:

(a) Relative magnitude of sediment accumulations considered likely to occur within project life (based on available information pertaining to subject basin and/or other areas in region).

(b) Apparent importance of measuring storage capacity depletions in subject reservoir.

(c) Use of index ranges in subject case.

(3) Category B: Ranges crossing reservoir arms and tributary channels. They are required as a basis for determining the magnitude of sediment deposits and the related changes to water surface profiles. Example criteria:

(a) Special problem areas; urban.

(b) Special problem areas; agricultural.

(c) Use of index ranges.

(4) Category AB: Ranges required for combining Category A and Category B conditions. Further examples include:

(a) Proposed network (include layout map and pertinent tabulations of data).

(b) Documenting horizontal and vertical controls.

(c) Estimates of initial installation costs.

(d) Discussion (review concisely any matters that may have a major bearing on decisions involved).

(5) Category C: Ranges crossing the stream channel and floodway within a limited reach immediately downstream from the dam as required for determining the nature and extent of cross-section changes. Example criteria for operation, planning, and design requirements:

(a) Description of principal problems considered; relative magnitude of channel changes expected as result of the construction and operation of subject project.

(b) Legal and public relations problems likely to arise from operation of subject project.

(c) Applicability of detailed study ranges vs. index ranges in subject case.

c. Classification According to Field Condition and Survey Method.

(1) Submerged: A range or range segment across a portion of a reservoir or channel that is frequently submerged for extended periods and therefore requires hydrographic survey methods.

(2) Overbank/Pool Combination Range: A range that is located at normal pool levels, to include both the reservoir pool and the portion of a range that is not submerged, located at higher elevation than the normal channel top of bank. Both hydrographic and terrestrial survey methods may be applicable.

(3) Dry Land: A range that ordinarily must be surveyed entirely by terrestrial survey methods.

G-3. Numbering Sediment Ranges. Ranges will be numbered as illustrated in Figure G-1 and described below.

a. Detailed Study Ranges. Ranges intended for detailed study purposes will be identified by appropriate serial numbers suffixed by the category letters applicable to the specific range. For example, a range numbered 2-A is number 2 in the sequence and is one in a general network intended for use in detailed studies of reservoir capacity depletions to be expected from sedimentation. The suffix "study" is implied, but not shown in this case. In the event a particular range is considered necessary to meet both Category A and B objectives, the number will be suffixed by both letters (such as range 11-AB as shown in Figure G-1).

b. Index Ranges. Ranges intended for index purposes only will be identified in the same manner as study ranges except that the word "index" will be added parenthetically (like 21-B (Index), 6-c (Index), etc.).

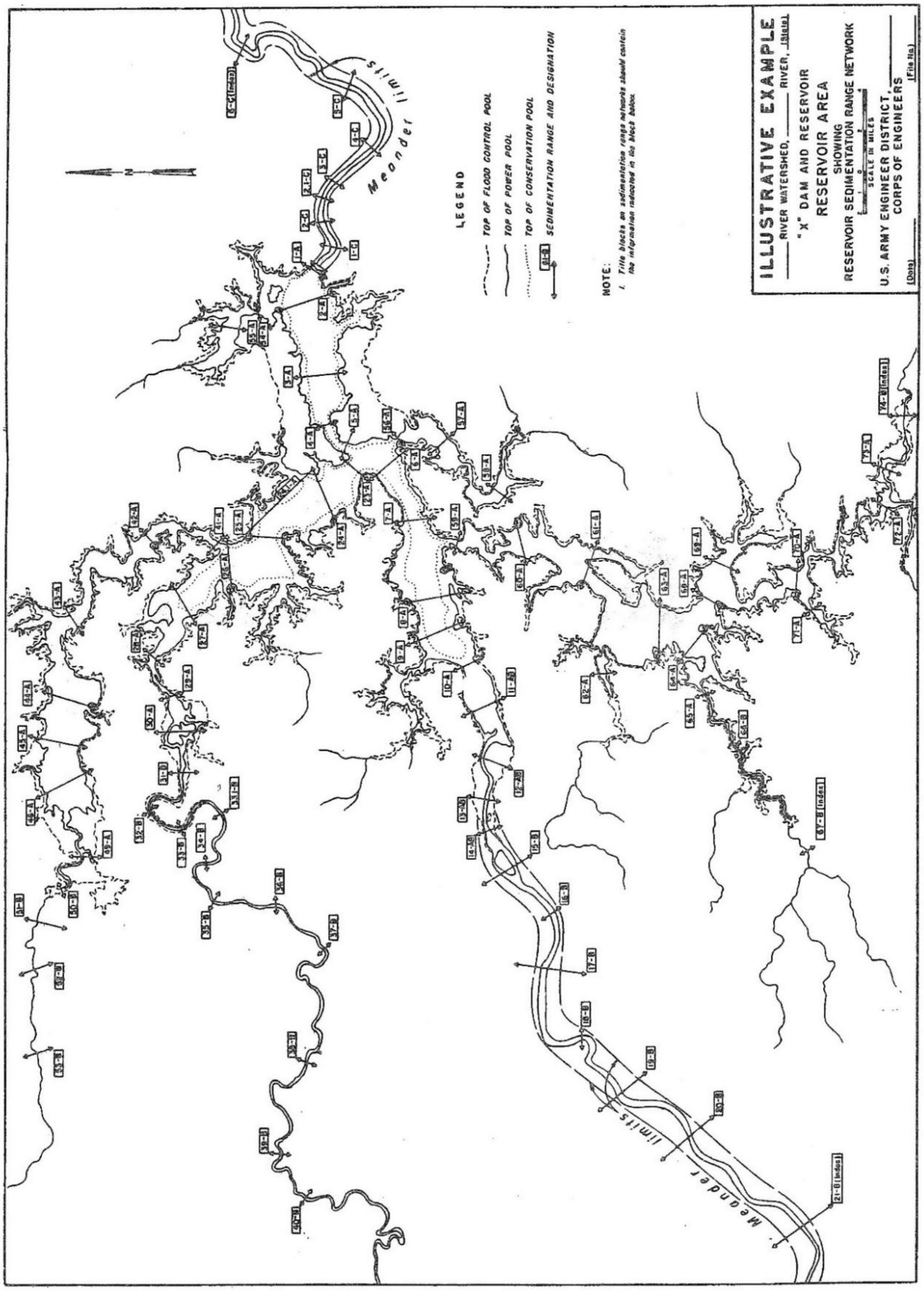


Figure G-1. Numbering of sediment ranges

Table G-1
Sediment Range Classification

	Reservoir Sedimentation Range Classification		
	Range Designation	**Definitions**	**General Considerations**
According to Scope of Study	Detailed Study Range	A range included in a general network of ranges suitably spaced to provide a basis for detailed surveys.	A network of detailed study (or study) ranges will be established where it is deemed necessary to measure sediment effects on a comprehensive basis.
According to Scope of Study	Index Range	An index range is the same as a study range, except that it may be more or less isolated from any correlated study network.	Index ranges are usually established at locations selected to: (1) provide index information for verification of the general magnitude of sedimentation effects without attempting to make detailed quantitative evaluations of such effects, and (2) conform as nearly as practical, with minimum costs for initial establishment and for resurveys. For example, to refute claims to the contrary or to realize reconnaissance purposes, index ranges may be established at certain locations simply to verify that sediment effects are negligible. At the same time, consideration should be given to the selection of locations with view to practical advantages such as ease of access and using established survey control points. (The latter consideration applies to establishment of all ranges, but the latitude is usually greater for choosing index range locations.)

	Reservoir Sedimentation Range Classification		
	Range Designation	**Definitions**	**General Considerations**
According to Technical Objectives	Category A	Ranges crossing the main body and principal arms of a reservoir that are required as a basis for determining storage capacity depletions resulting from sediment depositions with a degree of accuracy commensurate with general engineering needs.	A network of Category A ranges will be required when a detailed study of reservoir sedimentation is to be modeled. The ranges will be sufficient in number and suitably spaced to permit computation of sediment volumes by methods similar to those used in earthwork computations. It is impractical to prescribe rigid criteria for spacing and arranging Category A ranges, but observance of the technical objective involved will serve as a guide to judgment. In general, Category A ranges are established in directions that are approximately normal to the valley, and at points where distinct breaks in reservoir configuration or bottom slopes occur; random ranges at various angles are appropriate in some cases.
According to Technical Objectives	Category B	Ranges crossing reservoir arms and proximate reaches of tributary channels that are a basis for determining the magnitude of sediment depositions and related physiographic changes resulting from backwater reservoir influence.	Category B ranges will be established only where backwater influences are considered likely to create adverse effects on lands or cultural developments, or give rise to claims of adverse effects, but not at all elevations where the reservoir pool may exert hydraulic retardance on inflows during some phase of its operation. From a practical standpoint, it is necessary to evaluate the adverse effects of such backwater influences only in those reaches where it is reasonable to anticipate significant difficulties. Accordingly, proposals for establishing Category B ranges should be supported by appropriate comments indicating why the proposed ranges are considered advisable in the specific instances.

	Reservoir Sedimentation Range Classification		
	Range Designation	**Definitions**	**General Considerations**
According to Technical Objectives	Category AB	Ranges having requirements under both Category A and Category B conditions.	This designation should not be applied unless it is considered that the elimination of the range from the network would be seriously detrimental to both objectives A and B. For example, if a particular range, considered essential for the proper study of backwater effects, would have incidental value in computing reservoir sedimentation volumes, but could be eliminated without seriously reducing the accuracy of such volume computations, it should be designated only as a Category B range.

	Reservoir Sedimentation Range Classification		
	Range Designation	**Definitions**	**General Considerations**
According to Technical Objectives	Category C	Ranges crossing the stream channel and floodway in a limited reach immediately downstream from the dam as required for determining the nature and extent of cross-section changes.	Category C ranges are usually considered as part of a general reservoir sedimentation survey program and serve as a means of observing channel changes regardless of the process causing the change (such as retrogression, aggradation, degradation, or simply erosion by hydraulic action). However, Category C range installations are usually limited to a reach extending downstream from the dam a sufficient distance to delineate any substantial lowering of the bed, and/or widening of the channel that is likely to occur as relatively sediment-free water, released from the reservoir, regains its normal transportable load of sediment. The reach involved usually extends initially only a few miles downstream but may be as long as 10 to 50 miles or possibly longer in exceptional cases. The length of a reach tends to increase as retrogression proceeds over a period of years. Category C ranges are usually established for a sufficient distance downstream to provide for studies anticipated within a reasonable period; additional ranges may be added later if needed.

	Reservoir Sedimentation Range Classification		
	Range Designation	**Definitions**	**General Considerations**
According to Field Conditions	Submerged Range	A range or range segment across a portion of a reservoir or channel that is frequently submerged for protracted periods and therefore requires the use of hydrographic methods for surveys usually involving the use of floating equipment including echo sounders, lead lines, etc.	The normal operation of multipurpose reservoirs necessitates the frequent submergence of certain portions of the reservoir area, thus causing a major portion of sediment deposition occurring in these areas. In some cases, records of sedimentation within the conservation-power pool zones are of greater practical significance than information regarding storage depletion at higher levels. In the interest of economy, it may be appropriate that certain ranges in these areas be terminated below elevations of the outer limits of the flood control pool.
According to Field Conditions	Overbank/ Pool Combination Range	A range that ordinarily must be surveyed using both hydrographic and terrestrial survey methods.	This designation applies to ranges that are located in the upstream reaches of the reservoir. These ranges may be critical to monitor depletion in the flood storage zone(s). A portion of the range may include the main or tributary arm inflow channels. Vegetation in the delta deposition zone may create difficult survey conditions.
According to Field Conditions	Dry-Land Range	A range that ordinarily is surveyed by terrestrial methods.	This designation generally applies to ranges in all detention-type reservoirs or channels that are not ordinarily submerged to sufficient depths for long enough periods of time to permit advantageous use of floating equipment for sediment surveys. Deposition zones and frequent saturation conditions may create vegetation levels that create difficult survey conditions.

(1) Ranges Upstream from Dam. A single series of consecutive numbers will be used to identify ranges in Categories A and B”

(2) Ranges Downstream from Dam. A separate series will be used to identify ranges in Category C beginning with No. 1-C for the range nearest the dam and progressing downstream.

(3) Ranges Added to the Network. Ranges subsequently added to an existing network will be identified by a decimal series added to the range number. For example, inserting a new range between existing ranges labeled as 24-A and 25-A would be labeled as 24.1-A.

(4) Numbering Option. It is sometimes advantageous to use a numbering system reflecting the name of the stream that the range crosses. For example, a sedimentation range on New Hope River would be referenced as NH1-C downstream of the dam and upstream of the dam with an A or B suffix.

G-4. Simplified Methods for Numbering Sediment Ranges.

a. General. Review of USACE Districts nationally determined that implementation at most locations was simplified from the detailed number system previously described. Simplification was desirable for a number of practical reasons. Foremost, the number of installed sediment ranges at most USACE projects did not warrant the development of the complex numbering system. In addition, the sediment range data collection was performed by separate organizational staff, typically field offices, at most USACE projects. Other reasons for simplification included limiting the number of characters for record keeping, coding sediment range numbers within formatted computer cards, and use with computer software to avoid combined alphanumeric fields.

b. Numbering by River Miles. Most USACE projects have been constructed on large rivers and streams that have an established main channel river mile. In this case, numbering by river mile where the sediment range crosses the alignment was often employed. An advantage of this numbering system is that the river mile also provides location information. Figure G-2 is an example of this type of numbering system.

Figure G-2. Degradation sediment range numbering by river mile

c. Numbering for Smaller Projects. Sediment range numbering for smaller reservoirs or streams often consisted of a simplified USACE project letter code and range number. Figure G-3 provides an example where the two-letter code is “CS,” indicating the USACE project Cottonwood Springs, followed by a hyphen and the sequential sediment range number. The numbers begin at the first sediment range upstream from the dam and increase in the upstream direction.

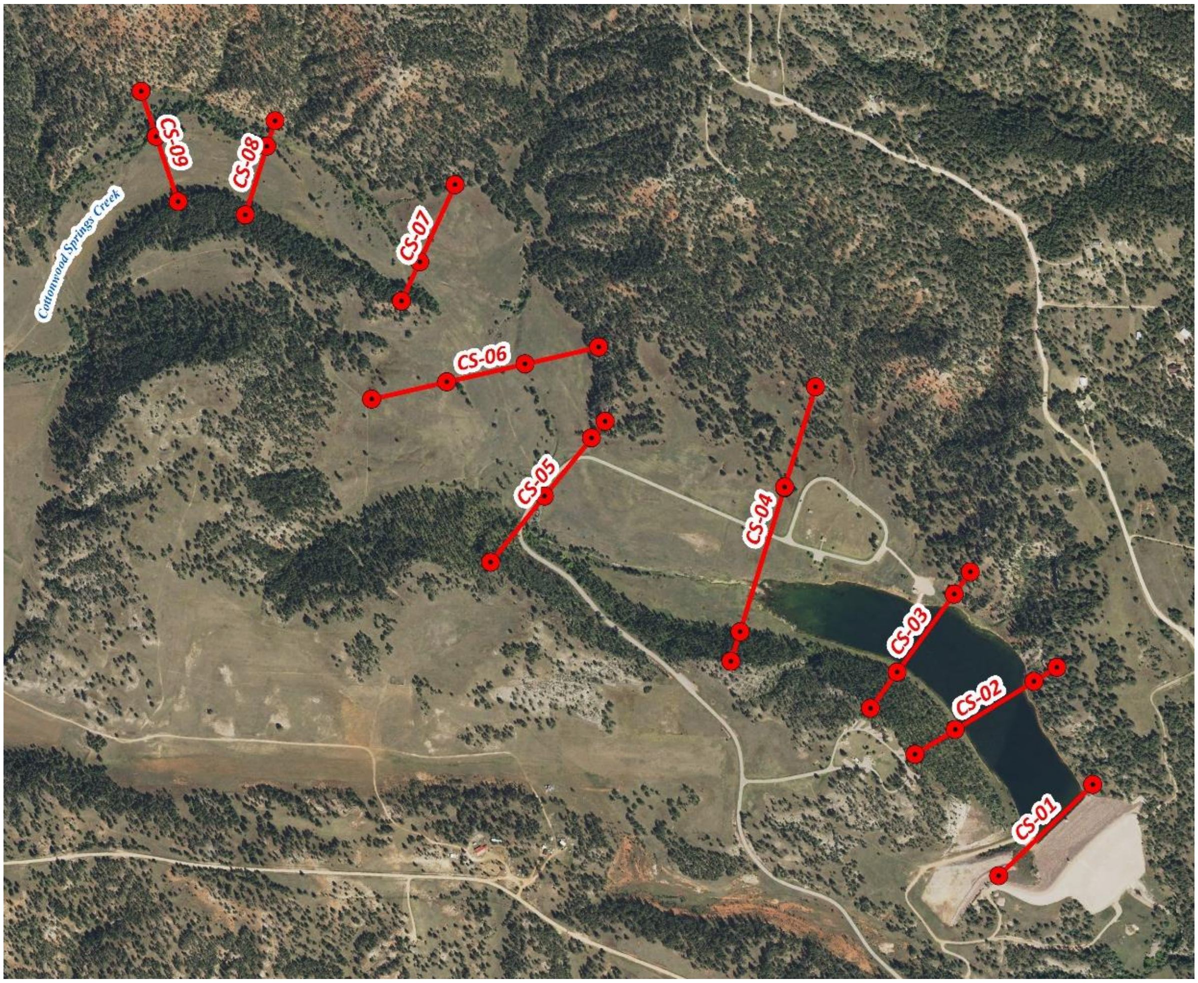


Figure G-3. Numbering of sediment ranges at smaller projects

G-5. Locating and Spacing Ranges.

a. General Principles.

(1) Sediment ranges should be located with respect to the irregular boundaries of the reservoir so that volume computation methods can be used to reconstitute the initial volume in the reservoir and the volume depletion measured by sedimentation resurveys of those same ranges.

(2) Equally important, ranges should be located to monitor the predicted delta profile so sediment deposition in this area will be accurately reflected in the resurveyed sediment range cross sections.

(3) The computation schemes for the volume in the reservoir and the volume of sediment deposited must be compatible.

(4) The ranges should be located by persons who understand the computational procedures that will be used to calculate reservoir capacity and sediment depletion, and who know the sedimentation forecasts for the reservoir in question. A field reconnaissance of the area is essential.

(5) The end points at each range should be monumented using established survey procedures to ensure the repeatability of future surveys. However, responsible survey personnel should be authorized to change the proposed range locations, within reasonable limits, so monuments can be set in the most favorable locations.

b. Range Layout Upstream from Dam.

(1) Networks are useless if ranges are incorrectly positioned or if they are located too far apart to provide the necessary resolution for the computation scheme. Moreover, the layout should be such that ranges can be easily located in the field and the topographic/hydrographic survey can be conducted with minimum effort.

(2) The first range should be placed at the toe of the dam. Continue along the main stem and include major tributaries. Extend the ranges past the limits of the reservoir pool area to the limits of the study area as described in Chapter 8, Reservoir Sedimentation, in this manual. Experience has shown that deposition is occurring upstream of the sediment range network established historically at many USACE reservoirs.

(3) Ranges should be oriented normal to the anticipated streamflow pattern after impoundment.

(4) These objectives suggest that developing the range network requires two passes through the reservoir. On the first pass, locate ranges where required to calculate reservoir volume. On the second pass, add those ranges where extra detail is needed. For example, deposition downstream from major sediment-producing tributaries may require extra detail.

(5) It is assumed that total sediment yield during the life of the project has been determined by the time range network design begins, and that the volume and distribution of the sediment deposit has been calculated. If such basic information is not available, take a conservative attitude, tending toward closer spacing, when planning the sedimentation ranges.

(6) Since the objective is to determine volume and aerial distribution of sediment deposits in specific areas, ranges must be close enough together to provide records of sediment depths over-representative subdivisions of those areas. That spacing varies somewhat with the type of pool as follows:

(a) Spacing in Conservation and Water Supply Pools. The storage capacity of water supply pools is usually small relative to the capacity allocated to flood control. Therefore, sediment deposition in these pools will cause noticeable problems more rapidly than in the larger, flood control pool. Consequently, advanced information on storage depletion is needed for planning storage reallocation or alterations to hydraulic structures. As a rough guide, a “substantial” sediment accumulation may be assumed to mean 20% of the original conservation-water supply storage.

(b) Spacing in Power Pools. The necessity for locating Category A ranges in the power pool are somewhat less urgent than the cases cited above for water supply and conservation pools, but these ranges will be important where sediment accumulation during project life are expected to exceed 10% to 20% of the original capacity.

(c) Spacing in Single-Purpose Flood Control Reservoirs. In single-purpose flood control reservoirs, A (Index) ranges will satisfy operational needs if the volume of accumulated sediment in 100 years is expected to be less than 20% of the original capacity.

(d) Spacing in the Flood Control Pool of Multipurpose Reservoirs. In multipurpose reservoirs where flood control is a primary purpose, the need for Category A ranges crossing the flood control pool is greater because of the difficulty of reallocating storage among multiple functions.

c. Range Layout Downstream from Dam. Sediment range layout downstream from the dam criteria is as follows:

(1) Ranges should start just downstream from the dam as safety permits.

(2) In stable channels, characterized by erosion-resistant rock beds and banks, a few Category C (Index) ranges at selected locations will provide a satisfactory database to verify that degradation is not a problem to stilling basin performance. In addition, cross sections should be located at all bridge crossings within the study reach. Determining study area boundaries is a complex task with the objective that range extent should be sufficient to monitor the expected future degradation reach for expected USACE needs.

(3) In alluvial channels, Category C ranges will be closely spaced near the dam with spacing increasing in the downstream direction. Locate ranges at hydraulic controls and sediment controls.

(4) Special Problem Areas. Ranges are advisable in areas where major sedimentation problems are expected or where visible sedimentation problems may be annoying to reservoirs users and the public at large.

d. Modifications to Existing Range Networks. As sedimentation increases in a reservoir, the location of problem areas may change, necessitating the addition of ranges not originally required. The need for additional ranges may also become apparent after some experience in operation of a particular reservoir has been gained. It is anticipated that modifications of established range networks will be made at the discretion of the District Engineer, as needs arise. However, the need for such additions should be minimized by adequate initial installations. All modifications should be reported to higher authority.

e. Range Monuments and Supplemental Markers.

(1) The vertical and horizontal controls used in resurveys must conform exactly to those governing the initial survey. Consequently, the end points of each range should be monumented with permanent survey control points following the guidelines established in EM 1110-1-1002. The end point monuments should be placed above the anticipated highest pool elevation for each reservoir.

(2) A system of secondary survey control monuments may be desirable. These secondary monuments should be set above the normal reservoir operating elevation or normal streamflow and away from areas susceptible to bank erosion. The purpose of these secondary survey control monuments is for datum checks during the resurvey process, and in highly vegetated areas will provide line-of-sight and control for the use of optical survey instruments.

(3) Without exception, accurate, pertinent records of the initial surveys and all survey control points must be preserved.

f. Permanent Monuments. Permanent monuments consist of bronze tablets set in stone or concrete emplacements on firm foundations. The base of such a monument should be buried sufficiently to prevent movement by frost action or accidental blows but may be incorporated in exposed structures if these are known to be stable.

g. Semi-Permanent Range Markers. Semi-permanent monuments will be used only where restoration could be accomplished at reasonable cost. They are structurally similar to permanent monuments, except that stability and survey accuracy requirements are reduced. These usually consist of exposed metal posts (angle irons, etc.) firmly set in the ground, possibly with concrete, on the line of sediment ranges. Elevation benchmarks are established at the base of these markers. Such range markers are used near the edge of permanent pools, or near pool levels attained fairly frequently, to reduce land survey requirements associated with underwater sedimentation surveys. Colors are desirable for identification purposes.

h. Temporary Markers. Various forms of temporary markers (flags, painted fence posts, etc.) are used in connection with actual surveys; these are supplementary to more permanent type markers and monuments.

i. Horizontal and Vertical Control for Range Monuments and Surveys. Sediment ranges will be surveyed in their entirety from the beginning end point left to the final end point station right. Real-time kinematic (RTK) GPS should be used wherever possible for determining position and elevation. However, some range surveys may require using an optical instrument to survey heavily wooded areas. An electronic fathometer should be used to determine underwater bathymetry. The installation of survey control and all range surveys should conform to the standards laid out in the following USACE engineer manuals: EM 1110-1-1002, EM 1110-1-1003, EM 1110-1-1005, and EM 1110-2-1003.

(1) Geodetic Survey Controls. This term applies to conventional triangulation or closed traverse ground surveys with a degree of accuracy of fourth order triangulation standards or better. With advances in survey methodology and the use of electronic surveying techniques (such as RTK with GPS), these methods are rarely used.

(a) Geodetic survey methods have the advantage of providing a highly reliable basis for relocating monuments in the future, regardless of changes in terrain or other modifications in conditions.

(b) Geographic positioning facilitates plotting the ranges on maps for calculating sediment volumes.

(c) Therefore, such methods are preferred in positioning range monuments where the cost is commensurate with the purposes to be served.

(2) Landmark Survey Controls.

(a) In some locations, closed traverse points are so far from sediment ranges that extensive ground surveys would be required to establish geodetic survey controls. In such cases, it may be advisable, for the sake of economy, to monument range locations to fixed landmarks or permanent structures. The exact geographic position of the references is not essential, provided the location of the ranges involved can be accurately positioned in the field. The locations of ranges on maps can usually be approximated satisfactorily for study purposes.

(b) The disadvantage of the landmark survey method is the risk that the landmarks used as the base location may be destroyed or modified during the project life.

(3) Witness Posts. All range survey control points should be marked “witnessed” with a commercial fiberglass marker, fence post, or other type of marker to facilitate location in the field.

j. Inspection and Maintenance of Field Facilities. Maintenance of each range survey control point may be necessary. Some survey control points may erode into the lake/stream or become buried beyond use by flooding and may have to be replaced. Maintenance is also required due to vandalism and natural events. Maintenance should occur during each resurvey or at some period time when funding and workforce permit a special maintenance effort. Whatever the approach, document it in the plan for the reservoir investigation program.

k. Removal of Vegetation along Range Lines. Line-of-sight clearing is often required for entire range lengths where vegetation may inhibit the use of GPS surveying methods and land survey methods using optical survey instruments must be used. In general, keep the clearing of vegetation to the minimum necessary to complete the survey.

l. Bank and Bar Movement. Changes in banklines, bar formations, and other channel features downstream from a dam are not adequately shown by ranges spaced at normal intervals. A closer spacing of ranges may produce the desired coverage, and in some cases, a mapped area may be advisable. Consideration should be given to the use of controlled aerial mosaics, LiDAR mapping, or other topographic mapping products. Periodic visits to the site, the collection of sediment samples from the stream bed, and visual observations of conditions in the problem areas should be included in the study plans. A data collection program for observing channel and bankline changes should include information on anticipated problems and proposed survey methods.

Appendix H
Sediment Range – Reservoir Capacity and Storage Depletion Computations

H-1. Introduction.

a. This appendix provides information on the reservoir capacity and depletion computations that are performed with sediment range survey data. Historically, reservoir capacity computations using the sediment ranges with the traditional average end area method or the MAEA with correction factor tables have been used at many USACE projects.

b. New remote sensing survey techniques and GIS-based computation methods provide both a cost effective and highly accurate alternative. GIS-based volume computation methods using remote sensing survey techniques are not addressed in this appendix. The remainder of this appendix provides historic information regarding reservoir capacity computations using sediment range survey data. Refer to Appendix G for additional information regarding use of sediment ranges within USACE.

H-2. Depletion Rates. Reservoir depletion rates refer to the rate at which storage capacity is lost due to sediment deposition within the reservoir. The most commonly used method for calculating volume of sediment deposits is by subtracting the resurvey capacity from the original capacity. Heinemann and Rausch (1971) stated that the sediment deposits may change in average density because of compaction between successive surveys and could possibly give erroneous sedimentation rates. In addition, variation in sediment source could affect sediment density. Therefore, differences in successive reservoir capacities, and corresponding depletion rates, should include adjustments to the sediment density.

H-3. Contour Area Methods. The contour area methods assume that the area encompassed by a contour line and the contour interval can adequately represent the volume between any successive contour elevations. The smaller the contour interval, the more accurate the method. Experience at USACE projects has shown that 2-foot contour intervals are adequate for most volume computations. There are four contour area methods: stage area, modified prismoidal, average contour area, and Simpson's Rule.

a. Stage Area Method. This method requires an accurate stage area curve. The stage area curve is developed by measuring via planimeter or other method the area inside a contour line and plotting it against the contour elevation as shown in Figure H-1. Reservoir volume is calculated by integrating the area between this "contour area curve" and the y-axis as indicated by the shaded area of Figure H-1.

b. Modified Prismoidal Method. This method is based on an average of the areas of two successive contour elevations. Figure H-2 shows the concept for this method. It is expressed mathematically as:

$$V = \left(\frac{L}{3}\right) \cdot (A + SQRT(A \cdot B) + B) \qquad \text{Equation H-1}$$

where:

V = volume between two contour elevations
L = contour interval
A = area of lower contour
B = area of upper contour

c. Average Contour Area Method. This method uses the average of two contour areas multiplied by the contour interval and is represented by the following equation. The variables are the same as previously defined.

$$V = \left(\frac{L}{2}\right)(A + B) \quad \text{Equation H-2}$$

d. Simpson's Rule. This method requires the contour interval to be constant if using contour area data. If cross-section-area data are used, the cross sections must be parallel and evenly spaced. Both require an even number of segments, therefore, if there is an odd number of segments, another method must be used for the last interval. The general equation is:

$$V = \left(\frac{1}{3}\right) h[A_0 + A_n + 4(A_1 + A_3 + \cdots + A_{n-1}) + 2(A_2 + A_4 + \cdots + A_{n-2})] \quad \text{Equation H-3}$$

where:

V = capacity in acre-feet
A = area of contour or cross section in acres
H = interval spacing between contours or cross sections
n = total number of contours or cross sections

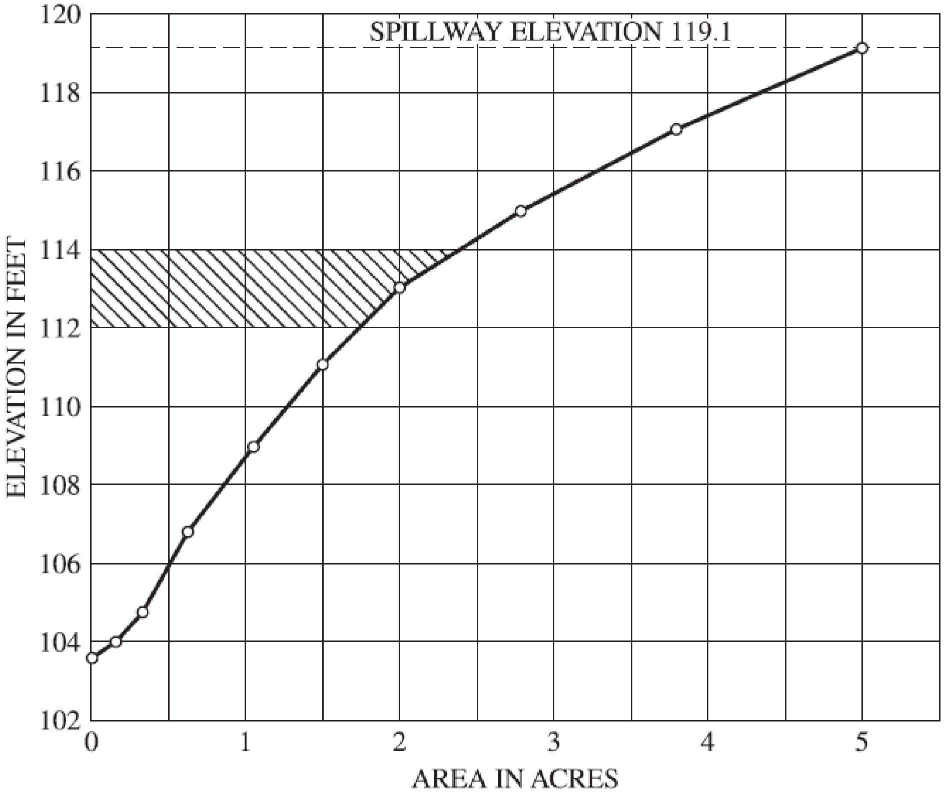


Figure H-1. Reservoir area vs. elevation (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

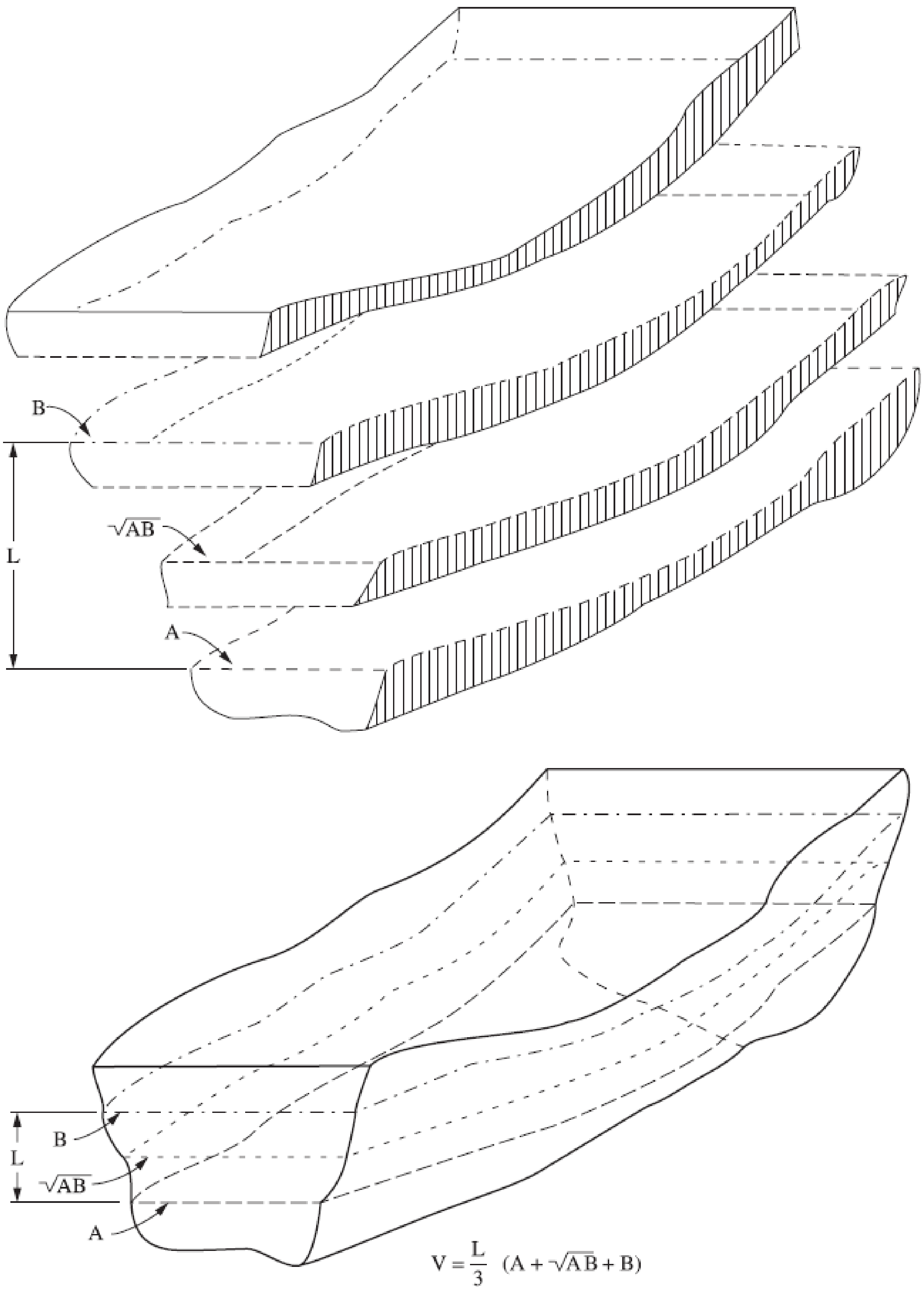


Figure H-2. Modified prismoidal method (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

H-4. Cross-Sectional Area Methods. Cross-sectional area methods require the areas of cross sections (ranges) and distance between them that necessitate the careful selection of range location and orientation to properly represent the topography. Four basic methods use cross-sectional areas: average end area, cross-sectional area vs. distance from dam, Eakin's range end formula (Eakin 1936), and Simpson's rule. Simpson's rule using cross-sectional area has previously been described in paragraph H-3d.

a. Average End Area Method. Using this method involves averaging the end areas of successive ranges and multiplying by the distance between the ranges to obtain the intermediate volume. The total volume is computed by adding each intermediate volume for the entire reservoir length.

b. Cross-Sectional Area vs. Distance from Dam. A plot of cross-sectional area (ordinate) vs. distance from the dam (abscissa) is first constructed in this method. A smooth curve, Figure H-3, is drawn through the plotted points and the area under the curve represents the total volume. Cross sections are assumed to be oriented parallel to the dam, and the distance from the dam is represented by a line perpendicular to the dam and cross section.

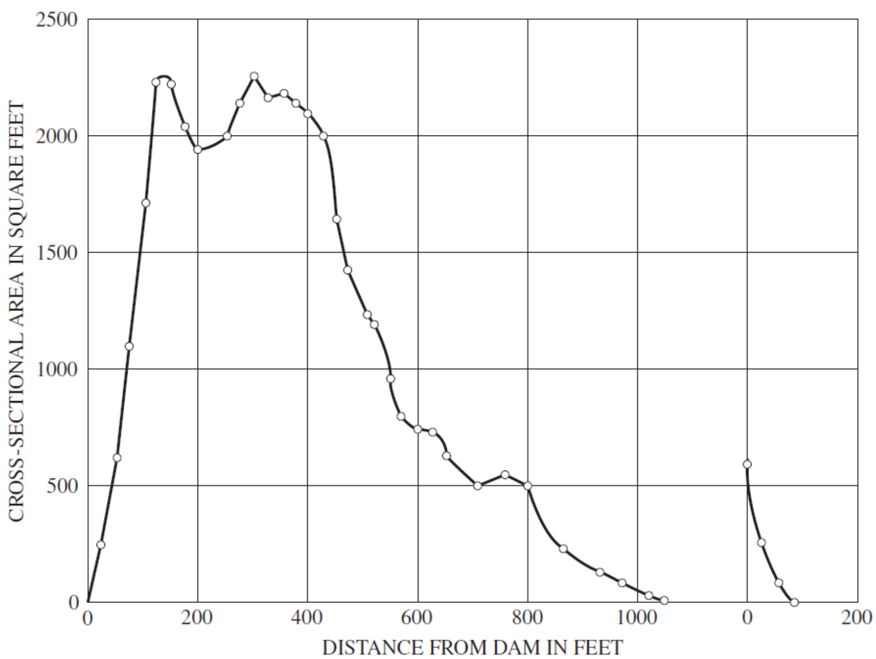


Figure H-3. Cross-sectional area vs. distance from dam (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

c. Eakin's Range End Formula.

(1) Eakin's method is an adaptation of the prismoidal formula and is shown in Figure H-4. The basic equation is:

$$V = \left(\frac{A}{3}\right) \cdot \left[\frac{E_1+E_2}{W_1+W_2}\right] + \left(\frac{A'}{3}\right) \cdot \left[\frac{E_1+E_2}{W_1+W_2}\right] + \frac{h_3 \cdot E_3 + H_4 \cdot E_4}{3} \cdot 43560 \qquad \text{Equation H-4}$$

where:

v = capacity between ranges, in acre-feet
A = total surface area of the segment at crest contour elevation, in acres
A′ = total surface area of quadrilateral (abed) formed by the intersections of the range with the crest elevation in acres
E = range cross-sectional area below crest elevation, in square feet
w = width of range at crest elevation
h = perpendicular distance from a tributary range to the junction of the tributary with the main stem or to the junction of the tributary with the downstream range, whichever is shorter, in feet. See Figure H-4.

(2) If the ranges are not parallel, A′ must be computed by substitution of line segments ab and cd by 12 and 11 respectively, where:

11 = perpendicular distance from the downstream range to the upstream range at its intersection (right side looking upstream) with the crest elevation
12 = perpendicular distance from the upstream range to the downstream range at its intersection (left side looking upstream) with the dam crest elevation

(3) The last term in Eakin's formula contains the contributing volume from the most downstream tributary range to the main stem and may be omitted if there are no tributaries with the ranges. The formula can be applied again from the downstream tributary range to the next upstream tributary range if there are more than one tributary range.

H-5. Combination Cross-Section Contour Area Method. Burrell (1951) developed a constant factor method that uses both contour and cross-section area information to directly compute deposited sediment volumes. In his method, the volume portion between ranges and bounded by the dam crest elevation is termed a segment and that portion in the segment between contour planes is termed a sub-segment. The volume of each sub-segment is then defined as:

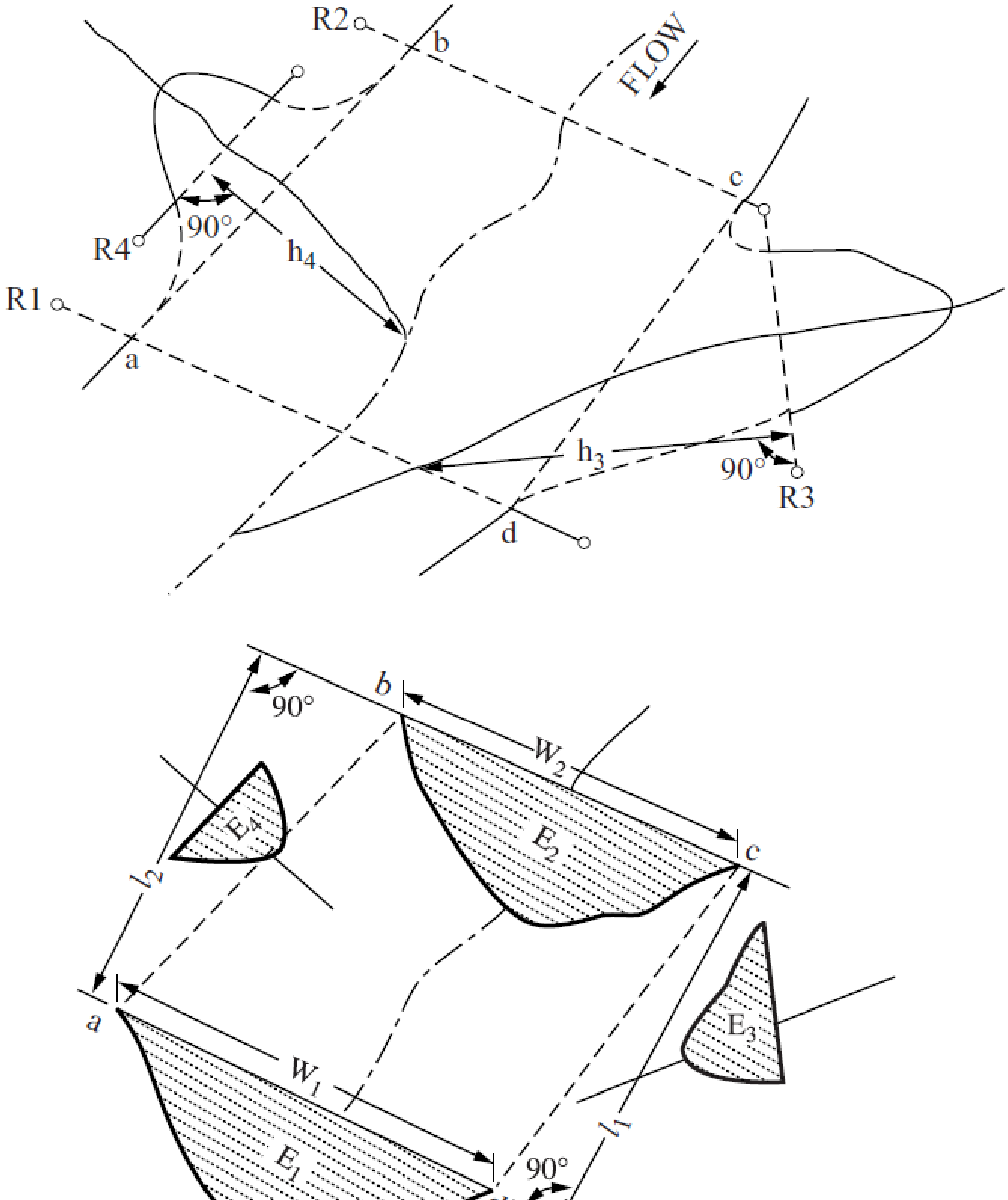
$$V_s = V_0 \cdot \left(\frac{A_s' + A_s''}{A_0' + A_0''}\right) \qquad \text{Equation H-5}$$

$$= F \cdot (A_s' + A_s'') \qquad \text{Equation H-6}$$

where:

F = $V_o/(A_o' + A_o'')$
V_s = sediment deposited in a sub-segment
V_o = original segment volume
A_o = original cross-section area
A_s = sediment area of sub-segment

′ = upstream cross section
″ = downstream cross section



Basic Equation

$$V = \frac{A'}{3}\left(\frac{E_1 + E_2}{W_1 + W_2}\right) + \frac{A}{3}\left(\frac{E_1}{W_1} + \frac{E_2}{W_2}\right) + \frac{h_3E_3 + h_4E_4}{3\ (43,\ 560)}$$

Figure H-4. Eakin's range end method (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

H-6. Accuracy of Methods. Heinemann and Dvorak (1965) determined reservoir capacity of several small reservoirs using stage area modified prismoidal, Eakin's range formula, Simpson's Rule (range cross-sectional area method), average contour area, and cross-sectional area vs. distance from dam methods. They found all these methods to be fairly accurate with the greatest deviation coming from comparisons of skewed and parallel ranges depicting the same reservoir shape. They also considered the stage area method to be the most direct, simple, accurate, and uniformly adaptable method.

H-7. Normal Usage. The stage area, prismoidal, average contour area, average end area, Eakin's Range End formula, and combination cross-section contour area methods are restricted for use by the spacing or orientation of sediment ranges. Contour methods are generally used for original volume computation because of availability of contour maps or the relative ease of obtaining more accurate contour maps by aerial photometric procedures. Cross-sectional area methods are generally used for resurveys because reservoir ranges, which are used in these methods, have previously been established.

Appendix I
Instructions for Compilation of Reservoir Sedimentation Data Summary – Form 1787

I-1. Introduction. The following instructions were prepared by members of the Subcommittee on Sedimentation, Advisory Committee on Water Information (revised March 1966), as a guide for completing ENG Form 1787, Reservoir Sediment Data Summary. Figures I-1 through I-4 provide examples illustrating ENG Form 1787 content.

a. Purpose. The purpose of the form is to provide a means for the uniform compilation and dissemination of pertinent basic data obtained in connection with reservoir sedimentation surveys.

b. Approach. The intent is to prepare a summary for each reservoir on which one or more sedimentation surveys has been made.

(1) The form should be reproduced, consistent with administrative provisions of the originating agency, in a sufficient number of copies to meet the needs of each agency represented on the subcommittee. This will permit each agency to accumulate a file of basic data prepared in a uniform manner that can be subjected to statistical treatment and interpretation.

(2) The Subcommittee recognizes that the Summary requests items of data that are not available for every reservoir. However, the timely compilation and dissemination of available data are preferred to delaying the publication until some of the more obscure items are collected.

(3) A new summary should be prepared when a new survey is made, and should bring forward the results of previous surveys, as indicated in the instructions. The new summaries can then be substituted for the older summaries in the office files.

I-2. Instructions for Compiling the Summary.

a. General notes for summary preparation are:

(1) In all cases where data are estimated or assumed, indicate by asterisk, and show an asterisk with the word “assumed” at the bottom of the form.

(2) Where other information is presented that needs clarification, footnotes should be used and shown by numbers, as 1, 2, etc. All footnotes are to be explained in the space provided under Item 47.

(3) All data should be shown to at least three significant digits, if available, and if accuracy of the survey warrants it. For example, for Item 14: 167,624; 16,762; 1,676; 168; 16.8; 1.68.

(4) Items 31, 32, 37, 38, 40, and 41: Where the sedimentation survey of a multiple purpose reservoir has covered only the pool level or levels used for storage most of the year (as irrigation, power, inactive) and has not covered the flood control pool above such levels, the data

should be shown for the pool levels surveyed. However, any data obtained concerning sedimentation in the flood control pool (not including surcharge storage) should be shown under the above items with a footnote reference of explanation under Item 47.

(5) Use continuation sheets when all data cannot be placed on one sheet.

b. Top section of the form includes:

(1) Name of Reservoir: Give official or most commonly used name. If dam has another name, give it in parentheses (for example, Lake Mead (Hoover Dam)).

(2) Data Sheet No.: Leave blank. The Subcommittee on Sedimentation will supply the Data Sheet Number.

c. Specific items on the Reservoir Sediment Data Summary (SCS-34 Rev. 6-66 or ENG Form 1787) are:

(1) The name of the person or the organization that owns or operates the structure. If a Federal or State government, give both the department and agency having supervision or control over operation of the dam. (Abbreviate as necessary).

(2) If the reservoir is located on a small stream, the name of which is not known, list as a tributary of the next largest stream. For example, “Trib. of Rock R.”

(3) If the dam lies in two states, both states should be given, the first state being that in which the headquarters for operation of the dam are located.

(4) Give the location of the dam by section, township, and range.

(5) Give the name of the nearest post office. If space permits, adding the distance in miles and direction of the dam from the nearest post office helps to pinpoint the location of the dam, as Tulsa 2 SE.

(6) Give the county in which the dam is located. If the dam is in two counties, the first-named county should be the one in which headquarters for operation of the dam are located, followed by a hyphen and the name of the second county.

(7) Give the latitude and longitude of the dam in degrees and minutes (seconds, if known).

(8) The elevation of the top of the dam, which is equal to the highest spillway elevation (Item 9) plus freeboard (specify vertical datum in Item 47, Remarks and References. All stated elevations on the entire form must be in a single vertical datum.

(9) This is the elevation of the highest spillway. If spillway is topped by movable gates, give the elevation of top of the gates in closed position, with an explanatory footnote in Item 47, Remarks and References. (See Item 2 under General Notes.)

(10) The sub-items under item 10 designate the purpose of the storage space allocation. All data corresponding to storage allocations a through g refer to original storages in the reservoir, if these data are available, or otherwise, to the first accurate capacities determined after the beginning of storage. Show revisions of initial storages if recent surveys yield more accurate data than the early surveys.

(a) Self-explanatory.

(b) Multiple use storage space refers to that which is purposely varied, seasonally, or alternately, as required, to serve two or more purposes. Use a footnote to explain the specific uses in Item 47.

(c) This item ordinarily refers to storage for hydroelectric or direct power development. However, storage developed or allocated specifically for cooling purposes in steam power plant operation should be listed under this item with a footnote explanation in Item 47.

(d) This item refers to water supply for municipal, industrial, domestic or livestock use, and fire protection.

(e) This item refers to storage space allocated specifically for water used to irrigate agricultural land.

(f) This item refers to storage allocated for regulation of low water flow of streams, navigation pools, recharge of ground water, recreation, fish, and wildlife, etc. Specify by footnote.

(g) This refers to storage below the lowest outlet in the dam that cannot be withdrawn for any consumptive or beneficial use and is not generally considered to be of significant value for any purposes listed under Conservation. This pool elevation in small reservoirs generally is considered by the Department of Agriculture to be sediment pool elevation. It is the level below which sediment is generally continually submerged and above which the sediment deposits tend to be more compacted due to periodic exposure to the air.

(11) The top of pool elevations in items a through g correspond to storage allocations listed under Item 10. Reference to mean sea level, if known. Otherwise, an assumed elevation or local datum should be given as relative elevation to the streambed level, the top of the dam, or the spillway crest. If regulation schedules provide for variation (seasonal or otherwise) in the top of pool levels, the maximum elevation should be shown with a reference to the footnote explanation of the other pertinent pool levels.

(12) Give the original surface area in acres at the elevation of the top of pool shown in Item 11.

(13) Give the original storage capacity in acre-feet for each allocation.

(14) Give the total original accumulated storage in acre-feet from the bottom of the reservoir to the top of each pool elevation indicated. Thus, the uppermost item recorded should be the original capacity of the reservoir below the spillway crest elevation shown in Item 9.

(15) Give the date when water was first impounded (month, day, and year, if possible).

(16) Give date (month, day, and year, if possible) that the initial operation for any function started.

(17) Give the length of reservoir, from the dam to the head of the backwater of the contributing stream.

(a) If the reservoir is composed of two or more principal arms, give the sum of the lengths, and specify the length of each main arm in a footnote in Item 47.

(b) Give the average width by dividing the surface area by the summation of the lengths.

(18) Give the entire flow-contributing drainage area above the dam.

(19) Give the drainage area exclusive of the surface area of the reservoir at the spillway crest elevation (Item 9) and exclusive of the upstream non-contributing basins or the watersheds above the larger reservoirs that are effective sediment traps.

(20) Drainage area length and width:

(a) Give the length of the total drainage area along the center line of the main stream valley.

(b) The average width is the area in Item 18 divided by the length in Item 20.

(21) Maximum and minimum elevations:

(a) The maximum elevation is the highest point of the watershed boundary.

(b) The minimum elevation of the watershed should be the lowest original stream bed elevation at the axis of the dam. This elevation is used to determine the height of the dam.

(22) Give the longest available recorded mean value. If known, include in parentheses the number of years of record. Give the average annual precipitation value for the total drainage area. If the mean annual precipitation varies widely for different parts of the watershed, record the range of values (for example, 18–35).

(23) Mean annual runoff in inches may be obtained from direct measurement; from published reports such as USGS Water Supply Papers; by transposing known data from similar adjacent watersheds; or from average annual runoff maps such as USGS Circular 52. For

precipitation, state the longest available recorded mean value and the number of years of record. The source of data may be shown by footnote with explanation under Item 47.

(24) The mean annual runoff in acre-feet may be obtained by multiplying Item 23, mean annual runoff in inches, by Item 18, total drainage area in square miles, times the conversion factor 53.33.

(25) The mean annual temperature and the average annual range in temperature should be given in degrees Fahrenheit.

(26) Give the date of the beginning of storage, if used to compute sedimentation, or the average date (month, day, and year) of the first reservoir survey, and of all succeeding surveys used in computing sedimentation. The original data from which the sedimentation record begins and subsequent data should be given under Items 26, 29, 30, 31, 32, and 33, but the original data should not be repeated under Item 26 below or in parallel boxes from Item 34 through Item 42, inclusive.

(27) Give the elapsed period between the beginning of storage or the first survey used to compute sedimentation (whichever is the more recent date) and between the average dates of each succeeding sedimentation survey. Compute to the nearest 0.1 year. If computations have been carried out to the nearest 0.01 year, two decimal places may be shown.

(28) Give the accumulative period from the beginning of storage or the first survey used to compute sedimentation (whichever is the more recent date) to each succeeding sedimentation survey. Compute to the nearest 0.01 year; two decimal places may be shown.

(29) Indicate Range or Contour and Detailed or Reconnaissance, as applicable. Detailed may be shown by the symbol (D), reconnaissance by (R).

(a) A detailed range survey is defined as one in which instrumental control of all sounding and spudding positions in the lake was maintained. Where this was not done, the survey should be labeled as (R).

(b) In a few cases, where instrumental control was not maintained, but the number of ranges and observations per range were substantially the same as those made on a detailed survey the designation Semi-Detailed may be used. The symbol for this should be (S).

(c) A contour survey to be labeled (D) should conform to at least standards of third order accuracy for topographic mapping (1 in 5,000). If the contouring was of an unsure or very generalized nature, designation should be (R). All contouring done with Kelsh plotters and similar equipment will be considered (D), but sketching of contours with portable stereoscope will be considered (R).

(30) Give the number of ranges or the contour interval. If a reconnaissance survey, give the number of individual measurements. The letter (M) should follow to indicate that they are

measurements and not ranges. Where a combination range and contour survey is utilized, the symbol (R) should follow the number of ranges and "(CI)" should follow the contour interval.

(31) The surface area at the spillway crest elevation (use the elevation of Item 9 to obtain the first entry). If the areas of different allocated storages have been determined, each should be referenced with a footnote to be shown in Item 47.

(32) The first figure entered should be the original capacity (below the spillway crest elevation, Item 9). If the capacities for different allocated storages have been determined, these should be shown and each one referenced with a footnote in Item 47. If the original capacity was not determined, give the first accurate capacity determined after the beginning of storage and note the date.

(33) Capacity-Inflow Ratio. C/I = Item 32/Item 24. Use the maximum capacity for the date (Item 32) for which the C/I ratio is being calculated and divide by the mean annual runoff in acre-feet (Item 24). This ratio should be adjusted if there are one or more upstream reservoirs that have a significant TE and control a substantial part of the drainage area (usually more than 25%).

(34) Give the mean annual precipitation over the drainage area for each period of years given in Item 27. If there is a substantial variation in precipitation for different parts of the drainage area, give the range as 10–23.

(35) In 35a, give the average annual water inflow to the reservoir in acre-feet for each period of years given in Item 27. The highest annual inflow for each period in acre-feet is to be given in Item 35b, and the total for each period is given in Item 35c.

(36) Give the water inflow in acre-feet to the reservoir for the accumulated periods of years given in Item 28.

(37) In Item 37a, give the volume of capacity loss below crest (Item 9) for the periods of years given in Item 27. Item 37b is obtained by dividing the volume given in Item 37a by the corresponding period of years shown in Item 27. Item 37c is obtained by dividing the value in 37b by the net sediment contributing area shown in Item 19.

(38) Item 38a gives the accumulative total sediment deposits below crest for the period or periods of years given in Item 28. Item 38b is obtained by dividing the value of Item 38a by the corresponding accumulative years shown in Item 28. Item 38c is determined by dividing Item 38b by the net sediment contributing area shown in Item 19. If the above-crest deposits exist and are measured, add their volume to the below crest deposits in Items 38a, b, and c, and also give these total values just under the other values. Where above-crest deposits are included, they should be referenced with a footnote and explained in Item 47, Remarks and References (see General Notes 3 and 4).

(39) Average dry weight of the deposited sediment in the reservoir, pounds per cubic foot. Since the dry weight of deposits tends to increase with time as silts and clays consolidate, dry

weight should be determined during each survey. If assumed values are used, indicate by asterisk (see General Note 1).

(40) Compute values as follows:

Item 40a = for first survey, Item 38c x Item 39 x 21.78
Item 40a = for subsequent surveys:
[(Item 38a x 39 latest) – (Item 38a x 39 previous)] x 21.78
Divided by (Item 27 for latest period) x (Item 19)

It is imperative that samples of the sediment representative of the entire period of sediment accumulation be obtained at the time of each survey:

Item 40b = Item 38c x Item 39 x 21.78

(41) Compute the values as follows:

$$\text{Item 41a} = \frac{\text{Item 38b x 100}}{\text{Item 14 (Maximum value in item)}}$$

$$\text{Item 41b} = \frac{\text{Item 38a x 100}}{\text{Item 14 (Maximum value in item)}}$$

(42) Compute the values as follows:

$$\text{Item 42a} = \frac{\text{Item 40a x Item 27 x Item 19 x 10,000,000}}{\text{Item 35c x 1359}} = \text{PPM by weight}$$

$$\text{Item 42b} = \frac{\text{Item 38a x Item 39 x 1,000,000}}{\text{Item 36b x 62.4}} = \text{PPM by weight}$$

(43) If elevation-capacity curves are developed, select the appropriate intervals in feet below and above the crest. Give the percentage of the total sediment deposits located within each depth designation (elevation zone). For example:

122–100	100–85	85–70	70–60	60–50	50–40
4	5	6	7	7	9

40–30	30–20	20–10	10–Crest	Crest–+10	+10–+20
10	12	15	18	5	2

(44) The sediment distribution in percent according to distance from the dam. The reach designation is the percent of the distance from the dam to the maximum upstream extent of the spillway crest contour at the elevation given in Item 9 at the date of the beginning of storage.

Thus, 20% would be 1/5 of the distance from the dam to the head of backwater at the original crest stage.

(45) List the maximum and minimum water elevations and the total inflow in acre-feet for each water year of record.

(46) Give data from the elevation-capacity curve for the latest survey shown on Item 26. Be sure to label each survey data on the form. If space permits give data from the elevation-capacity curve for the original survey.

(47) List here the vertical datum and horizontal control for the latest survey. Provide a list of all published and unpublished reports on sedimentation surveys of this reservoir. All footnote explanations are to be shown in this space. Also note and give any pertinent data, including dates of abnormal operational occurrences, such as reservoir evacuation; sluicing out sediment; releasing density currents; extreme floods and droughts; changes in spillway crest elevation; use of flash boards; and the installation of upstream control structures. Briefly describe the sediment and any available textural analyses. If needed, use continuation sheets.

(48) Give the department, agency, and division, branch, or field office responsible for each survey.

(49) Give the agency and department reporting the data.

(50) Give the date this form was prepared by the office listed in Item 49.

d. Prepared by the following agencies represented on the Subcommittee on Sedimentation of the Advisory Committee on Water Information (https://acwi.gov/).

DEPARTMENT OF AGRICULTURE
Agricultural Research Service
Forest Service
Natural Resource Conservation Service

DEPARTMENT OF THE ARMY
Corps of Engineers

DEPARTMENT OF COMMERCE
Bureau of Public Roads
Coast and Geodetic Survey

DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
Geological Survey
Bureau of Land Management

FEDERAL ENERGY REGULATORY COMMISSION

TENNESSEE VALLEY AUTHORITY

DEPARTMENT OF HEALTH, EDUCATION, and WELFARE
Public Health

	RESERVOIR SEDIMENT DATA SUMMARY						DEPARTMENT OF THE ARMY	
							U.S. ARMY CORPS OF ENGINEERS	
	PIPESTEM LAKE							
	NAME OF RESERVOIR				DATA SHEET NUMBER			
DAM	1. OWNER	USACE, OMAHA DISTRICT		2. STREAM	PIPESTEM CREEK		3. STATE	NORTH DAKOTA
DAM	4. SECTION	SEC. 10 - TWP. 140N - RANGE R64W		5. NEAREST P.O.	JAMESTOWN, ND		6. COUNTY	STUTSMAN
DAM	7. LATITUDE	46°57' 42"	LONGITUDE	98° 45' 16"	8. TOP DAM EL.	1507.5	9. SPILL. CREST EL.	1496.3
RESERVOIR	10. STORAGE ALLOCATION		11. ELEV. - TOP OF POOL	12. ORIGINAL SURFACE AREA	13. ORIGINAL CAPACITY, AC-FT	14. GROSS STORAGE, AC-FT	15. DATE STORAGE BEGAN	
RESERVOIR	a. FLOOD CONTROL		1496.3	4,718	142,505	177,144	7/1/1973	
RESERVOIR	b. MULTIPLE USE		1442.4	841	9,113			
RESERVOIR	c. POWER						16. DATE NORM. OPER. BEGAN	
RESERVOIR	d. WATER SUPPLY							
RESERVOIR	e. IRRIGATION						5/2/1974	
RESERVOIR	f. CONSERVATION							
RESERVOIR	g. INACTIVE		1415.0	28.0	55.0			
RESERVOIR	17. LENGTH OF RESERVOIR		5.5	MILES	AVERAGE WIDTH OF RESERVOIR		0.5	MILES
WATERSHED	18. TOTAL DRAINAGE AREA		611	SQUARE MILES	22. MEAN ANNUAL PRECIPITATION		19.0	INCHES
WATERSHED	19. NET SED. CONTRIBUTING AREA		399	SQUARE MILES	23. MEAN ANNUAL RUNOFF		1.20	INCHES
WATERSHED	20. LENGTH, MI	N/A	AVE. WIDTH, MI	N/A	24. MEAN ANNUAL RUNOFF		25,521	ACRE-FEET
WATERSHED	21. MAX. EL., FT	1650.0	MIN. EL., FT	1400.0	25. AN. TEMP. F°	40.0	RANGE	-22 to 99°F

SURVEY DATA

26. DATE OF SURVEY	27. PERIOD IN YEARS	28. ACCUMUL. YEARS	29. TYPE OF SURVEY	30. NO. OF RANGE LINES	31. SURFACE AREA, ACRES	32. CAPACITY, ACRE-FEET	33. CI RATIO AC-FT PER AC-FT
1973	0	0	RANGE LINE	21	841	9,113	0.3571
1980	7	7	RANGE LINE	21	843	8,840	0.3464
1990	10	17	RANGE LINE	21	841	8,979	0.3518
2002	12	29	RANGE LINE	21	822	8,354	0.3273
2014	12	41	RANGE LINE	21	832	8,423	0.3300

26. DATE OF SURVEY	34. PERIOD ANNUAL PRECIP.	35. PERIOD WATER INFLOW, ACRE-FEET			36. WATER INFLOW TO DATE, AF	
		a. MEAN ANNUAL	b. MAX. ANNUAL	c. PERIOD TOTAL	a. MEAN ANNUAL	b. TOTAL TO DATE
1973	*****	*****	*****	*****	*****	*****
1980	124	4,967	61,288	178,815	4,967	178,815
1990	164	2,380	52,579	238,042	1,737	416,857
2002	251	6,464	149,792	930,887	1,783	1,347,744
2014	236	6,241	319,838	961,738	1,480	2,309,482

26. DATE OF SURVEY	37. PERIOD CAPACITY LOSS, ACRE-FEET			38. TOTAL SEDIMENT DEPOSITS TO DATE, ACRE-FEET		
	a. PERIOD TOTAL	b. AVE. ANNUAL	c. PER MI²/YEAR	a. TOTAL TO DATE	b. AVE. ANNUAL	c. PER MI²/YEAR
1973	0	0	0.0	0	0	0.0
1980	-300	-43.0	-0.1	-300	-43	-0.1
1990	-92	-9.2	0.0	-392	-23	-0.1
2002	1,225	102.0	0.3	833	29	0.1
2014	-79	-6.6	0.0	755	18	0.0

26. DATE OF SURVEY	39. AVE. DRY WEIGHT, LBS/FT³	40. SED. DEPLET. TONS-MI²/YEAR		41. STORAGE LOST PERCENT		42. SEDIMENT INFLOW - ppm	
		a. PERIOD	b. TOTAL TO DATE	a. AVE. ANNUAL	b. TOTAL TO DATE	a. PERIOD	b. TOTAL TO DATE
1973	NO DATA			0.000	0.000		
1980	NO DATA			-0.029	-0.029		
1990	NO DATA			-0.010	-0.012		
2002	NO DATA			0.058	0.000		
2014	NO DATA			0.000	0.000		

ENG FORM 1787, NOV 1966 — *PAGE 1 OF 2*

Figure I-1. Example Reservoir Sedimentation Data Summary form ENG 1787 (page 1 of 2)

43. DEPTH DESIGNATION RANGE IN FEET BELOW AND ABOVE THE CREST ELEVATION

26. DATE OF SURVEY	130-110	110-90	90-70	70-50	50-40	40-30	30-20	20-10	10-CREST	CREST - +10	+10 - +20
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN DEPTH DESIGNATION										
1973			0.6%	5.2%	6.1%	8.4%	10.9%	14.1%	18.1%	25.1%	11.5%
1980			0.7%	5.1%	6.1%	8.4%	10.9%	14.1%	18.1%	25.1%	11.5%
1990			0.6%	5.2%	6.2%	8.4%	10.9%	14.1%	18.1%	25.1%	11.4%
2002			0.4%	5.0%	6.1%	8.6%	11.2%	14.2%	18.1%	25.1%	11.3%
2014			0.4%	5.1%	6.2%	8.6%	11.1%	14.1%	17.9%	25.2%	11.4%

44. REACH DESIGNATION PERCENT OF TOTAL ORIGINAL LENGTH OF THE RESERVOIR

26. DATE OF SURVEY	0-10	10-20	20-30	30-40	40-50	50-60	60-70	70-80	80-90	90-100	110
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN REACH DESIGNATION										
1973											
1980											
1990											
2002											
2014											

45. RANGE IN RESERVOIR OPERATIONS

WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW (acre-feet)	WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW (acre-feet)	WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW (acre-feet)
1973	NA	NA	NA	1990	1443.04	1441.76	2,065	2007	1459.65	1442.09	51,386
1974	NA	NA	NA	1991	1442.30	1440.64	1,925	2008	1447.02	1442.37	14,618
1975	1466.06	1440.59	59,554	1992	1443.43	1439.68	8,593	2009	1492.20	1441.98	191,686
1976	1447.39	1439.99	15,973	1993	1472.64	1439.65	85,481	2010	1474.70	1441.52	119,729
1977	1442.10	1439.97	3,923	1994	1463.54	1442.61	74,914	2011	1488.71	1442.67	319,838
1978	1454.90	1442.00	25,316	1995	1479.54	1442.69	134,211	2012	1449.98	1442.36	20,893
1979	1468.35	1441.92	61,288	1996	1475.87	1442.02	100,300	2013	1474.08	1441.53	101,424
1980	1443.47	1442.41	12,761	1997	1487.01	1440.89	149,792	2014	1460.77	1442.11	59,356
1981	1443.48	1442.35	10,122	1998	1457.76	1442.44	53,369				
1982	1457.43	1442.43	34,274	1999	1479.30	1441.98	130,426				
1983	1459.94	1442.11	47,362	2000	1464.18	1442.81	68,739				
1984	1456.88	1442.24	39,772	2001	1474.18	1441.91	111,235				
1985	1444.30	1441.16	8,811	2002	1445.26	1442.43	11,852				
1986	1447.90	1441.18	26,550	2003	1449.51	1442.04	16,456				
1987	1466.28	1442.48	52,579	2004	1456.14	1442.16	32,360				
1988	1445.22	1441.30	11,099	2005	1451.74	1442.61	23,069				
1989	1443.90	1441.45	5,407	2006	1443.33	1442.01	10,923				

46. ELEVATION & AREA-CAPACITY DATA

ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY
1417.8	0	0	1450	1,311	16,514	1485	3,485	97,761			
1420	40	54	1455	1,588	23,776	1490	3,922	116,254			
1425	149	489	1460	1,870	32,394	1495	4,531	137,117			
1430	301	1,579	1465	2,161	42,486	1500	5,444	161,786			
1435	489	3,531	1470	2,441	53,999	1505	6,486	191,661			
1440	711	6,497	1475	2,733	66,907	1510	7,505	226,636			
1445	1,000	10,692	1480	3,081	81,370						

47. REMARKS AND REFERENCES

1. Survey Units: U.S. Feet; Horizontal Datum: State Plane Coordinates, NAD83, North Dakota South, Zone 3302, U.S. Feet; Vertical Datum: NGVD29, U.S. Feet
2. The watershed consists of relatively thin loam cover with glacial moraine characteristics in the southwestern portion of the basin. Land use and soil type percentages not available. Topography of the basin is flat to gently rolling, consisting of glacial till, old lake beds, and low glacial moraines.

48. AGENCY MAKING SURVEY:	U.S. ARMY CORPS OF ENGINEERS, OMAHA DISTRICT	50. DATE
49. AGENCY SUPPLYING DATA:	U.S. ARMY CORPS OF ENGINEERS, OMAHA DISTRICT	2015

ENG FORM 1787, NOV 1966	*PAGE*	*2 OF 2*

Figure I-2. Example Reservoir Sedimentation Data Summary form ENG 1787 (page 2 of 2)

	RESERVOIR SEDIMENT DATA SUMMARY						DEPARTMENT OF THE ARMY	
							U.S. ARMY CORPS OF ENGINEERS	
	NAME OF RESERVOIR					DATA SHEET NUMBER		
DAM	1. OWNER			2. STREAM			3. STATE	
	4. SECTION			5. NEAREST P.O.			6. COUNTY	
	7. LATITUDE		LONGITUDE		8. TOP DAM EL.		9. SPILL. CREST EL.	
RESERVOIR	10. STORAGE ALLOCATION		11. ELEV. - TOP OF POOL	12. ORIGINAL SURFACE AREA	13. ORIGINAL CAPACITY, AC-FT	14. GROSS STORAGE, AC-FT	15. DATE STORAGE BEGAN	
	a. FLOOD CONTROL							
	b. MULTIPLE USE							
	c. POWER						16. DATE NORM. OPER. BEGAN	
	d. WATER SUPPLY							
	e. IRRIGATION							
	f. CONSERVATION							
	g. INACTIVE							
	17. LENGTH OF RESERVOIR			MILES	AVERAGE WIDTH OF RESERVOIR			MILES
WATERSHED	18. TOTAL DRAINAGE AREA			SQUARE MILES	22. MEAN ANNUAL PRECIPITATION			INCHES
	19. NET SED. CONTRIBUTING AREA			SQUARE MILES	23. MEAN ANNUAL RUNOFF			INCHES
	20. LENGTH, MI		AVE. WIDTH, MI		24. MEAN ANNUAL RUNOFF			ACRE-FEET
	21. MAX. EL., FT		MIN. EL., FT		25. AN. TEMP. F°		RANGE	
SURVEY DATA	26. DATE OF OF SURVEY	27. PERIOD IN YEARS	28. ACCUMUL. YEARS	29. TYPE OF SURVEY	30. NO. OF RANGE LINES	31. SURFACE AREA, ACRES	32. CAPACITY, ACRE-FEET	33. CI RATIO AC-FT PER AC-FT
	26. DATE OF	34. PERIOD ANNUAL PRECIP.	35. PERIOD WATER INFLOW, ACRE-FEET			36. WATER INFLOW TO DATE, AF		
	OF SURVEY		a. MEAN ANNUAL	b. MAX. ANNUAL	c. PERIOD TOTAL	a. MEAN ANNUAL	b. TOTAL TO DATE	
	26. DATE OF	37. PERIOD CAPACITY LOSS, ACRE-FEET			38. TOTAL SEDIMENT DEPOSITS TO DATE, ACRE-FEET			
	OF SURVEY	a. PERIOD TOTAL	b. AVE. ANNUAL	c. PER MI²/YEAR	a. TOTAL TO DATE	b. AVE. ANNUAL	c. PER MI²/YEAR	
	26. DATE OF	39. AVE. DRY WEIGHT, LBS/FT³	40. SED. DEPLET. TONS-MI²/YEAR		41. STORAGE LOST PERCENT		42. SEDIMENT INFLOW - ppm	
	OF SURVEY		a. PERIOD	b. TOTAL TO DATE	a. AVE. ANNUAL	b. TOTAL TO DATE	a. PERIOD	b. TOTAL TO DATE
ENG FORM 1787, NOV 1966							*PAGE*	*1 OF 2*

Figure I-3. Blank Reservoir Sedimentation Data Summary form ENG 1787 (page 1 of 2)

26. DATE	43. DEPTH DESIGNATION RANGE IN FEET BELOW AND ABOVE THE CREST ELEVATION										
OF SURVEY	130-110	110-90	90-70	70-50	50-40	40-30	30-20	20-10	10-CREST	CREST - +10	+10 - +20
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN DEPTH DESIGNATION										

26. DATE	44. REACH DESIGNATION PERCENT OF TOTAL ORIGINAL LENGTH OF THE RESERVOIR										
OF SURVEY	0-10	10-20	20-30	30-40	40-50	50-60	60-70	70-80	80-90	90-100	-110
	PERCENT OF TOTAL SEDIMENT LOCATED WITHIN REACH DESIGNATION										

45. RANGE IN RESERVOIR OPERATIONS											
WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW (acre-feet)	WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW (acre-feet)	WATER YEAR	MAX. ELEV.	MIN. ELEV.	INFLOW (acre-feet)

46. ELEVATION & AREA-CAPACITY DATA											
ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY	ELEVATION	AREA	CAPACITY

47. REMARKS AND REFERENCES	
48. AGENCY MAKING SURVEY:	50. DATE
49. AGENCY SUPPLYING DATA:	
ENG FORM 1787, NOV 1966	*PAGE 2 OF 2*

Figure I-4. Blank Reservoir Sedimentation Data Summary form ENG 1787 (page 2 of 2)

Appendix J
Specific Weight of Deposits

J-1. Specific Weight of Deposits. The conversion of the incoming weight of sediment to volume requires knowledge of the average dry specific weight of a mixture. The density of deposited material in terms of dry mass per unit volume is used to convert total sediment inflow to a reservoir from a mass to a volume. The conversion is necessary when total sediment inflow is computed from a measured suspended and bedload sediment sampling program. For material coarser than 0.1 mm, the specific dry weight of the mixture remains practically constant.

a. Basic factors influencing density of sediment deposits in a reservoir are: (1) the manner in which the reservoir is operated; (2) the size of deposited sediment particles; and (3) the compaction or consolidation rate of deposited sediments. The reservoir operation is probably the most influential of these factors. Sediments that have settled in reservoirs subjected to considerable drawdown are exposed to air for long periods and undergo a greater amount of consolidation. Reservoirs operating with a fairly stable pool do not allow the sediment deposits to dry out and consolidate to the same degree.

b. The size of the incoming sediment particles has a significant effect on density. Sediment deposits composed of silt and sand have higher densities than those in which clay predominates. The accumulation of new sediment deposits on top of previously deposited sediments changes the density of earlier deposits. This consolidation affects the average density over the estimated life of the reservoir (USBR 2006a).

c. There are three steps in calculating the specific weight of a sediment deposit:

(1) Determine the initial specific weight of each type of material (sand, silt, and clay).

(2) Calculate the compaction over time.

(3) Calculate the composite specific weight of the mixture of materials in the deposit.

J-2. Initial Specific Weight.

a. Lane and Koelzer (1943) correlated the specific weight of sand, silt, and clay with reservoir operation with results expressed as a table of coefficients. Lara and Pemberton (1963) updated that work based on the analysis of 1,300 samples. While retaining the basic concept of Lane and Koelzer, they modified the coefficients. Table J-1 shows that result.

b. High variability in deposits requires field measurements at each site, but the values in Table J-1 are satisfactory for planning purposes. The initial weight, W, for sand, silt, and clay is shown in columns W_s, W_{sl}, and W_{cl}. Columns K_s, K_{sl}, and K_{cl} are the consolidation coefficients associated with the relevant size fractions.

Table J-1
Constants for Estimating Specific Weight of Reservoir Sediment Deposits (adapted from Lara and Pemberton 1963)

Type	Reservoir Operation	Sand*				Silt*				Clay*			
		W_s		K_s		W_{sl}		K_{sl}		W_{cl}		K_{cl}	
		lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3	lbs/ft^3	kg/m^3
1.	Sediment always submerged	97	1,554	0	0	70	1,120	5.7	91	26	416	16	256
2.	Normally moderate to considerable drawdown	97	1,554	0	0	71	1,140	1.8	29	35	561	8.4	135
3.	Reservoir normally empty	97	1,554	0	0	72	1,150	0	0	40	641	0	0
4.	River bed sediments	97	1,554	0	0	73	1,170	0	0	60	961	0	0
*Based on the AGU classification scale where:		0.62 mm < Sand < 2 mm				0.004 mm < Silt < 0.62 mm				0.002 mm < Clay < 0.004 mm			

J-3. Consolidation of Deposits with Time.

a. Two cases are important for consolidation: (1) the consolidated specific weight at the end of a specified time; and (2) the average consolidated specific weight during that period.

b. Equation J-1 yields the consolidated specific weight at the end of a specified time:

$$W = W_i + K \cdot \log(T) \qquad \text{Equation J-1}$$

where:

K = consolidation coefficient (see columns K_s, K_{sl}, and K_{cl} in Table J-1)
T = age of deposit in years
W_i = initial specific weight of deposited material
W = specific weight at time T

c. Assuming continuous uniform settling during a period of years, the integrated equation developed by Miller (1953) can be used to obtain the time-averaged consolidated dry specific weight during a period of time, T:

$$W(T) = W_i + K \cdot \left[\left(\frac{T}{T-1}\right) \cdot \log(T) - 0.4343\right] \qquad \text{Equation J-2}$$

where:

$W(T)$ = average unit weight over T years of operation
W_i = initial specific weight of deposited material
K = the consolidation coefficient from Table J-1

J-4. Variation by Particle Size. As shown in Table J-1, the initial dry specific weight of a deposit varies by particle size of the deposit. Julien (2010) presented Figure J-1, using data from Wu and Wang (2006) that relates median particle diameter to specific weight.

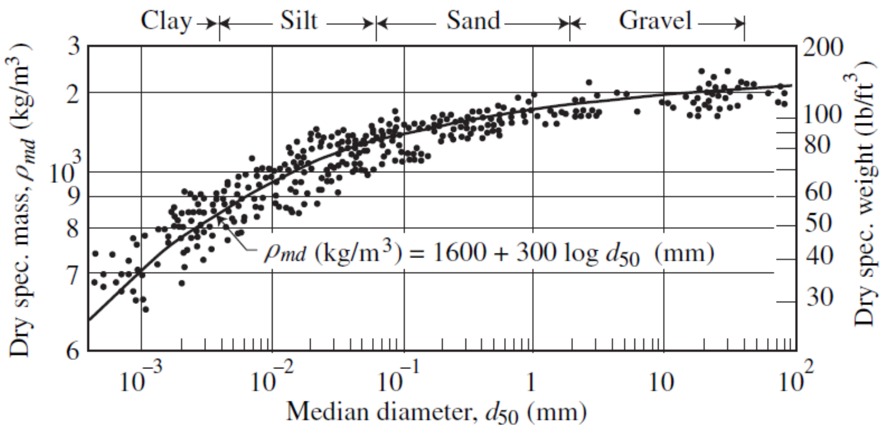


Figure J-1. Dry specific mass of sediment deposits vs. median diameter (Julien 2010; modified after Wu and Wang 2006)

J-5. Composite Specific Weight of a Mixture. The composite specific weight of a mixture of deposited sediments is estimated by:

$$W_C = \frac{1}{\left[\left(\frac{P_S}{W_S}\right)+\left(\frac{P_{SL}}{W_{SL}}\right)+\left(\frac{P_{CL}}{W_{CL}}\right)\right]} \qquad \text{Equation J-3}$$

where:

P_s = percent sand in mixture expressed as decimal
P_{sl} = similar quantity for silt
P_{cl} = similar quantity for clay
W_c = composite specific weight of mixture
W_s, W_{sl}, and W_{cl} = initial weights of sand, silt, and clay, respectively as given in Table J-1. Equivalently, the average specific weight can be calculated by multiplying the specific weight of each sample by the volume it represents, summing the values, and dividing the results by the total volume.

a. Example 1:

Determine the composite, initial specific weight for the following deposit:
Reservoir operation = Type 2
Inflowing sediment size analysis: 25% clay, 40% silt, 35% sand.
By Equation J-3:

$$W_C = \frac{1}{\left[\left(\frac{0.25}{35}\right)+\left(\frac{0.40}{71}\right)+\left(\frac{0.35}{97}\right)\right]}$$

$$= \frac{1}{[0.0071 + 0.0056 + 0.0036]}$$

$$= 61\,\frac{lbs}{ft^3}$$

b. Example 2:

Determine the average consolidated specific weight of the deposit during 50 years of operation using data given in Example 1.

Calculate the average specific weight for each class of material using Miller's equation (Equation J-2).

Sand: $W_s(50) = 97+ 0[(50/49)*\log(50) – 0.4343] = 97$

Silt: $W_{sl}(50) = 71 + 1.8[(50/49)*\log(50) – 0.4343] = 73$

Clay: $W_{cl}(50) = 35 + 8.4*[(50/49)*\log(50) – 0.4343] = 46$

Calculate the composite specific weight of the mixture using Equation J-3.

$$W_C = \frac{1}{\left[\left(\frac{0.25}{46}\right)+\left(\frac{0.40}{73}\right)+\left(\frac{0.35}{97}\right)\right]}$$

$$= 69\,\frac{lbs}{ft^3}$$

J-6. Measurement of Specific Weight of Deposits. In situ measurement is the preferred method of determining specific weight of deposits. Sediment samplers such as gravity core samplers, piston core samplers, and spud rod samplers can be used to take undisturbed samples of sediments. Alternately, measurements can be made with nuclear devices using gamma rays either with direct transmission or backscatter techniques.

Appendix K
Trap Efficiency of Reservoirs

K-1. Introduction. Determining the trap efficiency (TE) of a reservoir allows designers and managers to plan the daily operation of the reservoir and to estimate when dredging, sluicing, or other methods of sediment bypass might become necessary. Refer to paragraph 8-4b for additional discussion regarding the use of TE.

a. The TE of a reservoir is defined as the percentage of the total inflowing sediment that is retained in the reservoir in a given increment of time.

$$\text{TE} = \frac{[S_{inflow} - S_{outflow}]}{S_{inflow}} \qquad \text{Equation K-1}$$

where:

TE = trap efficiency expressed as decimal
S = sediment load in weight units for a given period of time

b. TE is of particular importance when determining the annual sedimentation rate or capacity loss as expressed by the equation:

$$C_{sa} = \text{TE}\left(\frac{S_a}{C}\right) \qquad \text{Equation K-2}$$

where:

C_{sa} = annual sedimentation rate, in percent
TE = trap efficiency, in percent
S_a = annual net sediment yield from the drainage area
C = original reservoir storage capacity in same units as S_a

c. Large reservoirs have higher trapping efficiency than smaller reservoirs. As sediment is trapped, the reservoir storage capacity is decreased and in turn, the TE decreases. For practical purposes in large reservoirs, the initial TE can be used as a constant up to 50% storage depletion. However, if storage depletion is rapid, the TE should be updated at time increments with an adjustment of C to reflect the sediment retained.

d. USBR (2006a) recommends that when the anticipated sediment accumulation is larger than 10 percent of the reservoir capacity, the TE be analyzed for incremental periods of the reservoir life. If the varying TE during reservoir life is critical to USACE reservoir operations, a downstream flood risk management project, or similar, then performing refined computations where the TE is updated in small increments of storage loss is usually warranted.

K-2. Factors Affecting Trap Efficiency.

a. Factors influencing TE are the characteristics of the reservoir and the inflowing sediment, as shown in Figure K-1.

b. Reservoir characteristics are of prime importance in calculating the retention time, the time it takes water to pass from the inlet of a reservoir to its outlet. Retention time can be influenced by the amount and character of inflow, reservoir shape, outlet configurations and reservoir operations. Longer retention times equate to lower average velocity and greater sediment deposition.

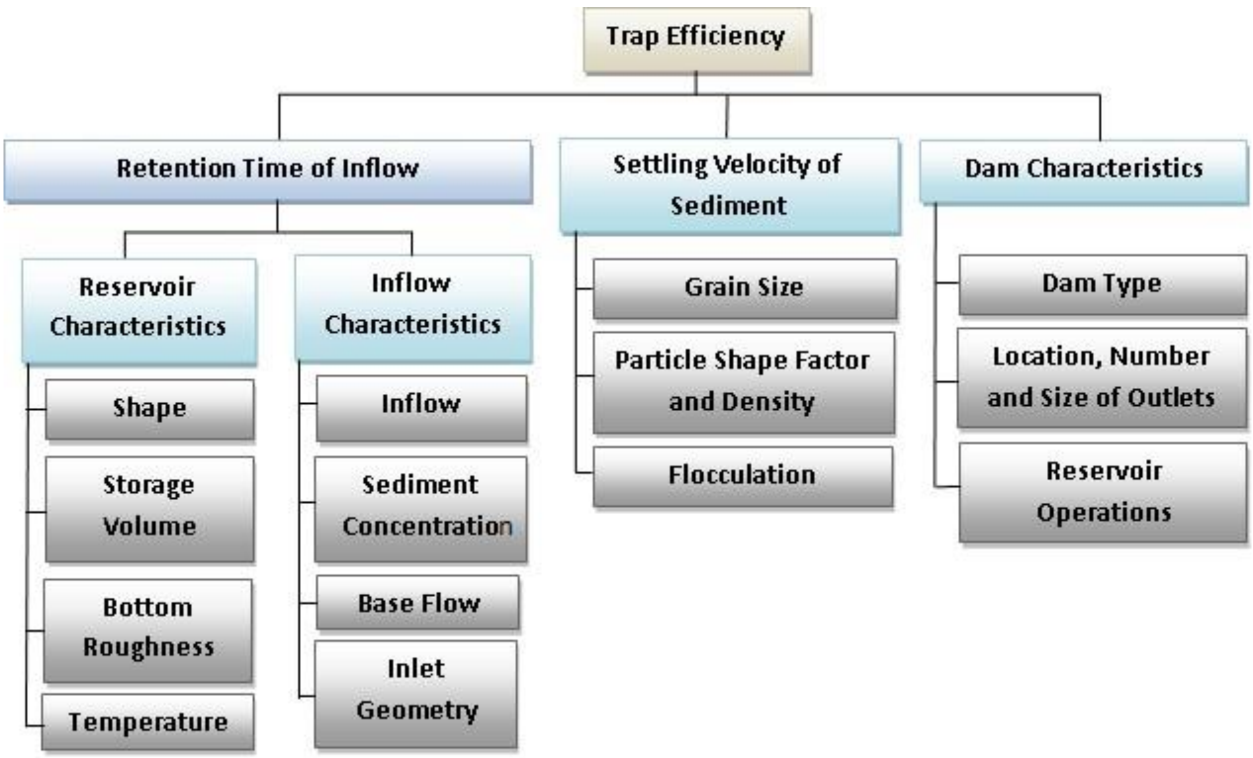


Figure K-1. Conceptual diagram of the variables and considerations that affect reservoir trap efficiency

c. An approximate measure of retention time for a reservoir is the capacity-inflow ratio, computed by dividing the reservoir capacity by the inflow rate of water. This approximation assumes that inflowing water (and sediment) utilizes the full capacity of the reservoir (for example, that there is no "short circuiting"), and inflows are uniformly mixed with the water already present.

$$R = \frac{C}{I} \qquad \text{Equation K-3}$$

where:

R = retention time
C = reservoir storage capacity (volume)
I = inflow (volume per unit time)

d. The shape of the reservoir plays a large role in determining the effective retention time. A short or sinuous reservoir, and resulting ineffective flow areas, can cause “short circuiting” in which the effective time becomes much less than the retention time (Figure K-2). Small reservoirs in panels (a) and (b) have the same storage capacity, but the reservoir in panel (b) may have a shorter retention time at the same inflow because of the shorter distance between inlet and outlet and potentially larger ineffective flow areas to the right and left of the main flow. Reservoir geometry should be reviewed to verify the underlying assumptions of the effective retention time and uniform mixing within the reservoir.

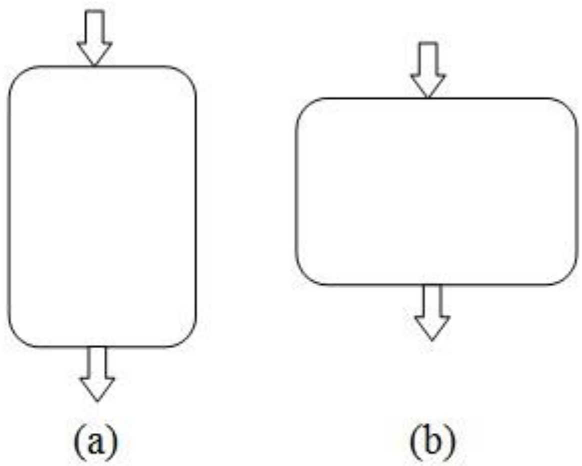


Figure K-2. Reservoir shape schematic

e. The outlet type, size, and location affect retention time. Placement of bottom outlets, particularly if they are opened to pass density currents out of a reservoir, can reduce TE for fine sediments. Outlet releases compared to reservoir inflow rates directly correlates to retention time. Many USACE projects have complex intake towers with low level intakes for normal flows augmented by inlets/weir structures at higher levels. Examination of all reservoir release structures is required.

f. Dam operations also affect TE. For example, lowering of the pool elevation decreases retention time, which subsequently decreases TE. This can be effective for passing sediments through a smaller reservoir if done during periods of high flow with high sediment concentration. The sluicing of sediments by reservoir operations may be limited by storage requirements and environmental requirements.

g. Sediment characteristics affecting TE are: (1) particle size distribution of the inflowing sediment load, (2) particle shape and density, and (3) the behavior (particularly flocculation) of fine sediments under varying temperatures, concentration, water chemical composition, secondary currents, and turbulence. Grain size distribution and particle shape determine particle fall velocities, and in conjunction with water depth and detention time, determine the percentage of the sediment that deposits or remains in suspension. Coarser material has a higher settling velocity, and less time is required for it to be deposited. Fine sediments (clay and silt sizes) are usually the only sediments that remain in suspension long enough to reach the outlets. Temperature, sediment concentration, and water chemical composition affect the aggregation properties of fine sediments, which determine the flocculation, settling, and resuspension of deposited sediments, and whether they are transported to the dam.

K-3. Use of Empirical Trap Efficiency Methods.

a. Trap efficiency can be most accurately computed by measuring sediment inflows and outflows. Moreover, the information from such a data collection effort (particularly if including both grain size and concentration) could prove invaluable for future modeling efforts. However, direct measurement may be cost-prohibitive or too time-intensive for initial estimates of trapping efficiency.

b. Modeling of sediment dynamics in a reservoir can also yield the quantities of interest for calculating TE. In practice, however, project needs, timelines, and budget constraints may not justify detailed modeling. In this case, several empirical methods are available to estimate reservoir TE.

c. Heinemann (1984) gave an overview of the many empirical models that could be used for predicting TE. An overview of the theoretically based TE models is provided by Haan et al. (1994). Verstraeten and Poesen (2000) provided an overview of the different methods available to estimate the trap efficiency of reservoirs and ponds. Common methods are presented below.

d. Capacity-Watershed Method (Brown's Curve). Brown (1943, 1950) developed a curve relating the ratio of reservoir capacity (C, in acre-feet) and watershed area (W, in square miles) to TE (E, in percent).

(1) When expressed in a general relationship, the curves presented by Brown have a coefficient which varies from 0.046 to 1.0, with a recommended design coefficient of 0.1. Brown (1943) suggested that values are close to 1 (such as high TE) for reservoirs in regions with smaller and more variable runoff and for those that store flood flows. The coefficient increases with retention time, average grain size, and for reservoir operations that limit sediment release through sluicing or reduce sediment movement toward the outlet due to pool regulation.

(2) Brune (1953) stated that reservoirs with the same C/W ratio could have completely different TEs if their catchments produced different runoff volumes due to other hydrological characteristics. This explains why there is a very high range in TE at lower C/W ratios. Brown's

curve is useful if the watershed area and reservoir capacity are the only parameters known (Figure K-3).

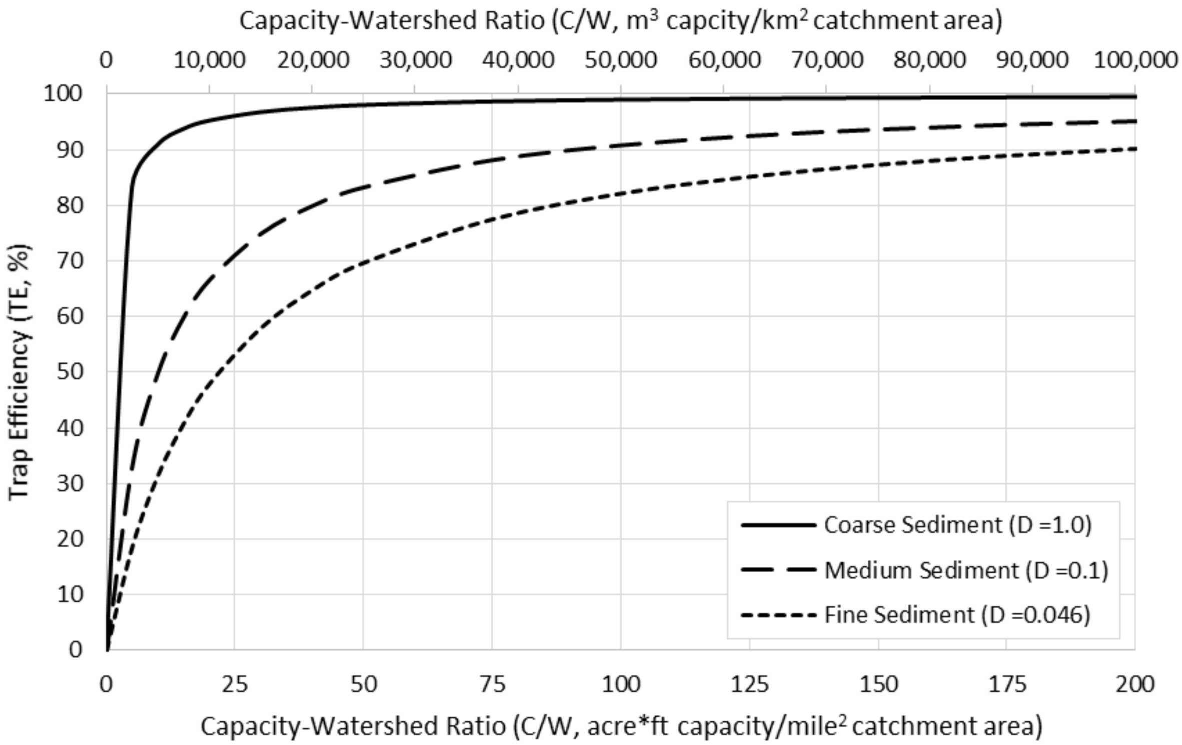


Figure K-3. Trap efficiency curve redrawn from Brown (1943) and Verstraeten and Poesen (2000)

e. Capacity-Inflow Method (Brune's Curve).

(1) Brune (1953) developed an empirical relationship for estimating the long-term reservoir trap efficiency based on the capacity-inflow ratio. He used data from 37 reservoirs with drainage areas ranging from 1.5 to 184,600 sq miles located primarily in the South, Southwest, and Midwest. The data includes 38 data points from 33 normal ponded reservoirs (some reservoirs had measurements over multiple time frames), two desilting basins, and two semi-dry reservoirs.

(2) Since the curves shown in Figure K-4 were generated by the use of data primarily from normally ponded reservoirs, they are not recommended for use in determining trap efficiencies of de-silting basins or dry reservoirs. Modified curves are presented by Verstraeten and Poesen (2000) that also include data for desilting basins and semi-dry reservoirs that illustrates wide variance from ponded. Heineman (1981) modified Brune's relationship using data from 20 normally ponded small agricultural reservoirs. Heineman (1981) concluded that his curved predicted a lower TE than the one of Brune (1953).

(3) Dendy (1974) added more data to Brune's curve and developed a prediction equation for the median curve:

$$TE = 100\left(0.97^{0.19^{\log\left(C/I\right)}}\right) \qquad \text{Equation K-4}$$

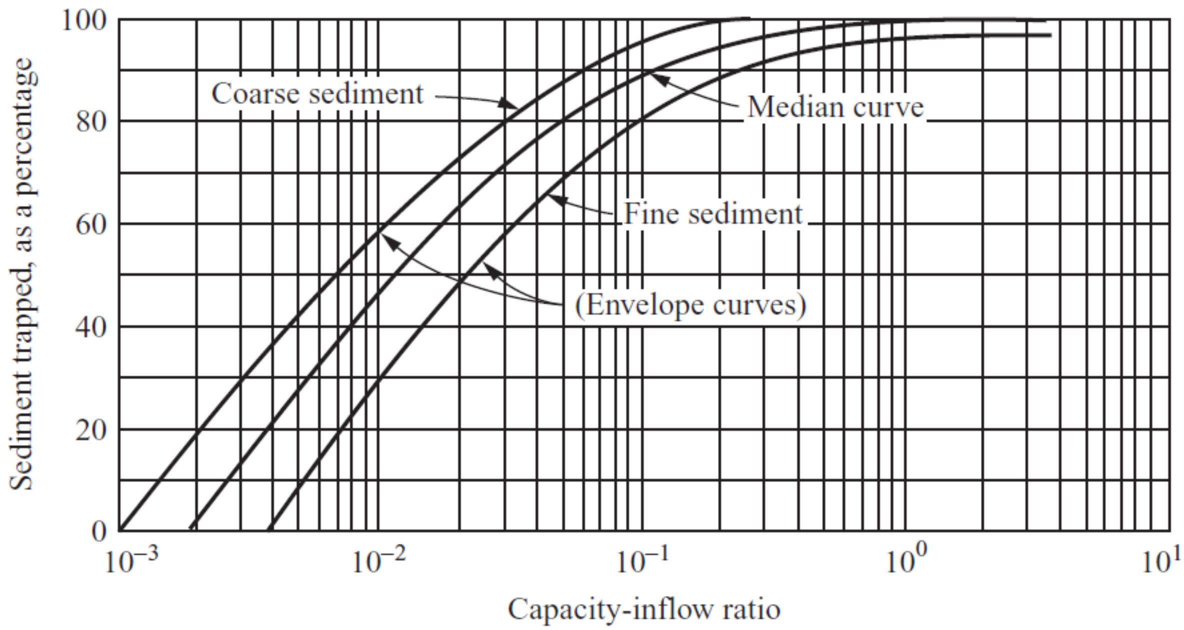


Figure K-4. Brune's (1953) trap efficiency curve (Vanoni 1975, 2006)

(1) Greimann (2015) created equations for all three curves:

$$TE = c\left(1 - 2e^{-dV_*^{0.35}}\right) \qquad \text{Equation K-5}$$

where V* = the capacity-inflow ratio, (reservoir volume/annual volume of inflow) and the coefficients *c* and *d* are taken from Table K-1 below.

Table K-1. Coefficients for Equation K-5

	Brune Curve		
Constant	**Low**	**Medium**	**High**
c	95	97	100
d	5.37	6.42	7.71

f. Sediment Index Method (Churchill's Curve).

(1) Churchill (1948) presented a relationship between the percent of incoming sediment passing through a reservoir and the sedimentation index (SI) of the reservoir. The relationship,

shown in Figure K-5, was developed using 25 data points from five Tennessee Valley Authority reservoirs. The “local silt” curve was developed from three reservoirs, and the “upstream reservoir” curve was developed from only two reservoirs.

(2) The SI of a reservoir is the period of retention divided by the mean velocity of flow in the reservoir. If the retention time or mean velocity cannot be obtained from field data, an approximation can be made by assuming the effective retention time to be equal to the retention time as computed by using the C/I ratio. This can be written mathematically as a series of relations:

$$\mathrm{SI} = R/V \qquad \text{Equation K-6}$$

$$\mathrm{R} = C/I \qquad \text{Equation K-7}$$

$$\mathrm{V} = I/A \qquad \text{Equation K-8}$$

$$\mathrm{A} = C/L \qquad \text{Equation K-9}$$

$$\mathrm{SI} = C\,A/I^2 = (C/I^2)\,(C/L) = (C/I)^2/L \qquad \text{Equation K-10}$$

where:

R = time of retention in seconds
C = capacity, in cubic feet, of the reservoir at the mean operating pool elevation
I = average daily inflow rate in cubic feet per second
V = mean velocity in feet per second
A = average cross-sectional area in square feet
L = reservoir length in feet at the mean operating pool elevation

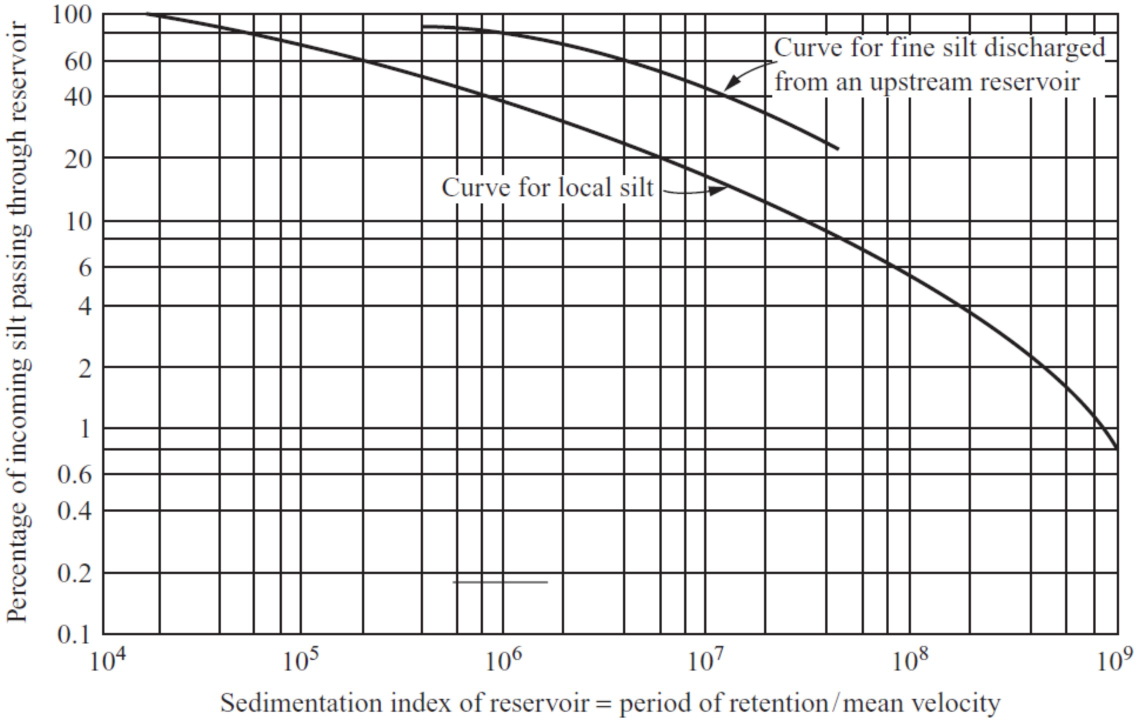


Figure K-5. Churchill (1948) passing efficiency curve (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

(3) Note that in the Churchill curves, the "average velocity" is the average in the entire reservoir over the complete hydrograph. Note also that Churchill's relationship has "percentage of incoming silt passing through reservoir" on the ordinate, which necessitates determining the difference between the value obtained and 100% to get the TE. The term "silt" on the ordinate axis means both silts and clays.

K-4. Evaluation of Empirical Methods.

a. Historically, the two most common approaches are the relationships developed by Brune (1953) and Churchill (1948). The empirical relationships for both methods are based on measured TEs from normally ponded reservoirs and specify the use of average annual inflow that experience relatively regular flows. These relationships may not be applicable in climate zones with widely variable inflows such as experienced in the tropics.

b. If the annual inflow rate is known, Brune's curves are generally considered to be widely applicable. Churchill's method requires the additional information of average velocity and reservoir length. Neither method explicitly accounts for sediment characteristics; therefore, judgment must be exercised in using these methods. A comparison of the Brune and Churchill methods was presented by USBR (2006a) as shown in Figure K-6.

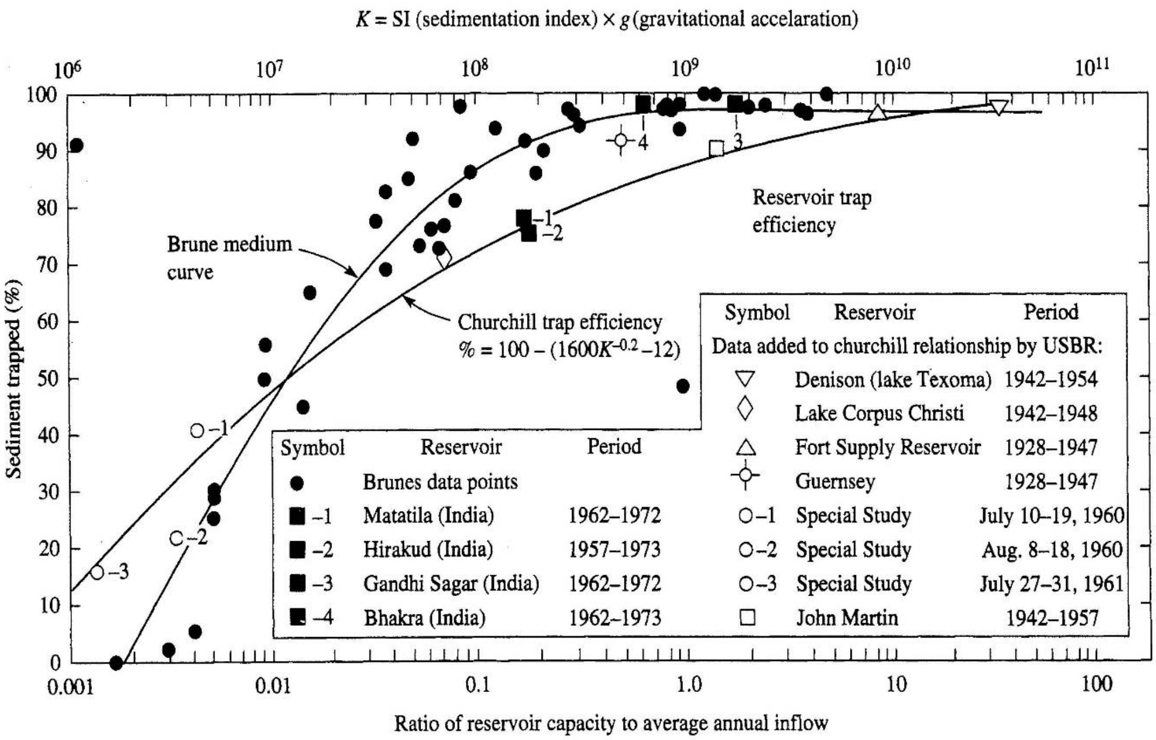


Figure K-6. Comparison of trap efficiency curves by Churchill (1948) and Brune (1953), from USBR (2006a)

c. Trimble and Carey (1990) compared the Churchill curves (1948) and the Brune curves (1953) for 27 reservoirs in the Tennessee River Basin. They concluded that, for a system of reservoirs, the Churchill method, which accounts for sediment received from an upstream reservoir, provides a more realistic estimate of sediment yields than the Brune method.

d. The popularity of the Brune curves over the Churchill curves may be due to the ease of obtaining the input data (Verstraeten and Poesen 2000). Moreover, the Brune curves include data from more diverse geographic regions.

e. A general guideline is to use the Brune method for large storage or normal ponded reservoirs and the Churchill curve for settling basins, small reservoirs, flood retarding structures, semi-dry reservoirs, or reservoirs that are continuously sluiced (USBR 2006a).

f. Empirical methods should be applied with caution and maybe suitable only during the concept study phase. Hydrology, sediment, and reservoir characteristics may have a significant effect on the depositional environment of the reservoir. In order to perform reasonably accurate USACE project design and alternative comparison, it may be more prudent to use physical measurements and physically based numerical modeling to estimate TE.

K-5. Example Application of Empirical Methods.

a. Table K-2 lists data for the example application.

Table K-2
J. Percy Priest Reservoir (Stones River, Tennessee)

	Value	**Units**
Capacity (summer power and recreation pool)	392,000	Acre-feet
Inflow rate	1,070,400	Acre-feet/year
Length	41.8 (220,700)	Miles (Feet)

(1) Brown's method:

K = 0.1

C/W = 392,000/892

= 439

$$TE = 100\left[1 - \frac{1}{1 + 0.1(439.5)}\right]$$

= 100 [1 − 0.022]

= 97.8%

(2) Brune's method (assume median curve):

C/I = 392,000/1,070,400

= 0.366

$$TE = 100\left(0.97^{0.19^{\log(0.366)}}\right)$$

$$TE = 100(0.97^{2.066})$$

= 93.9%

(3) Churchill's method:

C = 392,000 ac-ft x 43,560 ft^2/acre

= 1.71 x 10^{10} ft^3

I = 1,070,400 ac-ft/year x 43,560 ft^2/acre

= 4.66 x 10^{10} ft^3/year

$$= 4.66 \text{ x } 10^{10} \text{ ft}^3\text{/year x 1 year/3.1536 x } 10^7 \text{ seconds}$$

$$= 1{,}480 \text{ ft}^3\text{/sec}$$

$$C/I = (1.71 \text{ x } 10^{10} \text{ ft}^3) / (1{,}480 \text{ ft}^3\text{/sec})$$

$$= 1.16 \text{ x } 10^7 \text{ seconds}$$

$$SI = (C/I)^2/L$$

$$= (1.16 \text{ x } 10^7 \text{ seconds})^2 / 220{,}700 \text{ ft}$$

$$= 6.10 \text{ x } 10^8 \text{ (s}^2\text{/ft)}$$

From Figure K-6:

Percent of silt passing = 1.6%

TE = 100 – 1.6 = 98.4%

or from the equation shown in Figure K-6:

$$\text{Percent of silt passing} = 800(SI)^{-0.2} – 12.0$$

$$= 800(6.044 \text{ x } 108)^{-0.2} – 12.0$$

$$= 2.0\%$$

TE = 100 – 2.0 = 98.0%

b. In practice, TE could then be given as a range or, depending on whether the reservoir fits well with the reservoirs used to develop a particular curve, more weight can be given to the outcome of that method.

Appendix L
Analytical Methods for Estimating the Distribution of Sediment Deposits in Reservoirs

L-1. Introduction.

a. With the advance in numerical modeling, the methods illustrated in this appendix are no longer suitable for most USACE studies that require detailed evaluation of reservoir sediment deposition and associated impacts. Furthermore, sufficient time has elapsed since dam closure for most USACE reservoirs such that depositional patterns can be measured rather than estimated.

b. However, many historic USACE studies were performed using these historic procedures. The purpose of this appendix is to provide technical background documentation for historic USACE studies and to serve as an aid for understanding legacy computations when necessary. Refer to the current EM 1110-2-4000 chapter on Reservoir Sedimentation for a discussion of current methods.

c. Historically, agencies used several analytical relationships to compute the longitudinal pattern of sediment deposition. These methods fit curves to reflect future depositional patterns based on sediment yield rates, sediment properties, reservoir geometry and, occasionally, operational parameters. The area reduction method was the most popular method, along with several other methods. With the addition of a few minor supplements, this appendix consists of the entire Appendix H of the 1989 version of EM 1110-2-4000.

L-2. Factors Affecting the Distribution of Deposits.

a. The factors Hobbs (1969) considered to be the most significant in reservoir deposition problems are:

(1) Reservoir size and shape.

(2) Sediment quantities and characteristics.

(3) Sediment sources.

(4) Progressive vegetative growth on frequently exposed deposits.

(5) Consolidation of deposits.

(6) Magnitudes, frequency, and sequences of hydrologic events.

(7) Reservoir regulation practices.

b. Hobbs (1969) stated that "These factors and other influences interact in ever changing combinations to produce the distribution of deposits at any given time." Modern, computer-based, numerical models allow the engineer to simulate those complex interactions, but in practice, simple, empirical methods are always useful as the first approximation for studying a problem. Such methods have the advantage of simplicity at the sacrifice of consideration for the unique interactions that govern specific problems. Consequently, if followed implicitly, these methods can produce misleading results.

L-3. Summary Update of Current Analytical Methods.

a. This section presents a summary update of current and previous analytical methods to provide additional context to the historic EM 1110-2-4000 content.

b. In the absence of sufficient data for a site-specific analysis or as a first approximation, the following empirically derived methods are available for predicting depositional geometry. These methods derive from observation that sedimentation patterns differ based on:

(1) Geometry and geomorphology of the reservoir.

(2) Reservoir operations.

(3) Sediment size.

(4) Incoming sediment load.

c. The variation in sediment patterns is seen in Figure L-1 where percent of the sediment deposition is plotted against the relative depth in the reservoir where the sediment is deposited. This figure highlights differences such as between lake-type reservoirs where sediments accumulate at higher elevations and gorge-type reservoirs where sediments accumulate in deeper water.

d. Borland and Miller (1960) and Lara (1962) used this variation in depth vs. deposition behavior in what is called the Empirical Area Reduction method (see the historic EM 1110-2-4000, paragraph M-4.a.(4)).

(1) In the Empirical Area Reduction method, an empirical curve is selected using the reservoir type from sedimentation data for USA reservoirs presented in Table L-1. Selection is based on reservoir type and operation, usually applied with equal weighting between classes. For cases where there is a choice of type, the grain size of sediments may be used as a weighting factor or engineering judgment may be used to evaluate the relative importance of reservoir shape vs. operation.

(2) After determining the inflowing sediment load and the height of sediment accumulation at the dam, the empirical curve is used to distribute sediment throughout the reservoir. The dashed line in Figure L-2 represents an early method for distributing sediments, the area-increment method, that assumes that an equal volume of sediment will be deposited at each depth within a reservoir. The area-increment method straight line curve is very similar to the Type II (primarily floodplain/foothills) curve of the area reduction method.

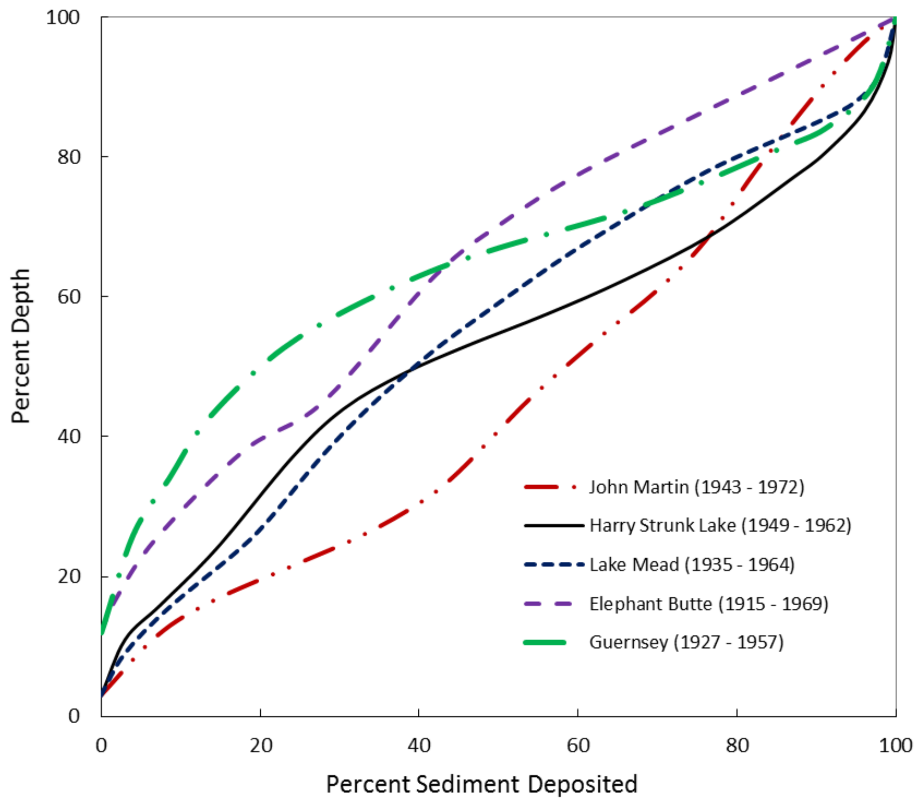


Figure L-1. Sediment deposition profiles of several reservoirs (redrawn from Lara (1962); Strand and Pemberton 1987)

Table L-1
Determination of Empirical Area Curve Using Reservoir Operations, Shape and Grain Size (redrawn from Lara (1962); Strand and Pemberton 1987)

Reservoir Operation		**Reservoir Shape**		**Type**
Class	**Type**	**Class**	**Type**	**Weighted Type**
Sediment Submerged	I	Lake Floodplain/Foothills Hill and Gorge	I II III	I I or II II
Moderate Drawdown	II	Lake Floodplain/Foothills Hill and Gorge	I II III	I or II II II or III
Considerable Drawdown	III	Lake Floodplain/Foothills Hill and Gorge	I II III	II II or III III
Normally Empty	IV	All Shapes		IV
Additional Criterion for grain size:			**Size**	**Type**
			Sand or coarser	I
			Silt	II
			Clay	III

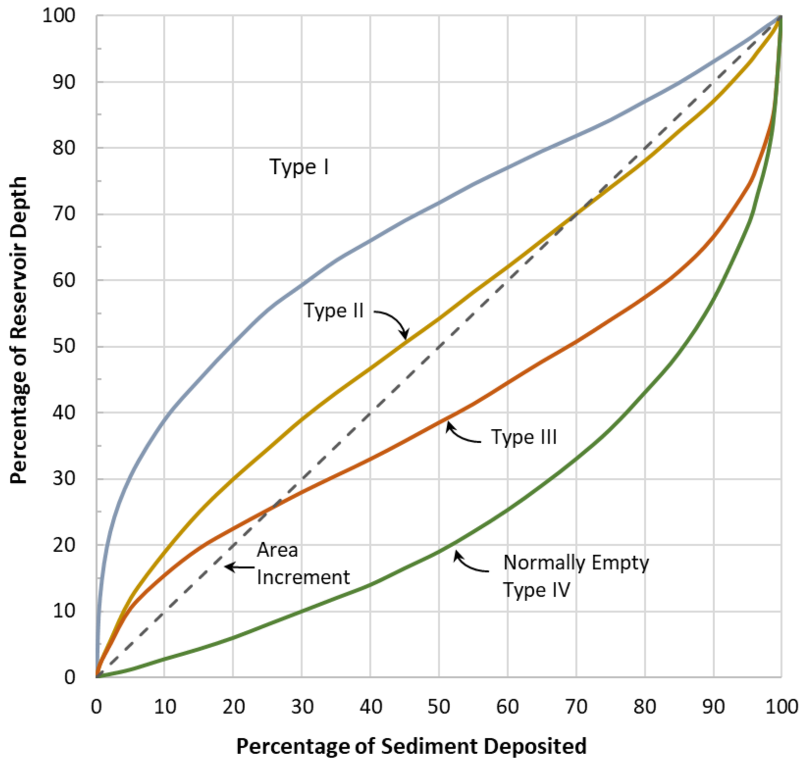


Figure L-2. Empirical curves for use with empirical area reduction method (redrawn from Lara (1962); Strand and Pemberton 1987)

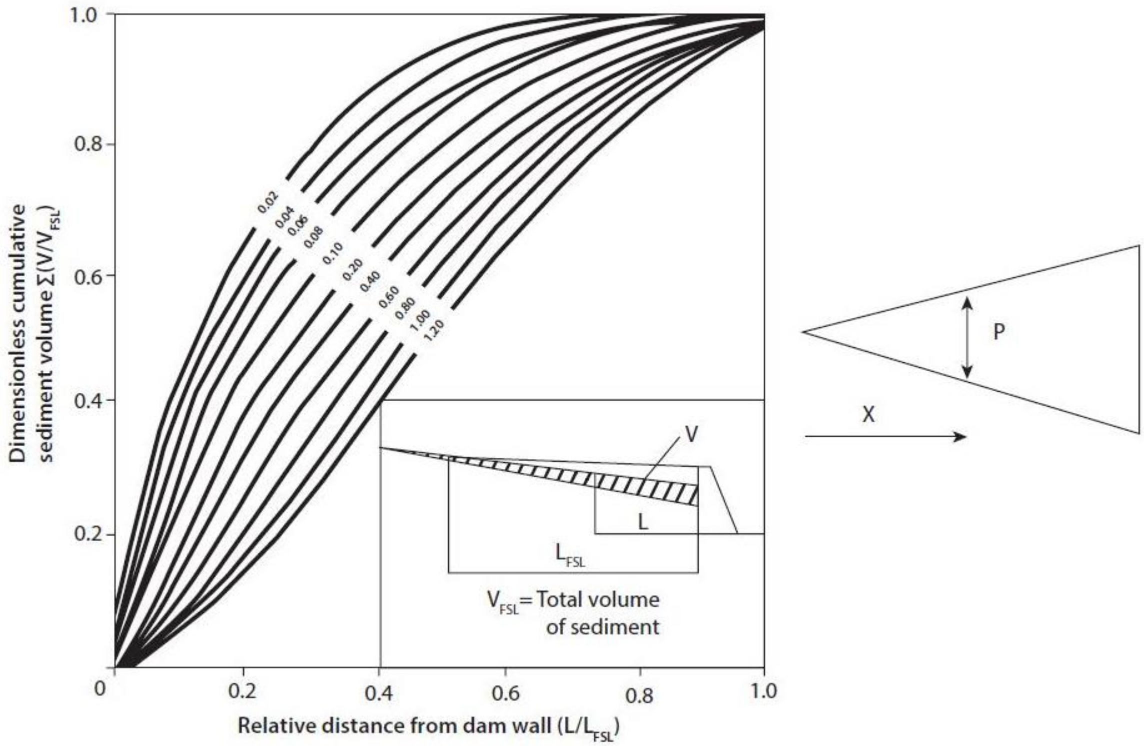
e. Annandale (1987) applied Yang's concept of stream power to predict sedimentation using data from 11 reservoirs in South Africa. Annandale's method builds on the theory that, for natural rivers and reservoirs, the concept of minimum stream power applies and can be used to compute the bed surface assuming constant shear velocity throughout the reservoir. Taking discharge as a conserved quantity, Annandale constructed a relation between the volume of sediment deposited at a given distance from the dam and the longitudinal gradient in wetted perimeter:

$$\sum \frac{V}{V_{FSL}} = f\left(\frac{L}{L_{FSL}}, \frac{dP}{dx}\right) \qquad \text{Equation L-1}$$

where:

- V = volume of deposited sediment at a given distance above the dam
- V_{FSL} = total volume of sediment deposited at full supply level (FSL) or maximum stage in the reservoir under normal operating conditions
- L = distance between cross section of interest and dam
- L_{FSL} = total length of reservoir at FSL
- P = wetted perimeter; dP/dx is the average or characteristic longitudinal gradient of wetted perimeter over the entire reservoir

f. Figure L-3 represents the sediment distribution below the FSL of the reservoir and Figure L-4 represents the sediment distribution above the FSL.



FSL = full supply level; L = variable length measured from dam; P = width of reservoir at variable distance x from upstream end of reservoir; V = volume of sediment between the dam and location L; L_{FSL} = total length of the reservoir at full supply level; V_{FSL} = total volume of sediment underneath the full supply level.

Figure L-3. Sediment distribution below FSL. Values between 0.02 and 1.20 represent the longitudinal gradient of wetted perimeter DpdP/dDx (Annandale et al., 2016)

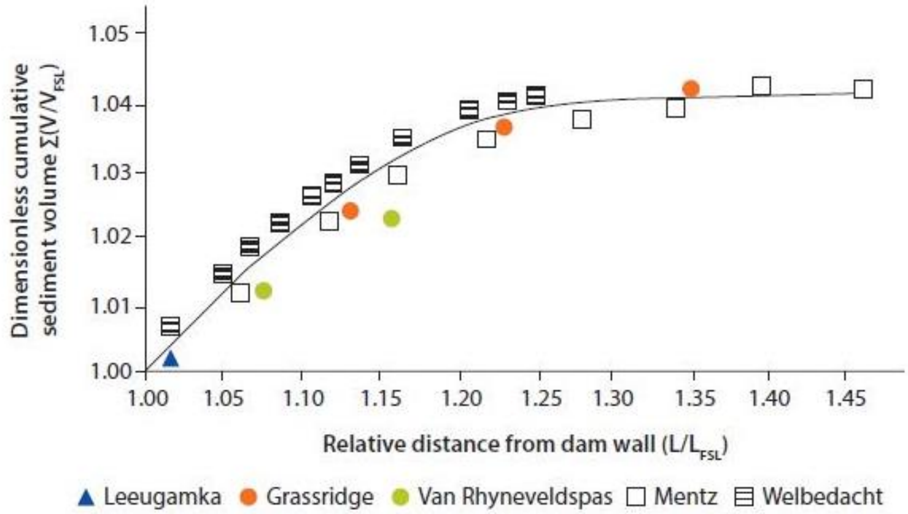


Figure L-4. Sediment distribution above FSL (Annandale et al., 2016)

g. Mohammadzadeh-Habili and Heidarpour (2010) proposed an empirical method for prediction of sediment distribution in reservoirs based on the original and sedimentation-reduced area-capacity data of 40 reservoirs in the United States. In their proposed method, sediment distribution in a reservoir is related to the sediment volume and original reservoir characteristics such as capacity, surface area, and the original depth capacity curve.

h. Rahmanian and Banihashemi (2012) refined the stream power approach of Annandale by subdividing the reservoir into different zones of wetted perimeter change and adding the influence of hydraulic radius or cross-sectional area into the calculation of sedimentation profiles.

L-4. Former EM 1110-2-4000 Choice of Methods. The remainder of this appendix consists of partial content from the historic EM 1110-2-4000 (1995, Appendix H).

a. Five empirical methods are presented. They are not all equally well suited for all projects. Therefore, where sediment deposition is expected to have a major effect on the design and operation of a reservoir project, it is prudent to use more than one method so that the variability in results from somewhat independent approaches can be used to allow for conservatism. Numerical sediment modeling, which was developed after these empirical methods, is the best approach because it calculates sedimentation, including the redistribution of deposits, based on hydraulics of flow and reservoir operation.

(1) Flood pool index method.

(2) Delta profile method.

(3) Area-increment method.

(4) Empirical area reduction method.

(5) Pool elevation duration method.

b. All depend on the same basic requirements for estimates of total sediment loads, average trap efficiencies, and gross volumes of sediment trapped during the period under consideration. None delineate developments at individual tributaries.

c. Note that only the volume of sediment trapped in the reservoir is to be distributed. This is of particular importance because if TE is low, and if the incoming sediment volume is used instead of the volume trapped, the predicted distribution would be overestimating the actual conditions.

d. Since sediment discharge is measured in units of weight, a conversion must be made to units of volume to be distributed. This conversion must consider the consolidation of the deposited sediment over time.

e. Methods other than those presented have been developed for prediction of sediment distribution. These include trigonometric, volume reduction, trial and error, USBR manual design curve, and Van't Hul methods. Most of these methods were superseded by progressively more accurate methods. The Van't Hul method was modified and eventually became the Empirical Area Reduction method, and along with the Area-Increment method, are the most widely used of all the analytical methods.

L-5. Flood Pool Index Method. This method divides deposits between those in the flood control pool and those below it. Figure L-5 shows the relationship between percent of time the reservoir operated in the flood control pool and the total sediment trapped in it. To use this method, calculate the flow pool index, read the percent trapped in the flood control pool from Figure L-5, and multiply that value by the total volume trapped.Delta Profile Method.

a. USBR (1987) proposed a procedure to predict the delta profile based on delta deposition patterns of resurveyed reservoirs. Figure L-6 shows a typical reservoir delta with the topset, foreset slope, and bottomset labeled.

b. To use this method, compute the topset slope using the Meyer-Peter, Muller Formula for beginning transport or the Schoklitsch equation for zero bed toad transport. The anticipated value is one-half the original channel slope, but that is a rule of thumb based on field observations at reservoirs and not a theoretical conclusion about reservoir delta deposits. In reservoirs where inflowing sediment concentration is high and the percentage of coarse particles is large, the slope may become parallel to the valley slope.

c. The intersection of the topset and foreset slopes forms a pivot point that can be a location of normal pool elevation. Historically, the extreme upstream limit of the delta was considered to be at the intersection of the maximum pool elevation and the original channel bed. A line is drawn from this point to the pivot point elevation to produce the topset slope for the delta. Experience has shown, however, that due to backwater effects, coarse sediment deposition migrates upstream over time, inducing deposition upstream of the maximum pool elevation.

d. Observations have shown that foreset slopes average 6.5 times the topset slope. Draw a line from the pivot point to the reservoir bottom at a slope 6.5 times the topset slope. Assuming the sediment is distributed uniformly across the reservoir, cross sections can be modified to show delta elevations and the volume of deposited sediment can be calculated using the average end area-reach length method.

e. The volume should agree closely with the volume of inflowing sand and gravel for the time period analyzed. Small differences can be rectified by changing the topset slope while retaining the pivot point elevation. If differences are large, retain the topset and foreset slopes and move the pivot point along the pivot point elevation line.

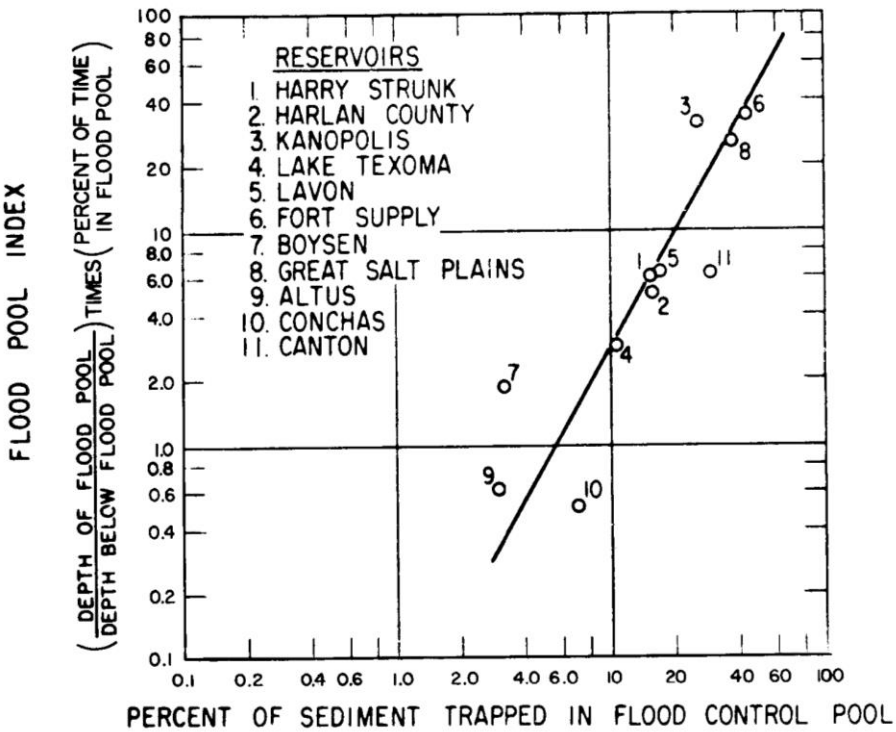


Figure L-5. Relationship between flood pool index and percent of total sediment trapped (Strand and Pemberton 1987)

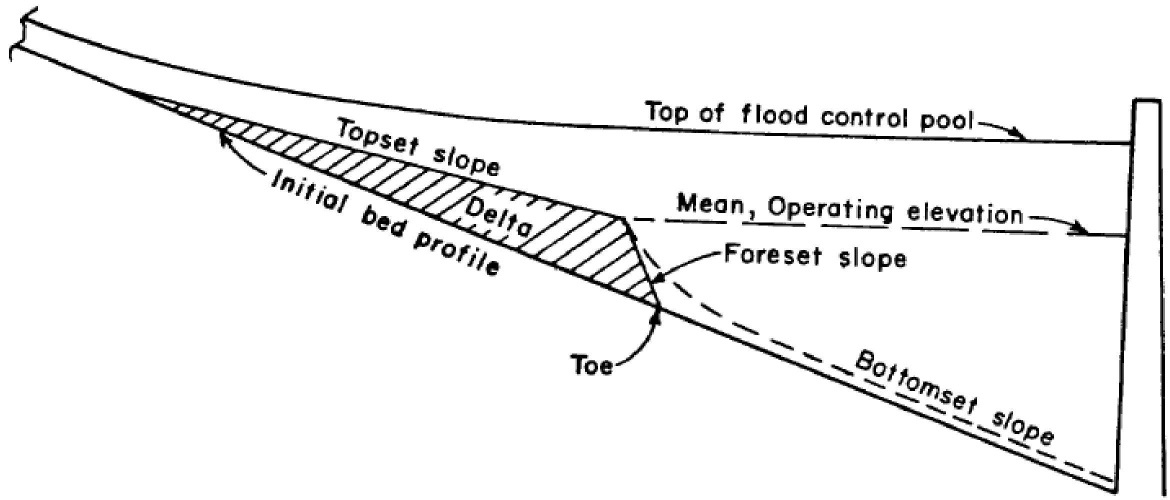


Figure L-6. Typical delta formation

L-7. Area-Increment Method.

a. USBR (2006a) developed the area-increment method, which assumes that the newly generated elevation-area curve, after sedimentation, is parallel to the original curve. This assumption is valid for most reservoirs if the storage depletion, as compared to the total capacity, is small. Significant errors can occur if there are large variations in reservoir pool elevations or if the inflowing sediment reservoir capacity ratio is large. A rule of thumb used by the USBR is to use this method only if the 100-year sediment accumulation is less than 15% of the total capacity.

b. Under extreme reservoir operation conditions or unusual reservoir shape, the empirical area reduction method should be used instead of the area-increment method.

c. Subject to the above qualifications, the area-increment method is considered satisfactory for determining storage loss in the conservation pool, however, both the area-increment method and the empirical area reduction method tend to over-predict the volume of deposits in the conservation pool.

d. The procedure is based on the following equation:

$$V_s = A_o \cdot (H - h_o) + V_o \qquad \text{Equation L-2}$$

where:

A_o = area correction factor that is the original reservoir area at the new zero elevation at the dam, in acres
V_o = sediment volume below the new zero elevation, in acre-feet
V_s = sediment volume to be distributed in the reservoir in acre-feet

H = reservoir depth at the dam-streambed to maximum normal water surface, in feet
h_o = depth to which the reservoir is completely filled with sediment-new zero elevation

e. This equation ensures that the incremental area adjustment at each elevation interval will produce the total capacity of the reservoir less the depletion from sediment accumulation. The procedure is not exact and requires trial and error to properly balance area and volume. Volume is calculated by the average end area or prismoidal formulas. If applied stringently, the area-increment method does not produce a smooth reduction in area from the original to the revised curve from the last few elevation increments to the maximum normal pool elevation. A correction could be made by placing a small amount of sediment above the maximum normal pool elevation and, starting at a few elevation intervals below the maximum normal pool elevation and, extending a few elevation intervals above the maximum normal pool elevation, making the area correction factor (A_o) progressively smaller for each increasing elevation interval such that the sediment volume (V_o) is conserved.

L-8. Empirical Area Reduction Method. Borland and Miller developed this method in 1958 for the USBR. Because it takes into consideration the shape of the reservoir more than the area-increment method, it is usually more accurate in predicting bed elevation change near the dam. Lara revised the original empirical area reduction method (Lara 1962) to include a correction for reservoir shape by classifying reservoirs according to Table L-2.

Table L-2
Reservoir Type Classification (redrawn from Lara 1962)

Reservoir Type	Classification	m
I	Lake	3.5–4.5
II	Flood plain foothill	2.5–3.5
III	Hill	1.5–2.5
IV	Gorge	1.0–1.5

a. Reservoir Type. Reservoir type is determined by plotting reservoir depth vs. reservoir capacity in Figure L-7. The plot is usually a straight line that indicates that the representative, reservoir cross section is similar to an inverted triangle.

b. Points of Caution.

(1) Some reservoirs have a shape that produce two straight lines. In those cases, careful examination should be made to determine where the volume change occurs with respect to normal operating pool elevation. For example, if the break is above the normal operating pool elevation, the lower line should be adopted. If the break is below that elevation, a combination of the two types should be considered.

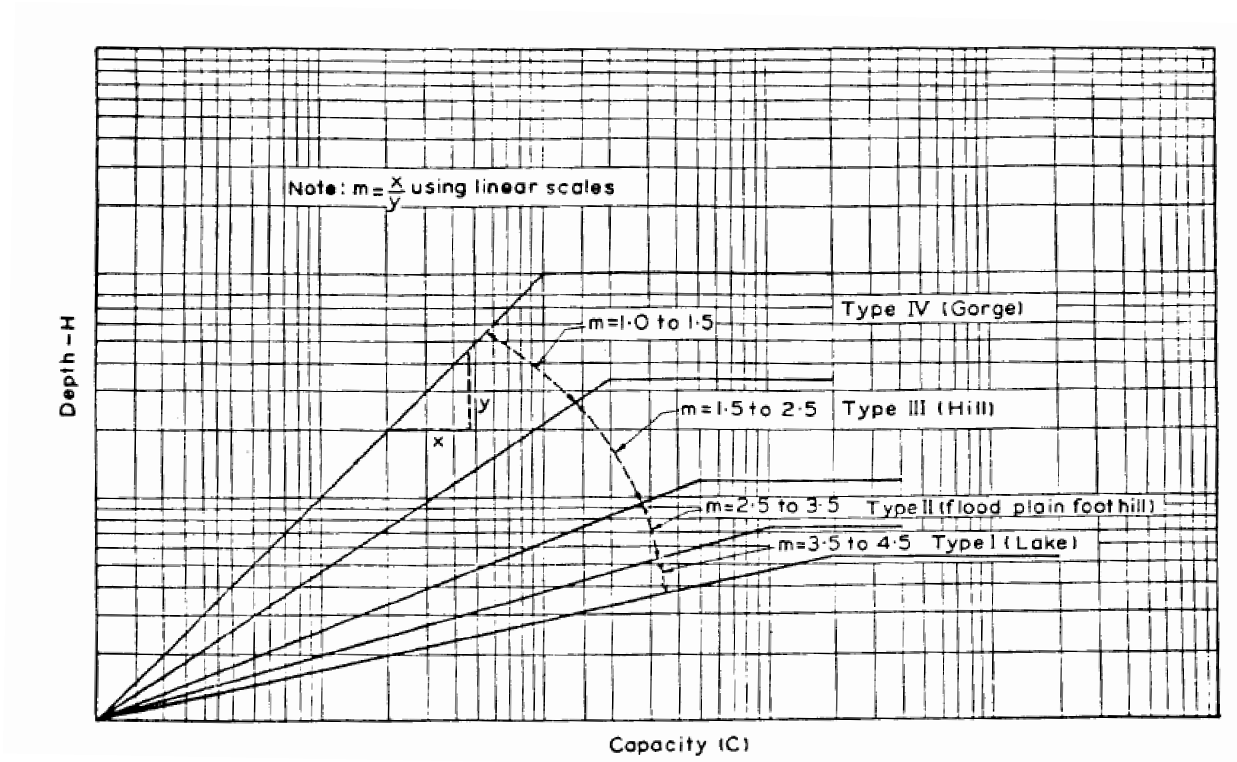


Figure L-7. Reservoir type relationship (adapted from Lara 1962)

(2) Extremities in reservoir operation and sediment characteristics should also be considered when classifying a reservoir. Although it may have a Type II classification based on the depth capacity relationship, an abnormally high percentage of clay in the inflowing sediment load could affect the movement of sediment such that a type III reservoir is more representative. A reservoir with an operation schedule that requires a substantial drawdown for long periods of time would have a higher classification number than that obtained by the depth capacity relationship. A low storage to water yield ratio tends to decrease the reservoir classification number because the resulting short detention time is similar to gorge-type reservoirs.

c. Design Curves. Based on the assumption that a relationship exists between percent of reservoir depth and total sediment volume, three design curves were developed using survey data from 30 reservoirs (Lara 1962):

(1) Sediment storage design curve (Figure L-8).

(2) Surface area design curve (Figure L-9).

(3) A relative depth of deposits at the dam (Figure L-10).

d. These design curves are used to develop future elevation-capacity and elevation-area curves based on the predicted sediment yield from the watershed.

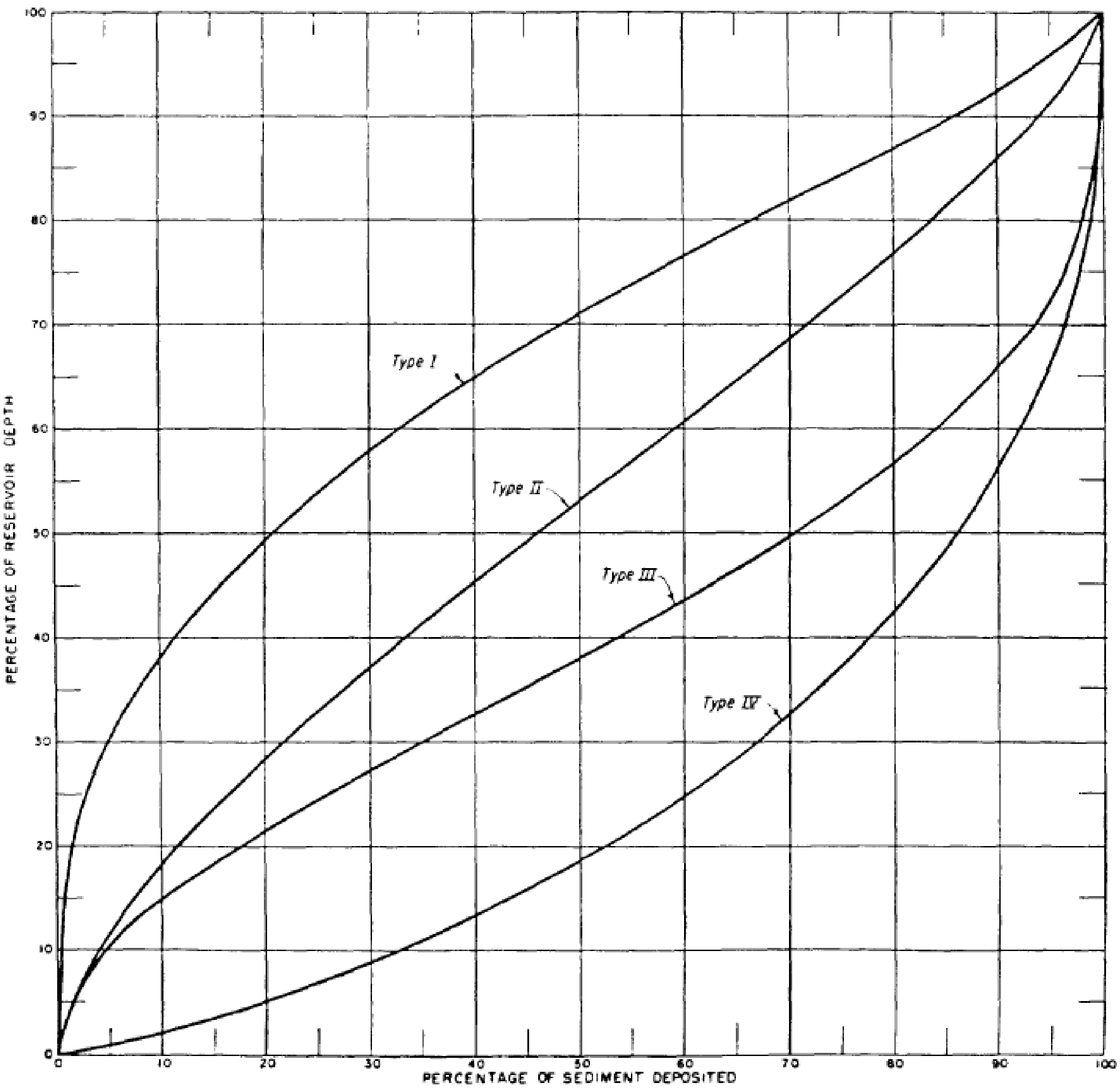


Figure L-8. Distribution of sediment deposits in the reservoir (adapted from Lara 1962)

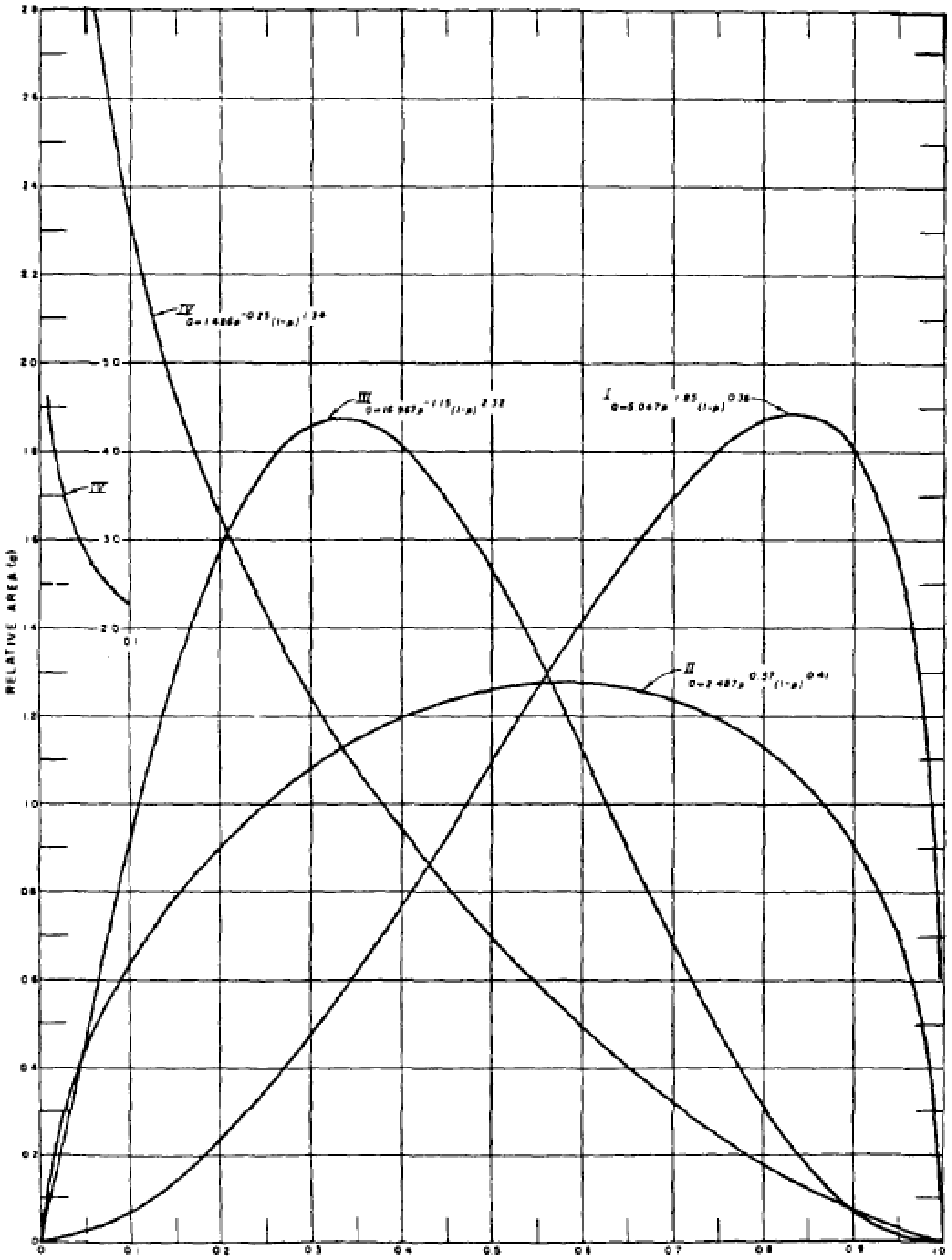
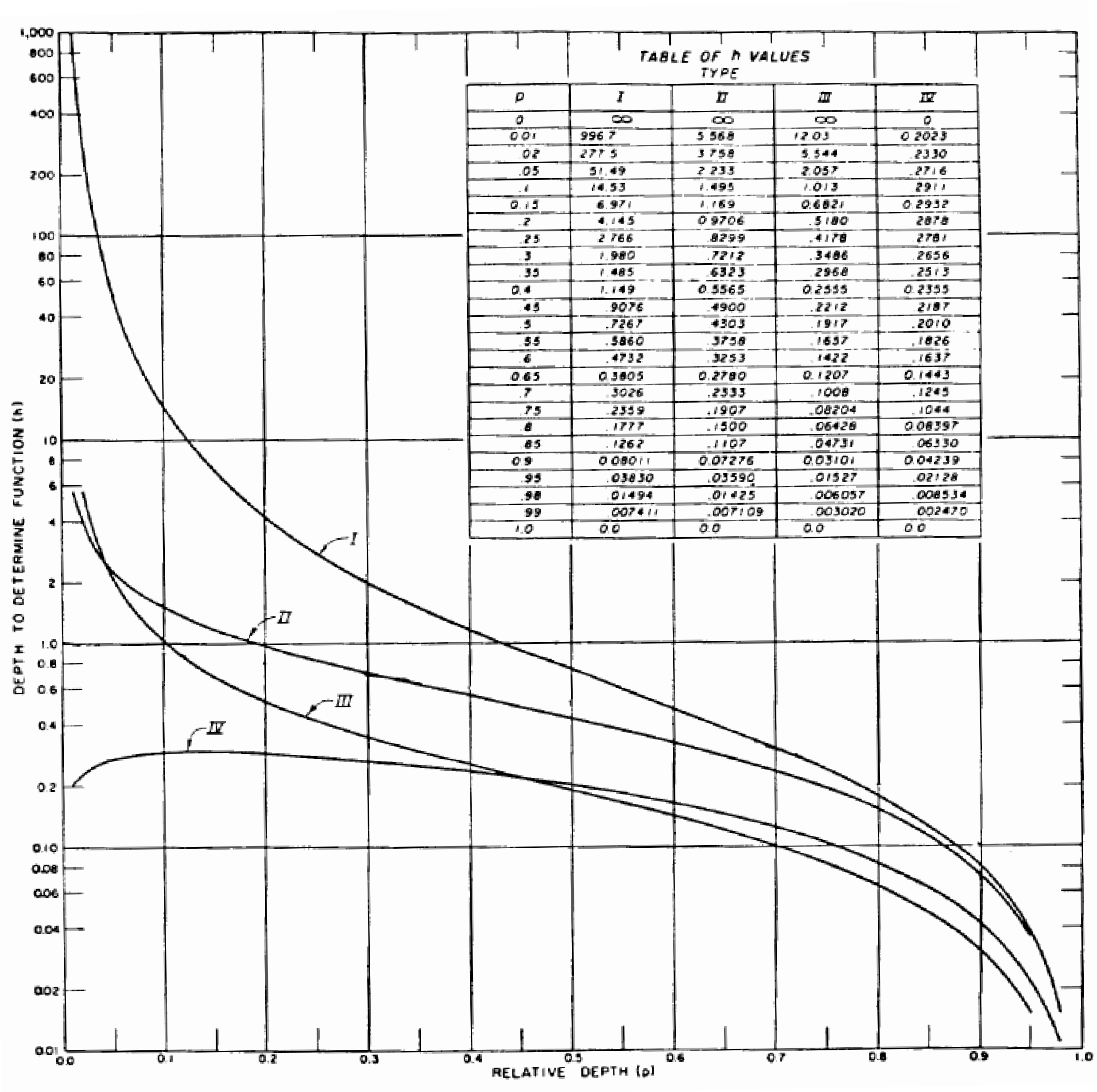


Figure L-9. Surface area of sediment deposits in the reservoir (adapted from Lara 1962)



p	I	II	III	IV
0	∞	∞	∞	0
0.01	996.7	5.568	12.03	0.2023
.02	277.5	3.758	5.544	.2330
.05	51.49	2.233	2.057	.2716
.1	14.53	1.495	1.013	.2911
0.15	6.971	1.169	0.6821	0.2932
.2	4.145	0.9706	.5180	.2878
.25	2.766	.8299	.4178	.2781
.3	1.980	.7212	.3486	.2656
.35	1.485	.6323	.2968	.2513
0.4	1.149	0.5565	0.2555	0.2355
.45	.9076	.4900	.2212	.2187
.5	.7267	.4303	.1917	.2010
.55	.5860	.3758	.1657	.1826
.6	.4732	.3253	.1422	.1637
0.65	0.3805	0.2780	0.1207	0.1443
.7	.3026	.2333	.1008	.1245
.75	.2359	.1907	.08204	.1044
.8	.1777	.1500	.06428	0.08397
.85	.1262	.1107	.04731	.06330
0.9	0.08011	0.07276	0.03101	0.04239
.95	.03830	.03590	.01527	.02128
.98	.01494	.01425	.006057	.008534
.99	.007411	.007109	.003020	.002470
1.0	0.0	0.0	0.0	0.0

Figure L-10. Depth of sediment deposits at the dam (adapted from Lara 1962)

L-9. Example Problem (Canton). The example, Canton Reservoir, is a multipurpose project owned and operated by the Tulsa District of USACE. It is located in Oklahoma. The problem is to predict the distribution of deposits and to determine how much the elevation-capacity relationship will change after 50 years of operation. The procedures and forms in this example are from the USBR (1987, 2006).

a. Pertinent Data. Pertinent data about the project:

(1)	Top of flood control pool elevation	1,630.0 feet
(2)	Elevation at base of dam	1,575.0 feet
(3)	Maximum depth of reservoir at the dam	55.0 feet
(4)	Expected sediment yield over 50-year life	48,000 acre-feet
(5)	Expected normal operation elevation range	1,595–1,625 feet

(6) Table L-3 provides elevation vs. reservoir capacity and reservoir surface area.

b. Reservoir Type. The depth capacity relationship from that data is plotted in Figure L-11 to develop the reservoir classification coefficient, m. The relationship did not plot a straight line. A value of 2.9 was computed for the lower part of the curve and 2.4 for the upper part. In Table L-2, 2.9 falls into the Type II category (2.5 to 3.5) and 2.4 is Type III (1.5 to 2.5). Since 2.4 is near the lower limit of Type III and 2.9 is almost in the middle of Type II, Type II is selected.

c. Depth of Deposit at the Dam. The next step is to determine the elevation of sediment deposited at the dam. The procedure, shown in Table L-4 and illustrated in Figure L-12, is to determine the relative depth of sediment deposited at the dam using the reservoir type calculated in the previous step. Note that Figure L-12 is a copy of Figure L-10 with the results from Table L-4 superimposed on it, where the column 2 table value is plotted on the abscissa and the column 6 table value is plotted on the ordinate.

(1) The two key constants in the computations, tabulated at the top of the table, were taken from the pertinent data information. They are the 50-year volume of sediment inflow, S, and the original depth to the top of the flood control pool at the dam, H.

(2) Assume an elevation, column 1.

(3) Calculate p, column 2 in Table L-4 by determining the height of the elevation in column 1 above the base of the dam and dividing that height by the depth of the flood control pool, 55 feet.

(4) Column 3 is the reservoir capacity obtained from Table L-3.

(5) Column 4 is calculated by subtracting column 3 from S.

(6) Column 5 is obtained by multiplying H by the area for that elevation in Table L-3.

(7) Column 6 is column 4 divided by column 5. It is then plotted on Figure L-12, and if it plots on the line for Type II, it is the result being sought. Otherwise, assume another elevation, column 1, and repeat the steps.

(8) The value P_o is the intersection of p vs. h′(p) curve with the Type II curve in Figure L-12. Sometimes the plotted curve does not intersect the reservoir type curve selected. If that happens, use the area-increment method to determine the height of the deposited sediment at the dam.

Table L-3
Canton Reservoir Area and Capacity Data

Elevation (ft)	Depth at Dam (ft)	Surface Area (acres)	Volume (ac-ft)
1,575	0	0	0
1,580	5	18	16
1,585	10	284	639
1,588	13	1,010	3,410
1,590	15	1,640	5,740
1,595	20	2,820	15,750
1,600	25	3,890	32,040
1,603	28	4,630	44,590
1,605	30	5,130	54,190
1,610	35	6,570	83,330
1,613	48	7,420	104,300
1,615	40	8,020	119,700
1,620	45	9,610	163,800
1,625	50	11,380	216,300
1,630	55	12,880	276,800

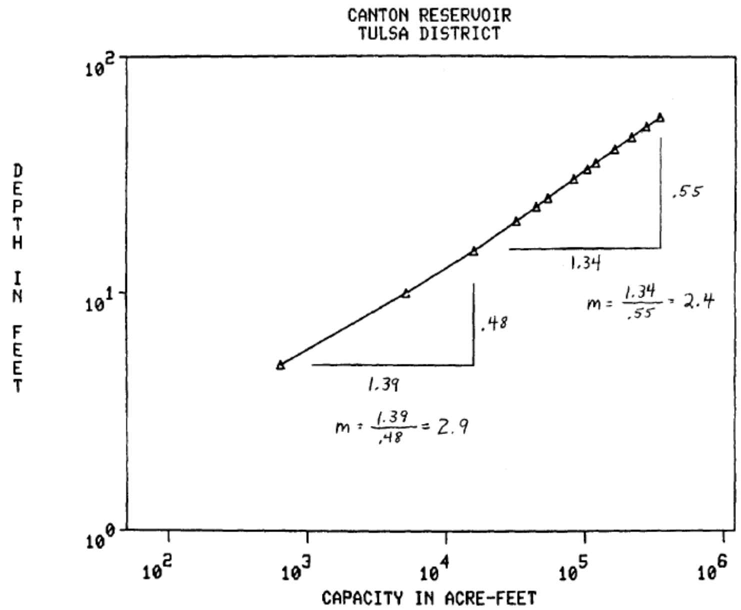


Figure L-11. Canton reservoir classification-type coefficient

Table L-4
Direct Determination of Elevation of Sediment Deposited at the Dam

Reservoir: Canton			**Project:**		
S = 48,000 ac-ft			H = 55 feet		
(1) Elevation (ft)	(2) p	(3) V (pH)	(4) S – V(pH)	(5) H A (pH)	(6) h′(p)
1,585	0.182	639	47,361	15,620	3.032
1,590	0.273	5,140	42,860	90,200	0.475
1,595	0.364	15,750	32,250	155,100	0.208
1,600	0.454	32,040	15,960	213,950	0.075
1,603	0.509	44,590	3,410	254,650	0.013

p_o = 0.24
p_o H = 13
Bottom elevation = 1,575
Elevation of sediment deposited at dam = 1,588
Notation of Symbols
p = relative depth of reservoir
V (pH) = reservoir capacity in acre-feet at a given elevation
S = total sediment inflow in acre-feet
H = height of dam
A (pH) = reservoir area in acres at a given elevation
h′(p) = a function of the reservoir and anticipated sediment storage expressed as:

$$h'(p) = \frac{S - V\,(pH)}{HA\,(pH)}$$

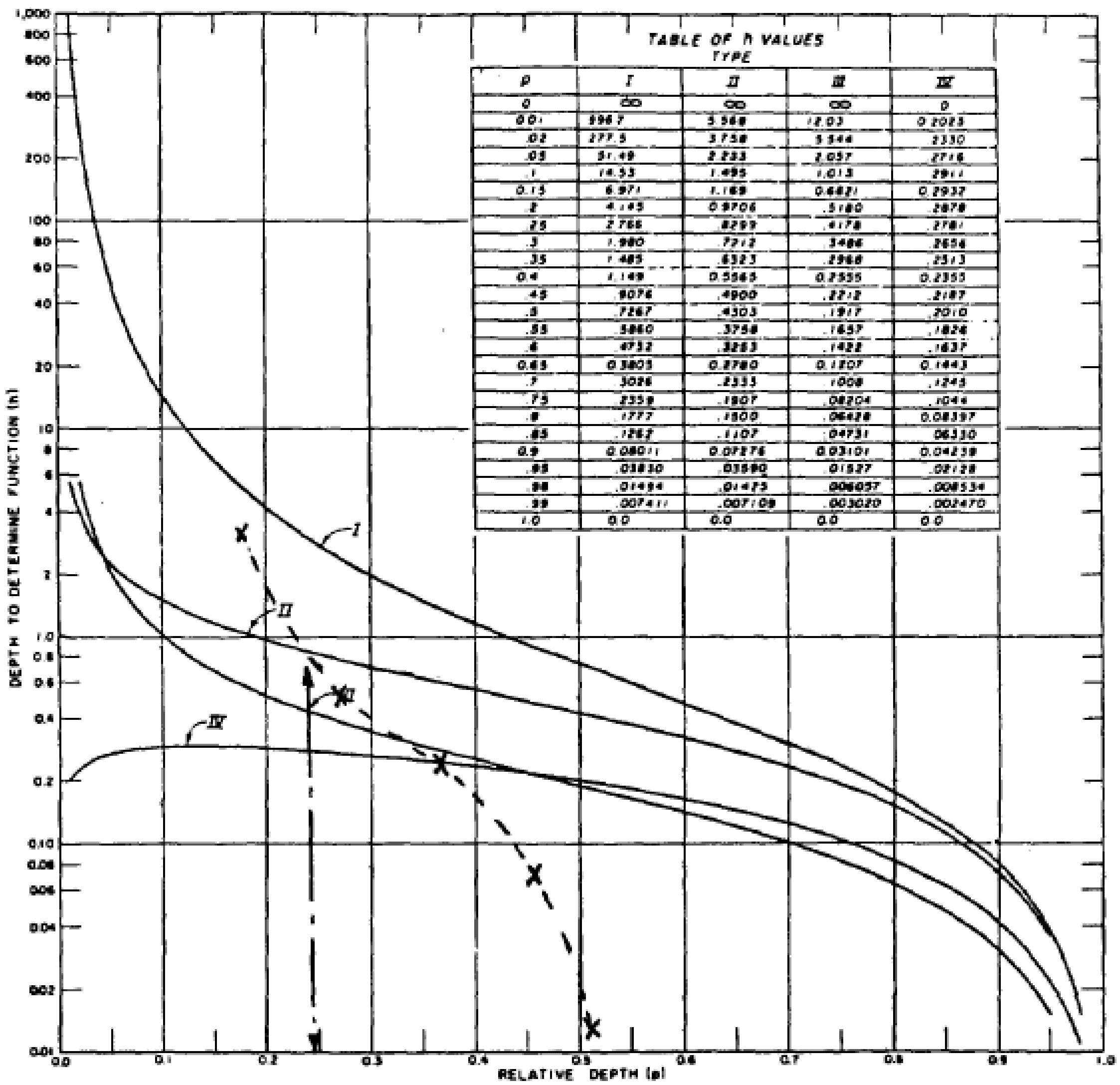


TABLE OF h VALUES

p	TYPE I	TYPE II	TYPE III	TYPE IV
0	∞	∞	∞	0
0.01	996.7	5.568	12.03	0.2023
.02	277.5	3.758	5.544	.2330
.05	51.49	2.233	2.057	.2716
.1	14.53	1.495	1.013	.2911
0.15	6.971	1.169	0.6821	0.2932
.2	4.145	0.9706	.5180	.2878
.25	2.766	.8299	.4178	.2781
.3	1.980	.7212	.3486	.2656
.35	1.485	.6323	.2968	.2513
0.4	1.149	0.5565	0.2555	0.2355
.45	.9076	.4900	.2212	.2187
.5	.7267	.4303	.1917	.2010
.55	.5860	.3758	.1657	.1826
.6	.4732	.3253	.1422	.1637
0.65	0.3805	0.2780	0.1207	0.1443
.7	.3026	.2333	.1008	.1245
.75	.2359	.1907	.08204	.1044
.8	.1777	.1500	.06428	0.08397
.85	.1262	.1107	.04731	.06330
0.9	0.08011	0.07276	0.03101	0.04239
.95	.03830	.03590	.01527	.02128
.98	.01494	.01425	.006057	.008534
.99	.007411	.007109	.003020	.002470
1.0	0.0	0.0	0.0	0.0

Figure L-12. Elevation of sediment deposit at canton dam

d. Table L-5 lists the last part of the sediment deposition computation. The following steps describe the procedure.

(1) Complete columns 1, 2, and 3 using the data from Table L-3 down to the new bottom elevation.

(2) Compute the relative depth values in column 4 by dividing the original depth, 55 feet, into the depths computed as the difference between elevation 1,575 and the elevations in column 1.

(3) Read relative area values from the Type II curve in Figure L-9 and list them in column 5.

(4) Compute the K in the supplement at the bottom of the table by dividing the reservoir area (column 2) by the relative area, Ap (column 5), at the elevation of the sediment deposited at the dam (1,588 feet).

(5) Complete column 6 by multiplying the values in column 5 by K.

(6) Compute the sediment volumes in column 7 using the average end-area method by averaging the areas in column 6 of two elevations and multiplying by the difference of the elevations.

(7) Starting with the storage for elevation 1,588, accumulate the volumes in column 7 to complete column 8. If the accumulated sediment volume does not equal 48,000 acre-feet, then calculate a new value for K using the following equation. Table L-5 actually shows trial 2. On the first trial, Kl was 1,031, which produced an accumulated sediment volume of 50,083 acre-feet. Using that result, K2 was computed as follows:

$$K2 = K1 \cdot \left(\frac{S2}{S1}\right) \qquad \text{Equation L-3}$$

where:

Kl = the relative distribution coefficient for trial 1
K2 = the relative distribution coefficient for trial 2
S1 = the sediment volume calculated with Kl in trial 1
S2 = the actual total sediment volume

$$K2 = 1031 \cdot \left(\frac{48000}{50083}\right) = 988 \qquad \text{Equation L-4}$$

(8) Steps 5 through 7 are then repeated, resulting in the values shown in columns 6 and 7.

(9) Compute column 9 as the difference between columns 2 and 6.

(10) Column 10 is the difference between columns 3 and 8. That is the new capacity curve for the project with 50 years of sediment storage.

(11) The new area and capacity curves for the project can be drawn from columns 1, 8 and 10.

Table L-5
Tabulated Sediment Deposition Computations, Canton

SEDIMENT DISPOSITION COMPUTATIONS
(Empirical Area - Reduction Method)

Reservoir Canton Project

Total Sediment Inflow 48000 acre feet Computed by DTW Date

① Elevation (Feet)	② Original Area (acres)	③ Original Capacity (acre-feet)	④ Relative Depth	⑤ A_p Type II	⑥ Sediment Area (acres)	⑦ Sediment Volume (acre-feet)	⑧ Accumulated Sediment Volume (acre-feet)	⑨ Revised Area (acres)	⑩ Revised Capacity (acre-feet)
1630	12880	276800	1.0	0	0		48000	12880	228800
						2175			
1625	11380	216300	.909	.88	870		46010	10510	170290
						4890			
1620	9610	163800	.818	1.10	1087		41120	8523	122680
						5730			
1615	8020	119700	.727	1.22	1206		35390	6814	84310
						2460			
1613	7420	104300	.636	1.27	1255		32930	6165	71370
						3780			
1610	6570	83330	.601	1.28	1265		29150	5305	54180
						6300			
1605	5130	54190	.546	1.27	1255		22850	3875	31340
						2480			
1603	4630	44590	.465	1.24	1225		20370	3405	24220
						3620			
1600	3890	32040	.418	1.21	1196		16740	2694	15300
						5855			
1595	2820	15750	.364	1.16	1146		10885	1674	4865
						5435			
1590	1640	5740	.273	1.04	1028		5450	612	290
						2040			
1588	1010	3410	.236	.98	1010		3410	0	0

SUPPLEMENT

1588	1010		.236	.98	$K_1 = 1010/.98 = 1031$

$K_2 = 1031\,(48000/50083) = 988$

L-10. Pool Elevation Duration Method. The key in successful application of these empirical methods is to identify the dominant factor in the problem, then select the method having that same dominant factor in the data sets used in its development. According to Hobbs, “regulation is one of the dominant factors affecting the location of sediment deposits.” Hobbs considered the first five factors in his list, paragraph H-l, to be governed in some degree by pool fluctuations. He illustrated this in Figure L-13 and Figure L-14.

a. Curve 3 of Figure L-13 shows a hypothetical suspended sand discharge entering a large reservoir on an alluvial stream during the design flood. Other data are the inflowing water discharge hydrograph, curve 1, reservoir outflow hydrograph, curve 2, and the pool elevation hydrograph, curve 4.

b. Curve 5 is the accumulated sand inflow expressed as a percent of the total. The reservoir was at the bottom of the flood control pool when the flood started, and 97% of the inflowing sand load entered before the maximum pool elevation was reached.

c. Coincidental values of inflow and pool elevations from those curves were plotted to show the upstream limits of backwater, Figure L-14, to demonstrate why most of the sediment delivered to a large flood control reservoir by any given flood is transported to elevations below the highest pool elevation attained during that flood. Also, it shows the source of energy that tends to redistribute material deposited during previous events.

d. As always, there is a considerable degree of ambiguity in designation of a reservoir as "large" or "small." The capacities of the reservoirs used to develop this method ranged from 60,000 to 20,000,000 acre-feet at the spillway crests.

e. This pool elevation duration method attempts to account for the influence of pool regulation by using an elevation duration curve. It considers the most dominant sediment property, particle size, by dealing with sands, and the size and shape of the reservoir are included in the approach. It also embodies the hypotheses that:

(1) Over a long period of time, sediment delivered by medium and moderate floods will establish some statistical order of coincidence with pool elevations between the maximum and minimum.

(2) Regulation of the rare floods, and therefore the distribution of sediment deposited in the higher elevation zones, will be similar. This suggests that there may be some reasonably definable relationships between duration of a given pool and the amount of sediment that will be deposited above and below the elevation of that pool.

f. The distribution of sediment deposits calculated by the pool-elevation duration method, have compared reasonably well with measured values. Some discrepancies, when checked out more closely, could be explained logically. For example, it is doubtful that conditions of deposition reported in Jemez Canyon Reservoir, New Mexico, could be predicted by any of the currently available empirical methods since substantial quantities of material have accumulated in elevation zones high above the maximum experienced pool elevation. That deposition appears completely unrelated to the reservoir.

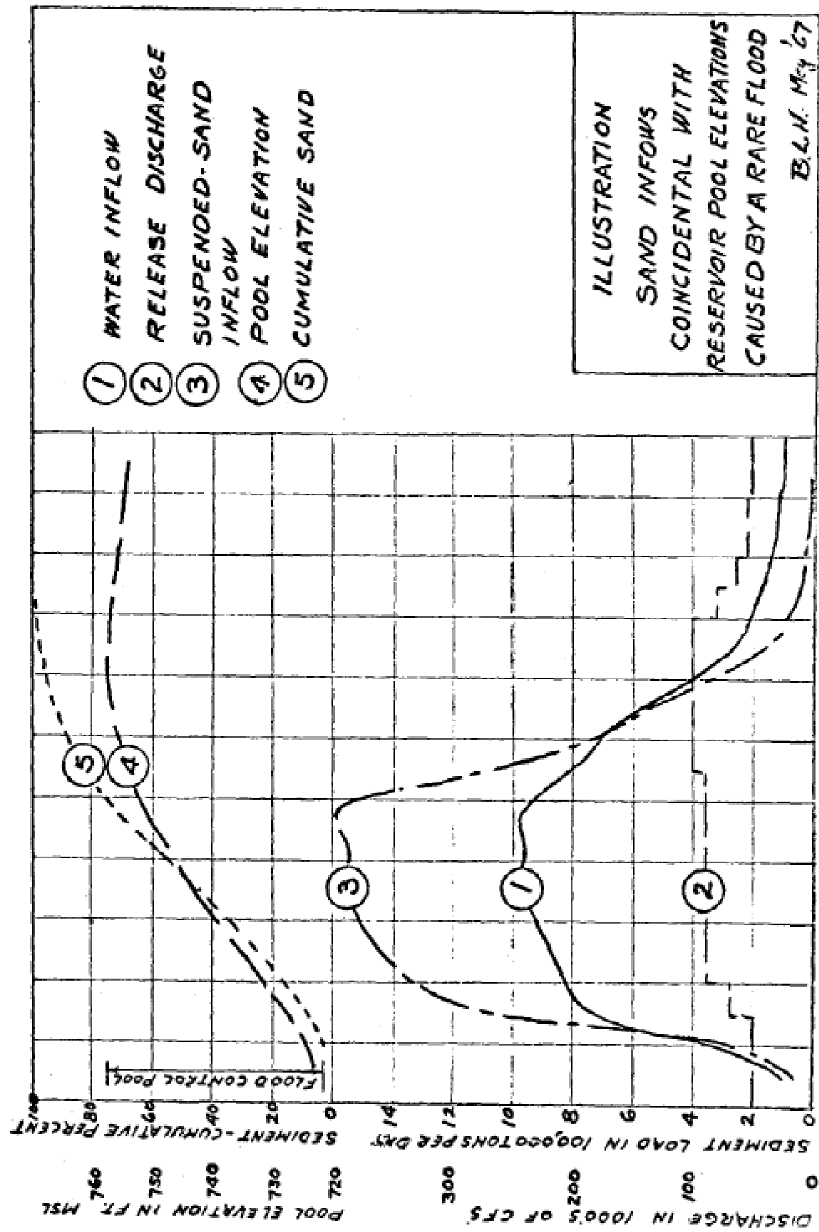


Figure L-13. Illustration, sand inflows coincidental with reservoir pool elevations caused by a rare flood (Hobbs 1969)

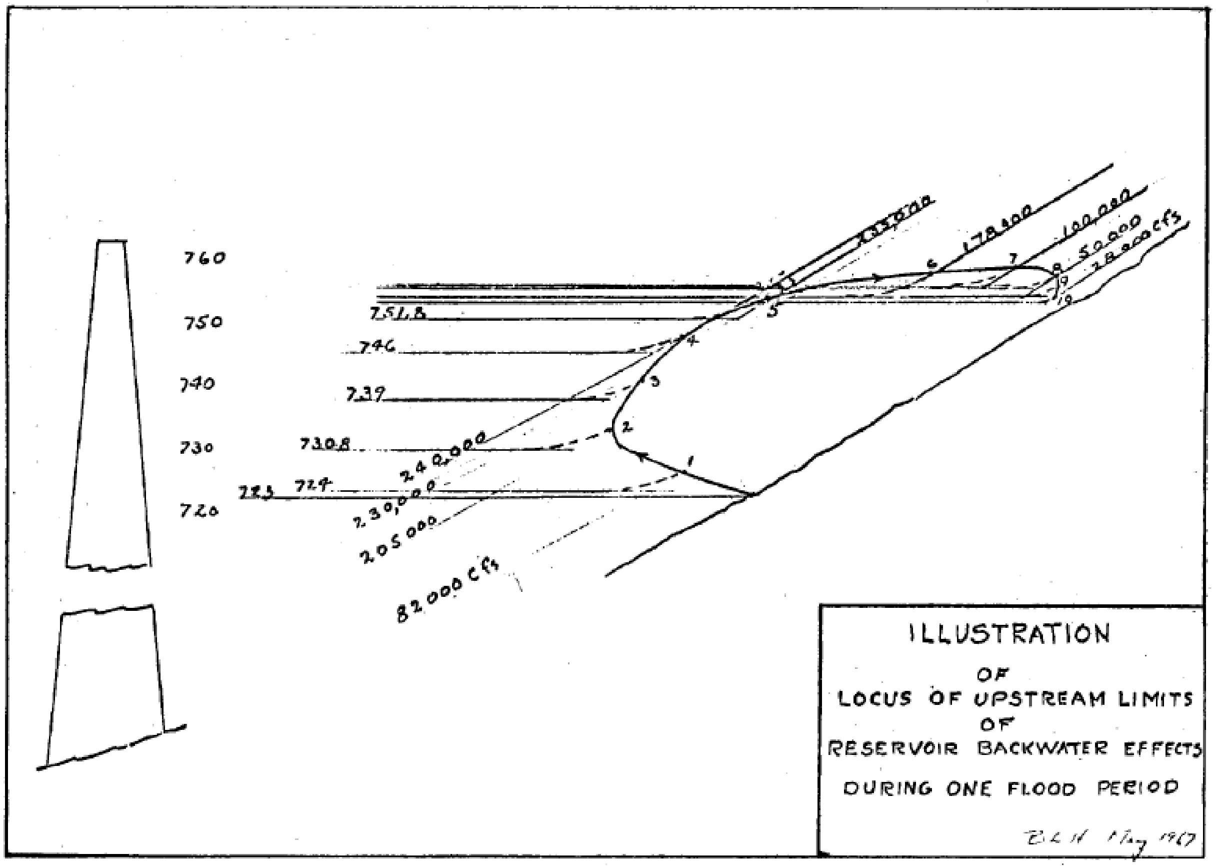


Figure L-14. Illustration, locus of upstream limits of reservoir backwater effects during one flood period (Hobbs 1969)

L-11. Example Problem (Fort Peck).

Using Fort Peck Reservoir data for explanation, the following information is required.

a. Pertinent Data.

(1) Pool elevation charts developed in connection with operation studies.

(2) Reservoir capacity, Table L-6.

(3) Estimated total sediment deposit during period under consideration.

(4) Estimate sand as a fraction of the total deposit.

b. Procedure. Plot the pool elevation duration curve, curve 1 in Figure L-15 from the pool elevation table.

c. Plot differences of capacity for increments of depth on log-log paper, Figure L-16. Five-foot increments were used here, but beware, the area-capacity table is in 10-foot increments.

Table L-6
Fort Peck Reservoir Condensed Area-Capacity Table (based on a 1961 Aggradation Survey, after Hobbs 1969)

Elevation (m.s.l.)	Depth (ft)	Area (Acres)	Capacity (Acre-Feet)
2,033	0	0	0
2,035	2	103	113
2,040	7	402	1,214
2,045	12	1,075	5,002
2,050	17	1,652	11,109
2,055	22	2,305	21,423
2,060	27	4,149	36,870
2,070	37	10,672	106,662
2,080	47	16,714	245,371
2,090	57	22,966	440,692
2,100	67	29,732	702,113
2,110	77	38,458	1,042,665
2,120	87	50,560	1,484,307
2,130	97	61,391	2,044,261
2,140	107	71,243	2,709,084
2,150	117	81,944	3,474,396
2,160	127	92,712	4,346,056
2,170	137	106,393	5,335,418
2,180	147	122,028	6,485,415
2,190	157	136,912	7,777,395
2,200	167	152,792	9,222,634
2,210	177	170,021	10,839,099
2,220	187	187,829	12,625,547
2,230	197	206,874	14,600,015
2,240	207	226,827	16,771,900
2,250	217	246,919	19,138,489
2,260*	227	270,200	21,704,684

*Extrapolated above elevation 2250.

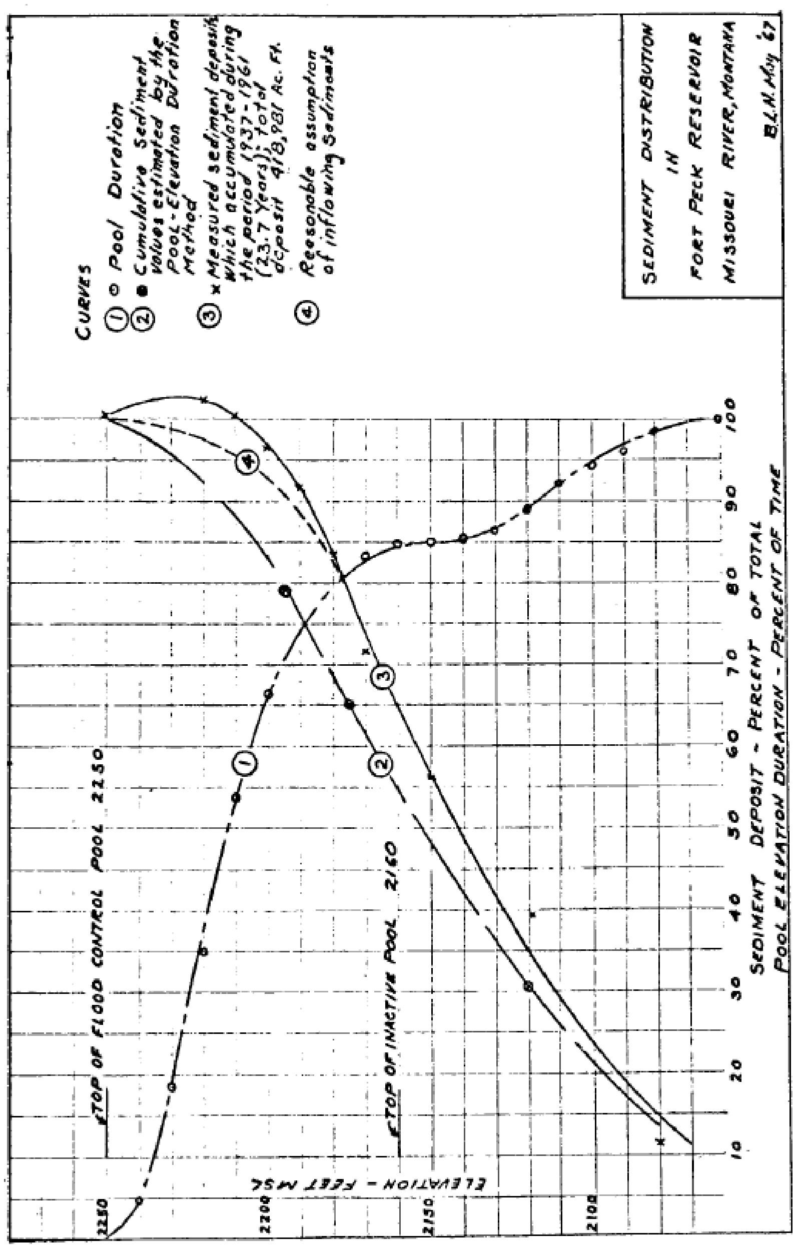


Figure L-15. Sediment distribution in Fort Peck Reservoir (Hobbs 1969)

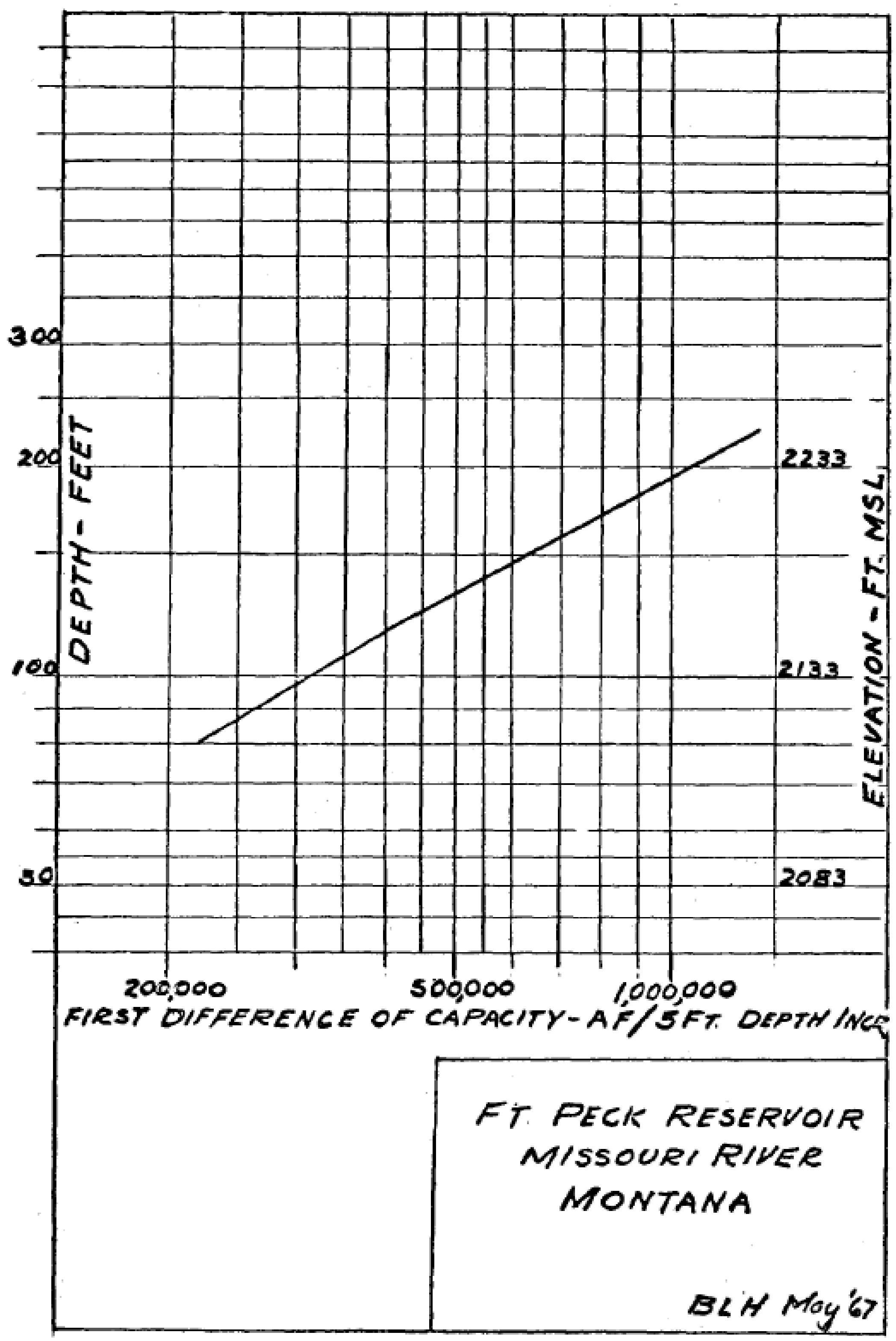


Figure L-16. Classification of Fort Peck Reservoir (Hobbs 1969)

d. Draw an estimated distribution curve on Figure L-17. In this case, a “right envelope” position was selected because of the low percentage of sand in the sediment deposit and the large capacities of pools in the operating range, from about 110,000 to 19,000,000 acre-feet. The position, in any case, is based on judgment. The sand scale shown on Figure L-17 is explained in paragraph (3) below.

e. Prepare Table L-7 as follows:

(1) Tabulate time durations (10%, 20% … 95% and 100%) in column 1.

(2) Tabulate pool elevations corresponding to the durations in column 2. Obtain values from curve 1 of Figure L-15.

(3) Tabulate initial differences of capacity, obtained from Figure L-16, in column 4.

(4) Compute ratios for “first differences of capacity” divided by the “first difference of capacity corresponding to the pool elevation that is exceeded only 5% of the time” and tabulate in Column 5.

(5) Enter the Fort Randall curve on Figure L-17 with ratios from column 5 and tabulate the corresponding values of cumulative percent of total accumulation in column 6. These values represent the estimated distribution of deposits. Measured values are tabulated in column 7 for comparison.

f. The percent sand scale on Figure L-17 is plotted from values taken from Figure L-18, which are a correlation of percent of sand with total deposits.

Table L-7
Estimate the Distribution of Sediment Deposits in Fort Peck Reservoir (adapted from Hobbs 1969)

Pool Elev. Duration (percent of time)[1]	Elev. (ft MSL)	Depth (ft)	First Diff of Capacity (Ac-Ft/5-ft Depth Increment)	Ratio[2] (Col. 4 + 1,125,000)	Sediment Distribution	Sediment Distribution
					Estimated (∑ %)	Measured (∑ %)[3]
(1)	(2)	(3)	(4)	(5)	(6)	(7)
10	2,117	84.0	236,000	0.27	30.0	32.0
20	2,116.5	143.5	580,000	0.52	65.0	78.5
30	2,195	162.0	722,600	0.64	79.0	92.0
40	2,208	175.0	828,000	0.74	88.0	95.5
50	2,212	179.0	862,000	0.77	90.0	97.8
60	2,218	185.0	915,000	0.81	94.0	98.8
70	2,225	192.0	987,000	0.88	97.0	99.5
80	2,230	197.0	1,030,000	0.92	98.5	99.8
90	2,236	203.0	1,090,000	9.97	99+	–
95	2,240	207.0	1,125,000	1.00	99.5	99.95
100	2,248	215.0	1,200,000	1.07	100.0	100.0

[1] Percent of time pool was at or below corresponding elevation shown in column 2.
[2] Ratio is 1.9 at the 95% pool.
[3] Values from Reservoir Sediment Data Summary.

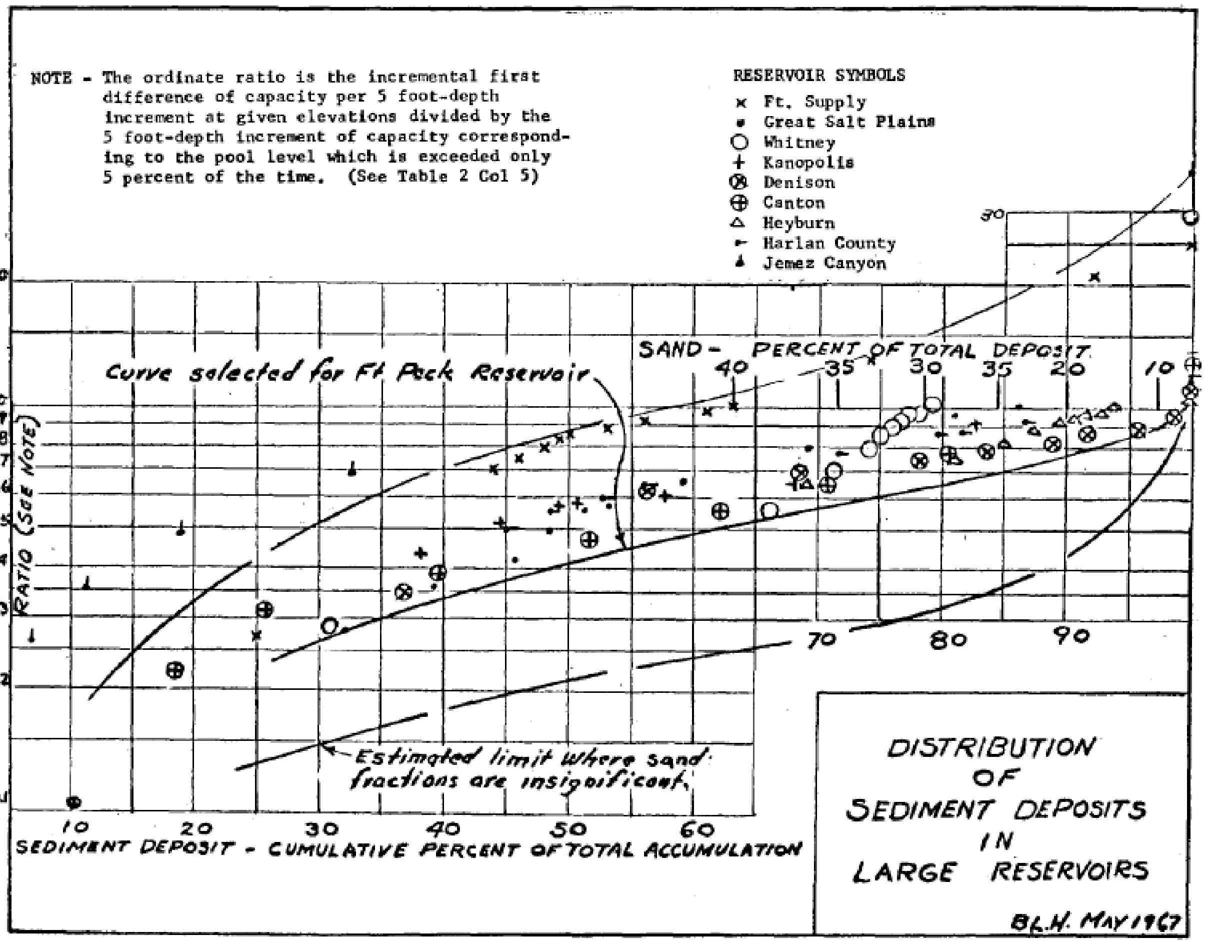


Figure L-17. Distribution of sediment deposits in large reservoirs (Hobbs 1969)

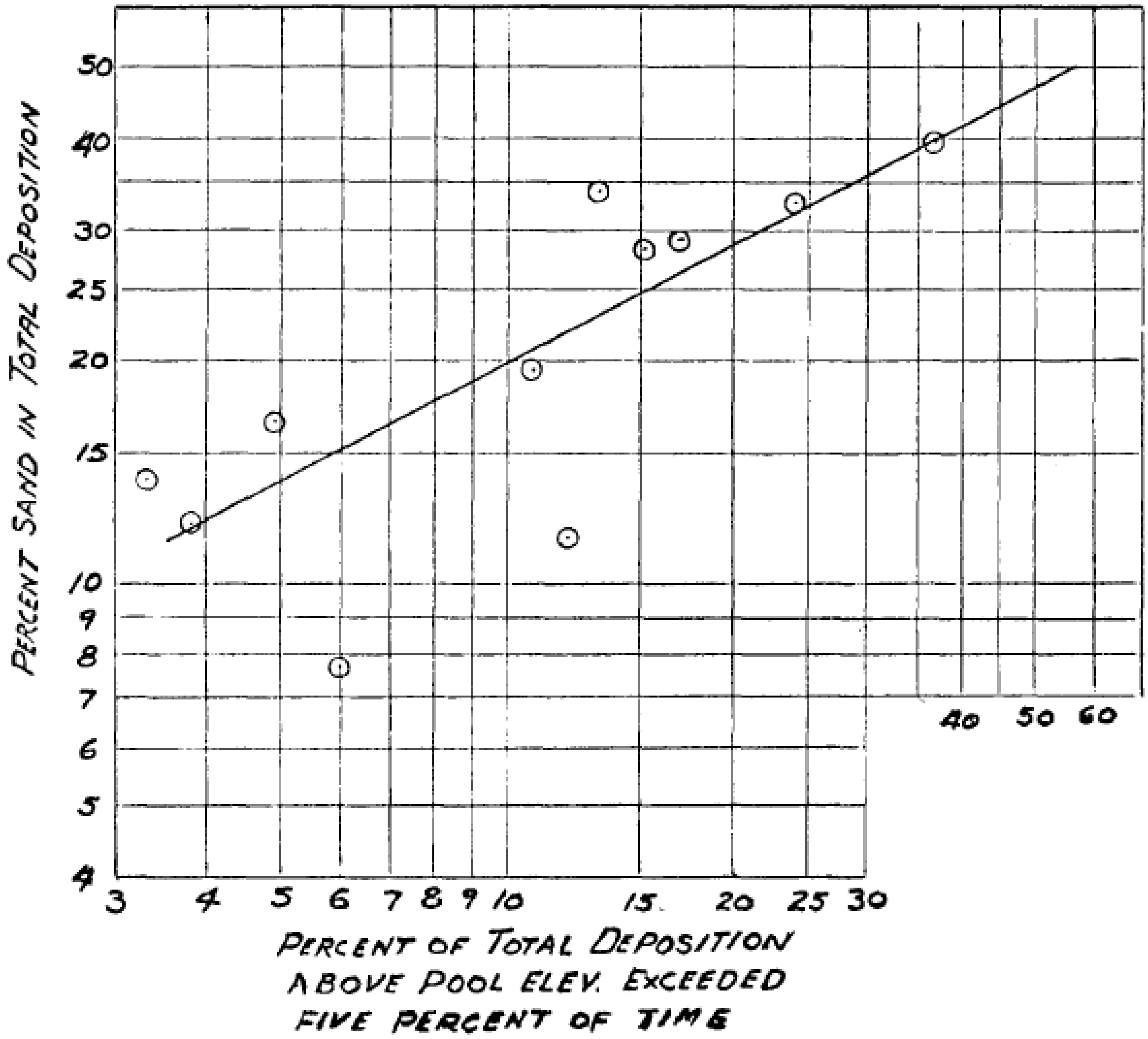


Figure L-18. Sand deposits above the 5% pool (Hobbs 1969)

Appendix M
Degradation of the Channel Downstream from a Dam

M-1. Introduction.

a. With the advances in numerical modeling, the methods illustrated in this appendix are no longer suitable for most USACE studies that require detailed evaluation of downstream channel degradation and associated impacts.

b. However, many historic USACE studies were performed using these procedures and can provide a quick estimate of the extent of degradation. The purpose of this appendix is to provide technical background documentation for historic USACE studies and to serve as an aid for understanding legacy computations when necessary. Refer to Chapter 8, Reservoir Sedimentation, for a discussion of current methods. With the addition of a few minor supplements, this appendix consists of the entire Appendix J, Degradation of the Channel Downstream from a Dam, extracted from the 1989 version of EM 1110-2-4000.

c. The rate and ultimate limit of channel degradation that can be expected downstream of a dam is dependent on the type of bed that comprises the channel. If the channel bed has a fairly uniform gradation and the largest particles present are easily capable of being transported at any time within the analysis time period, the stable slope method is recommended. For channel beds containing large enough particles in a significant amount to form an armor layer, the armor bed method is recommended.

d. The concept of dominant discharge is used in both methods and this discharge is defined as a representative single discharge, when allowed to flow indefinitely, would produce a channel very similar to that formed by a naturally fluctuating flow (Pemberton and Lara 1984). For an unregulated river, the dominant discharge is usually the bankfull discharge or the peak discharge having a recurrence interval of 1 to 2 years. The channel hydraulic properties to be used in both methods should be the average properties of the cross sections near the dam site for the dominant discharge.

M-2. Stable Slope Method.

a. General. Based on the assumption that the general character of the bed material does not change, the stable slope method, adapted from the USBR method (Pemberton and Lara 1984), is used only when there is insufficient coarse material to form an armored layer, the gradation of the bed material is the same down to the depth of degradation, and the bed material depth is greater than the expected degradation limit. If a stable stream slope can be defined as that slope at which no bed material is transported, bedload movement equations can be used to determine this slope by equating the bedload movement to zero and solving for the slope. The bedload equation selected should be tempered with judgment, compared with degradation limits of nearby dams with similar sediment/hydraulic characteristics, and compared with other equations.

b. Volume of Erosion. If there is not a limit to the degradation length, such as downstream structures or rock outcrops, the volume of expected eroded material must be estimated.

(1) For reservoirs with little flow regulation, the amount of coarse sediment, of the size found on the channel bed, that is trapped by the reservoir, is essentially the amount of sediment eroded downstream of the reservoir because of the stream's attempt to reach its transport potential. If the flow regulation is significant, the channel hydraulic and sediment properties for the dominant discharge must be used to calculate the sediment transport potential.

(2) Using an appropriate time interval, 1 to 5 years, the volume of sediment eroded can be estimated by multiplying the transport rate by the time interval. With this volume and the three-slope method, which is discussed later, the new average channel can be estimated, and its hydraulic properties can be used to estimate the sediment transport for the next time interval. The time interval increases as the change in bed elevation decreases. This procedure is continued until the design life of the reservoir is reached. The degradation length to be used is the lesser of the length computed as described or the distance to the nearest erosion preventing anomaly.

(3) The configuration of a degradation profile can be represented by the three-slope method as shown in Figure M-1. The volume of eroded sediment can be represented by:

$$Vol = A_t \cdot \frac{B}{43{,}560} \qquad \text{Equation M-1}$$

where:

Vol = volume of material to be eroded in acre-feet (equal to material trapped in coarse bed rivers)
A_t = longitudinal area of degradation in square feet
B = channel width in feet

c. Solving for A_t:

$$A_t = 43{,}560 \cdot \frac{Vol}{B} \qquad \text{Equation M-2}$$

(1) From Figure M-1:

$$A_t = \frac{(39 \cdot D^2)}{(64 \cdot del\ S)} \qquad \text{Equation M-3}$$

where:

D = depth of degradation immediately downstream of the dam
del S = difference between the existing and stable slope

(2) From Equations M-2 and M-3

$$43{,}560 \cdot \frac{Vol}{B} = (39 \cdot D^2)/(64 \cdot del\ S) \qquad \text{Equation M-4}$$

$$D = 267.4 \cdot SQRT\left[\frac{Vol \cdot del\ S}{B}\right] \qquad \text{Equation M-5}$$

(3) From Figure M-1 the degradation reach length, L4, is:

$$L4 = 13 \cdot \frac{D}{8 \cdot del\ S} \qquad \text{Equation M-6}$$

d. Stable Slope. The stable slope, S, can be computed using the following formulas.

(1) Meyer-Peter, Muller formula is recommended for coarse sediment.

$$S = 0.19 \cdot \left[\frac{n}{d90^{\frac{1}{6}}}\right]^{1.5} \cdot \frac{dm}{R} \qquad \text{Equation M-7}$$

where:

dm = effective size of bed material expressed as a weighted mean diameter, or d_{50}, in mm
d_{90} = particle size of bet material at 90% finer, in mm
R = hydraulic radius, for width-depth ratio greater than 40, use water depth in feet

(2) The Schoklitsch formula can also be used.

$$S = \left(0.00174 \cdot dm \cdot \frac{B}{Q}\right)^{0.75} \qquad \text{Equation M-8}$$

where:

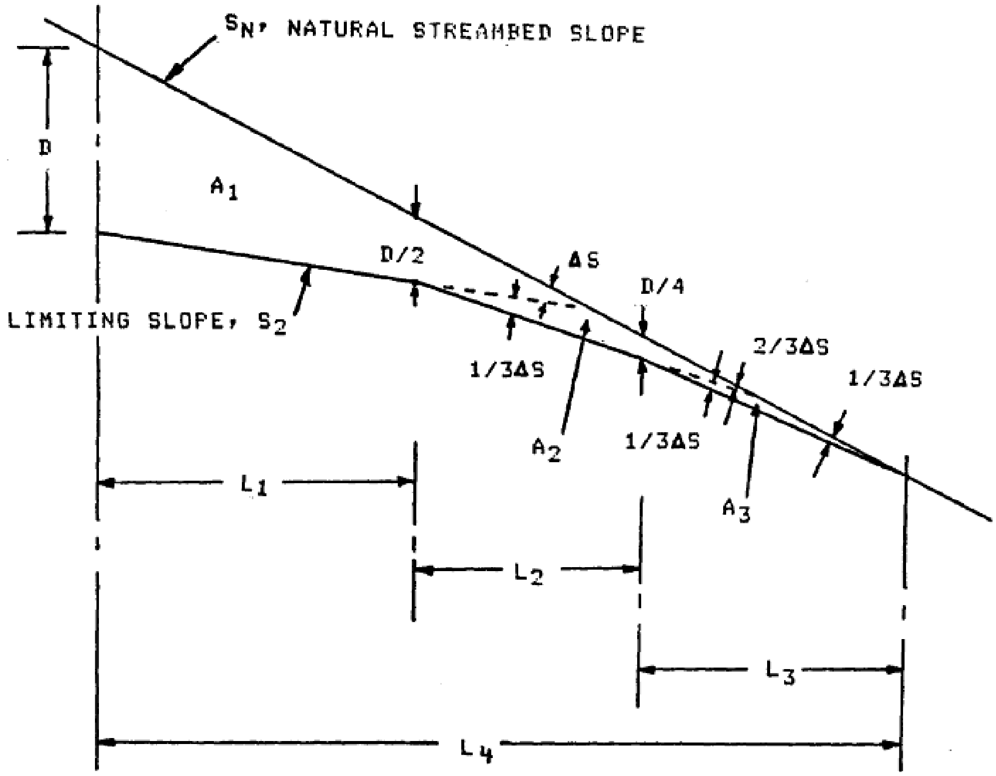
S = stable slope, in ft/ft
B = channel width, in ft
Q = dominant discharge, in ft^3/s

(3) Or the DuBoys formula.

$$s = \tau_{cr}/(gma \cdot R) \qquad \text{Equation M-9}$$

where:

gma = specific weight of water, in lb/cu ft
τ_{cr} = critical bed shear stress, in lb/sq ft, using dm and Figure M-1



D = Depth of degradation at the dam

$\Delta S = S_n - S_2$ in ft/ft

$$A_1 = \frac{3D^2}{8\Delta S} \qquad L_1 = \frac{D}{2\Delta S}$$

$$A_2 = \frac{9D^2}{64\Delta S} \qquad L_2 = \frac{3D}{8\Delta S}$$

$$A_3 = \frac{3D}{32\Delta S} \qquad L_3 = \frac{3D}{4\Delta S}$$

$$A_4 = \frac{39D}{64\Delta S} \qquad L_4 = \frac{13D}{8\Delta S}$$

Figure M-1. Three-slope method profile (Pemberton and Lara 1984)

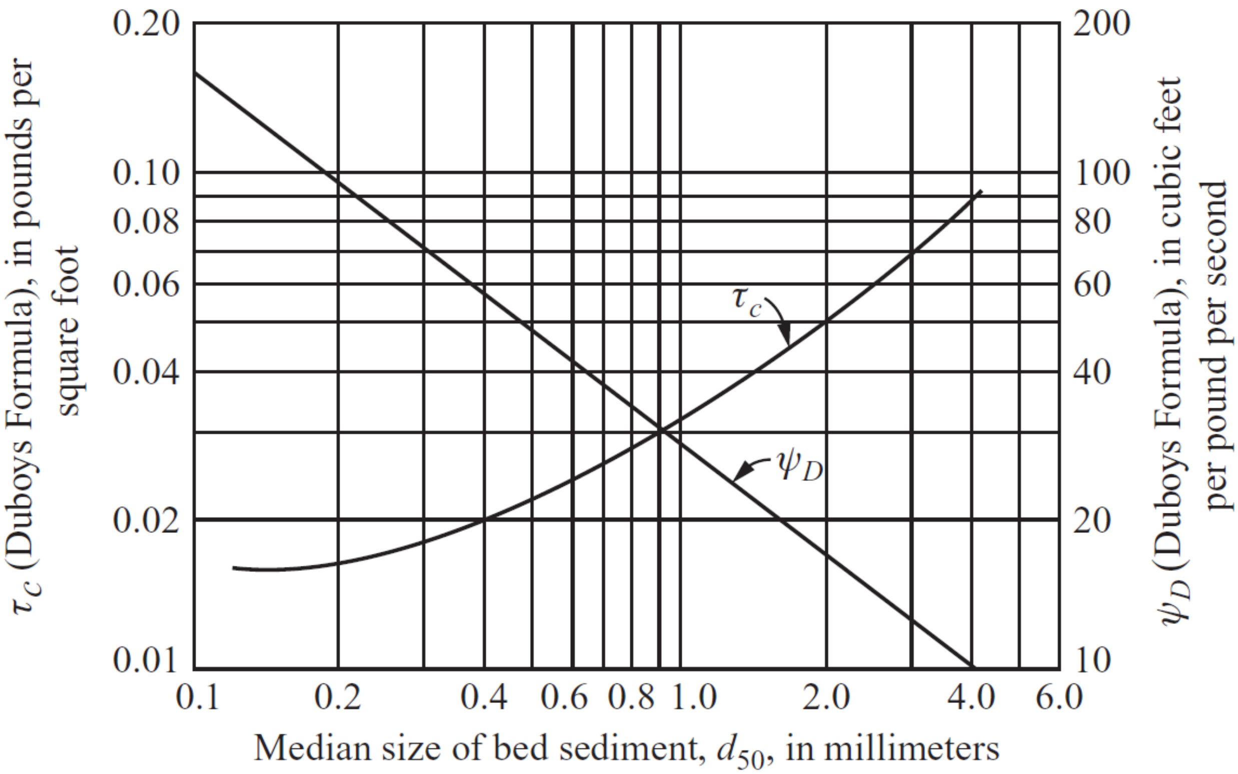


Figure M-2. Duboys relationship (Vanoni 1975, 2006)

e. Channel Width. The channel width, B, is the average width of the channel when degradation has reached its maximum. If the channel width increases with time, bank material will contribute to the volume being eroded and the anticipated amount must be subtracted from “Vol” used in Equation M-1. The extent of width change can be estimated by using stable channel design criteria to determine an equilibrium width and comparing it with the existing average width.

(1) Example 1.

Given:

Dominant discharge = 1,500 cfs.
Dm of the bed material = 0.5 mm.
Channel width = 400 ft.
d_{90} of the bed material = 1.5 mm.
Mean channel depth = 2 ft.
Manning’s n value = 0.03.
Existing stream bed slope = 0.0009.
Anticipated volume trapped by reservoir in 100 years = 3,000 acre-feet.

(2) Find. The stable channel slope, the depth of degradation, and the length of the degrading reach.

(3) Using the Meyer-Peter and Muller formula, Equation M-7:

$$S = 0.19 \cdot \left\{ \left[\frac{0.03}{1.5^{\frac{1}{6}}} \right]^{1.5} \right\} \cdot \frac{0.5}{2}$$

$$= 0.00022$$

(4) Using the Schoklitsch formula, Equation M-7:

$$s = \frac{0.00174 \cdot 0.5 \cdot 400^{0.75}}{1500}$$

$$= 0.0001$$

(5) Using DuBoys formula, Equation M-9 and Figure M-2:

$$\tau_{cr} = 0.022\ lb/sq\ ft$$

$$s = \frac{0.022}{62.4 \cdot 2}$$

$$= 0.00018$$

(6) Averaging the results from DuBoys and Meyer-Peter, Muller, but excluding Schoklitsch:

$$s = 0.0002$$

$$del\ S = 0.0009 - 0.0002$$

$$= 0.0007$$

(7) From Equation M-5, the depth of degradation is:

$$D = 267.4 \cdot SQRT \left[\frac{3000 \cdot 0.0007}{400} \right]$$

$$= 19.4\ ft$$

(8) From Equation M-6, the length of the degradation reach is:

$$L = 13 \left[\frac{19.4}{8 \cdot 0.0007} \right]$$

$$= 45{,}036\ ft\ or\ 8.5\ miles$$

M-3. Armor Bed Method.

a. This method requires the determination of "a minimum transportable representative particle size" for the hydraulic conditions of the dominant discharge. This particle size will become the primary particle size comprising the armored bed. Laboratory and field investigations have shown that the tractive force, tau (τ), exerted by moving water on the stream bed can be represented by:

$$\tau = gma \cdot R \cdot S \qquad \text{Equation M-10}$$

and this force will transport sediment particles up to a certain mean diameter size. The relationship between tractive force and mean particle diameter is shown by Figure M-3. By rearranging Equations M-7 and M-8, respectively, the mean armoring diameter for the dominant discharge can be computed.

$$dm = 5.26 \cdot S \cdot \frac{R}{\left[\left(\frac{n}{d90^{\frac{1}{6}}}\right)^{1.5}\right]} \qquad \text{Equation M-11}$$

and

$$dm = 4762 \cdot (S^{\frac{4}{3}}) \cdot \frac{Q}{B} \qquad \text{Equation M-12}$$

b. From the DuBoys formula, the mean diameter can be found by determining "tau" and using Figure M-2. The depth of degradation of an armoring bed is shown graphically in Figure M-4 and:

$$Y_a = Y - Y_d \qquad \text{Equation M-13}$$

where:

Y_a = thickness of armoring layer
Y_d = depth of degradation (depth from original stream bed to top of armoring layer)
Y = depth from original stream bed to bottom top of armor layer

$$Y_a = y \cdot [del\ p] \qquad \text{Equation M-14}$$

where [*del P*] is the decimal percentage of material larger than the armoring layer obtained from the bed material sieve analysis.

c. Combining Equations M-13 and M-14 gives:

$$Y_d = Y_a \cdot \left(\frac{1}{[del\ P]} - 1\right) \qquad \text{Equation M-15}$$

d. The depth, Y_a, is dependent on the particle size forming the armor layer and is generally considered to be 1 to 3 armoring particle diameters or 0.5 foot, whichever is smaller.

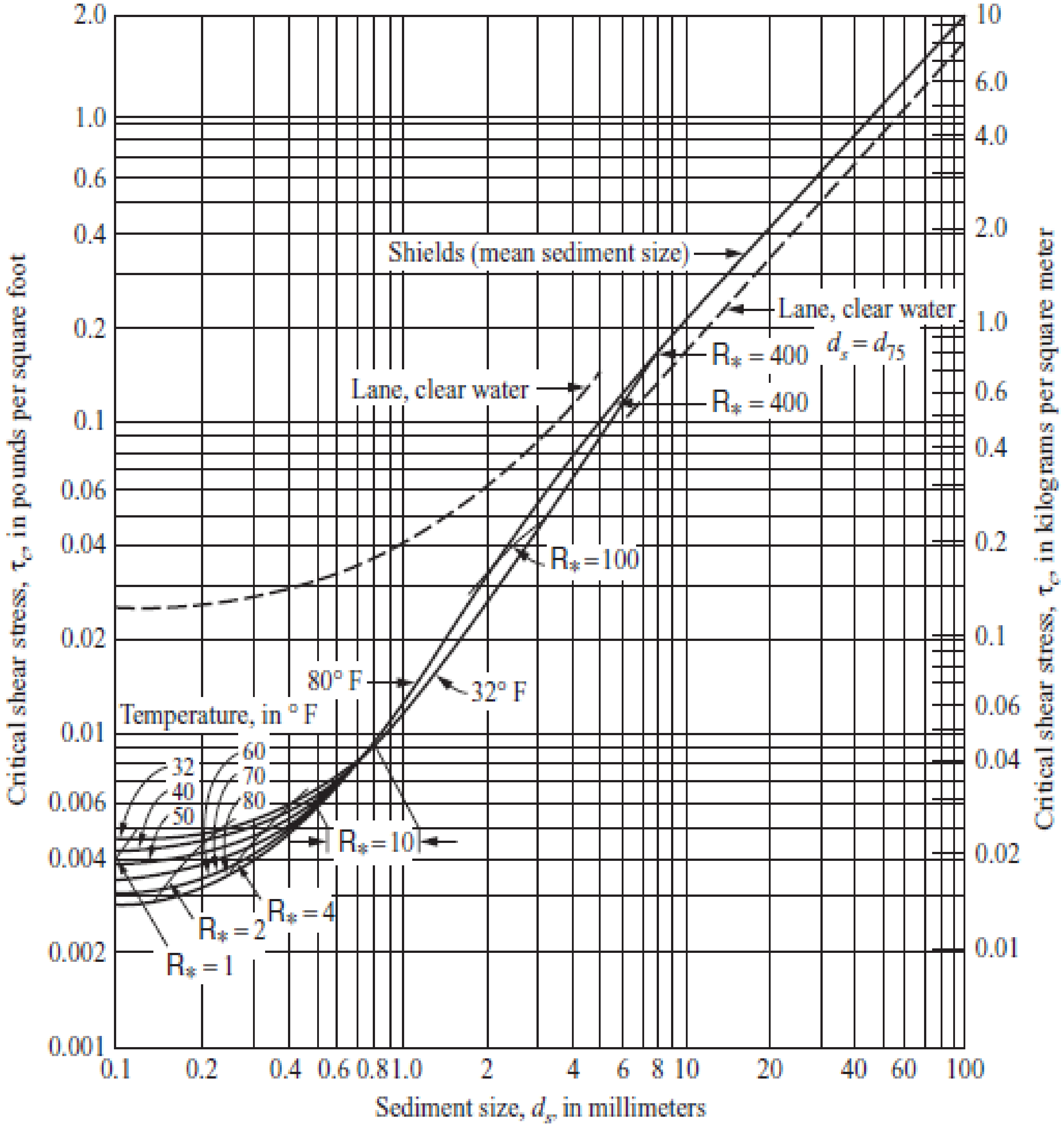


Figure M-3. Relationship of mean diameter and tractive force (Vanoni 1975, 2006)

Used with permission of ASCE, from Sedimentation Engineering, Manuals and Reports on Engineering Practice No. 54; ed. Vanoni, V.A., 1975, 2006; permission conveyed through Copyright Clearance Center, Inc.

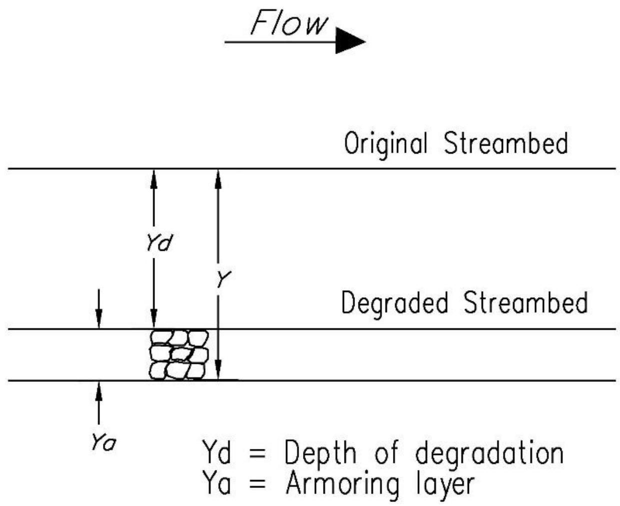


Figure M-4. Armoring definition sketch (revised from Strand and Pemberton 1987)

M-4. Armor Layer Example.

Given:

Dominant discharge = 1,000 cfs
Channel width = 75 ft
Hydraulic radius = 6 ft
Existing stream bed slope = 0.0015
Manning's n value = 0.03

Gradation of bed material is shown in Figure M-5.

a. Find the depth of erosion required to produce an armor layer.

(1) The tractive force is calculated using Equation M-10 as:

$$\tau = 62.4 \cdot 6 \cdot 0.0015$$

$$= 0.562\ lb/ft^2\ or\ 2{,}744\ g/m^2$$

(2) From Figure M-3, dm = 27 mm.

(3) From Equation M-11 and Figure M-5:

$$dm = \frac{5.26 \cdot 0.0015 \cdot 6}{\left[\frac{0.03}{40^{\frac{1}{6}}}\right]^{1.5}}$$

$= 22.9\ mm$

(4) From Equation M-12:

$$dm = 4762 \cdot (0.0015^{\frac{4}{3}}) \cdot \frac{1000}{75}$$

$= 10.9\ mm$

b. Using DuBoys' formula with tau = 0.562 lb/sq. ft and extrapolating Figure M-2

$dm = 22.4\ mm$

c. If 10.9 mm result computed from Equation M-13 is removed to produce a conservative estimate and averaging the remaining results:

$dm = 24\ mm$

d. The required armor layer thickness is:

$Y_a = 3d$

$= 72\ mm$

$= 0.24\ ft < 0.5\ ft$

e. From Figure M-5, del P = 0.20, expressed as a fraction:

$$Y_d = 0.24 \cdot \left[\left(\frac{1}{0.2}\right) - 1\right]$$

$= 0.96\ ft$

M-5. Dominant Discharge. The methods described are based on the dominant discharge being representative of equilibrium condition. However, if discharge is highly fluctuating and the peaks and troughs are significantly different from the dominant discharge, there could be scour and deposition along the stream that are transient in nature and disappear and reappear as other flows pass through. The long-term degradation will be as calculated, but if the fluctuation of the bed elevation along the degradation reach is important, the analysis should be made with a numerical modeling approach that simulates the actual hydrography. An extreme event analysis should be performed to evaluate the structural integrity of the project and downstream effects.

M-6. Bed Material Gradation. Bed material particle size is the primary factor responsible for the extent of degradation in a stream channel. Representative sizes are required for the study reach. “Representative sizes” do not denote constant or average sizes, but rather sizes that will control the degradation process. The process is not uniform; therefore, the representative size will vary along the stream. Typically the coarsest 5% of particles in the stream bed will control the rate and extent of bed degradation. Therefore, core and bulk samples should be taken at critical places.

M-7. Numerical Modeling Approaches. The degradation problem is too complex to rely on the simple analytical methods presented here for final design. More extensive analysis, such as provided by numerical modeling are required. Chapter 9 provides extensive discussion of the use of numerical models to estimate degradation.

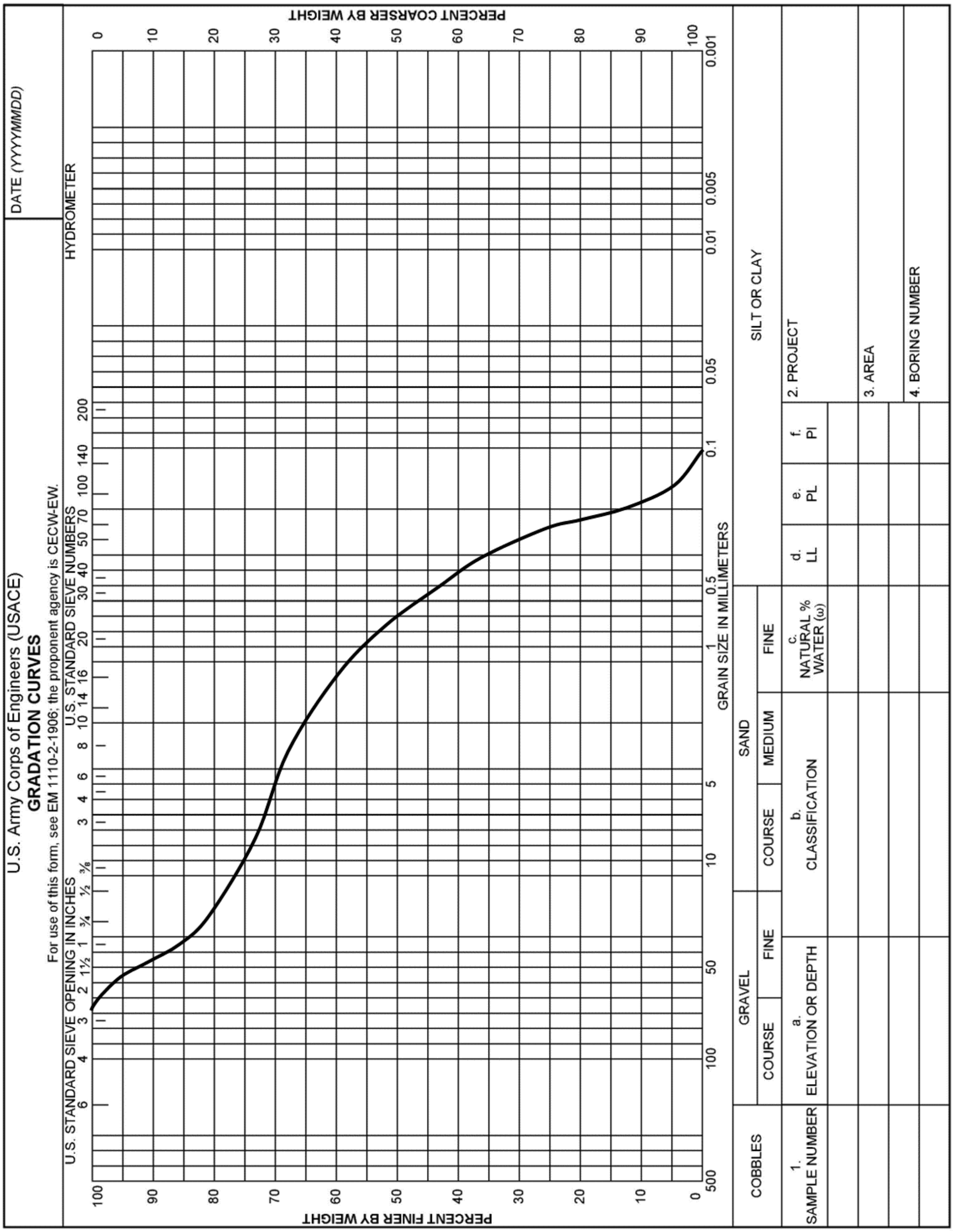


Figure M-5. Gradation of bed material

Appendix N
Case Studies

Case Study 2A
Puyallup-White River Basin Sediment Studies Work Plan

1. Case Study.

This case study provides an example Sediment Studies Work Plan (SSWP) that was developed by the Seattle District for the Puyallup-White River Basin (PWRB) in Washington State. The case study provides partial content that was condensed from the actual SSWP to illustrate the primary content and sections of an effective SSWP. The initial PWRB SSWP was prepared by USACE Seattle District in collaboration with the USGS Tacoma Water Science Center. The partial content demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

The Puyallup River Basin drains approximately 990 mi^2 of terrain between the northwestern slopes of Mount Rainer and Commencement Bay, and includes two major tributaries, the White and Carbon Rivers (Figure 1).

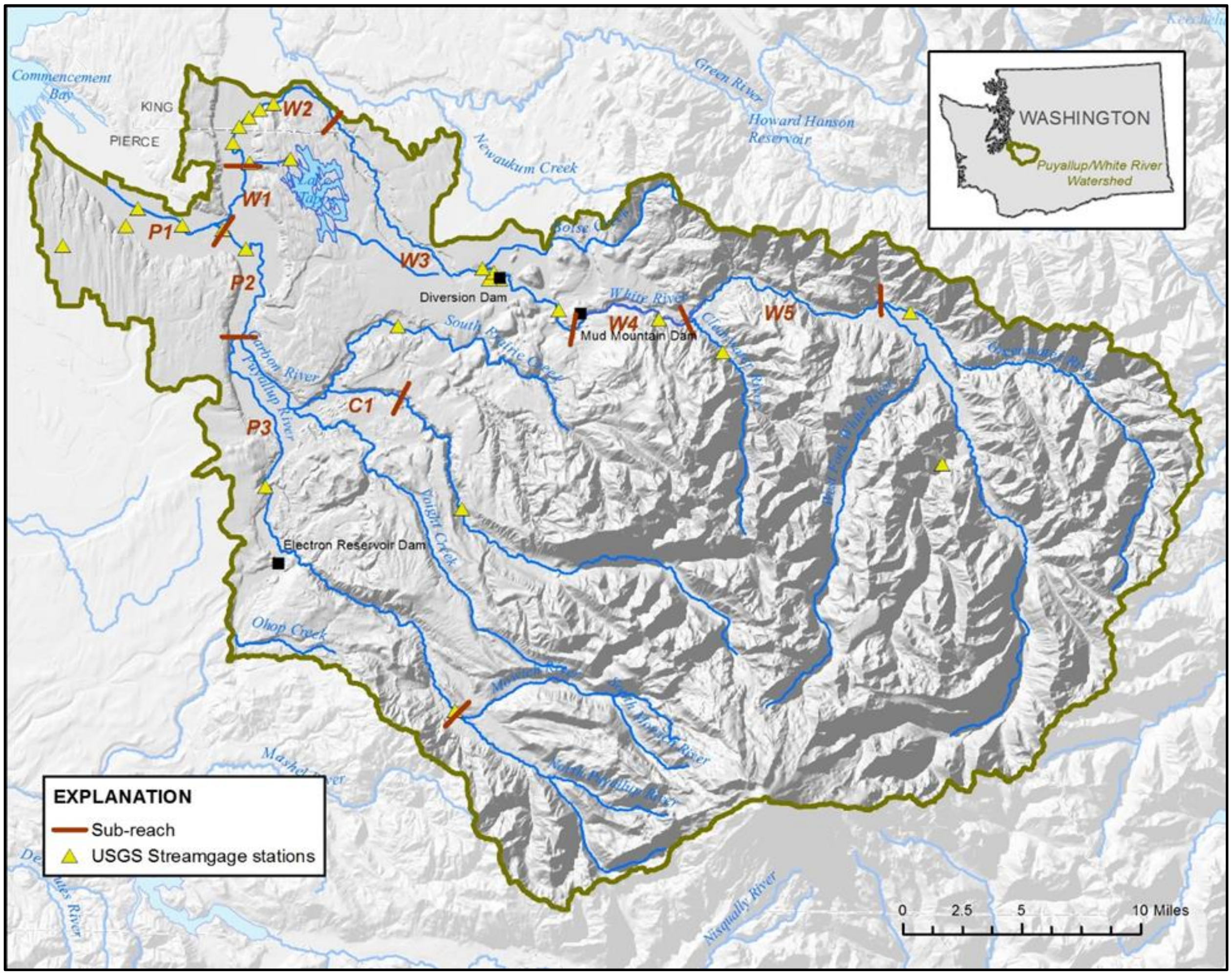
a. River Reaches. For this plan, the PWRB is divided into seven distinct river reaches and nine subsequent sub-reaches (Figure 1).

(1) All river miles (RM) extend in the upstream direction. The Puyallup River has three reaches: the Lower Puyallup River (P1; RM 0.0–RM 10.3), the Middle Puyallup River (P2; RM 10.3–RM 17.4) and the Upper Puyallup River (P3; RM 17.4–RM 29.6).

(2) The White River, which joins the Puyallup River at RM 10.3, is divided into three reaches, and five sub-reaches: the Lower White River (W1; RM 0–RM 3.6) from the Puyallup River confluence to Dieringer Canal, the Lower White River (W2; RM 3.6–RM 10.6) from Dieringer Canal to the mouth of the Auburn canyon, the Middle White River (W3; RM 10.6–RM 29.6) from the Auburn Canyon to Mud Mountain Dam, the Upper White River (W4; RM 29.6–RM 35.3) reservoir reach at Mud Mountain Dam (MMD), and the Upper White River (W5; RM 35.3–RM 45.8) above the MMD reservoir, between the Clearwater River and the Greenwater River.

(3) The Carbon River, which joins the Puyallup at RM 17.4, consists of a single reach and sub-reach (C1) that extends from the Puyallup River confluence to above the South Prairie Creek confluence (RM 0 to RM 5.9).

b. Flood Risk. Most of the flood risk is concentrated around the Puyallup River near Tacoma, the Puyallup River near Orting, and the White River near the town of Pacific. The rivers are levied through these communities. Levees have significantly confined the river, reducing its morphological footprint.



Case Study 2A Figure 1. Puyallup-White River basin area and SSWP reaches

c. USACE Nexus to PWRB. Multiple projects provide a link to sediment studies in the basin as discussed in the following sections.

(1) Large-scale Federal infrastructure projects involving significant taxpayer investments are in the construction phase (White River Diversion Facility replacement), or in the O&M phase (Mud Mountain Dam, Puyallup Federal Levees) in the PWRB. In addition, Pierce County and King County operate and maintain levees that are eligible for cost-shared emergency repair and rehabilitation assistance through Public Law 84-99 which provides for disaster preparedness, response, and recovery.

(2) Several major ecosystem restoration projects have also been constructed by the counties in recent years to improve salmonids habitat in the basin, and this effort is expected to continue. Critical port infrastructure (Tacoma Harbor, adjacent waterways) maintained by USACE are also impacted by sediment loading from the PWRB.

(3) In the upper watershed of the White River, Mud Mountain Dam (MMD) is located near Buckley, Washington, at RM 29.6. USACE operates MMD to manage flood risk along the lower 10 miles of the Puyallup River and the White River. The primary objective of MMD is to limit Puyallup River peak flows to less than 50,000 cubic feet per second (cfs). A secondary objective is to limit flows on the lower White River to less than 12,000 cfs.

(4) Currently, an approved Water Control Manual deviation request authorizes lower releases due to deposition that has reduced channel capacity along the lower White River at Pacific. MMD reduces peak flows on the White River during flood events until the peak on the Lower Puyallup River has passed. The MMD pool is then immediately evacuated to regain capacity for additional storms. The remainder of the basin is unregulated with only small hydropower diversions.

(5) All of these large-scale projects require some knowledge of, accounting for, or adaptation to the local or basin scale sediment erosion, transport, and depositional processes (referred to in this document as the "sediment regime") to achieve their authorized purposes. Because considerable effort, skill, and complexity are associated with understanding the sediment regime, work performed to date has focused primarily on problem areas, with more limited consideration for how data collected during these studies will be used by others in the future. This situation is not unique to the Puyallup Basin.

(6) To maximize the value of existing and future Federal investments related to sedimentation studies, USACE recommends that an SSWP be developed. Because several efforts are planned or underway that involve sediment data collection or studies, the Seattle District is following the recommendations of USACE's Channel Stabilization Committee (now referred to as the Committee on River Engineering) to develop an SSWP for the Puyallup River basin.

(7) The recommendations presented in this SSWP are based primarily on national guidance, best practices, and expectations of technological advancement. The staged sediment studies concept as outlined in this document, in which the complexity of a sediment study gradually increases from an impact assessment to a detailed investigation (in concert with the project phase), will be followed.

(8) Previous applicable studies by the USACE Seattle District include:

(a) Mud Mountain Dam: Sedimentation Surveys. Draft Summary Report for Water Year 2011 (2011).

(b) Memorandum for Record, Recommended Modifications to Mud Mountain Dam Reservoir Capacity Table (2012).

(c) Technical Memorandum Documenting Investigation of Mud Mountain Dam Modified Operations for Sediment management in the White River, Draft (2016).

(d) Technical Memorandum, Flood Inundation Mapping of the Lower White River (2017).

(e) Mud Mountain Dam Lahar Risk Management Plan (2018).

3. Current Problems.

There are significant challenges in the Puyallup-White basin for USACE, including long term institutional efforts, geologic conditions, and high sediment loads.

a. USACE Institutional Challenge. Given USACE's mission as an action agency, historical efforts to understand Puyallup-White basin sediment loads (by USACE) have primarily focused on localized conditions as problems arise, with less concern for maintaining continuity with future sediment data collection efforts in the basin (by USACE and others). Given the technical complexity of sediment studies, significant effort is expended to understand historical efforts when new studies arise, and oftentimes, when data is collected it is often done so with little consideration for how it might be used by others in the future. Combined, these missteps have resulted in gradual loss of valuable data and expertise, perpetuating the problem.

b. Geologic Challenge. The second major challenge is that the Puyallup-White basin is strongly influenced by active glaciers on Mount Rainier, one of the largest and highest stratovolcanoes in the continental U.S. Mount Rainier and surrounding areas supply large and highly variable sediment loads to the Puyallup basin and the study area, including substantial, very coarse bedload from sporadic rock falls at the glacial terminus. Transport of glacial material into the fluvial system is highly dependent on extreme rainfall events (USGS 2010). Sediment residence time, from glacial origins to the basin study area, can be on the order of decades to centuries (USGS 2010). A significant amount of sediment can also be produced from within the National Park boundaries.

c. Mudflows. Catastrophic but infrequent mudflows (lahars) originating from Mt. Rainier have had significant influence on basin-scale morphologic conditions downstream from Mt. Rainier, however, none of these events are in the historic record. Reoccurrence of a low-probability, moderate- to large-magnitude lahar poses significant concerns for the continued operation of USACE infrastructure in the basin (USACE 2018a). Monitoring that supports sediment studies increases situational awareness of lahar risks and helps improve emergency management planning and resilience.

d. Estimated Sediment Load. The USGS (2012) estimated historic annual sediment load between 860,000 and 1,200,000 tons/yr on the lower Puyallup. They estimated that around 500,000 tons/yr of that comes from the White River, with the remainder coming from the Upper Puyallup and Carbon Rivers (USGS 2012). High sediment loads will continue to characterize future conditions in the project study area.

e. Future Sediment Load. Climate change may even increase long-term sediment loads as glaciers recede, exposing new high-gradient sources. Prediction of future climate change in Washington State has included an 11.6% increase in runoff for 2010 to 2034, and an 18.1% increase in runoff for 2035 to 2059 (USGS 2012). An increase in runoff in the upper basin will transport more sediment from Mt. Rainier to leveed areas of the lower basin. The USGS has estimated that bedload in these rivers may increase on the order of 30% to 50% with increased flows (USGS 2012).

4. Sediment Studies Work Plan Purpose and Objectives.

a. Purpose. This SSWP identifies known PWRB sediment datasets, studies, and data gaps focused on river reaches with Federal projects (100% Federal or cost-shared). It compiles best practices for data collection, data storage, and quality control. It identifies collaborators and stakeholders for cost and data sharing. It provides recommendations for prioritizing data collection efforts and studies to provide water resource managers and decision-makers with timely and useful data to support existing and future efforts.

b. Goals. The main goal of the SSWP is to focus and streamline sediment data collection efforts and studies to maximize their existing and future value related to Federal actions by USACE in the Puyallup basin, with specific emphasis on the White River and Lower Puyallup River.

c. Objectives. The objectives of the SSWP are to:

(1) Establish a vision for success (ideal conditions for funding, data collection, data analysis, data sharing, data maintenance) for this work plan.

(2) Provide a clear basis for why a sediment studies work plan is needed to managers and planners inside and external to USACE.

(3) Develop a schedule, specifications, and cost estimate for data collection and model updates to support detailed feature design of flood damage reduction projects as part of the Puyallup General Investigation (GI) study.

(4) Develop a schedule for data collection and model updates to support related efforts by others throughout the basin.

(5) Maintain institutional knowledge and avoid unnecessary duplication of effort.

(6) Foster collaboration with local and regional experts to identify problems, opportunities, data gaps, current practices, best practices for sediment data collection, and modeling.

(7) Foster collaboration with stakeholders to ensure that their needs and interests are captured to craft a plan that is funded and implemented.

d. Desired End State. The desired impact of the SSWP is a plan, schedule, and budget to collect and support sediment data and analysis work in the PWRB for the next 10 years (2019–2029) that incorporates and leverages existing knowledge to increase the value and confidence of rapid low-scope analysis, and reduces the costs and increases the quality of higher scope/long-term efforts by minimizing duplication of effort and maximizing use of existing data and models and best practices. In addition, by identifying stakeholders and their interest in sediment data collection, cost and data sharing between agencies can be promoted.

5. Puyallup-White River Basin Sediment Studies Work Plan Components.

a. Components of the PWRB SSWP include the following:

(1) Background information on USACE activities in the basin, and purpose, need, goals and objectives of the SSWP (paragraphs 1, 2, and 3).

(2) Paragraphs 4 and 5 present that proposed components of the Puyallup-White basin SSWP, including the desired end state, execution strategy, and recommended activities by level of investigation. Proposed data collection efforts are provided by sub-basin, location, and frequency. Data collection efforts that are likely to be conducted by USACE are summarized, as are data collection efforts that could be cost shared with partners.

(3) Technical Exhibit 1 presents a proposed schedule and budget for SSWP activities by sub-reach between fiscal year (FY) 2019 and FY 2029. Technical Exhibit 2 provides a Puyallup Basin Reach Atlas Overview Map with Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) model sub-basins. Technical Exhibit 3 provides longitudinal profiles of the Puyallup River and major tributaries from the Puyallup Basin Corps Water Management System (CWMS) HEC-RAS (River Analysis System) model. Technical Exhibit 4 provides a table of surveyed cross-section locations.

b. SSWP Execution Strategy. Because the problems related to sediment are longstanding concerns in the PWRB, much effort has been expended to collect and analyze data supporting efforts to address or manage these issues. As such, this SSWP is primarily focused on identifying which historical data collection efforts should continue to inform status and trend questions (bed elevation, bed composition, general geomorphic conditions, changes in channel capacity, reach scale morphology and sediment transport, forecasted changes due to proposed channel modification efforts or watershed changes), and at what frequency these efforts should be conducted.

(1) In the case of the Puyallup-White system, a wide range of projects are undergoing planning and design for implementation and many more have already been implemented and are being operated (in the O&M phase). Thus, the purpose and need of most of the data collection and analysis products can be expected to support the needs and objectives of a wide range of projects and multiple stakeholders. For example, in the County Line reach of the Lower White River, Level I (qualitative geomorphic investigation based on existing data), Level II (quantitative geomorphic analysis based on new data), Level III (modeling analysis at reach scale), Level IV (modeling specific to a project under design), and Level V (monitoring) studies are underway concurrently.

(2) Due to the large number of stakeholders and efforts underway, there is diminished expectation that these efforts will conform to USACE SSWP and execution process. However,

by capturing the work done to date in the SSWP, USACE and its partner agencies gain knowledge on the status of the available studies and datasets, which increases the value and confidence of a USACE Level I analysis, for example, and reduces the costs and increases the quality of Level II–V analysis by minimizing duplication of effort and maximizing the use of existing data and models.

(3) The data collection schedules presented in this SSWP have not been vetted with external stakeholders and may reflect unrealistic assumptions related to funding. Given the large number of agencies involved in the basin with overlapping interests, it is clear that partnering and cost-sharing opportunities exist. The schedules are thus recommendations to support the wide range of scope and geographic extent of identified projects.

c. Level I Investigations for USACE Puyallup-White Sediment Management Projects.

(1) Applicability. The main objective of any Level I analysis is to develop an initial understanding of the nature and magnitude of a problem. As such, Level I analysis relates primarily to new projects or new issues. It is foreseeable that projects resulting from the Puyallup GI study (PED phase) or a new project study would usually require Level I analysis to better define the problems using available data. While a quantitative basin-wide sediment budget is not presently available, qualitative assessments by others that have identified aggrading, degrading, or stable reaches. These datasets and studies should be consulted as a first step in understanding reach scale conditions prior to undertaking detailed analysis (Level II or III).

(2) Qualitative Analysis of Existing Conditions and Potential Project Effects. All Level I analysis efforts should be focused on confirming the presence of a problem, beginning the process of understanding the magnitude of the issue, and considering a range of potential remedies and their qualitative impacts on the problem(s).

(a) The reference documents cited in this SSWP should be consulted before initiating any significant analysis as part of a Level I investigation. Wherever possible, adhere to the methods used previously by others to study a given reach to aid in trend analysis. Standardized data collection and reconnaissance forms should be developed and used.

(b) The need for Level I analysis is high, but the frequency that this will occur is unpredictable since it is related to new issues and new projects. Level I analysis is not included in the SSWP schedules since it is assumed to be conducted as part of any project effort. The list below provides a sample of items commonly found in a Level I Investigation Checklist:

- Project goals identified.
- Problem identified with possible remedies or mitigation (several options).
- Literature review/state of knowledge.
- Site reconnaissance and rapid data collection with standardized forms.

- If needed, geomorphic summary using available data (reconnaissance photos, LiDAR, aerial imagery, existing reports).

- Hydraulic summary using available hydrologic data, if needed (no modeling, just data outputs).

- Purpose and need for higher level analysis.

- Documentation with a technical memo or brief report.

(c) While EM 1110-2-4000 recommends developing a qualitative sediment budget for a Level I analysis, because the Puyallup-White basin geomorphic trends have largely been established by others (USGS 2010, 2012) the need for a sediment budget as part of a Level I analysis is limited, however current reach scale trends with respect to previous findings should be acknowledged.

d. Level II and III Investigations for Pending Projects.

(1) Applicability. Level II and III investigations result from Level I investigation finding that quantitative analysis and new data collection are necessary to meet the project objectives or to buy down risks to an acceptable level. This analysis builds on and verifies data and findings from the Level 1 investigation. Refer to EM 1110-2-4000 for guidance for performing Level I investigations.

(2) Analytical Analysis of Existing Conditions and Potential Project Effects. Sediment investigations conducted as part of a Level II and III investigation are typically complex and quantitative, relying on new datasets. The types of questions that these investigations can answer are:

(a) Status and trend of a river reach (geomorphic behavior over time, habitat availability).

(b) Channel capacity of a river reach.

(c) Long-term predictions of river conditions (sediment transport modeling).

(d) Documentation with a detailed report.

(3) The primary distinction between Level II and III analysis is the greater emphasis on quantitative analysis, advanced methods, numerical modeling, or more intensive data collection for Level III analysis. Refer to EM 1110-2-4000 for guidance for performing Level II and III investigations.

e. Level IV Investigations for Pending Projects. Level IV investigations are conducted as part of specific design efforts. These are not addressed as part of this SSWP. Refer to EM 1110-2-4000 for recommended sediment study guidance.

f. Level V Investigations for Completed Projects.

(1) Applicability. Level V investigations are conducted during the operations and maintenance and monitoring phase of a completed project. For the PWRB, this primarily includes Mud Mountain Dam, the White River Barrier (diversion) Dam at Buckley, the Lower Puyallup River Federal Levees, and the Tacoma Harbor.

(2) Analytical Analysis of Existing Conditions and Potential Project Effects. Sediment investigations conducted as part of a Level V investigation are closest in complexity to a Level I or Level II analysis. Sediment or geomorphic monitoring data that are typically collected as part of a Level V analysis include:

(a) Repeat channel surveys.

(b) Repeat aerial photography.

(c) Repeat sediment range surveys and reservoir capacity curve updates.

(d) Surface and sub-surface sediment grain size data.

(e) Suspended-sediment or bedload data collection.

(f) Documentation with technical memorandum or brief report.

(g) Refer to EM 1110-2-4000 for guidance for performing Level V investigations.

6. Proposed SSWP Data Collection and Work Products.

a. Overview. The following data types, locations, and collection frequency are proposed, and many of these do not have identified funding sources.

b. Reservoir Sediment Range Re-Occupation. USACE routinely monitors reservoirs for capacity losses related to sedimentation. The traditional approach is to resurvey permanent transects called sediment ranges. Mud Mountain Dam has 65 permanent sediment ranges that have been resurveyed at least eight times since dam completion. During the most recent survey of reservoir capacity, bare-earth LiDAR data was used due to the difficulty of using terrestrial land survey methods in the reservoir. This method will likely continue to be used in future monitoring efforts due to its high accuracy and relatively low cost, however, it does not capture bathymetric data for the low-flow channel. Surveying sediment ranges by boat during a pool used for debris operations is another method that can be used to speed up data collection in the low-flow channel.

c. Streambed Elevation Surveys.

(1) At present, USACE does not anticipate needing to conduct streambed elevation surveys (cross sections) at any reaches of the PWRB except for White River Reach 4 (reservoir),

but could benefit from work by others, especially in the dynamic aggrading Reach 2 of the Lower White River (W2, Figure 1) as channel capacity in this reach potentially affects Mud Mountain Dam releases.

(2) At present, King County has a robust monitoring program and has been sharing channel survey data with USACE to support its channel capacity and inundation mapping efforts for the Lower White River (W1 and W2). In addition, channel surveys at the new White River barrier dam near Buckley may become a routine occurrence if they are required to document channel maintenance activities related to operations and maintenance of the dam and fish facilities.

(3) The following information is provided for general guidance and requirements for channel surveys to support these efforts.

(a) Channel bed elevation surveys through individual cross sections or bathymetric surveys by boat provide information for quantitative trend analysis and flow conveyance capacity. The frequency of cross-section data collection should be tied to what is at risk (from channel changes) and how rapidly a reach is changing, in addition to vertical accuracy requirements for products that leverage the channel survey data. Generally, steeper channels require more cross sections surveyed to maintain vertical accuracy. Established cross sections for repeat surveys in each of the sub-reaches are identified in Exhibit 4. Re-survey efforts for the specific sub-reaches is shown in Table 1 and Exhibit 1.

(b) Universal requirements that include (1) tying the measurements to the North American Vertical Datum of 1988 (NAVD 88) vertical datum and (2) maintaining a minimum vertical accuracy of 0.3 feet should be established for any topographic survey. Site conditions dictate which survey methods are most advantageous. It is recommended that permanent section endpoints be established and maintained in reaches that are heavily monitored if terrestrial survey methods are preferred. RTK or RTN (Real Time Kinematic or Network) GPS tied to single-beam sonar are likely to be most advantageous for rapid data collection in non-wadeable reaches, and for terrestrial surveys in wide, highly visible areas.

(c) Cross-section frequency (spatially) should be tied to channel gradient, vertical accuracy requirements for modeling or change analysis efforts, and risk. Cross-section resurvey intervals should be based on the schedule/need for quantitative change analysis (volumetric trend and/or modeling).

(d) Cross-section surveys not performed in conjunction with LiDAR data acquisition should extend to the top of the bank to a fixed reference point for quality control purposes and to aid in volumetric comparisons. Cross sections performed at/near time of LiDAR acquisition can focus on bathymetric data collection, however, the locations surveyed should remain consistent with established survey lines. Note that King County has a well-regimented survey program on the Lower White (2-year interval) that can be used as a model for other data collection efforts.

(e) Establishing regular cross-section survey frequency requires dedicated funding sources from local agencies and cost-share sponsors. Currently, only the White River in King County between Sumner and Auburn (Reaches 1 and 2) is regularly surveyed. Due to the critical nature of the aggradation problem, this investment is warranted.

(f) Other reaches with lesser channel capacity concerns may require less frequent survey, but because of the high sediment loads of the basin and expectation of multi-decade benefits from flood damage and ecosystem restoration projects, resurvey is necessary to keep track of sediment trends. The Lower Puyallup Reach 1 has had channel capacity issues in the past and needs to be periodically monitored to verify channel capacity.

d. LiDAR Surveys. LiDAR surveys can be conducted for the different sub-reaches within the PWRB at the intervals shown in Table 1. As of 2017, there is nearly continuous coverage of the study area by LiDAR datasets, however, the quality of the data varies significantly by vintage. Regular data collection interval of approximately 5 years is recommended to anticipate the need of the basin-wide efforts underway.

(1) While LiDAR is known to be a significant expense, both the LiDAR data collection frequency and quality have increased in the last decade. These increases are due to recognizing that high-quality LiDAR can improve knowledge of geomorphic conditions and changes, and the quality of hydraulic modeling efforts. Costs are coming down as new sensors are increasing the point density, improving the vegetation filtering, and improving the vertical accuracy of bare earth models.

(2) Just as significantly, recent advances in green laser LiDAR now allow simultaneous above- and below-water data collection, and have been used with success by Seattle District on the Middle Green River and Skokomish River. However, the technology performs poorly under less than ideal conditions, such as those experienced during times of high sediment concentration when the glaciers are melting. Fortunately, there may be a narrow window in March when the Puyallup-White basin sediment concentrations reach minimum levels and water levels are much lower than average.

(3) There is a depth limit with this technique of about 2 meters so the lower White and Puyallup rivers (and deep pools) need to be surveyed with other methods. A significant advantage of this approach is the speed at which data can be collected, creation of a combined topographic/bathymetric digital elevation model (DEM), and reduced exposure for ground survey crews (safety).

(4) Without conducting a test data acquisition, it is not possible to determine if this technology can be successfully applied to the glacially fed reaches of the Puyallup-White basin. The MMD reservoir reach is a good test case, as is the canyon reach between Auburn and the dam due to their poor accessibility and infrequent survey. If the data acquisition is able to capture riffle sections with confidence, this method is suitable for data necessary to perform flood and sediment transport modeling in most reaches. All LiDAR products should be collected

while adhering to USGS LiDAR Base Specifications v1.3 at Quality Level 2 or better (USGS 2018), which corresponds to a minimum DEM cell size of 1 meter.

e. Photogrammetric Surveys.

(1) Photogrammetric surveys and Structure from Motion (SfM) analyses are useful for areas that lack vegetation and therefore are limited to exposed channel surfaces and high elevation regions of the PWRB. The accuracy, precision, and resolution all directly depend on photo acquisition, ground control, and processing methods. However, recent surveys of the proglacial zone of Mt. Rainier have yielded 0.5 meter DEMs with precision and accuracy (relative to 2008 LiDAR) of ±0.3 meter.

(2) Photogrammetric surveys can therefore be employed as an alternative or supplement to LiDAR surveys for river reaches with exposed (non-vegetated) active channels (such as W2, W3, W5, P3, and C1) and can occur over shorter time intervals (2–3 years relative to 5–10-year intervals, Table 1) given the reduced cost of this data acquisition relative to LiDAR.

f. Bed Sediment Data Collection.

(1) The bed sediment data collection locations from the USGS studies (2010, 1988) shown in Appendix B should be adhered to, at minimum, to inform modeling efforts and habitat change analysis at a frequency identified by Table 1 for each sub-reach.

(2) However, frequency of bed sediment data collection should include considerations on whether the locations capture major changes in grain size and thus are representative of current channel conditions. Project-specific studies should anticipate collecting more samples than reach scale modeling may necessitate. A mixture of surface grain size and sub-surface bulk samples are ideal.

(3) Use of image-based grain size analysis methods (Buscombe 2010; Buscombe and Rubin 2012; FHWA 2014) may be acceptable for low-risk reconnaissance efforts but should be coupled with measurements made with traditional approaches to develop correction factors if the data will be compared with historical measurements or used in modeling.

g. Sediment Discharge Data Collection.

(1) At present, the most likely locations where USACE would be interested in sediment discharge data collection are locations where this work has been conducted historically to support USACE operating projects: at the upstream and downstream ends of the Mud Mountain Dam reservoir (White River below Clearwater River, White River near Buckley, White River at Headworks above Flume). USACE has also been historically interested or directly involved in sediment discharge data collection efforts that inform channel capacity studies on the Lower White and Lower Puyallup Rivers.

(2) At present, USACE is primarily interested in data that will potentially impact reservoir operations. If channel condition change is likely to change significantly in the Lower White and

Lower Puyallup in the coming years, USACE would be more likely to participate in sediment discharge data collection activities, directly or as a cost-share partner.

(3) For the PWRB, ideal locations for sediment discharge data collection are co-located with a USGS streamgage station and occur at a bridge crossing to allow for multi-vertical sampling of either suspended-sediment or bedload using a reel and crane. These data are coupled with continuously operating surrogate sensors (turbidity, hydrophones, or simply discharge) to develop sediment discharge estimates.

(4) Suspended-sediment sampling should be conducted using isokinetic samplers that allow a water discharge-weighted sample that is representative of sediment traveling in suspension throughout the entire water column. In some cases, suspended-sediment sampling can occur at a cableway or by boat, although this is substantially more challenging.

(5) Sampling access from the bridge must provide sufficient space on the bridge to safely accommodate the crane and field personnel. Wide pedestrian walkways and shoulders are typically sufficient. In other cases where insufficient space is not available, options that include temporary closure of the bridge to traffic or using flaggers to manage traffic over the bridge are required.

(6) At a minimum, proposed locations for suspended-sediment monitoring include established USGS streamgage stations that capture the upstream and downstream regions of the river for the Puyallup River and White River. One location in the Carbon River is likely sufficient unless sediment issues develop in this river in the future that require more intensive monitoring. Proposed USGS stations for suspended-sediment monitoring are identified in Table 2.

(7) In addition, select locations for suspended-sediment monitoring could be implemented for specific, constrained study objectives. These locations could include sediment monitoring at the sub-reach scale. In particular, a local water purveyor monitors turbidity at intakes and returns associated with Lake Tapps, presumably as a water quality measure (such as USGS 12098700, 12101100, and 12101102). There is a potential that future concerns will result in estimating suspended-sediment transport at these locations.

(8) Bedload measurements are logistically more challenging, requiring more field personnel during the sampling period, typically three to four people instead of one or two for suspended-sediment sampling. Bedload measurements require a bridge crossing, are not feasible from a cable, and are extremely difficult by boat. In addition, while there are a variety of suspended-sediment samplers available to accommodate the range of sampling conditions at a location, the variety of bedload samplers is limited to 2 to 3 samplers at the date of this report.

(9) Current bedload samplers have had varied success at different sampling locations. In particular, under some field conditions, high streamflow velocities can prevent the bedload samplers from submerging and resting on the riverbed.

(a) Within the Puyallup River Basin, bedload sampling is feasible at the following locations, which are also identified in Table 2:

- Puyallup River near Orting, 12193500.

- White River at R Street, 12100490: Acoustic hydrophones for bedload monitoring were deployed for the 2018 water year.

(b) Potential new sites for bedload monitoring could include:

- Puyallup River near Puyallup, 12101500.

- White River Diversion Facility, although this would require additional study to determine feasibility (see text below).

(10) For purposes of measuring coarse sediment transport (bedload), note that the White River Diversion Facility flushes bed material regularly from the facility.

(a) Concerns by operators over where sediment accumulates and how to best bypass it are significant. Discussions with operations staff and designers on ways to measure sediment in the fore bay of the new barrier dam should be pursued at the earliest convenience, as this will provide a means to directly measure coarse sediment transport at an engineered constriction.

(b) If done in conjunction with fine sediment load measurements, this would provide a total load estimate upstream of the main area of concern (White River Reach 2). Since bedload deposition is the primary source of channel capacity change downstream, tracking coarse bed material flux through the facility may be beneficial in its own right.

h. Proposed Data Collection Activities Summary (Regardless of Funding Source). Table 1 and Table 3 provide a summary of recommended data collection and studies by reach. The data collection is primarily modeled after recent studies performed by the USGS (2010, 2012), USACE (2017) and King County (2017). The locations for data collection are generally the same as locations where data has been collected in the past, however, a limited number of new data collection locations are proposed where upcoming projects are occurring or where data gaps exist.

i. Proposed Technical Analysis Activities Summary. The proposed technical analysis activities are summarized as:

(1) Table 3 identifies the three major types of analyses expected to be conducted in the coming decades to establish status and trends of channel capacity and flood risk, habitat and geomorphic conditions, and sediment budgets.

(2) Previous studies should be used as templates for future efforts.

(3) For channel capacity and flood risk studies, it is reasonable to assume that updating the USACE GI study hydraulic model cross-section data is required in reaches that are experiencing ongoing channel change. This work can be completed in a relatively efficient manner provided that new survey data is available.

(4) Sediment transport modeling should be updated for any large-scale projects that will theoretically influence the upstream or downstream sediment loads over a long time period. Examples include the County Line levee setback project near the town of Pacific.

(5) In the lower Puyallup and White Rivers, periodic recalibration and updates to the sediment transport models should be considered to establish current trends to aid long-term planning.

Case Study 2A Table 1
Proposed Sediment Data Collection Activities in the Puyallup Basin

Basin	Sub-Reach	Geographic Extents	Cross section surveys		LiDAR	Sediment Size Data		Sediment Discharge Measurements
			#/mile	Resurvey Interval (years)	Resurvey Interval (years)	#/mile	Resurvey Interval (years)	Resurvey Interval (years)
Puyallup	**P1 Lower**	**Comm. Bay to White R**	**5**	**2**	**5**	**1**	**5**	**1***
	P2 Middle	**White R to Carbon R**	**7**	**5**	**5**	**3**	**5**	**5**
	P3 Upper	Above Carbon to above Champion Bridge	7	5	5	1	5	**10**
White	**W1 Lower**	**Puy R to Dieringer canal**	**10**	**2**	**2**	**1**	**5**	**5**
	W2 Lower	**Dieringer to Auburn Dam**	**10**	**1**	**2**	**3**	**2**	**1**
	W3 Middle	Auburn Dam to MMD	5	5	5	1	5	5
	W4 Upper	**Reservoir**	**7**	**10**	**5****	**1**	5	>10, <20
	W5 Upper	Above Reservoir	5	10	5	3	5	>10, <20
Carbon	C1 Lower	Puy R to above S. Prairie Cr	7	>10, <20	5	3	5	>5, <10
COLOR KEY: Likely funding source/proponent			**USACE**	**Pierce County**	**King County**			
			City of Sumner	City of Orting	unidentified			

*Long term suspended-sediment monitoring could occur at USGS streamgage station, Puyallup River at Puyallup (ID 12101500) with limited annual suspended-sediment sampling (for example, ~4–6 sampling annually) to maintain existing rating curve.
**Reservoir could be surveyed with photogrammetric imagery during summer baseflow conditions, which is less expensive than LiDAR but equivalent resolution, for topographic differencing analysis.

Case Study 2A Table 2
Identified USGS Streamflow Gaging Stations for Sediment Discharge Monitoring

		Sediment Discharge	
Reach	**Sub-Reach**	**USGS streamgage station**	**Notes**
P1 Lower	Comm. Bay to White R	Puyallup near Puyallup, 12101500	Established site for SSC, unknown feasibility of bedload.
P2 Middle	White R to Carbon R	Puyallup near Orting, 12093500	Established site for SSC and bedload measurements.
P3 Upper	Above Carbon to above Champion Bridge		
W1 Lower	Puy R to Deiringer canal	Proposed site: White R at 24th St E Bridge	Unknown feasibility for SSC and bedload measurements but LOCAL WATER PURVEYOR was interested in turbidity in White River downstream of Dieringer Canal.
W2 Lower	Dieringer canal to Auburn Dam	White at R St, 12100490	Established site for SSC and bedload measurements
W3 Middle	Auburn Dam to MMD	White R at Buckley	**Option** 1: LOCAL WATER PURVEYOR operates two different stations in close proximity. 12098700 is just water quality, including turbidity; 12099200 is discharge. Unfortunately, this site lacks a good bridge for SSC and bedload sampling. Nearest bridge is SR410 ~1 mile downstream, with no sidewalks. **Option 2**: Suspended and bedload monitoring system is designed for new Diversion Dam.
W4 Upper	Reservoir		
W5 Upper	Above Reservoir	White River below Clearwater River near Buckley, 12097850	Reach lacks bridges for sediment sampling.
C1 Lower	Puy R to above S. Prairie Cr	Carbon near Fairfax, 12094000*	USGS 12094300 is location for bedload measurement, but is not a USGS gage.

Case Study 2A Table 3
Proposed Sediment Analyses in the Puyallup Basin

Basin	Reach	Sub-Reach	Channel Capacity Flows & Inundation Mapping	Geomorphic Change Analysis & Status	Sediment Transport Model Updates
			Analysis Interval (years)	Analysis Interval (years)	Analysis Interval (years)
Puyallup	P1 Lower	Comm. Bay to White R	2	10	5
	P2 Middle	White R to Carbon R	5	5	5
	P3 Upper	Above Carbon to above Champion Bridge	5	5	10
White	W1 Lower	Puy R to Deiringer canal	1	5	2
	W2 Lower	Deiringer canal to Auburn Dam	1	5	2
	W3 Middle	Auburn Dam to MMD	10	5	5
	W4 Upper	Reservoir	10	10	10
	W5 Upper	Above Reservoir	>10; <20	10	>10; <20
Carbon	C1 Lower	Puy R to above S. Prairie Cr	>10; <20	>10; <20	>10; <20

7. USACE-Funded Sediment Studies Work Plan Data Collection and Work Products.

a. Overview. Operations and maintenance of Mud Mountain Dam and White River Diversion Facility constitute the primary funding source for sediment data collection by USACE in the Puyallup-White basin. Data that directly informs day-to-day operations, operational planning, or infrastructure rehabilitation projects are the most likely to be funded directly by USACE.

b. Activities.

(1) Reservoir Survey Plan. The Seattle District is required to routinely estimate the reservoir capacity at Mud Mountain Dam by re-occupation of established sediment ranges. Because of the run of river, operations and low-level sediment outlet coarse bedload is readily passed through the reservoir; however, some sediment accumulation is occurring during flood pools. The deposited sediments are primary fine-grained and mobilized by winter rainfall from the canyon walls in the reservoir. Some large deposits of coarse material that occurred in the mid-2000s have become vegetated with trees that can withstand periodic inundation.

(a) LiDAR surveys in the late winter when flows are at minimum levels or late summer are likely to be more cost effective than topographic surveys. The surveys should extend upstream to RM 35 to capture the reservoir backwater influence. The geographic extent of a LiDAR survey is about 2 square miles. At 2018 market prices, a reservoir LiDAR flight would cost about $30,000, using the average unit cost of $15,000 per square mile (per recent Seattle District project costs). Photogrammetric methods are not recommended due to the inability to filter out vegetation, which could bias the elevation data.

(b) If surveys are scheduled for February or March prior to onset of glacial melt, water clarity is likely to be good enough to recommend using topobathymetric LiDAR. This has the advantage of costing marginally more than terrestrial LiDAR, allows data collection during the leaf-off period, and would eliminate the need for costly low-flow channel sediment range surveys.

(c) The existing reservoir survey frequency is about once every 9 years. This is likely to be adequate for the foreseeable future for assessing reservoir capacity changes. Since the last surveys were conducted in 2011, the next scheduled survey is proposed for 2020. Comparison to historical data should be conducted at historical range lines to establish trends. Note that the frequency may need to be increased in response to large floods (on the order of November 2006 or February 1996) if there is evidence of significant geomorphic/volumetric changes.

(2) White River Barrier Dam Topographic Survey and Sediment Monitoring Plan.

(a) A turbidity monitoring station is in operation at the Cascade Water Alliance upstream of the barrier dam. It is assumed that this station will remain in operation following completion of the barrier dam replacement. This station can be used as a surrogate for suspended-sediment concentrations if measurements can be obtained over a range of flow events. This would allow establishment of the suspended-sediment load at the weir. Because this location is being modified as part of the barrier dam replacement project, it is expected that the river will become more constricted, and large quantities of coarse sediment will be trapped and sluiced routinely.

(b) If the quantity of bed material sluiced can be measured, and the suspended-sediment concentrations measured, a nearly complete estimate of the total amount and type of sediment passing by the barrier dam could be made, which could provide valuable insights in changes in sediment loads and potential downstream impacts. This data would also provide a direct measure of the amount and size of bed material passing through the 9-foot tunnel from year to year.

(c) At present, there is no sediment monitoring plan available to guide this work. It is assumed that operators will make real-time decisions based on current conditions and operational needs.

(d) It is recommended that a sediment monitoring program be initiated, focused on feeding real-time data to operations staff and tracking bedload flux through the facility, if technically feasible. This could potentially identify when and where sediment is building up prior to it becoming problematic, to help operators make better informed decisions instead of reacting to changing conditions.

(e) Due to the presence of floating debris, bed material passage, and fluctuating water levels, non-contact radar sensors are the most cost effective for tracking water surface changes. Methods for detecting bed elevations below the water surface include direct measurements (soundings), indirect measurements (bathymetric LiDAR, sonar) and indirect measurements combined with numerical modeling (inverse Computational Fluid Dynamics (CFD models, see Nelson et al., 2012).

(f) The cost to operate and maintain a semi-automated sediment flux monitoring system has not been determined as part of this SSWP. The technical complexity is high, and challenges are somewhat novel but not unsurmountable. A team should be established in FY 2019 to investigate the most feasible way to monitor sediment flux at this station. Ideally, it should consist of operating project staff, reservoir managers, and sediment monitoring SMEs at USACE and the USGS. It is possible that downstream stakeholders may be interested in funding some or all of the sediment load measurements.

(3) Mud Mountain Dam Operational Studies. The channel capacity of the White River in Reach 2 is expected to be highly variable in coming years due to changes caused by completion of new flood risk management infrastructure, combined with the natural depositional environment.

(a) Updating the channel capacity modeling and sediment transport modeling (USACE 2016, USACE 2017c) at regular intervals (every 1 to 3 years) is recommended to provide near-term situational awareness to reservoir and emergency managers. Work completed by USACE in 2016 suggested that dam (outflow) hydrograph shaping and magnitude had an influence on downstream sedimentation (higher flows were associated with reduced sedimentation), however, this assumes stationarity in the sediment load-flow relationship at R Street. Thus, USACE has an interest in sediment load data on the Lower White River.

(b) Continuation of investigations on how Mud Mountain dam operations may be influencing sedimentation should be continued, as they may provide opportunities to proactively manage channel capacity. This is recommended if monitoring data shows that the reach between A Street and Stewart Road begins aggrading again at historical rates or if sediment loads in this reach have markedly changed. King County and USACE have been collaborating in recent years on the modeling portion of this effort. This collaboration should continue.

(c) The annual cost to complete this work has averaged about $25k per study in recent years. The study cost will likely decrease over time because the effort will primarily involve re-cutting model geometry where new survey exists and updating inundation maps. If King County takes over the model maintenance work, the costs may drop significantly.

8. Proposed Cost-Shared SSWP Data Collection and Work Products.

a. Overview. The benefit of the SSWP is that it identifies opportunities for cost sharing data collection and studies. At present, USACE is most likely interested in cost-sharing efforts that inform conditions in Puyallup Reach 1 and White River Reaches 1 to 4 that impact or could potentially impact Mud Mountain Dam or Buckley Diversion Dam operations and maintenance.

(1) It is assumed that any cost sharing for data collection would occur in the same manner that USACE funds the USGS for stream gaging, while cost sharing for studies need to be conducted as part of the Economy Act Task Order under the current USACE/USGS Interagency Agreement. The partnerships do not preclude participation by other agencies or Tribal Nations.

(2) It is also possible that USACE could conduct data collection and studies on behalf of Tribal Nations under the Tribal Partnerships program independently or in collaboration with others.

b. Activities. Identified activities are organized in a framework of sediment datasets outlined in Appendix B and include data collection and analysis for topographic change, sediment discharge, and river bed grain size.

(1) Basin-Wide Analysis of Topographic Change. An analysis of channel and floodplain topographic change in all sub-reaches should occur at 25-year intervals to maintain up-to-date information. However, topographic change analysis for specific sub-reaches should occur more frequently (Table 1).

(a) This analysis should use the available cross-section, LiDAR, and photogrammetry datasets that, ideally, are timed at regular intervals for analysis across similar time periods. Data collection frequency varies from annual to 10-year intervals, depending on the sub-reach (Table 1). The last relatively large-scale analysis was for data up to 2009 (USGS 2010, 2012). Therefore, 2034 should be the approximate timing for the next basin-wide topographic analysis and should make use of potential data collection efforts beginning in 2020 (Exhibit 1).

(b) King County conducts annual topographic monitoring of the lower White River (W1, W2). No other funding sources are identified to support topographic data collection and analysis in the basin.

(2) Suspended-Sediment Discharge Measurements. Measurements are needed for maintaining or updating relations for suspended-sediment load estimates in Puyallup and White Rivers. Measurement interval varies by river reach.

(a) Suspended-sediment load estimates should be updated at varying frequencies, depending on the location in the river basin, and ranges from 5 years to more than 10 years (Table 1) to appropriately capture the current sediment dynamics in these rivers. At select locations, specifically the lower Puyallup and White Rivers, long-term, ongoing suspended-sediment monitoring could be considered given that monitoring at these locations provides information on basin-wide sediment conditions that is relevant to all sub-reaches.

(b) To establish a baseline bedload-discharge relation with which to calculate a suspended-sediment load estimates requires a 2-year effort of continuous discharge and turbidity sampling and 8 to 12 discrete suspended-sediment concentration samples over a range of sediment transport events during the 2-year study. Once a baseline is created, maintenance of the suspended-sediment concentration-turbidity or discharge relation could occur with 3 to 5 samples over a year at regular survey intervals (Table 1).

(c) Because suspended-sediment sampling ideally should occur at a channel-spanning bridge, with appropriate hydraulic conditions at the channel cross section, and be co-located with a USGS streamflow gage, the following USGS gages are established locations where suspended-sediment sampling could occur.

- Puyallup River at Puyallup, 12101500 (P1 Lower).*
- Puyallup River at Orting, 12093500 (P2 Lower).
- White River at R Street, 12100490 (W2 Lower).*
- White River at headworks above flume near Buckley, 12098700 (W3 Middle).
- Carbon near Fairfax, 12094000 (C1 Lower).

– In parentheses is the river sub-reach represented by sampling at each location.

– *Indicates locations for which a suspended-sediment concentration relation with turbidity already exists. Therefore, the 2-year baseline is not required and includes only maintaining the existing relation with limited SSC sampling (3–5) across a range of flows.

– In the case of the Carbon River, the identified location is the site of established bedload sampling and is expected to be a feasible site for suspended-sediment sampling, but is downstream of the USGS streamgage.

(d) There is ongoing turbidity monitoring at the White River at headworks near Buckley (USGS 121098700) by a local water purveyor, but no accompanying SSC sampling. Therefore, SSC sampling to establish SSC-turbidity relation could be a timely, cost-effective approach for load estimates downstream MMD that is relevant to the middle and lower White River sub-reaches.

(e) Turbidity monitoring on the lower White River associated with Lake Tapps Diversion at Dieringer (USGS 12101100 and 12101102) is also funded by a local water purveyor and could be leveraged with SSC sampling at these locations to understand suspended-sediment loads in the lower White River, although this information could potentially be redundant to monitoring at the USGS White River at R Street (USGS 12100490).

(f) No other funding sources are identified to support turbidity or suspended-sediment data collection and sediment load analysis in the basin at the time of this report.

(3) Bedload Sediment Discharge Measurements. Measurements are needed for maintaining or updating relations for bedload estimates in Puyallup and White Rivers. The measurement interval varies by river reach.

(a) Bedload estimates should be updated at varying frequencies, depending on the location in the river basin, and ranges from 5 years to more than 10 years (Table 1) to appropriately capture the current sediment dynamics in these rivers. Similar to suspended-sediment, to establish a baseline bedload-discharge relation with which to calculate bedload estimates requires a 2-year effort of continuous discharge sampling and 8 to 12 discrete bedload samples over a range of bedload transport events during the 2-year study. Once a baseline is created, maintenance of the bedload-discharge relation could occur with 4 to 6 samples over a year at regular survey intervals (Table 1).

(b) Bedload sampling ideally should occur at a channel-spanning bridge with appropriate hydraulic conditions at the channel cross section, and be co-located with a USGS streamflow gage, so the following USGS gages are established locations where bedload sampling could occur.

- Puyallup River at Orting, 12093500 (P2 Middle and P3 Upper).
- White River at R Street, 12100490 (W2 Lower).
- Carbon near Fairfax, 12094000 (C1 Lower).
- Puyallup River at Puyallup, 12101500 (P1 Lower).

– Listed in parentheses is the river sub-reach represented by sampling at each location.

– In the case of the Carbon River, the established bedload sampling site is downstream of the USGS streamgage.

– Feasibility of bedload monitoring in the lower Puyallup River at USGS streamgage Puyallup River at Puyallup, 12101500, is unknown, but could be considered to understand the total bedload transiting to Commencement Bay.

– Additionally, bedload monitoring could be incorporated at the new White River Diversion Facility (not listed), although this requires additional study to determine feasibility and cost.

(c) See Exhibit 1 for estimated costs by reach for bedload sediment data collection. No funding sources are identified to support bedload data collection and sediment load analysis in the basin at the time of this report. The counties are the obvious beneficiaries of this data, as it informs sediment transport model and channel capacity model updates that directly inform capital project planning.

(4) Basin-Wide Grain Size Characterization. Measurements are needed basin-wide for grain size characterization. The measurement interval varies by river reach.

(a) Bed material characterization should be updated at varying frequencies, depending on the location in the river basin, and ranges from 2 years to more than 10 years (Table 1) to appropriately capture the current sediment dynamics in these rivers. Locations should adhere to areas of previous data collection (see Appendix B) to track potential change through time. Approaches include both surface and sub-surface samples. Image-based data collection and analysis (such as Buscombe 2010) should be limited to reconnaissance efforts until these methods are further developed and tested to standards that confidently inform modeling efforts.

(b) See Exhibit 1 for estimated costs by reach for grain size data collection. No funding sources are identified to support grain size data collection and analysis in the basin at the time of this report. This data primarily informs sediment transport model updates and geomorphic trends analysis (channel capacity change, bank erosion, etc.).

(5) Channel Capacity Model Updates and Inundation Mapping.

(a) Hydraulic models should be updated at varying frequencies, depending on the location in the river basin, and ranges from 2 years in the lower White River to 10 years in the upper White and Puyallup Rivers or longer, as for the Carbon River (Table 1). Updating models rely on current topographic and bathymetric data.

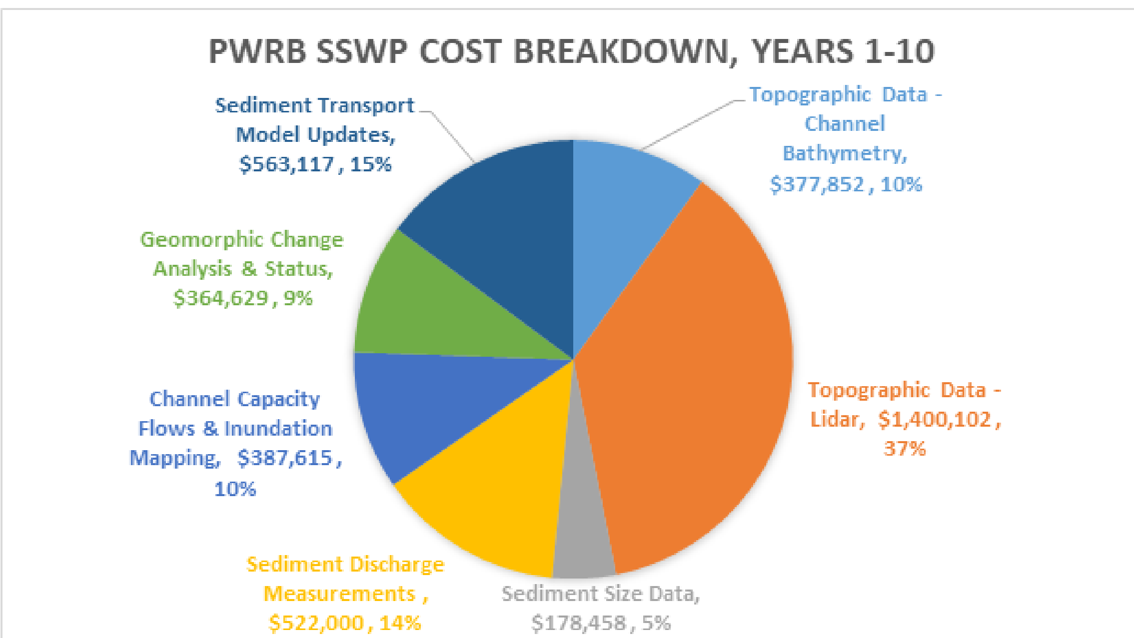
(b) No funding sources are identified to support updating hydraulic models in the basin at the time of this report, however, future maintenance of the PWRB CWMS model (Exhibits 2–4) are the primary beneficiary of this work and would be a logical funding source. See Exhibit 1 for estimated costs by reach for channel capacity and inundation map updates.

(6) PWRB SSWP Proposed Budget (2019–2029). The proposed SSWP budget for all activities is as follows.

(a) Exhibit 1 provides a schedule and budget by reach for all SSWP activities. Detailed budgets by activity are provided in Appendix A. All cost estimates are provided in 2018 dollars (non-escalated), and USACE's portion of the activity cost has not been determined. Note that no identified funding source is available to support these activities.

(b) Figure 2 shows that the primary activity by cost (37%) is topographic LiDAR. Because USACE missions are primarily interested in floodplain and channel data, USACE's cost share for this effort is likely much smaller. The counties are likely the primary cost-share partner for this work. The other activities inform study-specific questions, such as status and trends of geomorphic conditions and channel capacity or reservoir volume changes. USACE cost share for these activities would likely be higher.

(c) There are dependencies with the data collection activities. The sediment size data, channel bathymetry, LiDAR topography, and sediment discharge measurements are needed to provide accurate sediment transport model updates and predictions of future channel capacity changes, while only topographic and bathymetric data are needed to provide near-real-time channel capacity estimates.



Case Study 2A Figure 2. Cost breakdown of proposed sediment data collection and analysis activities, PWRB

9. Data Collection and Work Products by Others.

a. Overview. Frequently, USACE obtains data from others in the basin to support its studies and efforts, or shares its data with others. Periodic partnering sessions should be set up to align data collection locations and standards for maximum benefit to other stakeholders. A series of biannual basin sediment stakeholder meetings should be established to help coordinate activities of the various agencies and to provide a venue for status and trends findings.

b. USGS White River Study, Funded by King County. The objective of the study is to quantify the historical and current sediment supply and depositional trends in the White River to better inform capital flood risk reduction and environmental restoration projects along the Lower White River in King County. Specifically, this study uses existing datasets to understand sediment routing and its influence on channel morphology along the White River from the proglacial zone (upstream of W5) to the lower White River (W1). This project takes a combined approach that includes: (1) analyzing repeated topographic (LiDAR) surveys, (2) analyzing repeated aerial photographic surveys, (3) synthesizing existing information, (4) a specific gage analysis, and (5) applying existing empirical models of sediment transport and discharge to understand channel change in the lower White River. Project completion is expected at the end of 2018 calendar year.

(1) Lower White River Monitoring, Funded by King County. As previously indicated, King County operates robust monitoring of the lower White River that includes annual cross-section surveys and funds a series of USGS-operated stage-only gages. The stage gage is publicly available in real-time and survey data is shared with USACE.

(2) Water Quality and Discharge Monitoring in White River, Funded by Cascade Water Alliance. Ongoing turbidity and discharge monitoring by a local water purveyor is conducted at the following locations, and can be leveraged with additional suspended-sediment sampling at these locations to determine load estimates if this information were considered important.

(a) White River at headworks near Buckley, 121098700 (W2 Lower).

(b) White River at Lake Tapps diversion at Dieringer, 12101100 (W1 Lower).

(c) White River at 24th St E at Dieringer, 12101102 (W1 Lower).

10. Exhibits.

a. Exhibit 1 – Proposed SWPP 10-Year Schedule and Budget.

b. Exhibit 2 – Puyallup Basin Reach Atlas Overview Map with HEC-HMS Model Sub-Basins.

c. Exhibit 3 – Longitudinal Profiles of Puyallup River and Major Tributaries from the Puyallup Basin CWMS HEC-RAS Model.

d. Exhibit 4 – Table of Surveyed Cross-Section Locations.

Case Study 2A Exhibit 1
Proposed SWPP 10-Year Schedule and Budget (Base Year is 2018)

Puyallup River Sub-Reaches

Sub-Reach	Activity	Year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10
P1: Lower Puyallup - Commencement Bay to White River (RM 0.0 – RM 10.3). **Priority = 1**	Topographic Data - Channel Bathymetry		$ 22,571					$ 22,571				
	Topographic Data - Lidar		$ 170,225					$ 170,225				
	Sediment Size Data		$ 16,700					$ 16,700				
	Suspended Sediment Discharge Measurements		$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667
	Bedload Discharge Measurements		$ 29,500					$ 29,500				
	Channel Capacity Flows & Inundation Mapping			$ 24,584					$ 24,584			
	Geomorphic Change Analysis & Status			$ 43,824					$ 43,824			
	Sediment Transport Model Updates			$ 44,892					$ 44,892			
	Total by Year	$ -	$ 255,663	$ 129,967	$ 16,667	$ 16,667	$ 16,667	$ 255,663	$ 129,967	$ 16,667	$ 16,667	$ 16,667
P2: Middle Puyallup - White River to Carbon River (RM 10.3 – RM 17.4) **Priority = 2**	Topographic Data - Channel Bathymetry			$27,242					$27,242			
	Topographic Data - Lidar			$87,175					$87,175			
	Sediment Size Data			$7,640					$7,640			
	Suspended Sediment Discharge Measurements			$24,000					$24,000			
	Bedload Discharge Measurements			$29,500					$29,500			
	Channel Capacity Flows & Inundation Mapping									$16,946		
	Geomorphic Change Analysis & Status									$30,208		
	Sediment Transport Model Updates									$30,945		
	Total by Year	$ -	$ -	$ 175,557	$ -	$ -	$ -	$ -	$ 175,557	$ 78,100	$ -	$ -
P3: Upper Puyallup - above Carbon River to above Champion Bridge (RM 17.4 – RM 29.6) **Priority = 3**	Topographic Data - Channel Bathymetry				$ 50,856					$ 50,856		
	Topographic Data - Lidar				$ 102,410					$ 102,410		
	Sediment Size Data				$ 4,544					$ 4,544		
	Suspended Sediment Discharge Measurements											
	Bedload Discharge Measurements											
	Channel Capacity Flows & Inundation Mapping										$ 29,119	
	Geomorphic Change Analysis & Status										$ 51,908	
	Sediment Transport Model Updates										$ 53,174	
	Total by Year	$ -	$ -	$ -	$ 157,810	$ -	$ -	$ -	$ -	$ 157,810	$ 134,200	$ -

Activity Key
Topographic Data
Sediment Size Data
Sediment Load Measurements
Channel Capacity Flows & Inundation Mapping
Geomorphic Change Analysis & Status
Sediment Transport Model Updates

Case Study 2A Exhibit 1 (continued)

White River Sub-Reaches (*Note*: Presumes that King County continues channel monitoring program and shares data with USACE)

Sub-Reach	Activity	Year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10
W1: Lower White River - Puyallup River to Deiringer Canal (RM 0 – RM3.6) Priority = 2	Topographic Data - Channel Bathymetry		$ -		$ -		$ -		$ -		$ -	
	Topographic Data - Lidar		$ -		$ -		$ -		$ -		$ -	
	Sediment Size Data		$ 6,650					$ 6,650				
	Suspended Sediment Discharge Measurements		$ 2,000					$ 2,000				
	Bedload Discharge Measurements											
	Channel Capacity Flows & Inundation Mapping	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592	$ 8,592
	Geomorphic Change Analysis & Status											
	Sediment Transport Model Updates			$ 15,691		$ 15,691		$ 15,691		$ 15,691		$ 15,691
	Total by Year	$ 8,592	$ 17,242	$ 24,283	$ 8,592	$ 24,283	$ 8,592	$ 32,933	$ 8,592	$ 24,283	$ 8,592	$ 24,283
W2: Lower White River- Deiringer Canal to Auburn Dam (RM3.6 – RM 10.6) Priority = 1	Topographic Data - Channel Bathymetry	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
	Topographic Data - Lidar		$ -		$ -		$ -		$ -		$ -	
	Sediment Size Data		$ 7,550		$ 7,550		$ 7,550		$ 7,550		$ 7,550	
	Suspended Sediment Discharge Measurements	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667	$ 16,667
	Bedload Discharge Measurements		$ 29,500					$ 29,500				
	Channel Capacity Flows & Inundation Mapping	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708	$ 16,708
	Geomorphic Change Analysis & Status		Proposed Completion by USGS-King County					Proposed Completion by USGS-King County				
	Sediment Transport Model Updates			$ 30,509		$ 30,509		$ 30,509		$ 30,509		$ 30,509
	Total by Year	$ 33,374	$ 70,424	$ 63,884	$ 40,924	$ 63,884	$ 40,924	$ 93,384	$ 40,924	$ 63,884	$ 40,924	$ 63,884
W3: Middle White River - Auburn Dam to Mud Mountain Dam (RM 10.6 – RM 29.6) Priority = 3	Topographic Data - Channel Bathymetry				$ 54,600					$ 54,600		
	Topographic Data - Lidar				$ 201,025					$ 201,025		
	Sediment Size Data				$ 9,230					$ 9,230		
	Suspended Sediment Discharge Measurements				$ 24,000	$ 24,000						$ 24,000
	Bedload Discharge Measurements				$ 29,500	$ 29,500						$ 29,500
	Channel Capacity Flows & Inundation Mapping											
	Geomorphic Change Analysis & Status											$ 80,840
	Sediment Transport Model Updates											$ 82,811
	Total by Year	$ -	$ -	$ -	$ 318,355	$ 53,500	$ -	$ -	$ -	$ 264,855	$ -	$ 217,151

Case Study 2A Exhibit 1 (continued)

White River Sub-Reaches – Continued

Sub-Reach	Activity	Year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10
W4: Upper White River – **MMD** Reservoir (RM 29.6 – RM 35.3) **Priority = 2**	Topographic Data - Channel Bathymetry											
	Topographic Data - Lidar		$49,291.00					$49,291.00				
	Sediment Size Data		$9,230.00					$9,230.00				
	Suspended Sediment Discharge Measurements											
	Bedload Discharge Measurements											
	Channel Capacity Flows & Inundation Mapping											
	Geomorphic Change Analysis & Res. Capacity Status		$24,252					$24,252				
	Sediment Transport Model Updates			$24,843					$24,843			
	Total by Year	$ -	$ 82,772.89	$ 24,843.40	$ -	$ -	$ -	$ 82,772.89	$ 24,843.40	$ -	$ -	$ -
W5: Upper White River - above Reservoir RM 35.3 – RM 45.8 (Clearwater to Greenwater River) **Priority = 3**	Topographic Data - Channel Bathymetry											
	Topographic Data - Lidar				$ 51,590					$ 51,590		
	Sediment Size Data				$ 9,800					$ 9,800		
	Suspended Sediment Discharge Measurements											
	Bedload Discharge Measurements											
	Channel Capacity Flows & Inundation Mapping											
	Geomorphic Change Analysis & Status										$ 40,420	
	Sediment Transport Model Updates											
	Total by Year	$ -	$ -	$ -	$ 61,390.00	$ -	$ -	$ -	$ -	$ 61,390.00	$ 40,419.81	$ -

Case Study 2A Exhibit 1 (continued)

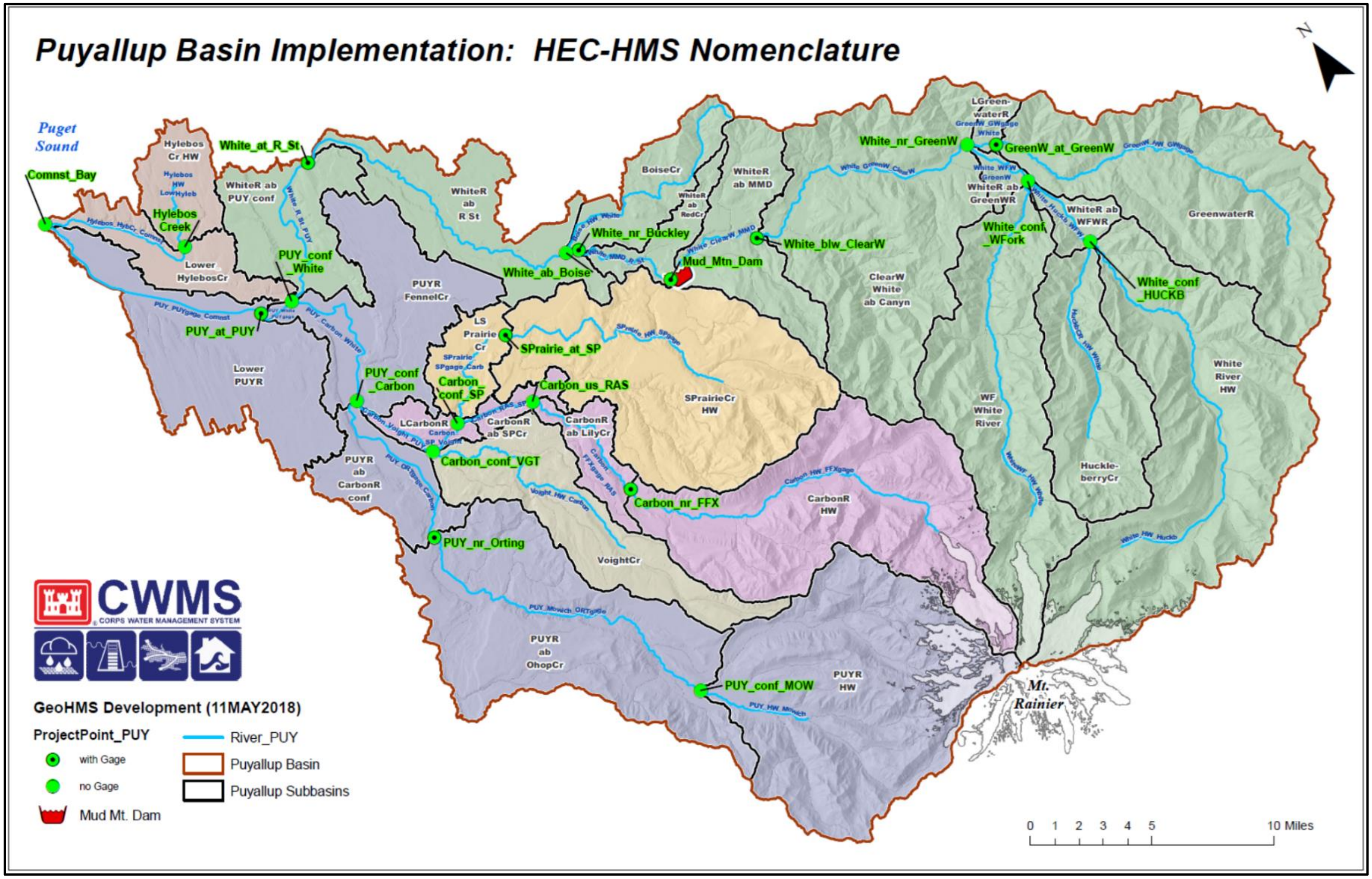
Carbon River

Sub-Reach	Activity	Year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10
C1: Lower Carbon River - Puyallup River to above South Prairie Creek (RM 0 - RM 5.9)	Topographic Data - Channel Bathymetry					$33,657.00					$33,657.00	
	Topographic Data - Lidar					$38,335.00					$38,335.00	
	Sediment Size Data					$6,560.00					$6,560.00	
	Suspended Sediment Discharge Measurements					$24,000	$24,000					
	Bedload Discharge Measurements					$29,500	$29,500					
	Channel Capacity Flows & Inundation Mapping											$14,082
	Geomorphic Change Analysis & Status											$25,103
	Sediment Transport Model Updates											$25,715
	Total by Year	$ -	$ -	$ -	$ -	$ 132,052.00	$ 53,500.00	$ -	$ -	$ -	$ 78,552.00	$ 64,900.00

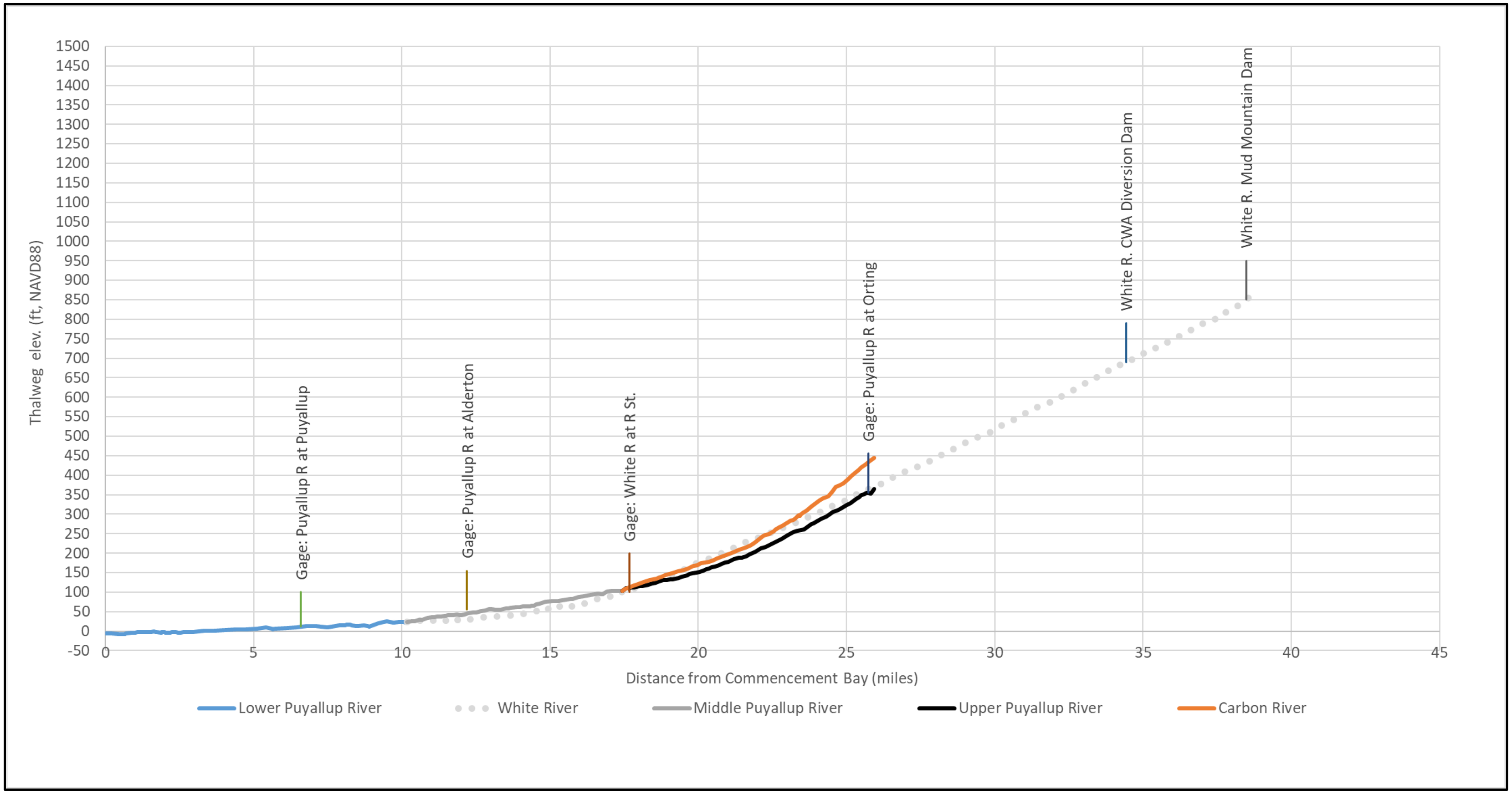
All Reaches Combined

Cost for all Activities by year, all reaches	Year 0	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10
	$ 41,967	$ 426,102	$ 418,533	$ 603,738	$ 290,385	$ 119,683	$ 464,752	$ 379,883	$ 666,988	$ 319,355	$ 386,884

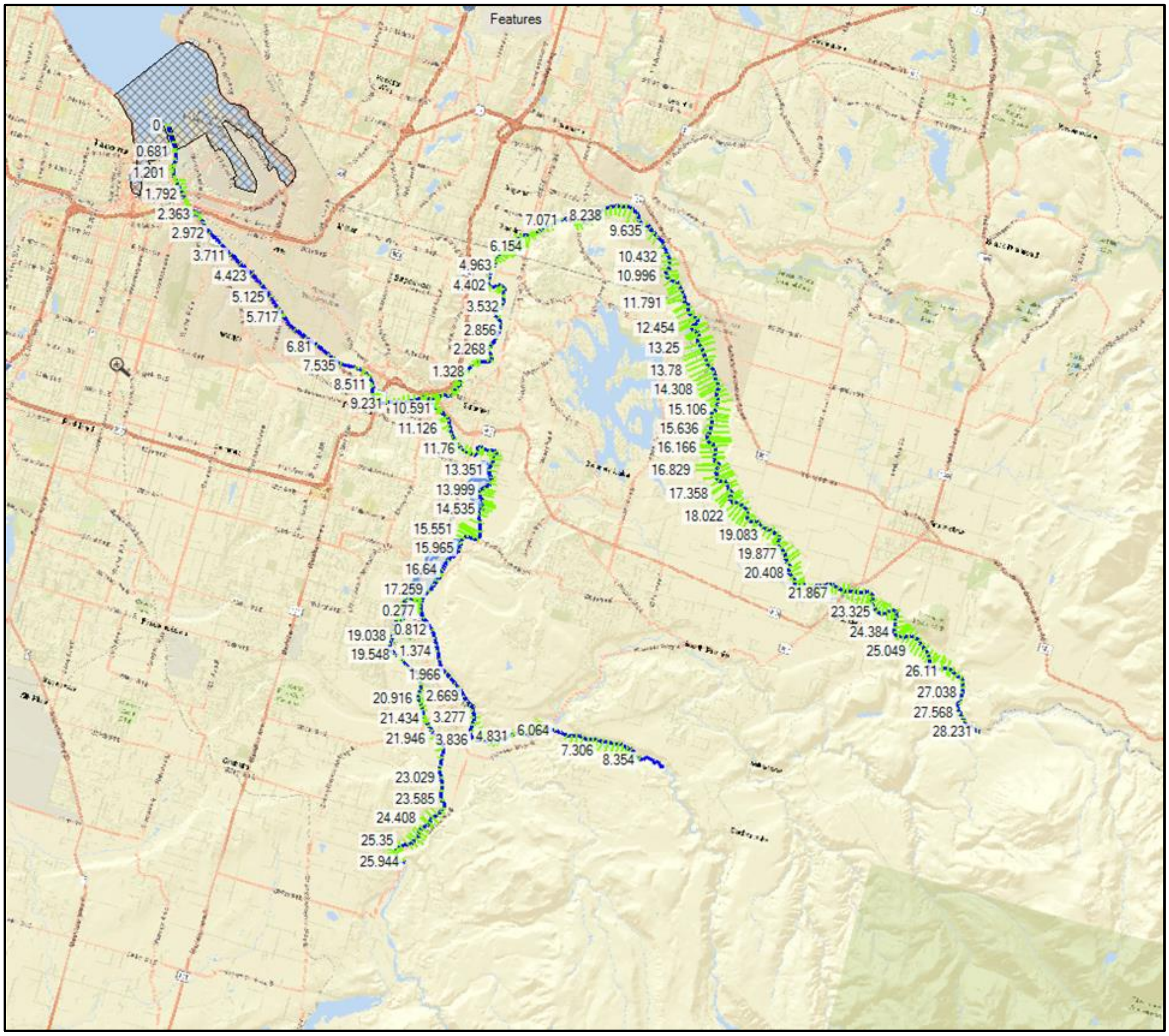
Case Study 2A Exhibit 2
Puyallup Basin Reach Atlas Overview Map with HEC-HMS Model Sub-Basins



Case Study 2A Exhibit 3
Longitudinal Profiles of Puyallup River and Major Tributaries from the Puyallup Basin CWMS HEC-RAS Model



Case Study 2A Exhibit 4
Puyallup-White River Basin Hydraulic Model Cross-Section Locations



Case Study 2A Exhibit 4 (continued)

Puyallup-White River Basin Hydraulic Model Cross-Section Locations by River Mile

PWRB CWMS model cross section stationing (miles)

Lower Puyallup River			Middle Puyallup River			Upper Puyallup River			White River							Carbon River		
0	2.307	8.293	10.354	13.881		17.546	20.184	23.223	0	2.012	5.295	8.09	14.308	19.877	25.447	0	4.734	8.055
0.234	2.328	8.358	10.474	13.999		17.569	20.269	23.336	0.093	2.115	5.35	8.238	14.444	20.011	25.579	0.065	4.831	8.186
0.449	2.363	8.511	10.53	14.094		17.599	20.276	23.367	0.114	2.117	5.421	8.381	14.585	20.144	25.704	0.166	4.917	8.354
0.631	2.428	8.531	10.591	14.264		17.648	20.358	23.585	0.134	2.268	5.453	8.527	14.726	20.276	25.844	0.277	5.008	
0.632	2.484	8.54	10.641	14.293		17.673	20.463	23.849	0.176	2.356	5.545	8.642	14.841	20.408	25.971	0.285	5.078	
0.635	2.488	8.543	10.642	14.296		17.681	20.536	23.856	0.297	2.436	5.655	8.769	14.973	20.541	26.11	0.431	5.184	
0.648	2.492	8.561	10.665	14.308		17.69	20.548	23.901	0.429	2.44	5.753	8.95	15.106	20.67	26.242	0.613	5.326	
0.668	2.552	8.764	10.69	14.343		17.691	20.58	24.016	0.445	2.641	5.829	9.139	15.253	20.851	26.375	0.812	5.395	
0.681	2.66	8.896	10.713	14.398		17.701	20.699	24.027	0.517	2.856	5.91	9.302	15.371	20.959	26.507	1.029	5.471	
0.867	2.756	8.91	10.886	14.469		17.706	20.706	24.186	0.566	3.002	5.95	9.473	15.503	21.074	26.652	1.041	5.495	
0.995	2.78	8.932	10.966	14.535		17.779	20.755	24.345	0.569	3.143	5.975	9.635	15.636	21.204	26.767	1.21	5.513	
1.059	2.817	8.96	11.028	14.564		17.788	20.804	24.408	0.574	3.289	6.053	9.767	15.769	21.337	26.911	1.221	5.578	
1.145	2.972	8.993	11.126	14.655		17.848	20.916	24.574	0.587	3.321	6.144	9.907	15.901	21.469	27.038	1.312	5.656	
1.201	3.358	9.061	11.222	14.867		17.916	21.018	24.696	0.623	3.45	6.147	10.187	16.034	21.602	27.172	1.374	5.68	
1.2904	3.711	9.231	11.35	15.054		18.031	21.123	24.785	0.723	3.532	6.154	10.432	16.166	21.734	27.297	1.559	5.727	
1.341	4.039	9.42	11.45	15.086		18.112	21.148	24.912	0.823	3.535	6.185	10.569	16.307	21.867	27.442	1.772	5.787	
1.387	4.315	9.536	11.5066	15.299		18.19	21.156	25.03	0.842	3.643	6.194	10.702	16.427	21.996	27.568	1.794	5.799	
1.408	4.423	9.6	11.563	15.416		18.201	21.22	25.165	0.897	3.778	6.214	10.849	16.564	22.135	27.698	1.966	5.808	
1.414	4.774	9.651	11.682	15.551		18.327	21.374	25.35	0.971	3.853	6.263	10.996	16.697	22.265	27.818	2.112	5.813	
1.444	5.125	9.766	11.76	15.676		18.454	21.434	25.454	0.977	4.008	6.28	11.129	16.829	22.397	27.976	2.292	5.824	
1.56	5.464	9.924	11.78	15.781		18.461	21.466	25.532	0.983	4.099	6.313	11.261	16.97	22.53	28.102	2.293	5.844	
1.677	5.659	9.98	11.891	15.844		18.568	21.469	25.61	1.035	4.11	6.396	11.394	17.094	22.667	28.231	2.423	5.85	
1.754	5.67	10.089	11.982	15.965		18.676	21.486	25.686	1.092	4.156	6.472	11.526	17.227	22.795		2.543	5.863	
1.792	5.676	10.165	12.006	16.012		18.797	21.492	25.764	1.098	4.192	6.585	11.659	17.358	22.928		2.59	5.95	
1.917	5.68	10.18	12.009	16.034		18.811	21.496	25.824	1.142	4.368	6.711	11.791	17.492	23.07		2.669	6.064	
1.932	5.717	10.218	12.013	16.133		18.924	21.502	25.883	1.145	4.398	6.8	11.896	17.625	23.197		2.812	6.217	
1.945	5.958		12.029	16.369		19.038	21.516	25.944	1.176	4.402	6.82	12.057	17.757	23.325		2.944	6.227	
1.988	6.138		12.137	16.64		19.133	21.533		1.227	4.551	6.906	12.189	17.89	23.458		3.056	6.301	
2.022	6.475		12.251	16.752		19.24	21.632		1.262	4.66	6.987	12.322	18.022	23.602		3.083	6.406	
2.034	6.81		12.399	16.764		19.342	21.751		1.272	4.765	7.071	12.454	18.152	23.723		3.277	6.514	
2.042	7.143		12.531	16.773		19.467	21.886		1.283	4.808	7.187	12.728	18.279	23.862		3.447	6.61	
2.068	7.316		12.679	16.781		19.548	21.946		1.328	4.81	7.331	12.852	18.42	23.988		3.637	6.713	
2.117	7.47		12.75	16.79		19.592	22.075		1.353	4.822	7.414	12.986	18.553	24.121		3.71	6.794	
2.157	7.535		12.849	16.823		19.639	22.121		1.486	4.83	7.42	13.117	18.685	24.248		3.836	6.947	
2.178	7.914		12.963	16.934		19.712	22.298		1.568	4.844	7.429	13.25	18.811	24.384		3.873	7.055	
2.187	7.979		13.057	17.075		19.798	22.519		1.592	4.846	7.434	13.384	18.95	24.519		4.035	7.161	
2.196	8.01		13.175	17.259		19.819	22.726		1.594	4.881	7.448	13.515	19.083	24.651		4.202	7.306	
2.214	8.093		13.351	17.441		19.894	22.742		1.612	4.963	7.455	13.648	19.209	24.784		4.31	7.452	
2.259	8.1		13.53	17.443		19.925	22.745		1.626	5.034	7.536	13.78	19.35	24.916		4.335	7.568	
2.268	8.11		13.609	17.449		20.045	22.845		1.673	5.125	7.666	13.913	19.477	25.049		4.406	7.703	
2.277	8.161		13.635			20.056	23.029		1.728	5.207	7.779	14.047	19.613	25.182		4.529	7.818	
2.286	8.226		13.689			20.138	23.213		1.926	5.293	7.932	14.178	19.746	25.314		4.639	7.916	

Case Study 2B
Sediment Activity Notes Report Example

This case study presents an example Sediment Activity Notes report prepared by the Omaha District in calendar year 2015. The example report content follows guidance presented in Chapter 2 (paragraph 2-7), Reporting Sedimentation Activities, for USACE District offices to report sediment activities. Document content and formatting may be adapted to meet specific needs of individual USACE District reporting offices.

US Army Corps of Engineers ®
Omaha District

NOTES ON SEDIMENTATION ACTIVITIES FOR CALENDAR YEAR 2015

Northwestern Division – Omaha District

Missouri River Near Glovers Point Bend, RM 714-710

Prepared By:
Engineering Division
Hydrologic Engineering Branch
River and Reservoir Engineering Section
January 2016

1 Omaha District Sedimentation Program Overview

1.1 The Omaha District's Sedimentation Program is managed by the River & Reservoir Engineering Section (CENWO-ED-HF) as a comprehensive maintenance and evaluation program for the purpose of systematically assessing the operating conditions of the six main stem and twenty-two tributary reservoirs as they relate to sedimentation issues and to recommend practical solutions to sediment related problems. Additional activities are conducted to assist with USACE civil works programs and investigations. An Omaha District project location map is shown in Figure 1.

1.2 The *Notes on Sedimentation Activities* is a reporting requirement of USACE Headquarters. USACE has a continuing need for reporting sedimentation activities including data gathered, studies performed, and relevant research activities. USACE reporting objectives are to provide a clear and concise summary of sedimentation activities to inform other offices of interesting efforts, to stimulate the exchange of information, and to inform management of ongoing activities.

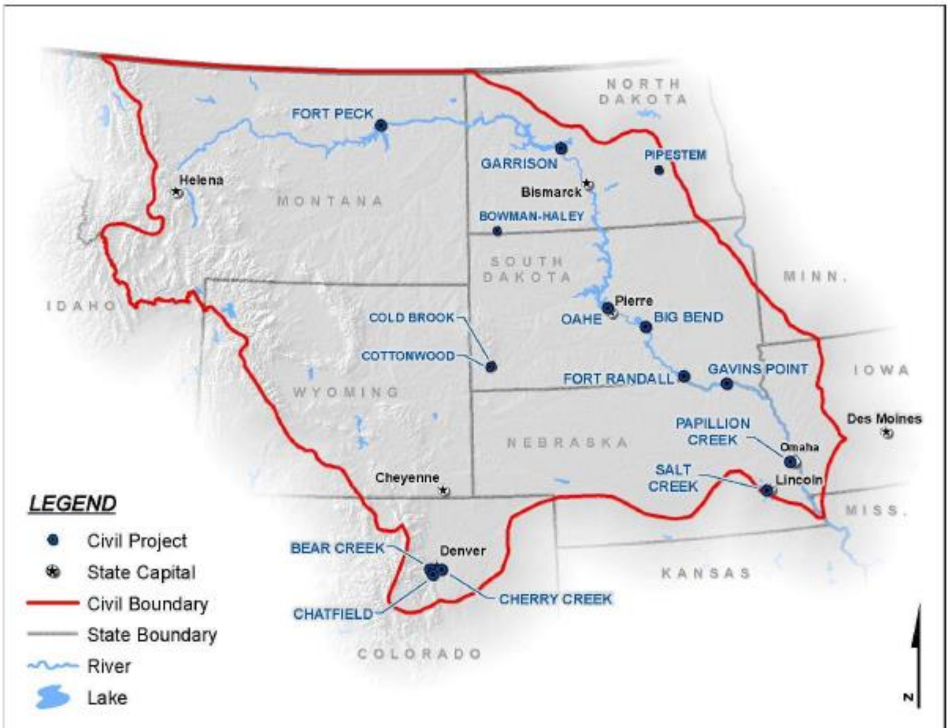


Figure 1 Omaha District Project Location Map

2 SEDIMENT SURVEYS

The following project specific survey data collection efforts related to sedimentation activities within Omaha District in calendar 2015.

2.1 Bathymetric Surveys - Coldbrook

A detailed bathymetric survey of the twenty-nine (29) sediment ranges at Coldbrook Lake in South Dakota was completed in 2015 by an in-house survey crew. New data will be used to calculate new reservoir surface area and storage capacity tables.

2.2 Topographic Surveys

No topographic surveys for the general purpose of sedimentation activities were completed in 2015.

2.3 Groundwater Observations

In 2015, only the groundwater observation wells in Pierre, SD and Niobrara, NE are being used. The other wells have historical data attached to them and can be rehabilitated and used in the future to report any changes in groundwater levels over time. All present records are obtained by Sedimentation Program personnel and entered into a HEC Data Storage System (HECDSS) in the RARE database for preservation.

3 Sediment Sampling

Sediment sampling investigations conducted in calendar year 2015 are detailed in the following sections.

3.1 Sediment Sampling Stations

The Sedimentation Program operated six (6) suspended sediment sampling stations in 2015. Three stations are on the Missouri River navigation reach while the other three are on the Yellowstone, Bad, and White River tributaries. Table 2-1 lists the sampling stations, locations, and published data. The USGS monitors, maintains, and publishes the data from these stations under a cooperative stream-gaging program funded yearly by the Omaha District.

Table 2-1 - Operating Sediment Sampling Stations in 2015

Stream/Station No.	Location	USGS Published Data
Yellowstone River Station No. 06329500	Sidney, Montana	Suspended sediment concentration Suspended sediment discharge
White River Station No. 6452000	Oacoma, South Dakota	Suspended sediment concentration Suspended sediment discharge
Bad River Station No. 6441500	Fort Pierre, South Dakota	Suspended sediment concentration Suspended sediment discharge
Missouri River Station No. 06486000	Sioux City, Iowa	Suspended sediment concentration Suspended sediment discharge
Missouri River Station No. 06610000	Omaha, Nebraska	Suspended sediment concentration Suspended sediment discharge
Missouri River Station No. 06807000	Nebraska City, Nebraska	Suspended sediment concentration Suspended sediment discharge

3.2 Bed Samples – Missouri River.

Bed samples were collected on the Missouri River navigation channel from River Mile 735 near Sioux City, IA, to River Mile 500 near Rulo, NE. Samples were collected with an in-house crew using the BM-54 sampler. Samples were collected at roughly a five-mile interval with three samples (left, center, right) at each location.

3.3 Bed Samples – Bear Creek.

Bed samples were collected at Bear Creek reservoir near Denver CO. Samples were collected within the reservoir using the BM-54 sampler (12 locations at established sediment ranges) and upstream in the dry using field collected grab samples (6 sediment ranges).

4 Sediment Investigations

Sediment related investigations conducted in calendar year 2015 are detailed in the following sections.

4.1 Garrison – New M.R.B. Sediment Memoranda No. 32

Garrison Dam – Lake Sakakawea is located on the Missouri River in northwestern North Dakota. New M.R.B. Report No. 32, *Lake Sakakawea Aggradation Study, 1954-2014,* was completed in 2015 that evaluated geomorphic conditions and sedimentation trends for Lake Sakakawea. Presented are project statistical data, range line cross section data, reservoir capacity and sediment depletion data, and comparisons of pre- and post-2011 flood conditions.

4.2 Oahe – Update M.R.B. Sediment Memoranda No. 15c

Oahe Dam – Lake Oahe is located on the Missouri River from downstream of Bismarck, ND to Pierre, SD, over a distance of 231 river miles. M.R.B. Report No. 15a, *Lake Oahe Aggradation Study, 1958-2010*, was completed by an A-E contractor in 2015. The report evaluates geomorphic conditions and sedimentation trends for Lake Oahe and its four major tributaries consisting of the Cannonball, Grand, Moreau, and Cheyenne Rivers using land and hydrographic survey data collected in 2010.

4.3 Regional Sediment Management (RSM) Program

The RSM program is supported by ERDC. The program aims to find cost effective and novel solutions to sediment management problems facing USACE. In 2015 a reservoir flushing model study was completed in conjunction with IWR-HEC. The model replicated active flushing that is ongoing at Spencer Dam on the Niobrara River in northeast Nebraska. This was the first use of the HEC-RAS model for flushing that included a detailed calibration dataset. An ERDC tech note will be published on the study in 2016.

4.4 Channel Stabilization Program – Missouri River Water Surface Profiles

Water surface profiles are informative to track stage-trend information within the Missouri River navigation channel. Two water surface profiles were completed by an in-house survey crew on the Missouri River downstream of Gavins Point Dam. The first profile was conducted in July at a Missouri River discharge (at Gavins Point Dam) of 27,000 cfs and covered the entire stretch from Gavins Point Dam to Rulo, NE (348 miles). The second profile was conducted in September at a discharge of 28,000 cfs.

Case Study 4A
Examples of ISSDOT Application for Bedload Transport Estimates from Repeated Multi-Beam Surveys

1. Case Study.

This case study documents application of the Integrated Section, Surface Difference Over Time (ISSDOT) method to estimate bedload transport. The ISSDOT method was applied by ERDC personnel on the Missouri River in the Omaha and Kansas City Districts. The case study content demonstrates several concepts but is not comprehensive of ISSDOT application for USACE study requirements.

2. Introduction.

The Integrated Section, Surface Difference Over Time, Version 2 (ISSDOTv2) method of computing bedload transport from time-sequenced bathymetric data has been used at eight different USACE Districts and on five major rivers in the United States as of December 2014. The ISSDOTv2 method produces consistent and repeatable results based on a sound physical and mathematical process.

a. This document presents examples of real-world data collection efforts conducted by the Engineer Research and Development Center – Coastal and Hydraulics Laboratory (ERDC/CHL) for various USACE Districts. The data was obtained between 2009 to 2014, and consisted of important physical parameters of river hydraulics and sediment transport on the Missouri, Ohio, and Mississippi Rivers. In addition, this procedure has been used on the Snake River, Red River, and in other rivers not listed. These are all large, sand-bed rivers for which traditional bedload samplers provide marginal to unusable results. In most cases, both suspended-sediment and bedload transport were quantified, together with a host hydrodynamic dynamic data.

b. This case study presents only the bedload data. These parameters were measured on different days at varying flow rates to allow the creation of flow vs. bedload rating curves. At some sites, in addition to computing average channel parameters, the data was collected in such a way as to facilitate sub-cross-sectional computations of hydraulic and sediment parameters.

c. The bedload transport was computed using the ISSDOTv2 method, which computes bedload transport from successive, multi-beam surveys of the same swath of river. For more detailed information see (Abraham et al., 2011; Abraham et al., 2015; McAlpin et al., 2016).

3. Data Collection.

Careful data collection procedures related to boat speed and direction, instrument settings, and maximum elapsed time between surveys must be followed for the data to be useable. Engineers interested in collecting bathymetric data to compute bedload should contact CHL for more information regarding data collection procedures.

a. For ISSDOTv2 calculations, data collection lines must be surveyed at least twice to compute a surface difference. However, to correct for a documented under-prediction bias (Shelley et al., 2013), at least three repetitions are needed, and four to six are preferred. Figure 1 shows the CHL survey boat used to measure bathymetric swaths of rivers for bedload measurement.

Case Study 4A Figure 1. Survey boat with multi-beam echo sounder

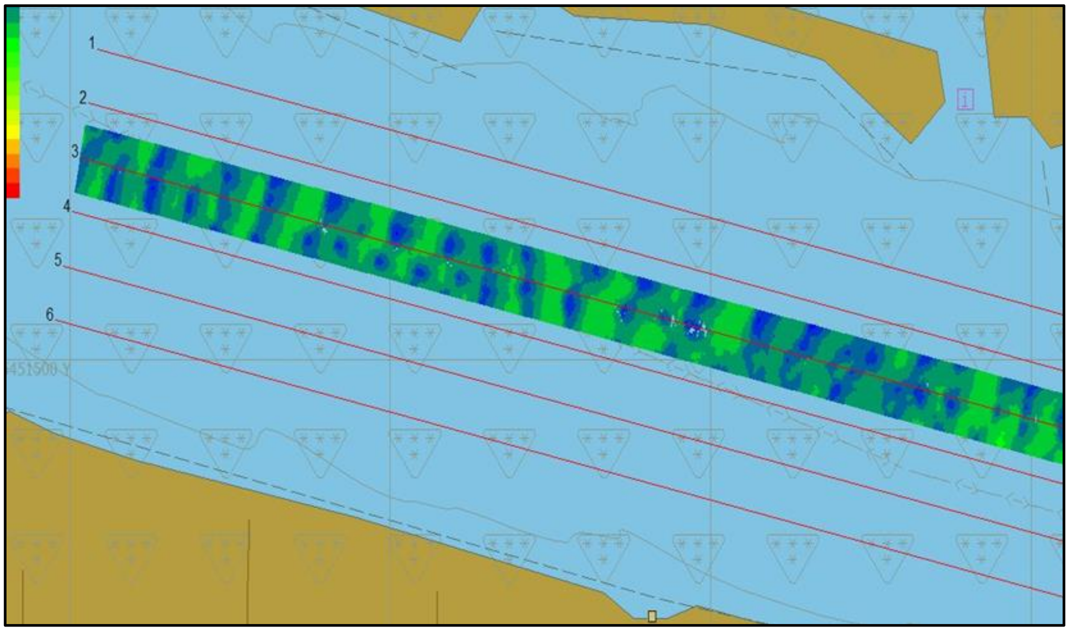
b. Figure 2 provides an example of one swath of collected bathymetric data. Bedload values from multiple swaths can be summed to compute the full bedload at a river location. (Figure 3).

c. Figure 3 shows four bathymetric swaths of raw data collected on the Missouri River near Kansas City, collected to create a flow/bedload rating curve at this location.

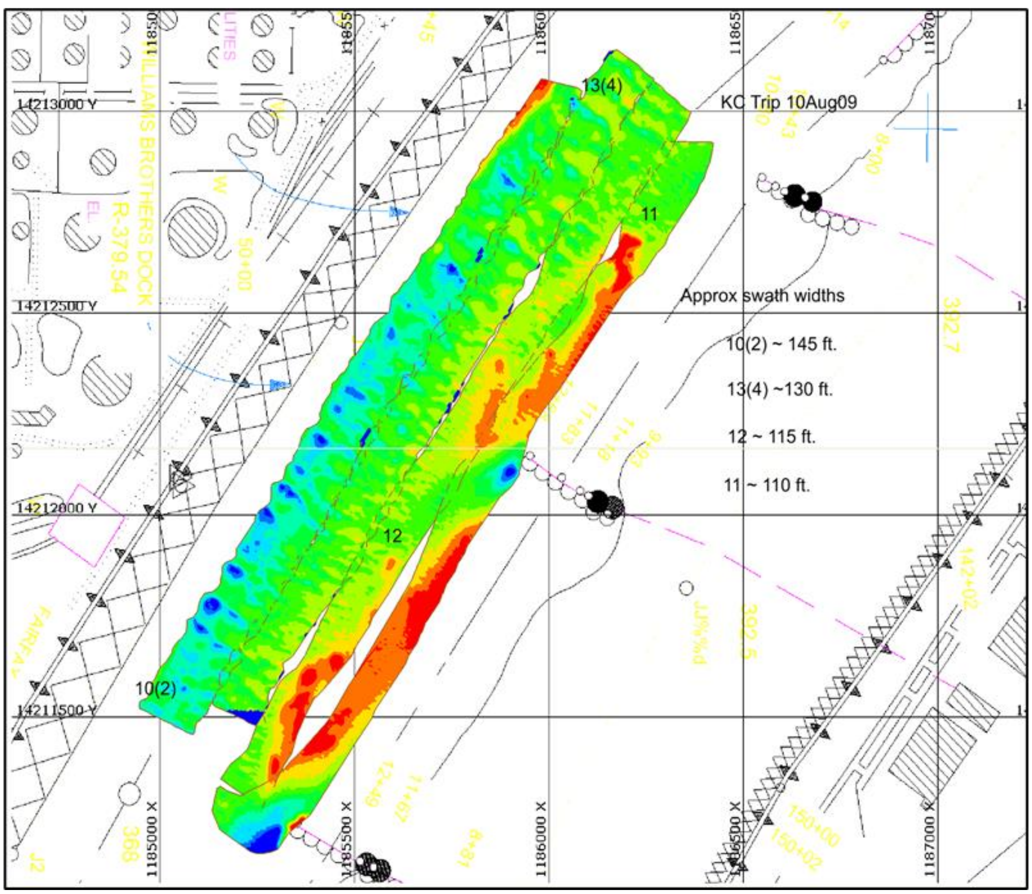
(1) The sand waves (dunes) are clearly visible in each swath. Comparing the dune sizes allows a first prediction for relative transport rates that can be used as a reasonableness check on the ISSDOTv2 results.

(2) Swath 10(2) is located on the right descending bank (flow is from the top of the figure) and located in the deepest part of the channel. At higher flows, the large sand dunes visible in swath 10(2) should transport large quantities of bedload.

(3) Swath 11 is located on the other side of the navigation channel, adjacent to the three training structures. The channel is shallower here and the dunes are much smaller. It is expected for this side of the channel (swath 11) to transport a smaller quantity of bedload than the other side (swath 10(2)).



Case Study 4A Figure 2. Example of a single bathymetric swath

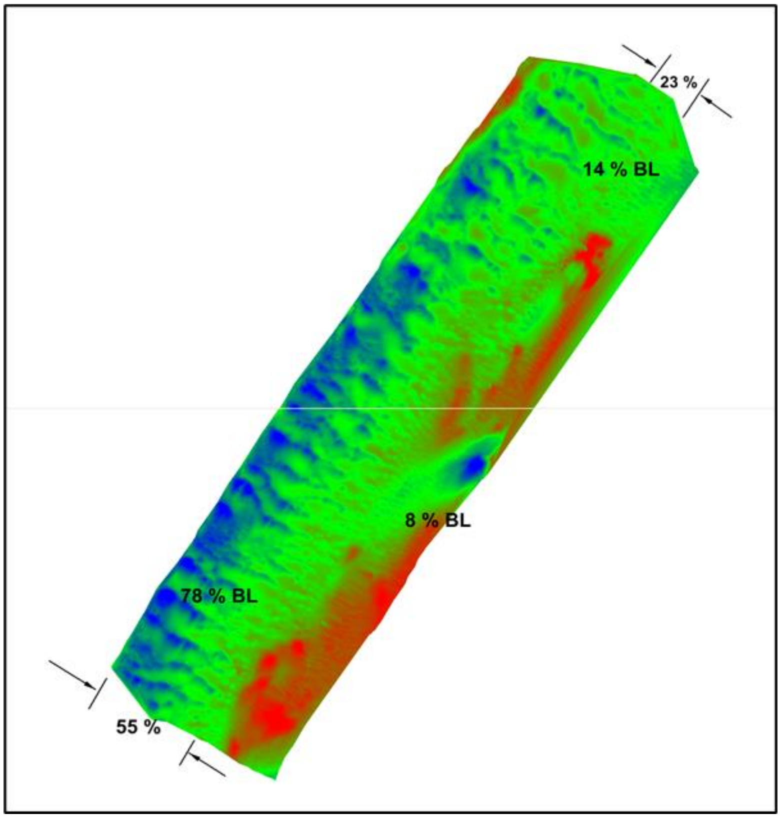


Case Study 4A Figure 3. Bathymetric data on the Missouri River near Kansas City

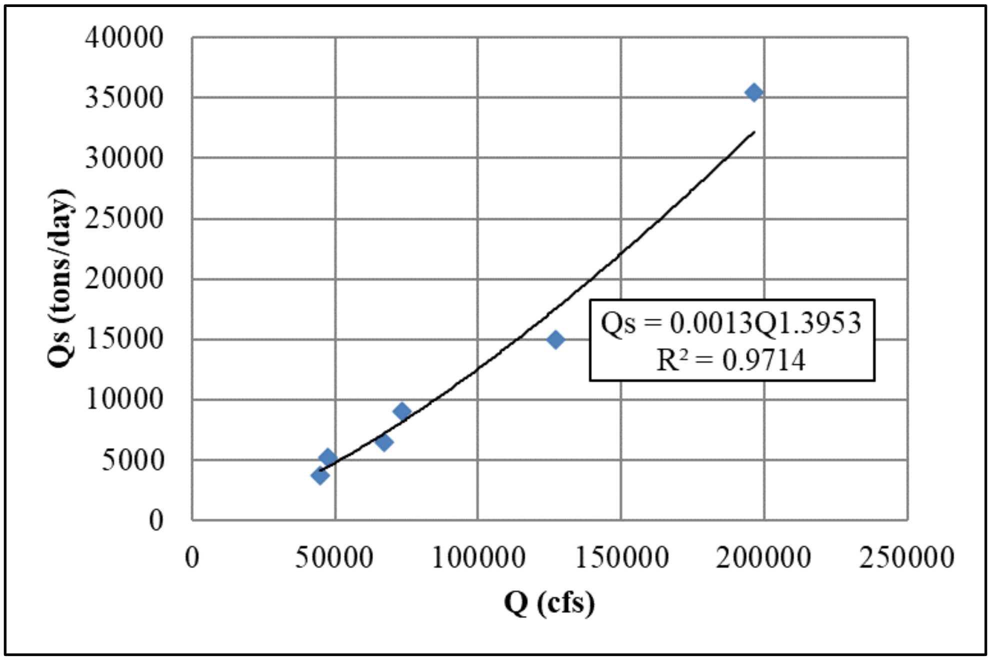
4. Evaluation.

After running the ISSDOT code using the raw data shown above (along with additional multi-beam measurements at various time steps), and applying the output results to each measured section of the river (four swaths), transport rates in each swath were determined. The sum of the four swaths (3,470 tons per day) was the sand transport for the entire measured portion of the river section.

a. The lateral variability in the bedload transport for the Missouri River near Kansas City data is shown in Figure 4. This figure illustrates the lateral variation of the bedload transport and the ability of the ISSDOTv2 method to quantify this variability. Almost 80% of the bedload transport occurs in only 55% of the channel. As expected, the larger dunes at the river thalweg transport more sediment than the smaller dunes near the training structures. Figure 5 presents the rating curve developed for the Missouri River near Kansas City.



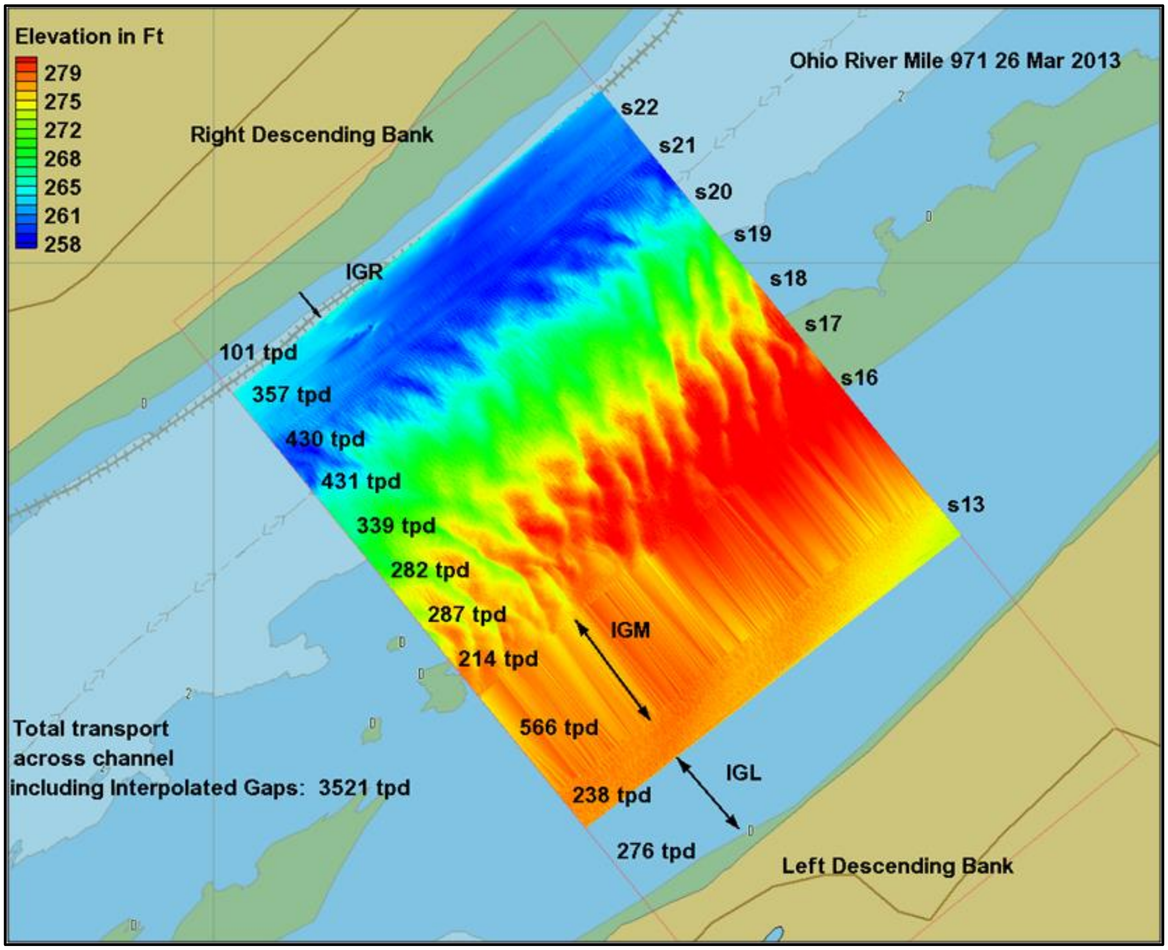
Case Study 4A Figure 4. Lateral variability of bedload transport on the Missouri River near Kansas City



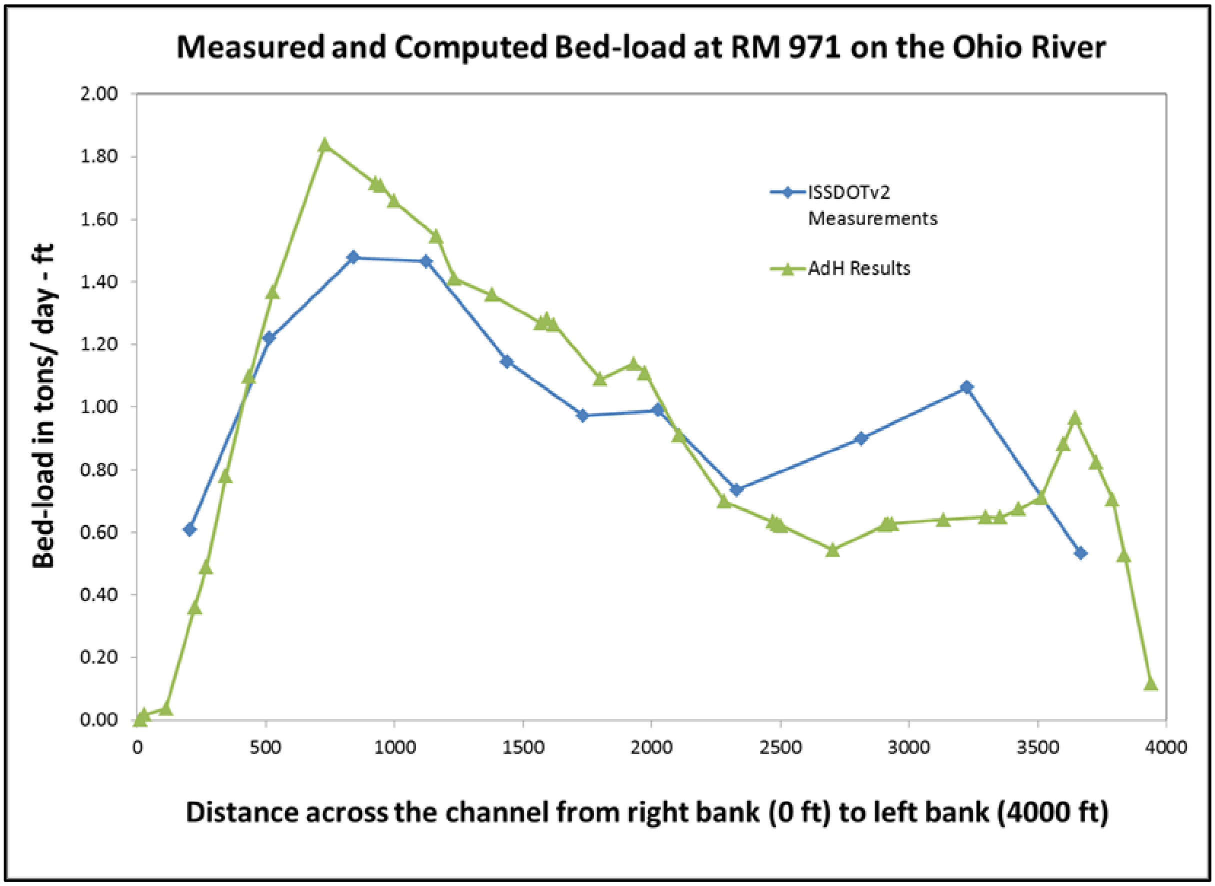
Case Study 4A Figure 5. Bedload rating curve developed for the Missouri River near Kansas City

b. Laterally varying bedload values computed with ISSDOTv2 can be used to validate numerical models. Figure 6 displays the bedload values computed for the Ohio River near Mound City, Illinois. The deepest water, largest waves, and highest transport are in the portion of the channel covered by swaths s21, s20, and s19. The entire section was not covered by the survey and the Figure 6 labeled sections of IGR, IGM, and IGL represent areas of interpolation (Abraham et al., 2015).

c. A numerical hydrodynamic and sediment transport model was built for this location using the Adaptive Hydraulics (AdH) code. The bedload transport function used in the numerical model was Meyer-Peter-Muller (1948) with the Wu Wang Jia Gravel Correction (Wu et al., 2000). Figure 7 indicates reasonable agreement between the numerical model computations for bedload (the green line with the triangles) and the ISSDOTv2 computed values (the blue line with the diamonds).



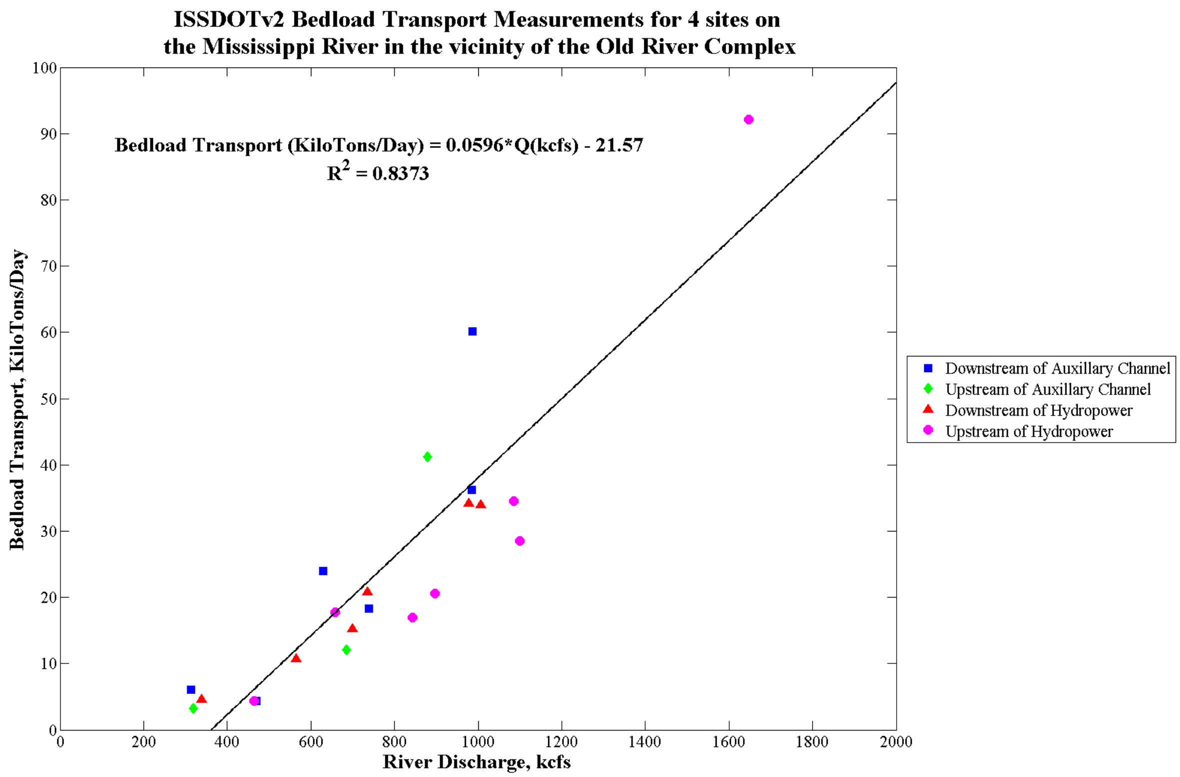
Case Study 4A Figure 6. Lateral variation in bedload transport on the Ohio River



Case Study 4A Figure 7. ISSDOTv2 calculations and AdH numerical results

d. The largest ISSDOT data set was collected at the Old River Control Complex (ORCC) on the Mississippi River (Heath et al., 2015). Twenty-two separate measurements were obtained at various flowrates (from ~470 kcfs to almost 1,600 kcfs) over a six-mile reach on the main stem Mississippi River. This data was collected at four different sites with a maximum distance of six miles from the farthest upstream and downstream measurement locations. Figure 8 shows a plot of the computed bedload transport in tons per day versus flow in cfs. A linear regression was performed to obtain a rating curve relating the river discharge to the bedload transport.

e. Rating curves such as this are immensely valuable to river managers for many of the reasons mentioned in the Introduction section of this document.



Case Study 4A Figure 8. Bedload rating curve for the Mississippi River near the Old River Control Complex

5. Summary.

The examples above illustrate that the ISSDOTv2 methodology provides consistent and repeatable results with practical applications. The results are based on sound physical and mathematical considerations, and agree with both the temporal and spatial expectations for bedload transport in rivers. This data can be used for creating sediment rating curves and for validating sediment transport models, as well as for numerous other purposes.

Case Study 4B
Missouri River Jet Erosion Testing at River Mile 760–780

1. Case Study.

This case study documents jet erosion testing on the Missouri River performed by U.S. Department of Agriculture, Agricultural Research Service (USDA-ARS) and USACE personnel on 15–17 October 2012. The testing was performed on the Missouri River in the Omaha District supporting bank erosion studies. The case study content demonstrates several jet testing concepts but is not comprehensive of erosion testing for USACE study requirements.

2. Jet Erosion Testing Process.

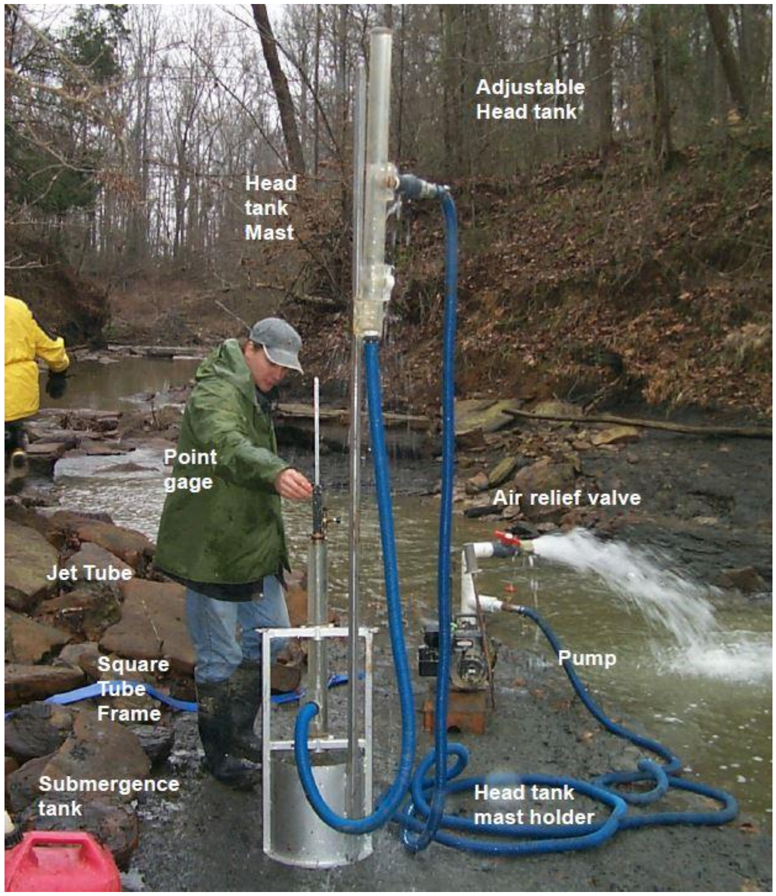
The jet erosion testing (JET) process is described in a series of steps. First, the process of jet erosion testing is explained. Second, the locations tested are described. Third, the results are presented. Finally, potential sources of error are listed and quantified and best practices for future testing are identified.

a. Location. On 15–17 October 2012, USACE-Kansas City, USACE-Omaha, and USDA-ARS personnel performed jet erosion testing on the Missouri River at selected sites between RM 760 and 780. The goal of this testing effort was to determine the erodibility and critical shear stress of bank and bar material in the Upper Missouri River.

b. Submerged Jet Test. These parameters were measured using the submerged jet erosion test described by ASTM Standard D5852. Two jet erosion testers belonging to the USDA Agricultural Research Service were used for this test.

(1) The JET consists of placing the testing device on the soil surface and allowing a submerged water jet to impinge on and scour a small hole into the soil surface. After a short and measured time period, the jet is shut off and the depth of the hole is measured. The applied shear stress is calculated from the scour depth, available head, nozzle dimensions, and frictional losses as the jet travels through the water to the bottom of the scour hole. The jet of water is then allowed to continue scouring the soil material for another short interval. As the scour hole deepens, frictional losses increase, the rate of erosion decreases, and longer jetting times are needed to register measurable change. Figure 1 illustrates the key components of the original JET apparatus (Hanson and Cook 2004).

(2) Eventually, the scour hole deepens sufficiently to lower the shear stress applied by the water jet to below the soil's critical shear stress, the minimum stress required to induce erosion. The scour hole approaches this maximum depth asymptotically and may take hours to months to actually reach a point where no measurable erosion takes place (Hanson and Cook 1997). In practice, the test is run sufficiently long enough to describe the shape of the asymptotic curve, and the final scour depth is extrapolated using a procedure developed by Blaisdell et al. (1981).



Case Study 4B Figure 1. Original JET apparatus (Hanson and Cook 2004)

Figure 2 from the report ‘Apparatus, Test Procedures, and Analytical Methods to Measure Soil Erodibility In Situ’ published by the American Society of Agricultural and Biological Engineers (ASABE). Used with permission.

(3) Two USDA JETs were used on 15–17 October 2012. One of the JETs is shown in Figure 2. Using two JETs allowed the collection of side-by-side tests to characterize site variability and provide better average values. A minimum of four tests per site were conducted, though not all tests were successful.

Case Study 4B Figure 2. USDA jet erosion tester

3. Testing Sites.

The following three sites were selected based on the presence of cohesive material: an eroding bank at RM 775, and sand bars with underlying cohesive material at RM 772.9 and RM 768. These locations are shown in Figure 3. Five additional sites were examined but were found to be composed of non-cohesive sands. Visual inspection over the course of three days confirmed that cohesive material is the exception rather than the rule for this reach of the Missouri River. Virtually all banks, bars, and islands are composed of sand.

Case Study 4B Figure 3. Jet erosion test site locations with approximate river mile

a. The first site tested, shown in Figure 4, was an eroding bank just downstream of the Vermillion Bridge. Nearby banks were armored. The material was 3.5% sand, 85.5% silt, and 11% clay, as evidenced by a particle size analysis of a sample taken on the day of survey.

Case Study 4B Figure 4. First jet erosion testing site: eroding bank downstream from Vermillion Bridge at approximately RM 775

b. The second site was a sand bar on the left bank at approximately RM 772.9, shown in Figure 5. The 2011 flood had deposited 0.5–1 foot of sand on this bar (1.7% medium sand, 91.3% fine sand). The sand was excavated with a hand shovel to allow testing of the underlying cohesive layer, as seen in Figure 6. The underlying layer was 36% sand, 53.6% silt, and 10.4% clay.


Case Study 4B Figure 5. Jet erosion testing site at approximately RM 772.9


Case Study 4B Figure 6. Excavation to cohesive layer at approximately RM 772.9

c. The third site at approximately RM 768 was a mid-channel bar with fresh deposition from the 2011 flood, shown in Figure 7. This bar was unique in that a significant amount of coarse gravel was also deposited on the bar surface. The top layer was 63% gravel, 37% sand, and less than 1% silts or clays. The top layers of non-cohesive material were excavated to allow testing of the underlying cohesive layer. The cohesive layer was 17.7% sand, 65.8% silt, and 16.5% clay.

Case Study 4B Figure 7. Testing of underlying cohesive layer of mid-channel bar at RM 768 (Note gravel deposition on bar surface)

4. Procedures/Results.

The jetting time started at 5 seconds and increased to as long as 20 minutes between measurements. At times, anomalies in scour hole development or equipment malfunction necessitated early termination of the testing. Total jetting time for tests used to compute the reported average values ranged from 14 to 82 minutes. Tests that terminated with less than 14 minutes of total jetting time were not used to compute average critical shear stress (τ_c) or erodibility (k_d). The average values for τ_c and k_d for the three sites are provided in Table 1, along with the 90% confidence intervals expressed as a percentage of the mean.

Case Study 4B Table 1
Results from Jet Erosion Testing

River Mile	τ_c (Pa)	k_d (m3/N-s)	# tests	τ_c 90% Confidence Intervals (±%)	k_d 90% Confidence Intervals (±%)	Location
775	0.47	1.84E-05	4	55%	32%	Eroding Bank
772.5	2.09	1.37E-06	2	3%	49%	Clay layer under sand bar
768	2.43	2.72E-06	4	32%	11%	Clay layer under sand bar

a. As seen in Table 1, the cohesive material at both bars possesses similar critical shear stress and erodibility, while the eroding bank exhibits lower critical shear and higher erodibility.

b. Hanson and Simon (2001) discuss cohesive material in terms of the diameter of non-cohesive sediment that would have a similar critical shear stress. Their analysis uses the following equation:

$$\tau_* = \tau_c/(\gamma_s - \gamma_o)d \qquad \text{Case Study 4B Equation 1}$$

where:

τ_* = dimensionless shear stress, with common values in the literature ranging from 0.03 to 0.06
τ_c = critical shear stress
γ_s = the specific weight of the sediment
γ_o = the specific weight of water
d = the diameter of the sediment

c. By assuming a value for τ*, the "equivalent diameter" of non-cohesive sediment for a given critical shear stress can be computed. Using common values for the dimensionless shear stress (τ*) the eroding bank has an equivalent critical shear stress of fine to medium sand and the cohesive bars, coarse to very coarse sand (Table 2).

Case Study 4B Table 2
Size of Non-Cohesive Particles with Equivalent Critical Shear Stress

River Mile	τ_c, Pa	d equiv, mm ($\tau_* = 0.03$)	d equiv, mm ($\tau_* = 0.047$)	d equiv, mm ($\tau_* = 0.06$)	Equivalent Sediment Size Description
775	0.47	0.97	0.62	0.48	Fine to medium sand
772.9	2.09	4.29	2.74	2.15	Very coarse sand to fine gravel
768	2.43	5.01	3.20	2.51	Coarse to very coarse sand

d. Table 3 provides more information on the measured grain sizes in both the overburden and the underlying cohesive layers. A comparison with Table 2 indicates no simple correlation between the sizes of the cohesive sediment, overburden sediment, or equivalent diameters. This is likely due to the very site-specific nature of erodibility parameters in cohesive sediments, but it may also be due to an insufficient sample size for determining a complex relationship that depends on both percent sand and percent clay.

Case Study 4B Table 3
Sediment Gradations at Sample Sites

River Mile	τ_c, Pa	D_{50} (mm)	% Sand	% Silt	% Clay	D_{50} (mm) Overburden
775	0.47	0.033	3.5%	86%	11%	NA
772.9	2.09	0.050	36%	53.6%	**10.4%**	0.207
768	2.43	0.037	17.7%	65.8%	16.5%	9.52

5. Sources of Error/Sensitivity Analysis.

Sources of error include curve-fitting and extrapolation per Blaisdell et al. (1981), shape of the scour hole, measurement error of jetting time, and measurement error of the head. Beyond these and other sources of error, soil properties are inherently variable and site-specific. Soils within a few feet of each other can have different erosional properties, not due to error, but rather to real differences in depositional and compaction history.

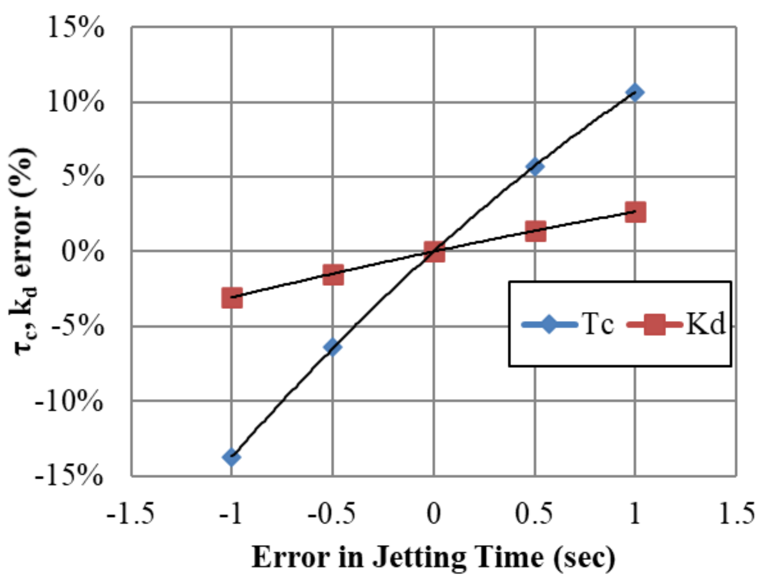
a. The Blaisdell et al. (1981) method for extrapolating the maximum scour depth, critical shear stress, and erodibility introduces potential error to the results. This error is inversely related to the total jetting time. Mazurek and Gheisi (2009) found that the Blaisdell method over-predicts the depth of the scour hole, under-predicts τc, and over-predicted k_d compared with tests run to completion.

b. Weidner (2012) found that the shape of the scour hole had a significant effect on the jet-testing results.

(1) Wide, shallow holes returned results generally consistent with the assumptions in the jet-testing equations, meaning that shear stress on the surface of the hole can be described using equations for jet impingement on a flat plate. Weidner found that wide, shallow holes tend to over-predict the critical shear stress and under-predict the erodibility by up to 10%.

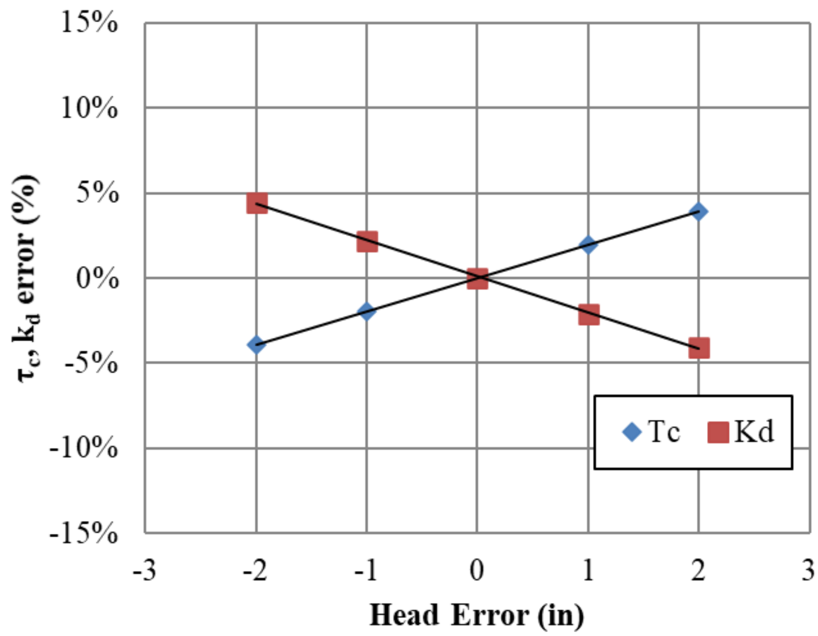
(2) Narrow, deep holes produce more friction, which leads to a significant over-prediction of applied shear stress. This translates into a much greater over-prediction of critical shear stress and under-prediction of erodibility. In contrast to extrapolation error, this error increases with increased jetting time, as over time the scour hole becomes narrower and deeper. During the field work, tests with narrow, deep scour holes were terminated early and their results were not used in computing average soil characteristics.

c. Error in the measurement of jetting time is limited to perhaps a half a second. However, with jetting times of as little as 5 seconds, a half second represents a 10% error in the jetting time. Sensitivity to the jetting time was assessed for Test #1 at RM 768 (Figure 8). Figure 8 was generated by adding or subtracting the seconds shown on the x-axis from each jetting time value, then re-computing in τc and k_d. τc is found to be more sensitive than k_d to small changes in jetting time. A half-second increase in jetting time leads to a 6% increase in τc. In the field, the deflector plate is rotated into and out of the path of the water jet in order to allow jetting for the desired period of time. Figure 8 emphasizes the precision required for engaging and disengaging the deflector plate to achieve accurate jetting times.



Case Study 4B Figure 8. Error in τ_c and k_d from error in jetting time

d. The sensitivity of the results to the measurement of head was assessed for Test #4 at RM 768, and is shown in Figure 9. Potential error in head measurement of up to two inches yields error of up to 4% in τ_c and k_d. Results are relatively insensitive to errors on the scale of potential errors in head measurement under field conditions.



Case Study 4B Figure 9. Insensitivity to potential errors in head measurement

e. Greater precision in starting and stopping the jet and in field measurements can reduce the potential errors displayed in Figures 8 and 9. Error inherent to extrapolating to the maximum depth can be reduced by longer total jetting times. Error due to the shape of the scour hole can be limited by only using tests with wide, shallow holes. A better description of real variability and a more representative average for a given feature or soil type require a greater number of tests on the same material in close proximity.

6. Conclusions.

Field reconnaissance of the Missouri River from River Mile 780 to 660 indicated that most banks, bars, and islands are composed of sand. Three cohesive soils were located and tested, one eroding bank and two mid-channel bars. The bank material was found to be more erodible than the bar material. The cohesive materials have critical shear stresses that are characteristic of fine to very coarse sands. Due to the preponderance of sands in this reach of the Missouri River, and due to the equivalent diameters of the cohesive material falling in the sand range, sand properties should be used for bank erosion modeling in this reach of the river in the absence of site-specific information.

Case Study 6A
Estimating Channel Forming and Effective Discharge

1. Introduction.

This case study presents excerpts from previously published USACE documents related to the determination of channel-forming and effective discharge.

a. For more information, refer to "Effective Discharge Calculation," ERDC/CHL Coastal and Hydraulics Engineering and Technical Note (CHETN)-VII-4 (Biedenharn and Copeland 2000), "Channel-Forming Discharge," ERDC/CHL CHETN-VIII-5 (Copeland et al., 2000), and "Effective Discharge Calculation: A Practical Guide," ERDC/CHL TR-00-15 (Biedenharn et al., 2000). Although not universally accepted, the concept of a channel-forming discharge is found in many sedimentation discussions on stability, including Copeland et al. (2001) and NRCS (2007).

b. Channel-forming discharge refers to the concept that natural alluvial streams experience a wide range of discharges and adjust their shape and size during flow events that have sufficient energy and duration to mobilize either the stream's bed or banks. This discharge, therefore, dominates channel form and process and may be used to make morphological inferences. Using a single representative discharge is the foundation of "regime" and "hydraulic geometry" theories for determining morphological characteristics of alluvial channels and rivers. This representative discharge has been given several names by different researchers, including dominant discharge, channel-forming discharge, effective discharge, and bankfull discharge (Biedenharn et al., 2000).

c. Using many similar terms to describe these concepts has led to some confusion. For USACE study purposes, the channel-forming discharge and the dominant discharge are generally equivalent, and are defined as a theoretical discharge that, if maintained indefinitely, would result in the same channel geometry as the existing channel subject to the natural range of flow event. This definition should be regarded as conceptual and likely not physically feasible since bankline vegetation, bank stability, and even the bed configuration would be different in a natural stream than in a constant discharge stream.

2. Concept Background.

a. While alluvial rivers have the potential to adjust their shape and dimensions to all flows that transport sediment, Inglis (1947) suggested that for rivers that are in regime, a single steady flow could be identified which would produce the same bankfull dimensions as the natural sequence of events (Biedenharn et al., 2000). Based on field observations, Inglis (1947) determined that the channel-forming discharge was approximated by flows at or about bankfull stage. This finding is supported by subsequent research (Nixon 1959, Simons and Albertson 1963, Kellerhals 1967, Hey and Thorne 1986).

b. These studies have not, however, explained why bankfull flow controls channel form. Wolman and Miller (1960) found that the effective discharge corresponds to an intermediate flood flow since frequent minor floods with both shorter durations and smaller peaks transport too small of a cumulative sediment load to have a marked impact on the gross features of the channel, while catastrophic events, which individually transport large sediment loads, occur too infrequently to be effective in forming the channel.

c. The potential for large floods to disrupt the regime condition and cause major channel changes is recognized by this concept, but large floods are not the channel-forming events, provided that the return period of these extreme events is longer than the period required for subsequent, lesser events to restore the long-term, average condition (Wolman and Gerson 1978). Subsequently, the flow transporting the most sediment was named the effective discharge (Andrews 1980).

d. Equivalence between bankfull and effective discharges for natural alluvial channels that are in regime has been demonstrated for a range of river types (sand, gravel, cobble, and boulder-bed rivers) in different hydrological environments (perennial, humid and slightly ephemeral, and semiarid) provided that the flow regime is adequately defined, and the appropriate component of the sediment load is correctly identified (Andrews 1980; Carling 1988; Biedenharn et al., 2000; Soar and Thorne 2001).

3. Channel-Forming Discharge Methods.

The channel-forming (dominant) discharge is defined as a theoretical discharge that, if maintained indefinitely, would produce the same channel geometry as the natural long-term hydrograph. Channel-forming discharge concepts are applicable to stable alluvial streams (such as streams that have the ability to change their shape while neither aggrading nor degrading). For channels in arid environments where runoff is generated by localized high-intensity storms and the absence of vegetation ensures that the channel will adjust to each major flood event, the channel-forming discharge concept is generally not applicable.

a. The content of this section is primarily condensed from “Channel-Forming Discharge,” ERDC/CHL CHETN-VIII-5 (Copeland et al., 2000) and “Hydraulic Design of Stream Restoration Projects,” ERDC/CHL TR-01-28 (Copeland et al., 2001).

b. Estimating Methodologies. Channel-forming discharge concepts are applicable to stable stream systems (such as streams that are neither aggrading nor degrading). While many techniques and methodologies are used to estimate a channel-forming discharge in stable alluvial channels, all can be characterized as one of four main types. The several methods to estimate channel forming discharge are based on (NRCS 2007) as listed in Table 1.

Case Study 6A Table 1
Channel-Forming Discharge Estimating Methods

Method	Description
Bankfull Indices	Most commonly defined as the maximum discharge that the channel can convey without flowing onto its floodplain
Effective Discharge	Defined as the discharge that transports the largest fraction of the average annual bed-material load; also, as the mean of the arithmetic discharge increment that transports the largest fraction of the annual sediment load over a period of years
Specific Recurrence Interval	Typically between the mean annual and five-year peak; varies with stream type and region
Drainage Area	Empirical relationships available for various basins that correlate dominant discharge to drainage area; offer a quick technique for assessing a dominant discharge; often highly variable

c. None of these methods should be assumed to provide the best estimate of the channel-forming discharge without confirmation using field indicators of geomorphic significance. Limitations of each of these methods must be considered by the user. The selection of the appropriate method should be based on data availability, physical characteristics of the site, level of study, and time and funding constraints. If possible, a recommended practice is that all methods be used and cross-checked against each other to reduce the uncertainty in the final estimate.

4. Bankfull Discharge.

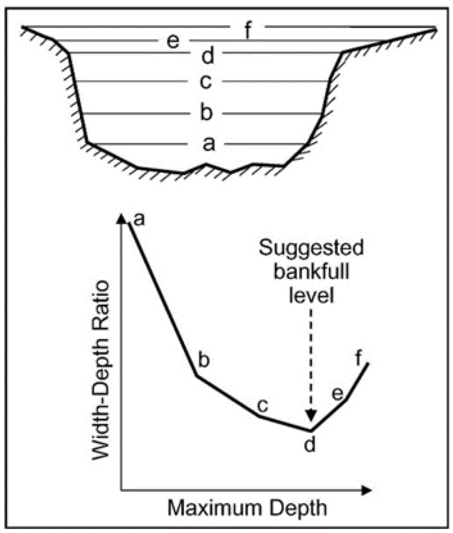
Bankfull discharge is the maximum discharge that the channel can convey without overflowing onto the floodplain. This discharge is considered to have morphological significance because it represents the breakpoint between the processes of channel formation and floodplain formation.

a. Bankfull discharge is determined by first identifying bankfull stage within the field and then determining the discharge associated with that stage. Identifying the relevant field features that define the bankfull stage can be problematic. In reaches where stage is dominated by a downstream backwater, determining bankfull discharge from bankfull stage may not be generally applicable. Many field indicators for bankfull stage have been proposed, but none appear to be generally applicable or free from subjectivity (Williams 1978).

b. The most common definition of bankfull stage is the elevation of the active floodplain (Leopold and Wolman 1957; Nixon 1959). Another common definition of bankfull stage is the elevation where the width-to-depth ratio is a minimum (Wolman 1955; Pickup and Warner 1976). This definition, diagramed in Figure 1, is systematic and relies only on accurate field surveys. In some cases, the highest elevation of channel bars may be used as an indicator of bankfull stage (Leopold and Wolman 1957).

c. Woodyer (1968) defines the bankfull stage of rivers having several overflow surfaces as the elevation of the middle bench. Wolman (1955) combines the width-to-depth ratio criterion with identifying a discontinuity in the channel boundary such as a change in its sedimentary or vegetative characteristics. Schumm (1960) defined bankfull stage as the height of the lower limit of perennial vegetation, primarily trees. Similarly, Leopold (1994) states that bankfull stage is indicated by a change in vegetation, such as herbs, grasses, and shrubs.

d. Given the number of criteria commonly used to define bankfull stage and the considerable experience required to apply them, it is not surprising that there can be wide variability in field determination of bankfull stage. Bankfull stage indictors in stable reaches should generally follow a longitudinal trend consistent with the energy gradeline of the stream and/or valley floodplain slope; plotting field-identified stage indicators longitudinally allows for developing an envelope that spans a reasonable range of natural variability (Figure 4). Subsequent hydraulic evaluation can be used to estimate the flow range associated with the range of bankfull stage variability and its relative significance.



Case Study 6A Figure 1. Bankfull depth using width-to-depth ratio (after Knighton 1984, from Copeland et al., 2001)

e. The field identification of bankfull indicators is often difficult and subjective, and should be performed only in stream reaches that are stable and alluvial (Knighton 1984). The stream reach should be identified as stable and alluvial before field personnel attempt to identify bankfull stage indicators. If the project reach is unstable (or non-alluvial), it may be possible to find indicators of bankfull stage in stable alluvial reaches upstream or downstream on the same stream. The process of identifying bankfull indicators is often an iterative process that involves a great deal of judgment (Copeland et al., 2001).

(1) If a reach is not stable and alluvial, indicators of bankfull stage are unreliable. Some examples are given below:

(a) If a reach is non-alluvial, then sediment transport capacity normally exceeds sediment supply, and deposits are missing or underdeveloped. Using underdeveloped deposits as bankfull indicators results in too low a channel-forming discharge. Deposits could also be relics of extreme flood events; in which case they normally give too high a channel-forming discharge.

(b) If the channel is degrading, then sediment transport capacity exceeds sediment supply, and the observations above for the non-alluvial channel hold true. In addition, since the bed of the channel is lowering, former floodplain deposits are being abandoned (they are in the process of becoming terraces). Using these features as indicators gives too high a channel-forming discharge.

(c) If the channel is aggrading, the in-channel deposits could be incorrectly mistaken for bankfull stage indicators. Since the bed of the stream is rising, using the existing floodplain as an indicator gives too low a discharge. (The floodplain will aggrade as well, but usually at a slower rate than the channel.)

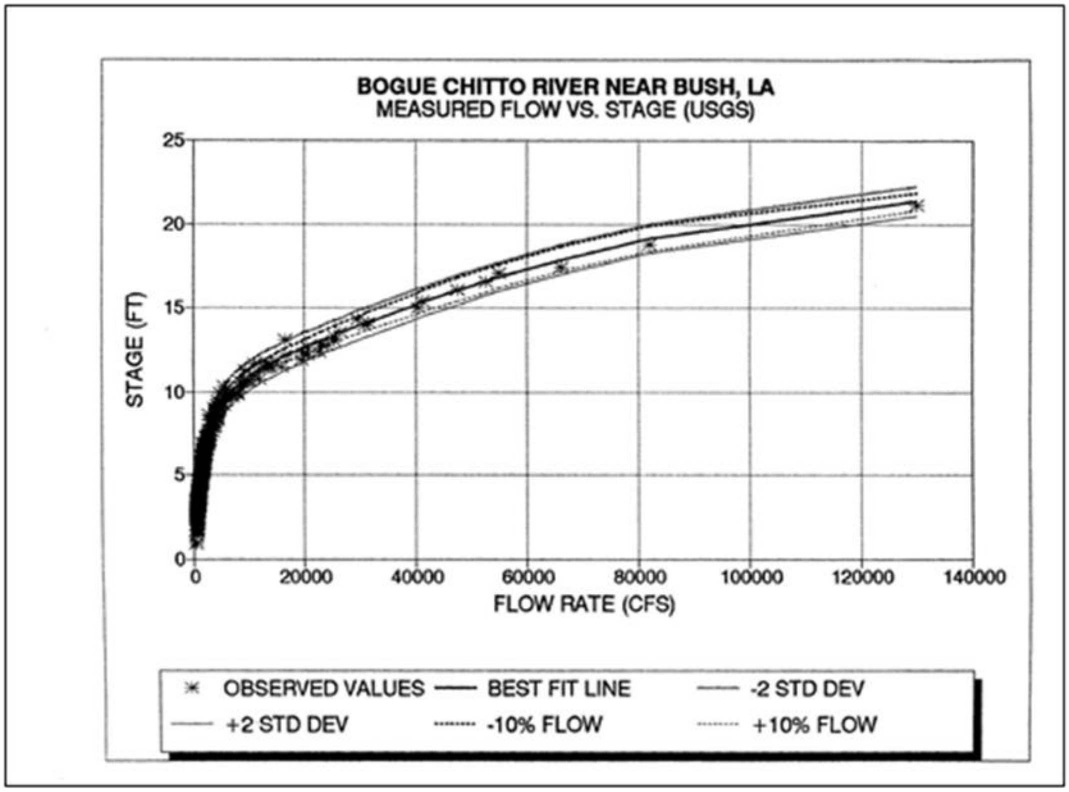
(d) If the channel hydrology is decreasing (such as multi-year drought or significant increases in upstream regulation or irrigation), then the channel may be slowly adjusting to develop a lower inset active channel, and the prior bankfull elevation (while appearing reasonable) may eventually be abandoned.

(2) Confusion often occurs when criteria suggest a bankfull stage at an elevation that is not close to the top of either bank. This condition suggests that the channel may not be in equilibrium, that the existing channel geometry may not be stable, and that the channel-forming discharge would be poorly approximated by the bankfull discharge.

(3) Since USACE projects for stream restoration are often implemented in unstable channels and watersheds (instability is often the reason for restoration), field determination of bankfull stage may be impractical or impossible. In fact, attempting to determine a channel-forming discharge from an unstable stream conflicts with the theoretical premise of a stable system that is the basis for the channel-forming discharge concept (Copeland et al., 2001).

(4) Once bankfull stages are estimated for a reach of the stream, then the corresponding bankfull discharge can be estimated. Ideally, the discharge associated with bankfull stage can be determined from a stage-discharge rating curve based on measured data at the project site.

(5) When floodplain conveyance is significant with respect to channel conveyance, there is a distinct break in the stage-discharge rating curve at bankfull stage as shown in Figure 2. The data scatter in Figure 2 occurs because stage is not a unique function of discharge in alluvial streams. It is therefore necessary to estimate a rating curve through the data scatter. It is best to consider that the bankfull discharge has a range rather than a single discrete value. Uncertainty associated with the stage-discharge relationship is addressed in EM 1110-2-1619 (USACE 1996).

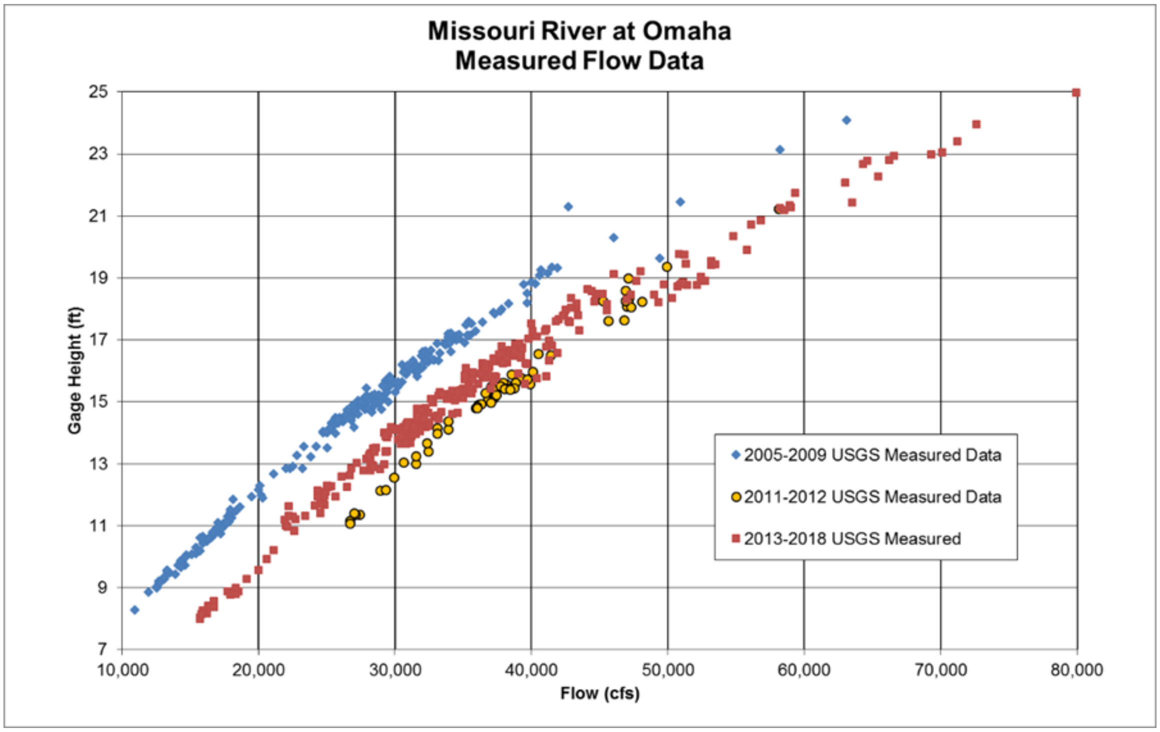


Case Study 6A Figure 2. Stage-discharge rating curve, Bogue Chitto River near Bush, Louisiana (Copeland et al., 2001)

f. In stream types where floodplain conveyance is not significant with respect to channel conveyance, there may not be a distinct break in the stage-discharge rating curve (Figure 3). In this case, the bankfull discharge may not have as much morphological significance as when floodplain flow is significant.

g. Figure 3 below illustrates an issue within a presumed stable system, the Missouri River at Omaha, Nebraska, due to the extreme event of 2011. The period before the 2011 event (blue triangles from 2005 to 2009) had a gage height of about 15.6 at 30,000 cfs. Following the 2011 event, the period from 2013 to 2018 also has a fairly stable gage height of about 13.8. The period of 2011 through 2012 shows an adjustment from the maximum degradation with a gage height of 12.6 at 30,000 cfs. Therefore, while some rebound occurred initially from the extreme flows of 2011, full recovery back to pre-2011 event stages has not occurred. In this case, two distinct rating curves from two time periods are present.

h. For comparison purposes, the 2-year event for the Missouri River at Omaha is about 64,000 cfs. Many anthropogenic actions have modified the Missouri River, including navigation channel structures, adjacent levees, and upstream reservoir systems with flow regulation and massive sediment retention. When long-term gage records are available, a comparison of rating shifts can help identify reaches that are cyclically stable versus on a diverging trend.

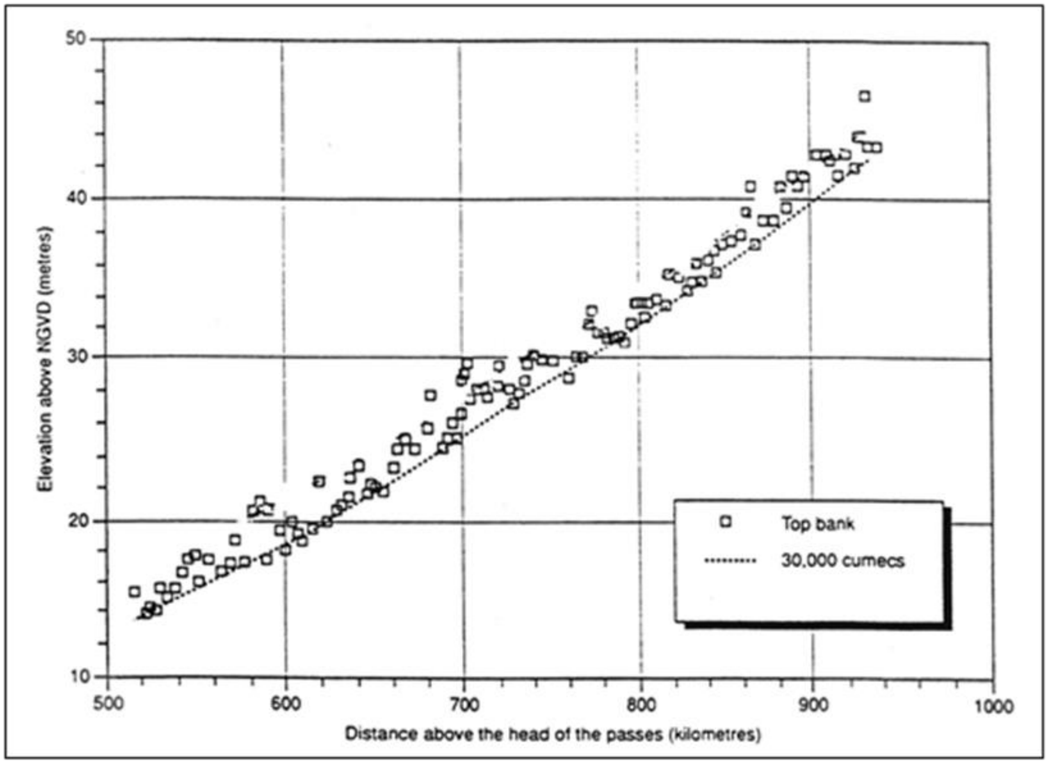


Case Study 6A Figure 3. Measured flows, Missouri River at Omaha, Nebraska

i. Lacking gage data at the project site, a stage-discharge rating curve can be determined via a hydraulic computational model using programs such as HEC-RAS.

(1) The accuracy of the computed rating curve depends on the model calibration to available data, uncertainties associated with assigned hydraulic roughness coefficients, and the cross-section geometry. Uncertainty is greatest when the stage-discharge rating curve is estimated from a single cross section. In this case, both hydraulic roughness and energy slope must be assigned. It is best if the determination of bankfull stage occurs over a reach of at least one planform wavelength or 10 channel widths, and should be measured at the riffle cross-over locations where the most stable geomorphic signals are typically located.

(2) An example of a comparison of bankfull stage and a computed water-surface elevation is shown in Figure 4. Note in Figure 4 that bankfull stage is taken to be at the bottom of the top-of-bank data scatter because this represents the elevation at which flow onto the floodplain begins. Also note that considerable variability in bankfull stage could be estimated if only a single top-of-bank point were used in the analysis. The Hydraulic Engineer determines what method is best suited to compute the bankfull discharge from the bankfull stage indicators. For example, hydraulic model computations may be required in some cases, while normal depth computations are sufficient in others.



Case Study 6A Figure 4. Long-channel variation in bank top elevations: Lower Mississippi River (Biedenharn and Thorne 1994; Copeland et al., 2001)

j. Guidelines relative to field determination of bankfull discharge and the use of bankfull discharge as the channel-forming discharge are provided as:

(1) Bankfull discharge is geomorphologically significant only in stable alluvial channels. Therefore, the reach where bankfull stages are determined should be stable and bed material load size fractions of the streambed should be mobile at bankfull flow.

(2) When the bankfull discharge is used to determine channel dimensions for the main channel, the field indicators used for the identification of the bankfull stage must be top-of-bank indicators. A stage identified by the edge of the active channel, the beginning of woody vegetation, or the top-of-channel bars may have value for designing those particular features in a restored channel, but should not be used for establishing the bank height of a stable channel. Only bankfull discharges, which are top-of-bank discharges, are morphologically significant in establishing the channel-forming discharge.

(3) An exception to the above rule is in a stable and alluvial incised stream that has formed a new floodplain within the incised channel. In this case, the top of the high bank is now an abandoned floodplain or terrace, and there should be newly formed top-of-bank features within the older incised channel. However, it is important to remember that the new floodplain may not yet be fully formed, that is, the channel may not be stable (it may still be aggrading). This gives misleading values for the bankfull discharge.

(4) Assuming that the bankfull discharge for one reach of a stream is the same as the bankfull discharge in another reach may not be appropriate. For example, the bankfull discharge taken from a reach with a narrow floodplain may be inappropriate for use on another reach on the same stream, which has a wide floodplain. The location of the break between the channel and the floodplain is influenced by many factors, including (but not limited to) the following:

(a) Confinement of the floodplain.

(b) Hydrologic regime.

(c) Sediment supply.

(d) Bed and bank sediment size and cohesiveness.

(e) Size and type of vegetation on the floodplain and within the channel.

(f) Controls on channel width, slope, and alignment.

5. Specified Recurrence Interval.

a. Due to difficulties in the identification of bankfull discharge and stage, many researchers have related the channel-forming discharge to a specific recurrence interval discharge. In these studies, the researchers have typically studied stable streams where bankfull stage could readily be determined and where streamgages were located nearby.

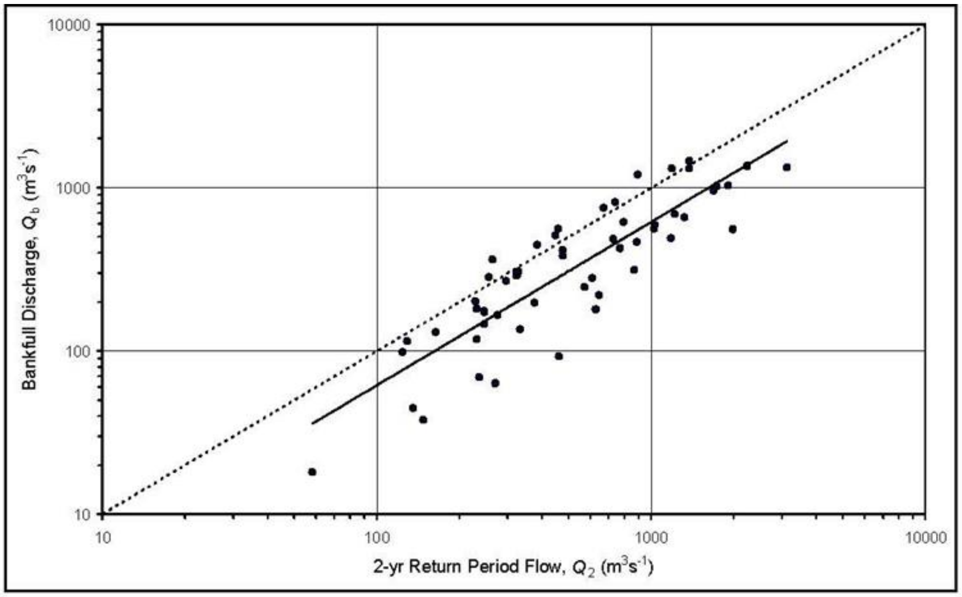
b. Under these conditions, bankfull discharge is assumed to be the channel-forming discharge, and most of the literature addressing specified return interval discharge use the two terms interchangeably. This can be confusing, as studies are actually comparing two methods for approximating the channel-forming discharge, and not actually comparing an approximation method to the true value.

c. In general, bankfull discharge in stable alluvial channels has been found to correspond to an annual flood recurrence interval of approximately 1 to 2.5 years and the 1.5-year recurrence flood has been shown to be a representative mean of many streams (Leopold 1994). However, there are many instances where the channel-forming discharge does not fall within the 1 to 2.5-year range.

d. Recurrence interval relations are intrinsically different for channels with flashy hydrology than for those with less variable flows. For instance, Williams (1978) clearly showed that of 35 floodplains he studied in the United States, the bankfull discharge varied between the 1.01- and 32-year recurrence interval, and that only about a third of those streams had a bankfull discharge recurrence interval between 1 and 5 years.

e. In a similar study, Pickup and Warner (1976) determined that bankfull recurrence intervals ranged from 4 to 10 years. Because of such discrepancies, many have concluded that recurrence interval approaches tend to generate poor estimates of bankfull discharge. Hence, field verification is recommended to ensure that the selected discharge reflects morphologically significant features.

f. As shown in Figure 5, the 2-year flow event is greater than the bankfull discharge in most cases, and provides an adequate upper bound to the range of bankfull discharges. The best-fit line in Figure 5 is linear at the 95% significance level, and represents bankfull discharges at approximately 60% of 2-year flow over the range of the data.



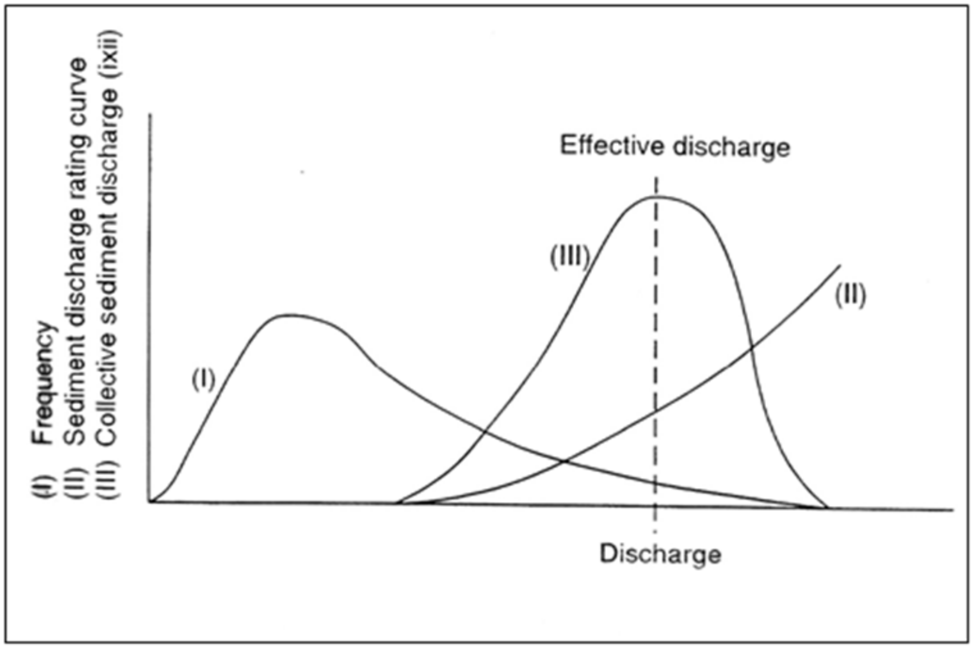
Case Study 6A Figure 5. Relationship between the 2-year return period flow, Q_2, and bankfull discharge (Q_b) for 57 U.S. sand-bed rivers (Copeland et al., 2001); solid line is the best-fit power relationship; dotted line is equality

6. Effective Discharge.

Effective discharge is defined as the mean of the arithmetic discharge increment that transports the largest fraction of the annual sediment load over a period of years (Andrews 1980). Effective discharge transports the largest fraction of the bed-material load. While often a good estimator for channel-forming discharge, it should not be assumed to be equivalent to the channel-forming discharge without confirmation using field indicators of geomorphic significance. The content of this section is primarily condensed from “Effective Discharge Calculation: A Practical Guide,” ERDC/CHL TR-00-15 (Biedenharn et al., 2000).

a. The effective discharge incorporates the principle prescribed by Wolman and Miller (1960) that the channel-forming discharge is a function of both the magnitude of the event and its frequency of occurrence. It is calculated by integrating the flow-duration curve and a bed-material-sediment rating curve. A graphical representation of the relationship between sediment transport, frequency of the transport, and the effective discharge is shown in Figure 6. The peak of curve C from Figure 6 marks the discharge, which is most effective in transporting sediment, and therefore hypothesizes that it does the most work in forming the channel.

b. Effective and bankfull discharges are not always equivalent, as reported by Benson and Thomas (1966), Pickup and Warner (1976), Webb and Walling (1982), Nolan, Lisle, and Kelsy (1987), and Lyons, Pucherelli, and Clark (1992). This suggests that the effective discharge may not always be an adequate surrogate for the channel-forming discharge.



Case Study 6A Figure 6. Derivation of total sediment load-discharge histogram (III) from flow frequency (I) and sediment load rating curves (II) (Copeland et al., 2001)

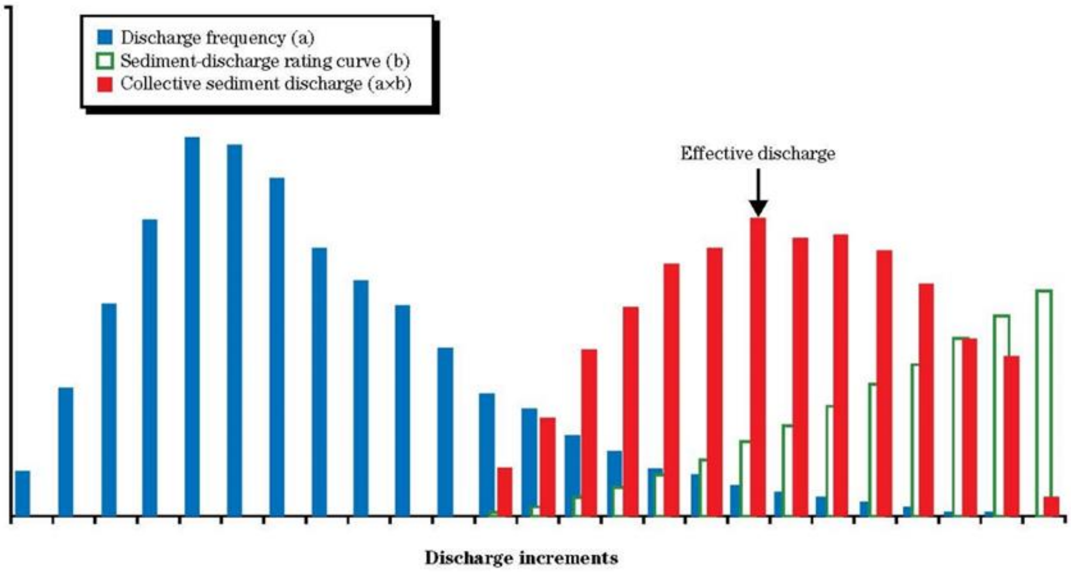
c. The following procedure for effective discharge calculations has been developed for a range of river types. It is a systematic method designed to have general applicability. The effective discharge procedure requires both hydrological and sediment data. The procedure to determine the effective discharge is executed in three major steps which are as follows:

(1) The flow-frequency distribution is determined from available flow-duration data.

(2) Sediment data are used to construct a bed-material-load rating curve.

(3) The flow-frequency distribution and bed-material-load rating curve are combined to produce a bed-material-load histogram which displays sediment load as a function of discharge for the period of record. The histogram peak shown in Figure 6 indicates the effective discharge.

d. Descriptive flow charts illustrating the methods in each of the three major steps are provided in “Effective Discharge Calculation: A Practical Guide” (Biedenharn et al., 2000). Figure 7 provides a graphical representation of the relationship between sediment transport, frequency of the transport, and the effective discharge.



Case Study 6A Figure 7. Graphical representation of relationship between flow, sediment rating, and sediment discharge (NRCS 2007)

Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

e. Flow-Frequency Distribution. The flow-frequency distribution is developed from a flow-duration curve (EM 1110-2-1415). The flow-duration curve can be developed from gage data from in or near the project reach or from physiographically similar watersheds.

(1) Using Gage Data. The record from a single gaging station can be used to develop the flow-duration curve if the gage is close to the project reach and the discharge record at the gage is representative of the flow regime in the project reach.

(a) It is important that watershed conditions have remained unchanged during the selected historical flow period.

(b) The period of record must be sufficiently long to include a wide range of morphologically significant flows, but not so long that changes in the climate, land use, or runoff characteristics of the watershed produce significant changes with time in the data.

(c) A reasonable minimum period of record for an effective discharge calculation is about 10 years, with 20 years of record providing more certainty that the range of morphologically significant flows is fully represented in the data.

(d) For gages with long periods of record, it can be helpful to discretize the record with a moving window of ~10 to 20 years to identify any long-term temporal trends.

(2) Mean daily discharges are conventionally used to construct the flow-duration curve. However, this can, in some cases, introduce error into the calculations because mean daily values

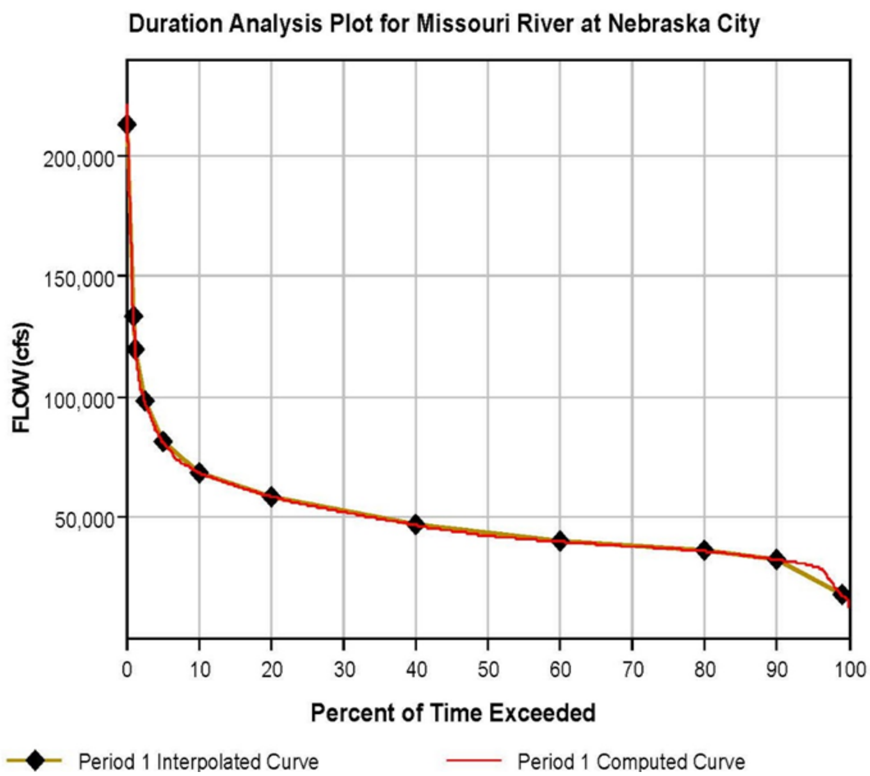
can under-represent the occurrence of short-duration, high-magnitude flow events that occur within the averaging period.

(a) On large rivers such as the Mississippi River, using the mean daily values is acceptable because the difference between the mean and peak daily discharges is negligible.

(b) On smaller streams, flood events may last only a few hours, so that the peak discharge is much greater than the corresponding mean daily discharge.

(c) The time base for discharges used to develop the flow-duration curve should be sufficiently short to ensure that short-duration, high-magnitude events are properly represented.

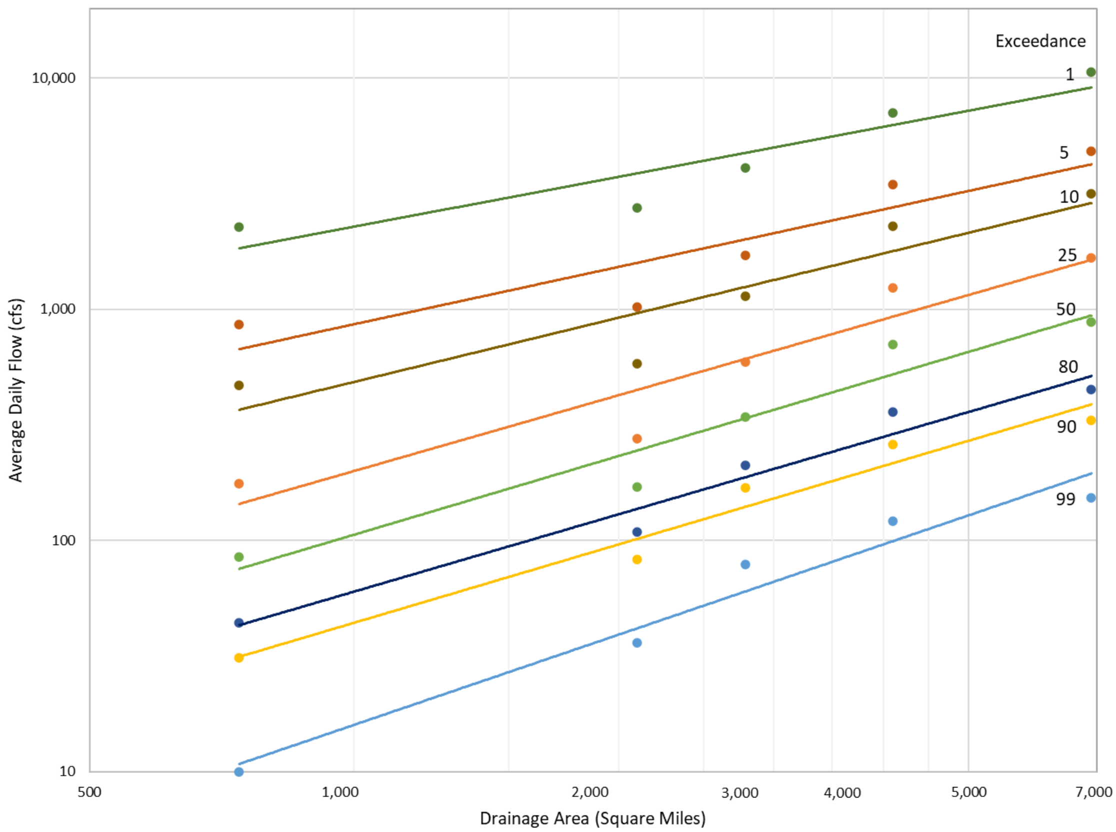
(d) Flow duration computations have been automated using available software such as HEC-SSP (USACE 2016d). Figure 8 illustrates results from an HEC-SSP flow duration analysis.



Case Study 6A Figure 8. Flow duration analysis using Hydrologic Engineering Center-Statistical Software Package (HEC-SSP), Missouri River at Nebraska City

(3) When Gage Data Is Not Available. At locations where gaging records are either unavailable or are found to be unrepresentative of the flow regime, it is necessary to synthesize a flow-duration curve. Two possible methods of doing this are: (a) use records from nearby gaging stations within the same drainage basin, or (b) develop a regionalized flow-duration curve.

(a) Using Records from Nearby Gage Stations. The drainage basin flow-duration method relies on the availability of gaging station data at a number of sites on the project stream. Flow-duration curves for each gaging station are derived for the longest possible common period of record. Provided there is a regular downstream decrease in the discharge per unit watershed area, then a graph of discharge for various exceedances against contributing drainage area should produce a power function with insignificant scatter about the best-fit regression line. Figure 9 shows this relationship for the Elkhorn River, Nebraska, using daily data from five USGS gage stations. This method enables the flow-duration curve at an ungaged site on that river to be determined as a function of its drainage area.



Case Study 6A Figure 9. Downstream daily flow-duration curve, Elkhorn Rive, NE (Various USGS Gage Stations)

(b) Regionalized Flow Duration Curve. A regional-scaling method based on data from watersheds with similar characteristics can be used to generate a flow-duration curve for an ungaged site.

- Emmett (1975) and Leopold (1994) suggest using the ratio of discharge Q to bankfull discharge Q_{bf} as a non-dimensional index Q/Q_{bf} to transfer flow-duration relationships between basins with similar characteristics.

– However, bankfull discharge does not necessarily have either a consistent duration or return period (Williams 1978).

– To avoid this problem, Watson, Dubler, and Abt (1997) proposed a non-dimensional discharge index by using the regionalized 2-year discharge Q_2 to normalize discharges as Q/Q_2. For ungaged sites, the 2-year discharge may be estimated from regionalized discharge frequency relationships developed by the United States Geological Survey (USGS 1993) on the basis of regression relationships between the drainage area, channel slope, and slope length. These relationships are available for most states.

– The dimensionless discharge index (Q/Q_2) can be used to transfer a flow-duration relationship to an ungaged site from a nearby gaged site. The gaged site may be in the same basin or an adjacent watershed.

- A flow-duration relationship can be transferred in a watershed by the following method.

– Develop the Regionalized Flow-Duration Curve. Using a flow-duration curve from a gaged site in a physiographically similar watershed, divide the discharges in the flow-duration relationship by the Q_2 for the gaged site to create a dimensionless flow-duration curve. If more than one gage site is available, an average dimensionless flow-duration curve for all the sites can be developed.

– Compute the Q_2 for the ungaged site.

– Calculate the flow-duration curve for the ungaged site by multiplying the dimensionless ratios from the regionalized flow-duration curve by the ungaged Q_2.

– Divide the flow-duration curve into discharge increments and compute an occurrence frequency for each increment.

f. Bed Material Load Rating Curve. Sediment data are required to generate the bed-material-load rating curve. This data may be obtained from measurements at a gaging station if the gage is close to the project reach and if size-class fractions are provided so that the bed-material portion of the measured load can be determined.

(1) A bed gradation from the project reach is required to determine the division between wash load and bed-material load, and to calculate sediment transport by size class if necessary. The wash load should be excluded from the data set used to develop the rating curve as it is generally not transport limited and typically does not influence alluvial channel morphology.

(2) If the bed-material load moves both as bedload and suspended load (typical of sands), then both bedload and suspended-load measurements are required to determine the bed-material load.

(3) If measured data are insufficient, appropriate equations in the SAM hydraulic design package (Thomas et al., 2002) HEC-RAS (HEC 2016) can be used to generate bed-material loads for selected discharges.

(4) In streams dominated by suspended load, a best-fit regression curve fitted to the data may be adequate to produce a bed-material load function. This frequently takes the form of a power function:

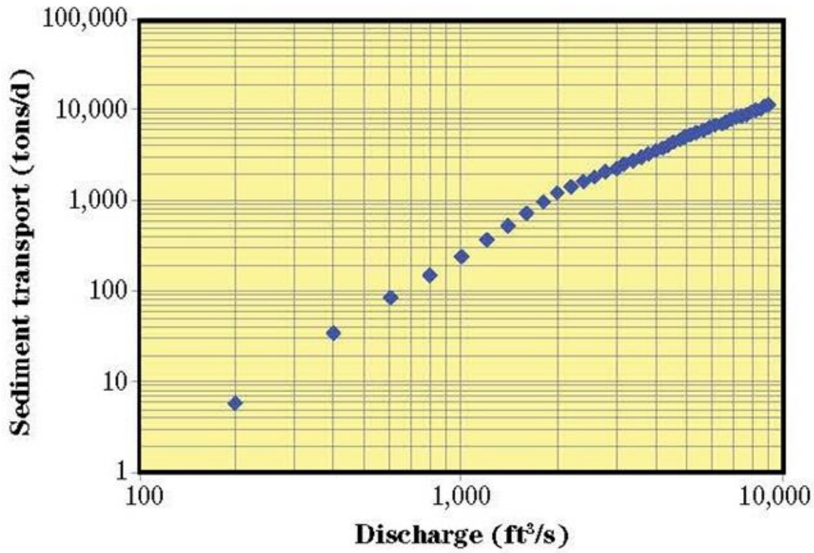
$$Q_s = a\,Q^b \qquad \text{Case Study 6A Equation 1}$$

where Q_s is the bed-material-load discharge, Q is the water discharge, *a* is a regression coefficient, and *b* is a regression exponent.

(5) However, a log-linear power function may not be appropriate in all cases. Sometimes, at high discharges the rate of increase in sediment concentration with discharge begins to decrease, especially for the finer sand sizes. In this case, it may be necessary to use a different curve-fitting function. In coarse bed streams, it is likely that a coarse surface layer will develop at lower discharges, significantly reducing sediment transport potential. This process involves both hydraulic sorting of the streambed and hiding of small particles behind bigger particles.

(6) Typically, calculated sediment-transport rating curves developed from a single bed gradation will overestimate sediment transport at low discharges. This is probably the most important reason for too much sediment being calculated in the lower discharge class intervals.

(7) An example computation is presented by the NRCS (2007). For the example, the sediment transport rating curve was calculated from data collected during field surveys. At this location, the bed material gradation in the upstream supply reach was determined from the average of three volumetric bulk samples taken laterally across the stream. The cross-sectional geometry and slope were surveyed, and hydraulic parameters were calculated assuming normal depth. The Meyer-Peter-Muller equation was chosen to make the sediment calculations because the bed material size was primarily gravel. Figure 10 illustrates the example computation of a bed material rating curve (NRCS 2007).



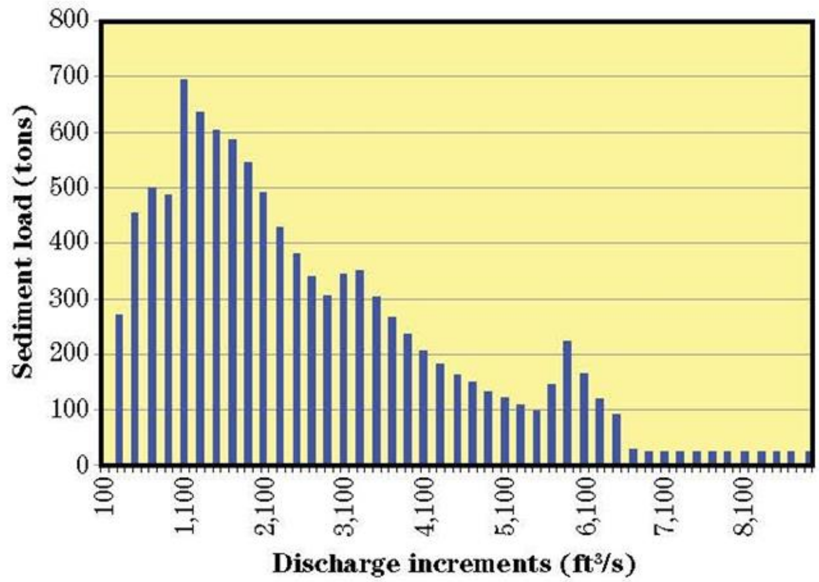
Case Study 6A Figure 10. Sediment transport rating curve calculated from bed material gradation (NRCS 2007)

Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

g. Bed Material Load Histogram. The discharges used to generate the bed-material-load histogram are the mean discharges in each arithmetic class in the flow-frequency distribution. Note that the selection of arithmetic bin size can influence subsequent calculations, and care should be taken to ensure that the bin size is not too large or small as to bias the results. The histogram is generated by using the representative discharges and the bed material load rating curve to find the bed-material load for each discharge class and multiplying this load by the frequency of occurrence of that discharge class. The results are plotted as a histogram representing the total amount of bed material load transported by each discharge class during the period of record.

(1) The bed-material-load histogram should display a continuous distribution with a single mode (peak), and the effective discharge corresponds to the mean discharge for the modal class (the histogram peak). If the modal class cannot be identified, try adjusting the discharge bin size. Alternatively, the effective discharge can be estimated by drawing a smooth curve through the tops of the histogram bars and interpolating the effective discharge from the peak of the curve. If the modal class of the bed-material-load histogram is the lowest discharge class, the indicated effective discharge is likely erroneous. In this case, modify the procedure by either increasing the number of discharge classes or modifying the bed-material rating curve with noted cautions.

(2) An example of results of the combination of the flow duration and bed-material-load rating curve are shown in Figure 11 (NRCS 2007). At this location, the discharge increment with the largest increment of sediment transport is between 1,000 and 1,200 cubic feet per second. The average annual sediment load, which is the sum of the sediment loads for each increment, is 10,677 tons. Ideally, this effective discharge range should be cross-checked with additional indicators as discussed in paragraph 6c below.



Case Study 6A Figure 11. Effective discharge example (NRCS 2007)

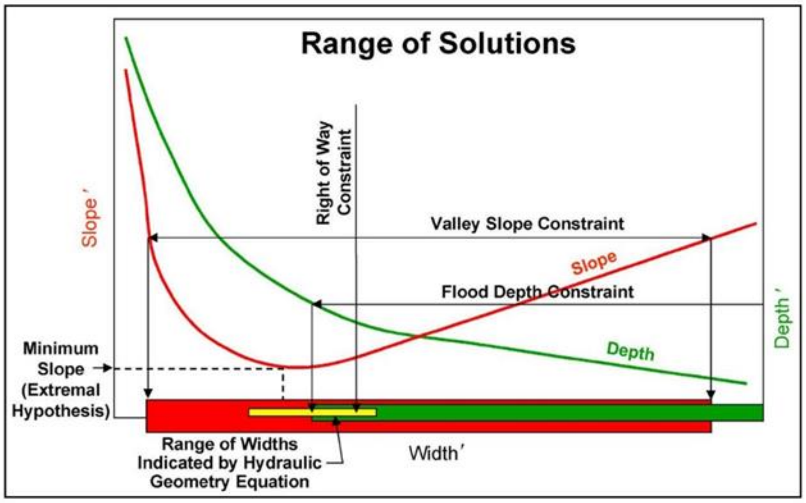
Image courtesy of USDA Natural Resources Conservation Service. No Federal endorsement implied.

7. Application to Channel Restoration Stable Channel Design.

Channel restoration design is the reconstruction of a river channel to a geometric configuration which is self-sustaining and in balance with imposed flow and sediment regimes. Techniques to design restored channels are often based on a combination of field experience, reference reach observations, and basin-wide regime-type curves. While these approaches can yield appropriate target channel dimensions in some cases, they do not explicitly account for upstream sediment supply into the restored channel and the capacity of the restored reach to transmit sediment to the channel downstream. If sediment transport continuity through the restored reach is not achieved, the restored channel dimensions will not be stable (Biedenharn et al., 2000).

a. The stable channel analytical method presented in “Hydraulic Design of Stream Restoration Projects,” ERDC/CHL TR-01-28 (Copeland et al., 2001) is based on the principle of effective discharge. The stable channel analytical method uses a family of solutions for slope and depth for specified widths for a selected discharge. These curves represent combinations of width, depth, and slope that satisfy the sediment transport and roughness equations.

b. Lacking project constraints, a hydraulic geometry relationship with confidence limits for width could be used to select a range of stable slopes and depths, or the extremal assumption can be applied, and the unique solution occurs at the minimum slope on the stable channel design curve. Figure 12 illustrates a generalized stability curve determined with the stable channel analytical method (Copeland et al., 2001).



Case Study 6A Figure 12. Stability curve from Stable Channel Analytical Method (Copeland et al., 2001)

c. Evaluate if the Effective Discharge Is Reasonable. At the end of the procedure, it is important to check that the effective discharge is a reasonable value for the project reach. This is accomplished by comparing the calculated effective discharge with other discrete approximations of the channel-forming discharge.

(1) The return period for the effective discharge is expected to vary between sites, depending on the flow and sediment-transport regime of the individual river or reach. For sites where annual maximum series flood-flow data are available, the return period of the calculated effective discharge may be checked to ensure that it lies within acceptable bounds. Experience indicates that it lies within the range 1.01 and 3 years with a preponderance between 1.01 and 1.2 years, regardless of the type of river (Copeland et al. 2001). Predicted effective discharge return periods outside the range of approximately 1 to 3 years should be queried.

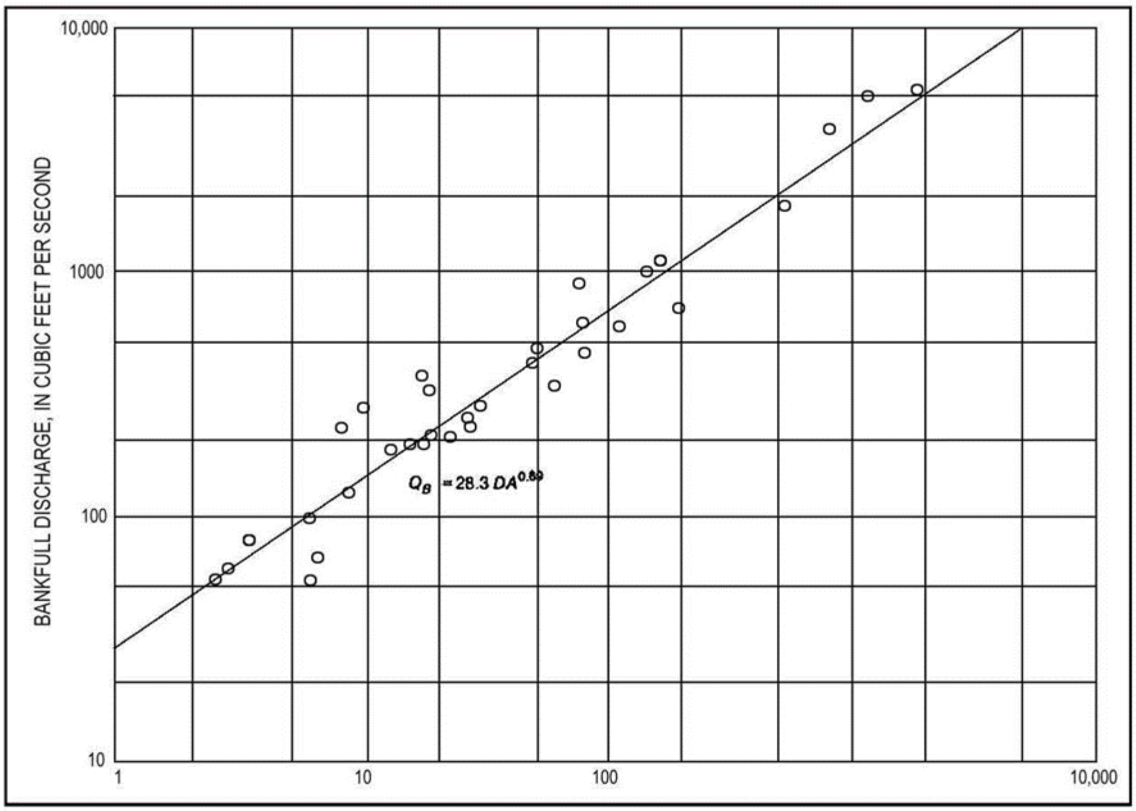
(2) A further check is to compare the duration of the effective discharge with basin area-flow duration curves. The percentage of time the effective discharge is equaled or exceeded should be compared to the expected range of values reported in the literature.

(3) Finally, a morphological check should be undertaken to compare the effective discharge to the bankfull discharge. This is best performed by identifying the bankfull stage at a stable cross section and calculating the corresponding discharge either from the stage-discharge relationship at a nearby gaging station or using the slope-area method.

8. Drainage Area and Channel-Forming Discharge.

Using regional regression curves for determining channel-forming discharge as a sole function of the drainage area is not recommended, as drainage area is only one of many parameters affecting runoff. However, within physiographically similar watersheds, it may be useful to develop a channel-forming discharge versus drainage area curve for use in that watershed.

a. Emmett (1975) developed such a curve for the Salmon River in Idaho (Figure 13). Emmett (1975) chose stable channel reaches for his study and assumed that bankfull discharge was equivalent to channel-forming discharge. Although the regression line fits the data in a visually satisfactory fashion, note that for a drainage area of about 80.6 sq km (70 square miles), the bankfull discharge varied between about 8.50 cu m/s (300 cfs) and 25.48 cu m/s (900 cfs). This large range should not necessarily be attributed to errors in field measurements, but rather to the natural variation in bankfull discharge with drainage area.



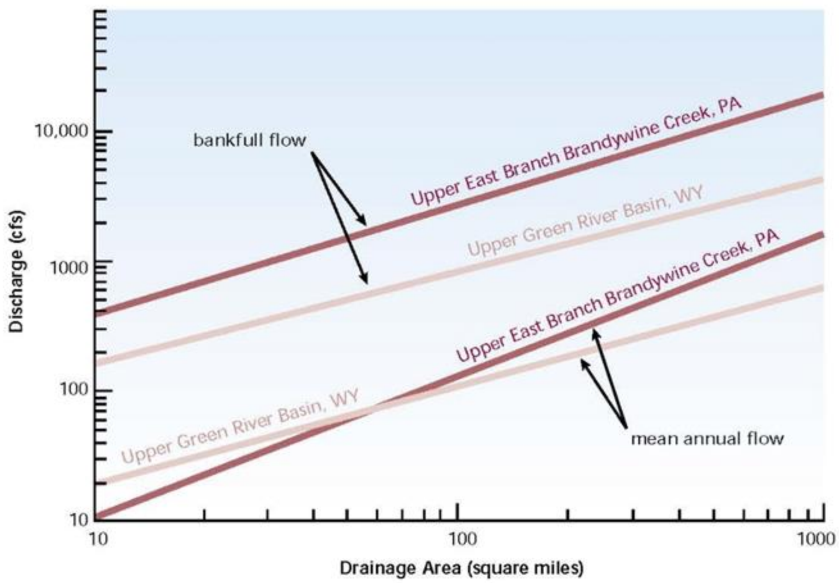
Case Study 6A Figure 13. Bankfull discharge as a function of Salmon River drainage area (Emmett 1975; Copeland et al., 2001)

b. The relationship between the bank full discharge and drainage area shown above takes this form:

$$Q_{bf} = a\, A^b \qquad \text{Case Study 6A Equation 2}$$

where Q_{bf} is the bankfull discharge in cfs, A is the drainage area in square miles, and a and b are regression coefficients. Values for the coefficients a and b may be available from regional evaluations in the study area.

c. Another surrogate for channel-forming discharge in empirical regression equations drainage area that has been developed uses the mean annual flow. Given that both bankfull discharge and the mean annual flow exhibit a similar functional dependence on drainage area, a proportionality is expected between these discharge measures within the same region. Leopold (1994) gives the following average values of the ratio Q_{bf}/Q_m for three widely separated regions of the United States: 29.4 for 21 stations in the Coast Range of California, 7.1 for 20 stations in the Front Range of Colorado, and 8.3 for 13 stations in the Eastern United States (FISWRG 1998). The bankfull discharge is generally greater than the mean annual flow, as shown in Figure 14.



Case Study 6A Figure 14. Regional relationships for bankfull and mean annual discharge as a function of drainage area (FISRWG 1998, after Dunne and Leopold 1978)

9. Summary.

Due to the limited scope of many stream restoration projects, hydraulic design has been attempted using only a single representative discharge.

a. Using a representative or channel-forming discharge may be appropriate for determining initial or preliminary design dimensions, but the difficulty in the determination of the channel-forming discharge and the uncertainty related to the concept itself makes its sole use untenable for reliable and effective hydraulic design.

b. However, the concept of channel-forming discharge is useful and has become an accepted part of channel restoration design and therefore methods to calculate this value are required. All three methodologies for estimating the channel-forming discharge present challenges.

c. The selection of the appropriate method is based on data availability, physical characteristics of the site, level of studies, and time and funding constraints.

d. It is recommended that all three methods be used and cross-checked against each other to reduce the uncertainty in the final estimate of the channel-forming discharge.

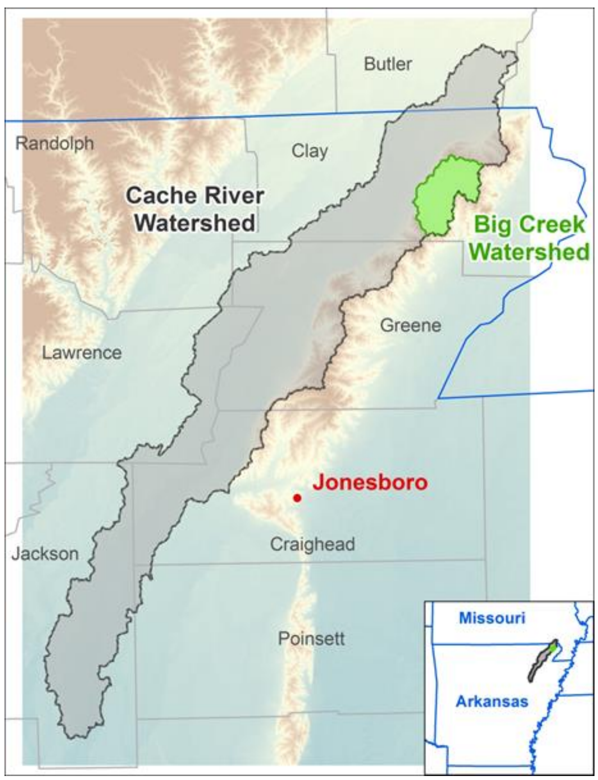
Case Study 6B
Cache River Watershed Sediment Reduction Study

1. Case Study.

This case study provides an example of analysis components involved in a sediment reduction study. The case study provides partial content that was derived from a USACE study conducted by the Memphis District in 2010. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

The sediment reduction study for the Cache River, Arkansas, applied the U.S. Department of Agriculture, Agricultural Research Service's (USDA-ARS) **Ann**ualized **Ag**ricultural **N**on-**P**oint **S**ource (AnnAGNPS) suite of models (Bingner et al. 2018). The Cache River is a major watershed in Northeastern Arkansas (Figure 1).



Case Study 6B Figure 1. Location of Cache River and Big Creek watersheds

a. Chronic sediment deposition in the Cache River watershed led to flooding. Preliminary assessments indicated gully erosion as a significant contributor of sediments. Detailed gully information was not available for the Cache River or Big Creek watersheds. Therefore, evaluating gully erosion for this effort required adaptations to the AnnAGNPS model to best utilize the limited available information.

b. These adaptations included model enhancements, developing general criteria for identifying gully locations, and estimating and calibrating model parameters. The objective of this study was to evaluate and integrate gully erosion into an AnnAGNPS pollution model using a generalized adaptation of the classical and ephemeral gully modules within AnnAGNPS. Criteria and methods for selecting gully locations using geographic information system technology were developed and tested on the Big Creek and Cache River watersheds. The developed gully information was applied in the AnnAGNPS model to assess proposed alternative conservation practices in these watersheds.

c. A viable solution to reduce flooding required a reduction in sediment produced from the watershed. Also, sediment reduction from the watershed was desirable to restore environmental function. Based on the results of the AnnAGNPS simulations, meeting a 50% reduction from base conditions required gully controls throughout the Cache River watershed. Without any gully control and using only Conservation Reserve Program (CRP) practices on steep landscapes, a 10% sediment reduction was predicted.

3. Watershed.

a. The Cache River watershed is bounded on the east by Crowley's Ridge, a prominent ridge in an otherwise featureless alluvial plain. The outlet of the Cache River watershed included in the plan of work is located near Grubbs, Arkansas. Grubbs is located at approximately RM 130 on the Cache River. The mouth of the Cache River watershed is about one mile northwest of Clarendon, Arkansas, where it joins the White River.

b. The elevation of the Cache River watershed ranges from 62 to 163 meters. The extent of Crowley's Ridge approximately follows the 100-meter line of elevation. The entire Cache River watershed is large, measuring approximately 134.5 km north to south and 91.6 km west to east. The area of the watershed is 242,878 hectares (600,143 acres). The entire Cache extends to the White River, where the present study includes only about one-half of the entire basin (938 square miles as compared to 2025 square miles, which includes the major Bayou DeView tributary, which contributes 694 square miles).

c. The Big Creek watershed (Figure 1) lies within the Crowley's Ridge area, but has a substantial extent in the alluvial plain. The mouth of the Big Creek watershed is located near Boydsville, Arkansas, where it enters the Cache River just downstream of State Highway 90. The elevation of the Big Creek watershed ranges from 90 to 163 meters. The Big Creek Watershed area measures 15.7 km north to south and 13.7 km west to east. The area is 11,889 hectares (29,378 acres).

4. Study Tasks.

a. The major tasks of the study were to catalog and build on previous research in the White River basin, the Cache River watershed, and the Big Creek watershed. The extent of the Cache River and Big Creek watersheds were to be defined and subdivided into smaller areas.

These homogeneous areas, called sub-watersheds or AGNPS cells, marked the areas where management was varied to analyze the impact of certain management practices on the watershed.

b. As gully activity was a major source of sediment discharge, gully locations were determined by manual and automated means, and the AGNPS model was modified to enable an analysis of gully impact on the watershed. The model also included simulation of channel sources of sediment and transport thereof.

c. A number of management scenarios were applied to all or part of the watershed, to determine the change in sediment discharge under different practices.

5. Model Development.

Limited detail regarding modeling and calibration is provided in this case study.

a. In order to evaluate the impact of gullies in the watershed, an assessment was first applied to the Big Creek watershed area where some limited information was available on the loadings from gullies. This information described the sediment size distribution of deposited sediment in the channel near the outlet of Big Creek as 97% sand, with very little silt or clay present. Approximate cleanout periods were recorded from 1991 through 2008 where sediment was removed from this channel at various intervals from 1991.

b. Only the period from 2004 to 2008 was determined to be reliable since only a portion of the channel was dredged, allowing upstream sand present in the channel to move into the dredged portion. This transported sediment was difficult to separate from what would have been produced from gullies during this period. An estimate of the total sediment removed as a result of deposition from gullies during this period from the channel was 38,000 tons. This was assumed to be the sand sediment load produced from landscapes and gullies in Big Creek watershed for 2004 to 2008.

c. With very little agricultural land in the watershed, the amount of sediment from the watershed was calibrated with this estimate of sediment produced by adjusting the coefficients of the gully components within AnnAGNPS. This resulted in a coefficient of 10 and an exponent of 1.0 that was subsequently used for all gullies in the Cache River watershed.

d. Using calibration results from the Big Creek watershed, it was necessary to test model performance in producing sediment loads at the outlet of Cache River watershed. Some limited data was useful in determining if the simulated results were reasonable.

(1) From a study near Cache River watershed by Hupp and Morris (1990), sediment deposition rates in wetland areas as a result of sediment from the Cache River prior to 1944 of 0.13 cm/yr were reported. From 1981 to 1990, deposition rates were reported as 0.29 cm/yr. This information does not provide the mass of sediment that could have been transported and deposited by Cache River.

(2) Hupp and Morris (1990) report that at the Patterson gage, approximately 88,000 tons of sediment was recorded from October 1987 to September 1988. AnnAGNPS simulated sediment load for this period resulted in 180,000 tons of sediment, using soybeans as the dominate crop instead of a rice-soybean rotation. While there is a difference in 90,000 tons between simulated and reported, it is not clear if the sediment reported at the gage included sand contributions, which AnnAGNPS reported as nearly 40,000 tons. Without better information, the results produced by the simulation were within limits that could be described by the uncertainty in measured results from the gage and the climate data.

6. Simulation Outputs.

The base condition was simulated by using the 2004 land use as a starting point and applied over 25 years of historical climate data (1984–2008). The base condition simulation resulted in average annual erosion over the entire watershed of 2.27 tons per acre per year.

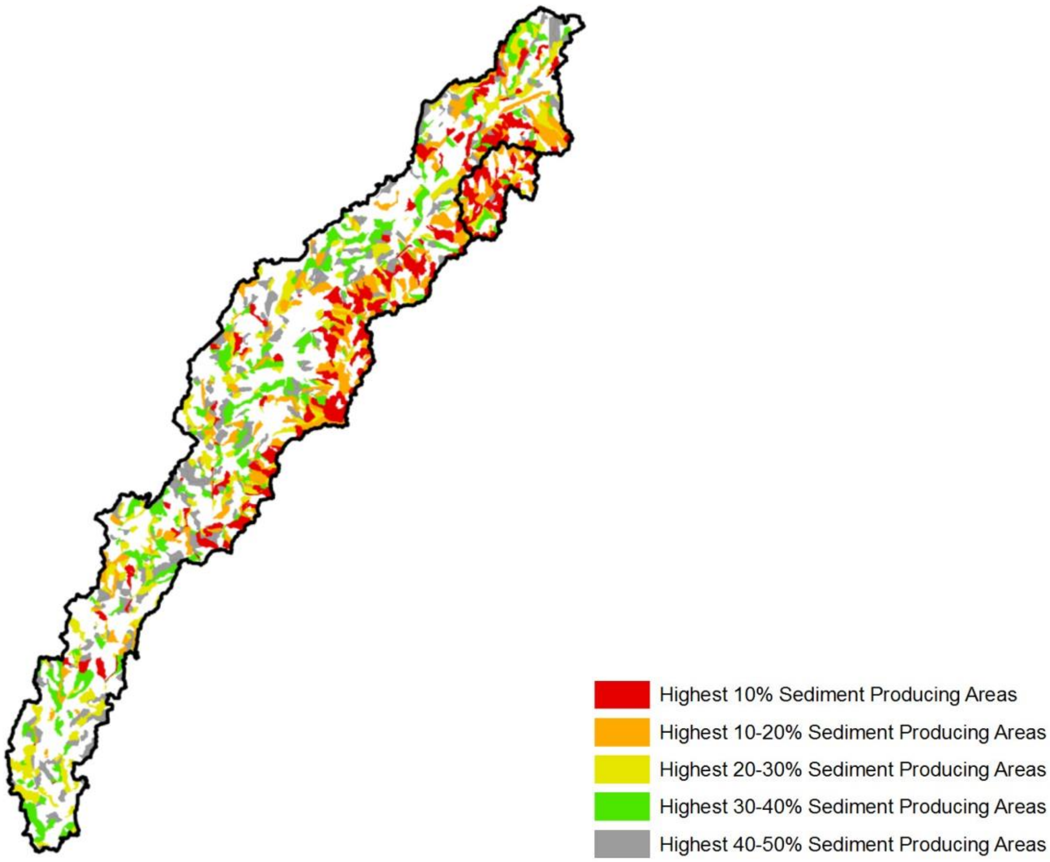
a. Table 1 summarizes base condition results. Of the 1,364,690 tons/yr of gross erosion in the watershed, the model indicates that only 48,543 tons/yr, or 3.6%, is delivered to the watershed outlet. This watershed delivery ratio is determined by the fact that the vast majority of the eroded sediment is redeposited on a field scale and never makes it to a stream, and sediment is also lost to deposition in the stream transport system.

Case Study 6B Table 1
Summary of Base Condition Simulation Output

Item	Amount	Units
Watershed Average Runoff	14.50	in/yr
Watershed Average Total Rate of Erosion	2.27	t/ac/yr
Watershed Total Tons of Erosion	1,364,690	t/yr
Watershed Sediment Yield to Streams	0.84	t/ac/yr
Sediment Loading Rate to Watershed Outlet	0.59	t/ac/yr
Sediment Loading Amount to Watershed Outlet	352,072	t/yr
Highest Erosion from an Individual Cell w/gully	38.14	t/ac/yr
Highest Erosion from Individual Cell-Sheet and Rill	17.86	t/ac/yr

b. The various base condition simulation results provide an indication of the location of higher and lower producing areas (Figure 2). Most of the watershed produces high volumes of runoff in the base condition. The high runoff-producing areas are a combination of agricultural areas that generally produced higher runoff rates than the other areas and soil types that are susceptible to high runoff.

c. High erosion rates generally exist in the steeper sloped areas of the watershed, with the flatter slopes along the river containing low erosion rates. The erosion rates correlate strongly with the RUSLE LS-factor (slope length) generated by the TOPAGNPS (Topographic Agricultural Non-Point Source) model. While the high erosion rates are scattered throughout the watershed, the areas where most of the sediment makes it to the outlet occurs in the eastern portion of the watershed. Sediment is deposited from the eroded areas in the river as it is transported throughout the system.

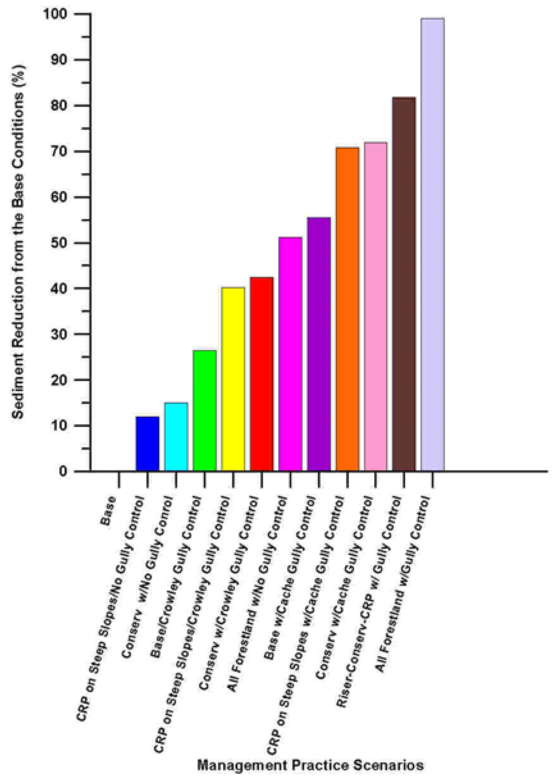


Case Study 6B Figure 2. Cache River watershed highest sediment producing areas at the top 10%, 20%, 30%, 40%, and 50% levels for the base conditions

7. Results from Management Alternatives.

Various management alternatives were evaluated, ranging from conventional tilled practices to no till practice, gully controls throughout the watershed, gully control only limited to the steeper landscapes of Crowley's Ridge, applying conservation reserve program conditions within steeper sloped cultivated fields where the LS factor exceeds 0.30, and conditions considered forests covered the entire watershed with existing gullies or no gullies at all.

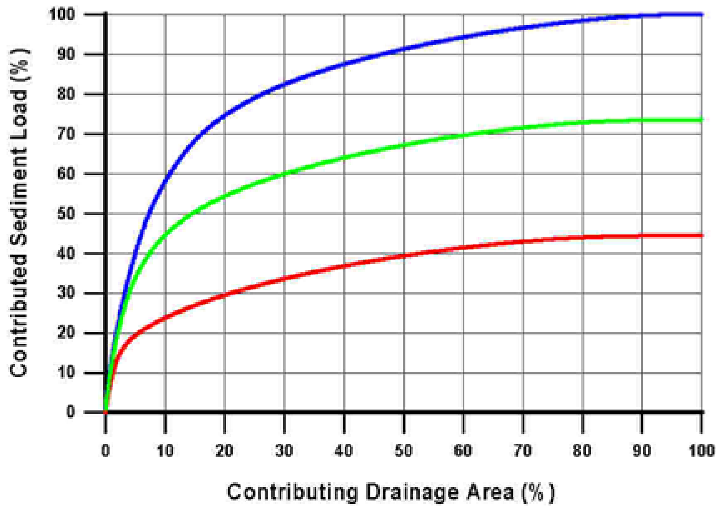
a. These alternatives were used to compare with the baseline condition for sediment loads, as shown in Figure 3. To meet a 50% reduction from base conditions requires gully controls throughout the Cache River watershed. Without any gully control and using only CRP practices on steep landscapes, then a 12% sediment load reduction was predicted. Conservation tillage using no till practices predicted only a 14% sediment load reduction.



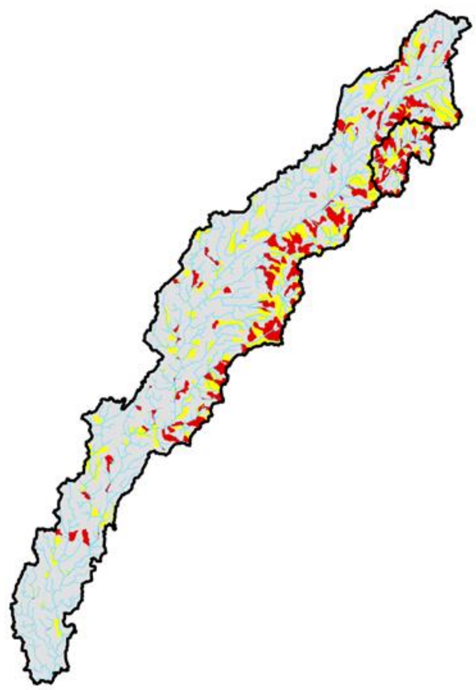
Case Study 6B Figure 3. Percent sediment reduction from the base level of base conditions for selected management alternatives

b. Nearly 60% of the annual sediment load at the Cache River watershed outlet is produced by only 10% of the area of the watershed (Figure 4). By targeting only 10% of the watershed for gully control measures, then the annual sediment load for the entire watershed could be reduced by 35% (Figure 5). Although, as a result of less sediment produced from gullies, sediment from other areas could now be transported to the outlet (Figure 6). If gully control measures are combined with CRP conservation measures on 10% of the watershed, then a 50% sediment reduction could be attained (Figure 7).

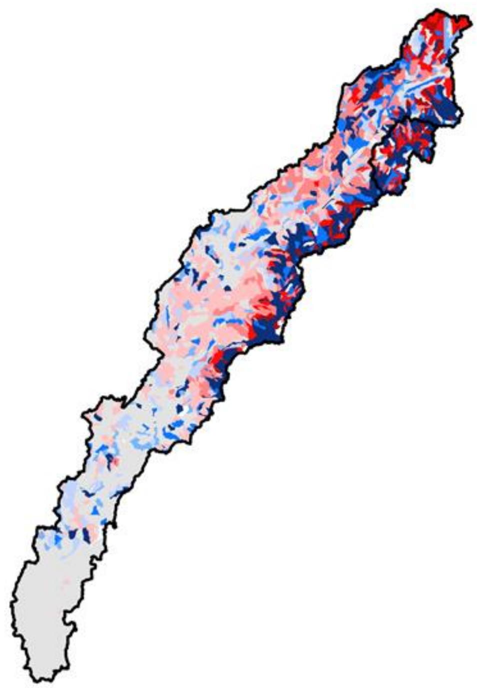
c. Additional practices of controlling all gullies, but in forested areas, utilizing riser boards, from 1 November through 31 January, and combining no-tillage conservation practices and CRP were also evaluated (Figure 8).



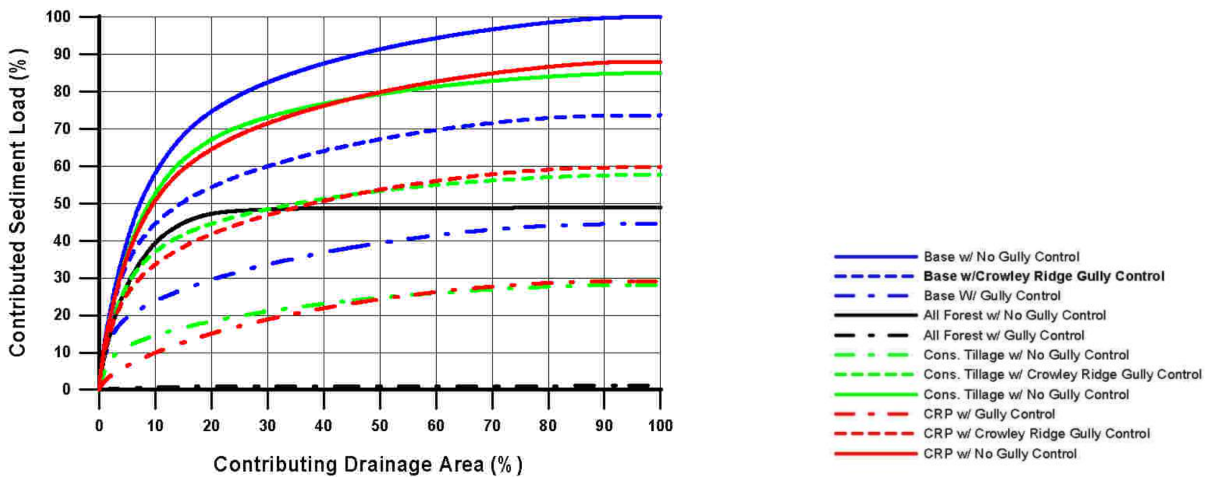
Case Study 6B Figure 4. Cache River watershed sediment load by contributing drainage area as determined from a unit area ranking ratio for base conditions with no gully control (blue), gully control for the entire watershed (red), and gully control only for Crowley's Ridge (green)



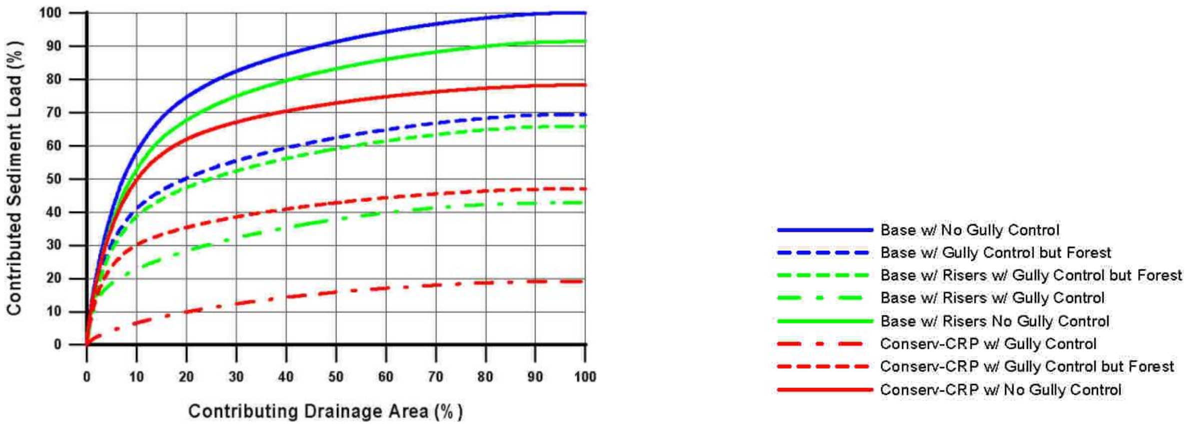
Case Study 6B Figure 5. Cache River watershed top 10% sediment load-producing areas in red, and areas that produce sediment higher than the watershed average (yellow) from the base simulation with no gully control



Case Study 6B Figure 6. Cache River watershed sediment load reductions (blue) and increases (red) from the base conditions with no gully control to the entire watershed containing gully control



Case Study 6B Figure 7. Cache River watershed sediment load by contributing drainage area for all conditions, including those with no gully control, gully control for the entire watershed, and gully control only for Crowley's Ridge



Case Study 6B Figure 8. Cache River watershed sediment load by contributing drainage area for additional conditions, including those with gully control for the entire watershed but forests, riser boards, and no-tillage conservation practices with CRP

Case Study 6C
Upland Sediment Production and Delivery in the Great Lakes Region Under Climate Change

1. Case Study.

This case study provides content condensed from studies performed by the Detroit District for the Great Lakes region related to sediment delivery for climate change scenarios. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Executive Summary.

USACE has 139 harbors in the Great Lakes. Many of these harbors sit at the outlets of rivers that convey large amounts of sediment and create periodic dredging requirements. At the outset of this study, it was unknown how dredging costs were affected by climate variability and future climate change.

a. This study examined the St. Joseph River and Maumee River watersheds to estimate the potential effects of climate change on dredging requirements. The St. Joseph River is located in Michigan and Indiana and enters Lake Michigan through Detroit District's St. Joseph Harbor. The Maumee River flows through Indiana, Michigan, and Ohio before entering Lake Erie through Buffalo District's Toledo Harbor.

b. A total of 346 climate change scenarios (112 Coupled Model Intercomparison Project Phase 3 (CMIP3) projections and 234 CMIP5 projections) were run through the Soil and Water Assessment Tool (SWAT) sediment yield models of each watershed. A second model, the Landscape Hydrology Model (LHM) was run to look at the impact of model selection. Based on the average results of the SWAT simulations, small increases in dredging costs can be expected for the Maumee River, and small decreases are likely on the St. Joseph River, but there is a significant range of variability in the results. In the most extreme cases, the change in dredging costs ranges from a decrease of more than $500,000 to an increase of $250,000 for the St. Joseph River, and from a decrease of $450,000 to an increase of $1,000,000 for the Maumee River.

c. Other important findings of this study include identifying potential biases in the downscaled climate data and significant differences between the older CMIP3 scenarios and the newer CMIP5 scenarios. These two findings indicate that older studies may need to be revisited in light of more recent climate models. This study also found very different responses to climate change between the two adjacent watersheds and between different modeling schemes. Therefore, future studies should extrapolate these results to other watersheds only with great caution and multiple models should be considered to better understand the true range of potential variability.

3. Background.

The Great Lakes and Ohio River Division (LRD) of USACE operates and maintains the U.S. portion of the Great Lakes Navigation System (GLNS), consisting of 139 projects (63 commercial and 76 shallow-draft), including three lock complexes, 104 miles of navigation structures, and over 600 miles of maintained navigation channels.

a. The GLNS is a complex deep-water navigation system stretching 1,600 miles through all five Great Lakes and connecting channels from Duluth, Minnesota to Ogdensburg, New York. In 2006, approximately 173 million tons of commodities were transported to and from U.S. ports located on the waterways of the Great Lakes system.

b. It is a non-linear system of interdependent locks, ports, harbors, navigational channels, dredged material disposal facilities, and navigation structures. The GLNS provides an estimated transportation rate savings benefit of $3.6 billion per year. Waterborne commerce is the most environmentally friendly and safest form of transportation for bulk commodities, producing lower emissions as well as less damage to property and a reduction in fatal and non-fatal injuries when compared to transportation by truck or rail.

c. A recent study concluded that pollution abatement savings resulting from the continued use of the GLNS exceeds $350 million annually. Many of these harbors sit at the outlets of rivers that convey large amounts of sediment and create periodic dredging requirements. At the outset of this study, it was unknown how dredging costs may vary with climate variability and future climate change.

d. This study specifically examines the St. Joseph River and Maumee River watersheds (Figure 1) to estimate the potential effects of climate change on dredging requirements. The St. Joseph River watershed covers 4,686 square miles in Michigan and Indiana. The river enters Lake Michigan through Detroit District’s St. Joseph Harbor. The Maumee River drains 6,570 square miles of Indiana, Michigan, and Ohio to Lake Erie through Buffalo District’s Toledo Harbor. These harbors were selected because of their sizable dredging requirements, and the existence of sediment production and transport models that could potentially be updated with new climate scenarios.

Case Study 6C Figure 1. Location map of the St. Joseph and Maumee River watersheds; the St. Joseph River watershed is on the left, the Maumee watershed is on the right

e. Many climate change models predict both increased temperature and increased precipitation in the Great Lakes region (Pryor et al., 2014). The increased precipitation and runoff, coupled with warmer temperatures, has the potential to significantly affect sediment production and transport in Great Lakes rivers, increasing the loadings to Federal harbors that already have a large dredging backlog. Additionally, a number of future climate scenarios predict lower water levels in the Great Lakes, which further exacerbates the impacts on harbors.

f. This project looked at two Federal harbors in the Great Lakes and their watersheds to examine potential impacts. The information gained from this work will help USACE make qualitative comparisons with current dredging requirements at Federal harbors in the Great Lakes.

4. Purpose and Scope.

The goal of this study was to assess how dredging cost requirements at Great Lakes harbors may vary in the future as the climate changes precipitation regimes and runoff characteristics. To approach this question, changes in watershed sediment yield and streamflow were modeled.

5. Methodology and Approach.

Sediment for the two study areas was estimated using models created with SWAT. SWAT is a lumped parameter hydrology model that was developed by the U.S. Department of Agriculture and is particularly good in agricultural areas. At the start of the project, SWAT models of both the St. Joseph and Maumee Rivers were identified. Unfortunately, further investigation revealed that the existing models were created in older versions of SWAT and could not be easily updated. The existing data was used to create new models instead.

a. Both models were created using ArcSWAT 2012.10.0.7 and SWAT 2012, Rev. 622. Data included 30 meter DEMs from the National Elevation Dataset, 2006 land use/land cover data from the USGS, and SSURGO soils data from the Natural Resources Conservation Service (NRCS). Observed precipitation and temperature data for 1950–2010 were obtained from the USDA-ARS for stations in all counties covered by the study watersheds. This dataset is missing data for January 2002, so all of year 2002 was excluded from all comparisons with the historical simulation.

b. ArcSWAT then determined which gaging station to use for each sub-watershed. All model outputs, including calibration and validation, were on a monthly basis. Each model included a minimum of a five-year warm-up period to avoid biases resulting from the initial states. The number of sub-watersheds for each SWAT model was kept low to minimize computational time. The St. Joseph River was split into 33 sub-basins and 338 Hydrologic Response Units, or HRUs. The Maumee River was modeled using 24 sub-basins and 307 HRUs.

c. Since dams can act as sediment sinks and affect the hydrology of a river, significant dams were included in the SWAT models. Physical data for the dams was primarily obtained from the National Inventory of Dams. All dams were modeled in SWAT as uncontrolled spillways, due to a lack of information on operation procedures.

d. Calibration.

(1) The St. Joseph River SWAT model was calibrated to USGS flow data at Niles, Michigan (Gage 04101500). A sediment rating curve for this gage was obtained from a previous study and combined with the monthly flow data to generate monthly sediment data.

(2) SWAT parameters used for the hydrologic calibration were: baseflow alpha factor (ALPHA_BF); deep aquifer percolation fraction (RCHRG_DP); groundwater delay (GW_DELAY); SCS runoff curve number (CN2); and surface runoff lag time (SURLAG). Parameters used for sediment calibration were initial sediment concentration in reservoirs (RES_SED); median particle diameter of reservoir sediment (RES_D50); normal sediment concentration in reservoirs (RES_NSED); and number of days to reach target storage from current reservoir storage (NDTARGR).

(3) Both calibration and validation Nash-Sutcliffe efficiencies for flow, seen in Table 1, are in the range characteristic of “good” or “very good” model fits, according to Moriasi et al.

(2007). The percent bias of sediment for both calibration and validation is well within the “very good” range.

(4) The Maumee River model was calibrated to USGS flow gages at both Waterville, Ohio (Gage 04193500) and Defiance, Ohio (Gage 04192500). Monthly sediment data for the USGS gage at Waterville was used to calibrate the sediment portion of the SWAT model. The model was calibrated for hydrology using the GW_DELAY, ALPHA_BF, and CN2 parameters in SWAT.

(5) Sediment calibration was accomplished using RES_SED, RES_D50, RES_NSED, and NDTARGR. Summary statistics for the calibration and validation of the model can be found Table 1. The Nash-Sutcliffe efficiency for the flow is “very good” for both the calibration and validation scenarios, as is the percent bias of the sediment.

Case Study 6C Table 1
Summary of Calibration and Validation Results for SWAT Models

<table>
<tr><th colspan="3"></th><th>Calibration</th><th>Validation</th></tr>
<tr><td rowspan="6">St. Joseph River at Niles, Michigan</td><td colspan="2">Years</td><td>1990–1999</td><td>2000–2009</td></tr>
<tr><td rowspan="2">Flow</td><td>R2</td><td>0.78</td><td>0.83</td></tr>
<tr><td>N-S</td><td>0.78</td><td>0.71</td></tr>
<tr><td rowspan="3">Sediment</td><td>R2</td><td>0.58</td><td>0.42</td></tr>
<tr><td>N-S</td><td>0.51</td><td>0.21</td></tr>
<tr><td>% Bias</td><td>4.6%</td><td>-12.2%</td></tr>
<tr><td rowspan="6">Maumee River at Waterville, Ohio</td><td colspan="2">Years</td><td>1991–1999</td><td>2000–2001</td></tr>
<tr><td rowspan="2">Flow</td><td>R2</td><td>0.86</td><td>0.87</td></tr>
<tr><td>N-S</td><td>0.79</td><td>0.79</td></tr>
<tr><td rowspan="3">Sediment</td><td>R2</td><td>0.53</td><td>0.55</td></tr>
<tr><td>N-S</td><td>0.53</td><td>0.55</td></tr>
<tr><td>% Bias</td><td>-3.8%</td><td>-2.5%</td></tr>
<tr><td rowspan="2">Maumee River at Defiance, Ohio</td><td rowspan="2">Flow</td><td>R2</td><td>0.84</td><td>0.85</td></tr>
<tr><td>N-S</td><td>0.79</td><td>0.79</td></tr>
</table>

e. Climate Projections.

(1) Binary-Coupled Statistically Downscaled (BCSD) climate projections were obtained from the Downscaled CMIP3 and CMIP5 Climate and Hydrology Projection archive (http://gdo-dcp.ucllnl.org/downscaled_cmip_projections/). Projections from both the World Climate Research Programme’s CMIP3 multi-model dataset and the newer CMIP5 multi-model ensemble datasets were downloaded.

(2) These datasets have already been downscaled using BCSD methodology and the gridded temperature and precipitation data for 1950–1999 compiled by Maurer et al. (2002) and are described by USBR (2013). The gridded, monthly temperature and precipitation data were further downscaled to daily time steps for each of the gages utilized by the SWAT models. This was accomplished by creating MATLAB code to implement the BCSD technique described in Maurer and Hidalgo (2008). A total of 112 CMIP3 projections and 234 CMIP5 projections, generated by different combinations of global circulation models (GCMs) and emissions scenarios, were downscaled.

(3) The downscaled climate data was run through the SWAT models and compared to the model results when run with the historical temperature and precipitation observations. To gain a more refined understanding of the potential climates, three 20-year sequences were compared: 1989–2008 (historical); 2011–2030 (projected near future); and 2031–2050 (projected far future). Results for each of the future time periods are reported as a percentage change from the historical (1989–2008) simulation for that particular climate run.

f. Dredging.

(1) The SWAT models produced an estimate of the sediment flowing out of each watershed, in metric tons per month, but not all of this sediment is dredged. Some sediment moves through the harbor without settling out and some locations within a harbor are not dredged due to either a lack of interest in maintaining the depths at that location or limited funding.

(2) Dredging costs were estimated by developing correlations between the simulations of the historical data and records of recent dredging operations. For St. Joseph Harbor, estimates were developed separately for both the inner and outer harbors. Dredging quantities were also calculated separately for the Maumee River Navigation Channel and Maumee Bay dredging sites and then added together. Dredging data from 1989 to 2009 was used for St. Joseph Harbor and data from 1990 to 2009 was used for both the Maumee River and Maumee Bay dredging sites. All costs were adjusted for inflation to 2009 dollars using Consumer Price Index data from the U.S. Bureau of Labor Statistics.

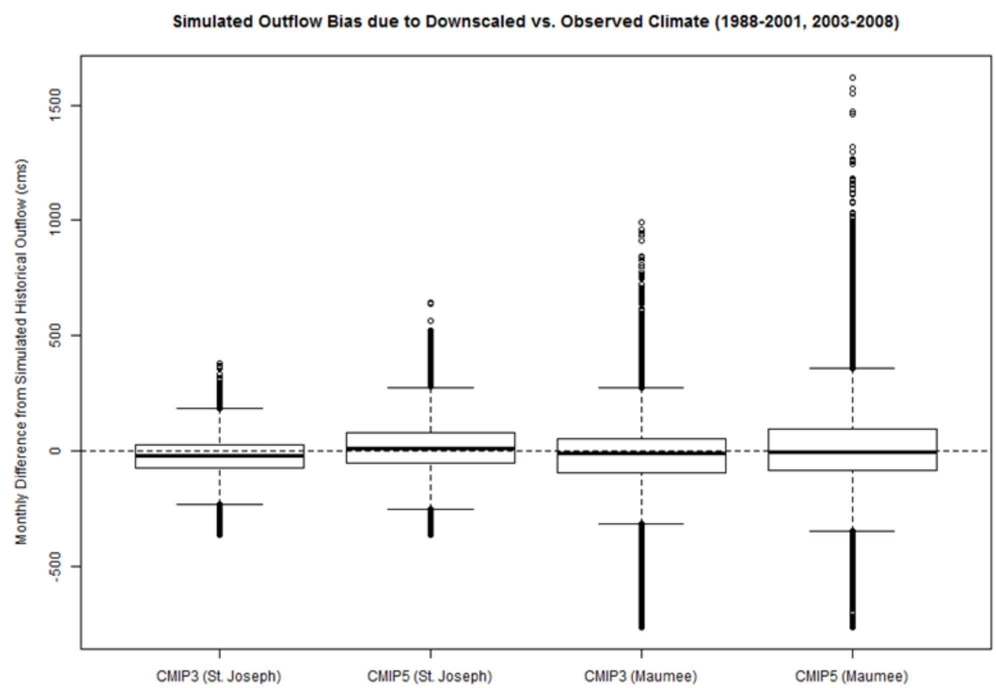
g. To investigate possible variability based on model selection, sediment yields were also calculated using the LHM. LHM is a fully distributed hydrologic model developed by Michigan State University for integrated surface and groundwater studies at the watershed to regional scale. For this study, the same Modified Universal Soil Loss Equation (MUSLE) was implemented as is used in SWAT. Since LHM takes much longer to run, approximately 12 days of computing time for a 70-year simulation, only a single scenario was run. The scenario presented here is an ensemble of the CMIP5 Representative Concentration Pathway (RCP) 6.0 inputs used for the rest of the study.

6. Results.

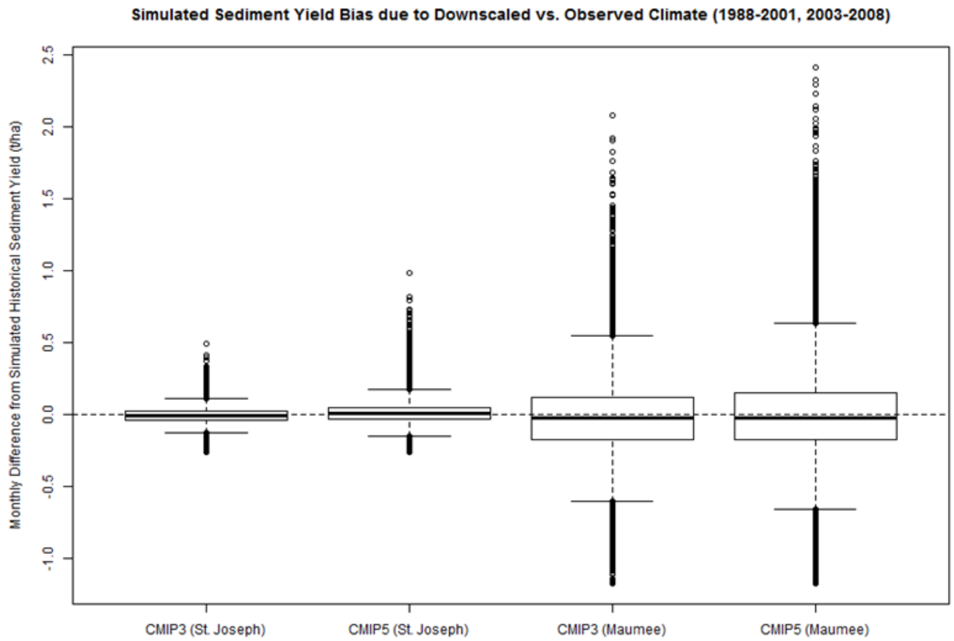
Results from the models run using each of the downscaled climates were initially compared to the model results using the available observed data for the same time period (1988–2001, 2003–2008), to look for potential biases in the GCMs.

a. Results are seen in Figure 2, where box plots on the left are for the St. Joseph River and those on the right are for the Maumee River. Similar graphics for sediment yield (Figure 3) and sediment outflow (Figure 4) are also presented. The average and standard deviation of the biases is also presented in Table 2 and Table 3. On average, the CMIP3 scenarios under-predict outflow, while the CMIP5 scenarios tend to over-predict outflow. The set of CMIP5 scenarios tend to over-predict both sediment yield and delivery in the St. Joseph, but under-predict sediment delivery in the Maumee. Caution should be used when comparing the CMIP3 and CMIP5 standard deviations, because there are roughly twice as many CMIP5 scenarios as CMIP3 scenarios.

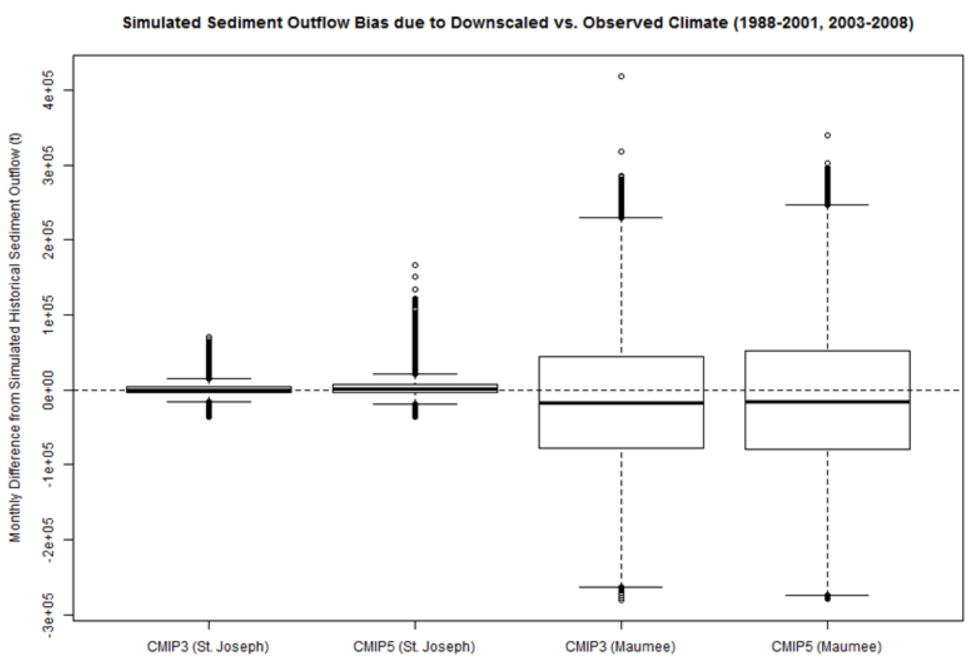
b. To avoid the biases in the climate data, all results in the remainder of this report are presented as changes relative to the 1989–2008 time period of the respective climate change scenario.



Case Study 6C Figure 2. Bias in simulated outflow (climate change scenarios: gaged precipitation and temperature inputs)



Case Study 6C Figure 3. Bias in simulated sediment yield (climate change scenarios: gaged precipitation and temperature inputs)



Case Study 6C Figure 4. Bias in simulated sediment delivery to harbors (climate change scenarios: gaged precipitation and temperature inputs)

Case Study 6C Table 2
Modeled Climate Scenario Flow Bias for 1989–2008

	Flow			
	CMIP3		CMIP5	
	Avg.	St. Dev.	Avg.	St. Dev.
St. Joseph	-15.4%	5.6%	+8.6%	8.4%
Maumee	-15.2%	7.0%	+8.0%	9.7%

Note: Calculated as the average difference from the historical simulation, divided by the average of the historical simulation.

Case Study 6C Table 3
Modeled Climate Scenario Sediment Bias for 1989–2008

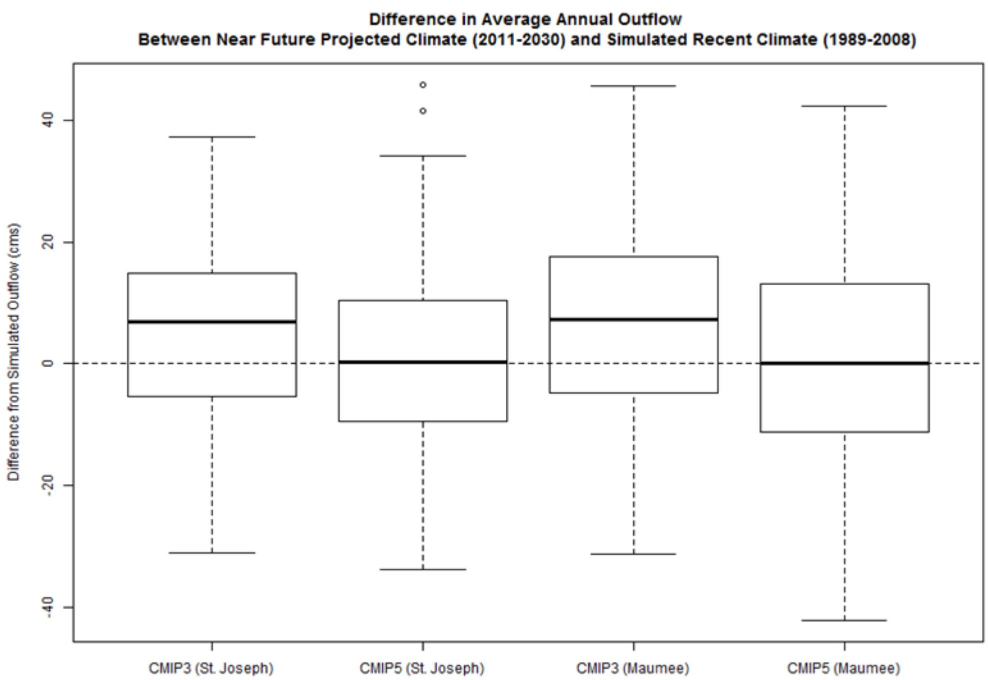
	Sediment Yield				Sediment Delivered to Outlet			
	CMIP3		CMIP5		CMIP3		CMIP5	
	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.	Avg.	St. Dev.
St. Joseph	-7.3%	7.0%	+29.4%	12.1%	-2.0%	8.9%	38.5%	14.2%
Maumee	-9.8%	6.4%	+0.7%	7.5%	-13.6%	5.5%	-11.2%	5.0%

Note: Calculated as the average difference from the historical simulation, divided by the average of the historical simulation.

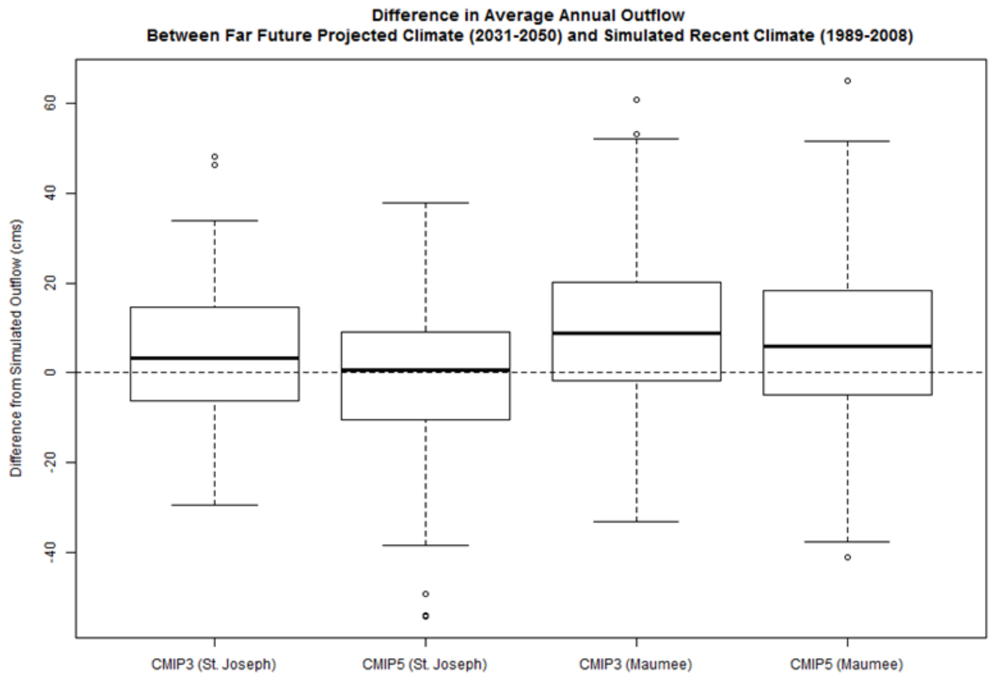
c. Relative Flow Change. The distribution of relative flow changes for the near future scenarios (2011–2030 of the climate input data) are shown in Figure 5, while the far future (2031–2050) results can be seen in Figure 6. In both figures, the histogram on the left is the St. Joseph River and the one on the right is the Maumee River. Summary statistics are presented in Table 4. Generally, the CMIP3 scenarios result in small increases in average flow. The CMIP5 scenarios result in flows very similar to current conditions, except for the far future simulations of the Maumee, which show small increases.

Case Study 6C Table 4
Modeled Flows Relative to 1989–2008

			St. Joseph		**Maumee**	
		Years	**2011–2030**	**2031–2050**	**2011–2030**	**2031–2050**
Flow	CMIP3	Min	-24.0%	-23.1%	-28.3%	-24.1%
		Avg	+4.0%	+3.3%	+6.0%	+8.0%
		Max	+28.7%	+37.3%	+44.4%	+53.7%
	CMIP5	Min	-18.2%	-33.0%	-23.9%	-24.3%
		Avg	+0.6%	+0.1%	+1.1%	+4.8%
		Max	+41.5%	+28.7%	+46.9%	+57.6%

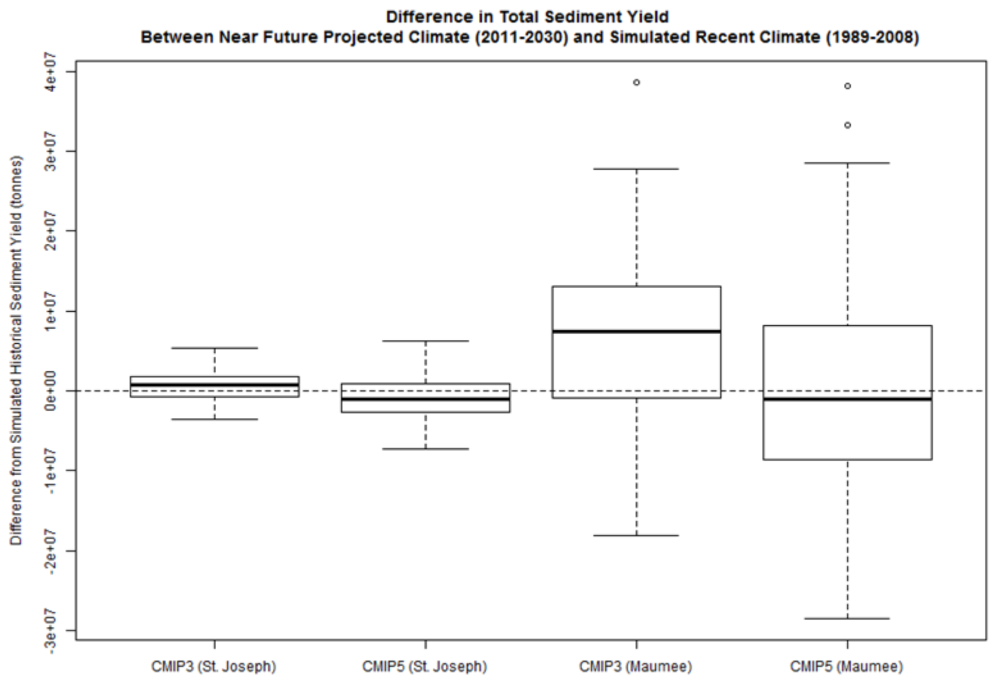


Case Study 6C Figure 5. Modeled change in flow between present (1989–2008) and near future (2011–2030)



Case Study 6C Figure 6. Modeled change in flow between present (1989–2008) and far future (2031–2050)

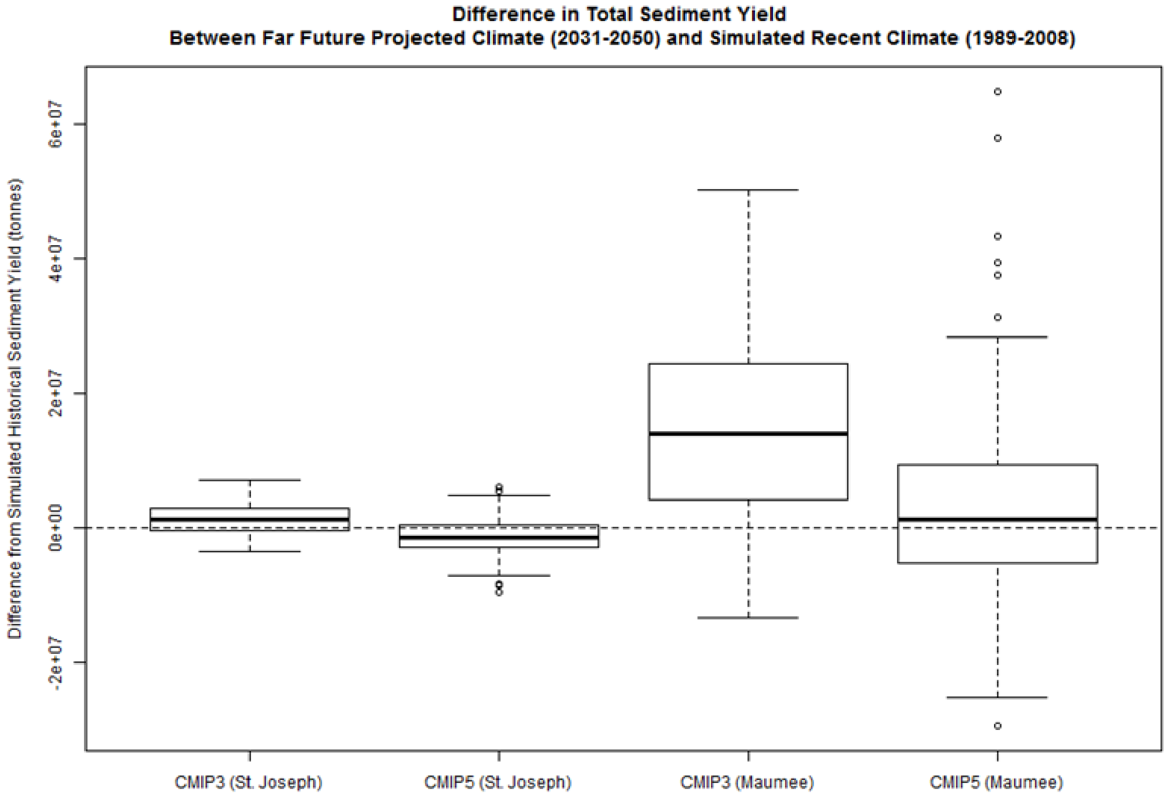
d. Sediment Yield. Sediment yield for the near future climate scenarios is shown in Figure 7. Figure 8 shows the far future sediment yields. These graphics, along with the summary statistics in Table 5, show that the difference in the effects of the CMIP3 and CMIP5 input data on sediment yield are significant. The CMIP3 scenarios produce an increase in sediment yield that increases over time. The CMIP5 scenarios for the St. Joseph River watershed, however, result in a decreased sediment yield. In the near term, the CMIP5 scenarios result in very little change to sediment yield in the Maumee, but a small increase for the later time period.



Case Study 6C Figure 7. Modeled change in 20-year sediment yield between present (1989–2008) and near future (2011–2030)

Case Study 6C Table 5
Modeled Sediment Yield Results Relative to 1989–2008

		Years	St. Joseph		Maumee	
			2011–2030	**2031–2050**	**2011–2030**	**2031–2050**
Sediment Yield	CMIP3	Min	-22.1%	-23.1%	-19.4%	-14.7%
		Avg	+4.4%	+9.0%	+6.9%	+15.4%
		Max	+38.8%	+41.5%	+44.5%	+57.4%
	CMIP5	Min	-27.5%	-38.7%	-22.8%	-24.6%
		Avg	-3.3%	-5.4%	+0.3%	+2.8%
		Max	+48.7%	+43.2%	+49.1%	+82.9%

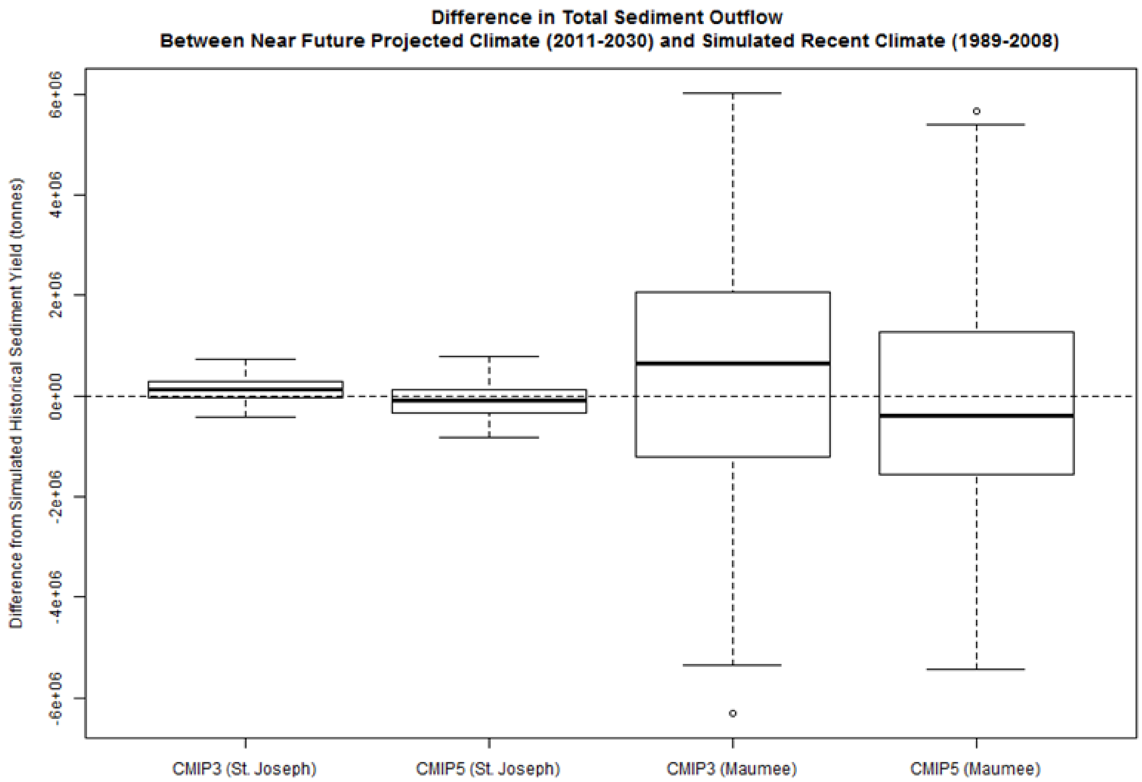


Case Study 6C Figure 8. Modeled change in 20-year sediment yield between present (1989–2008) and far future (2031–2050)

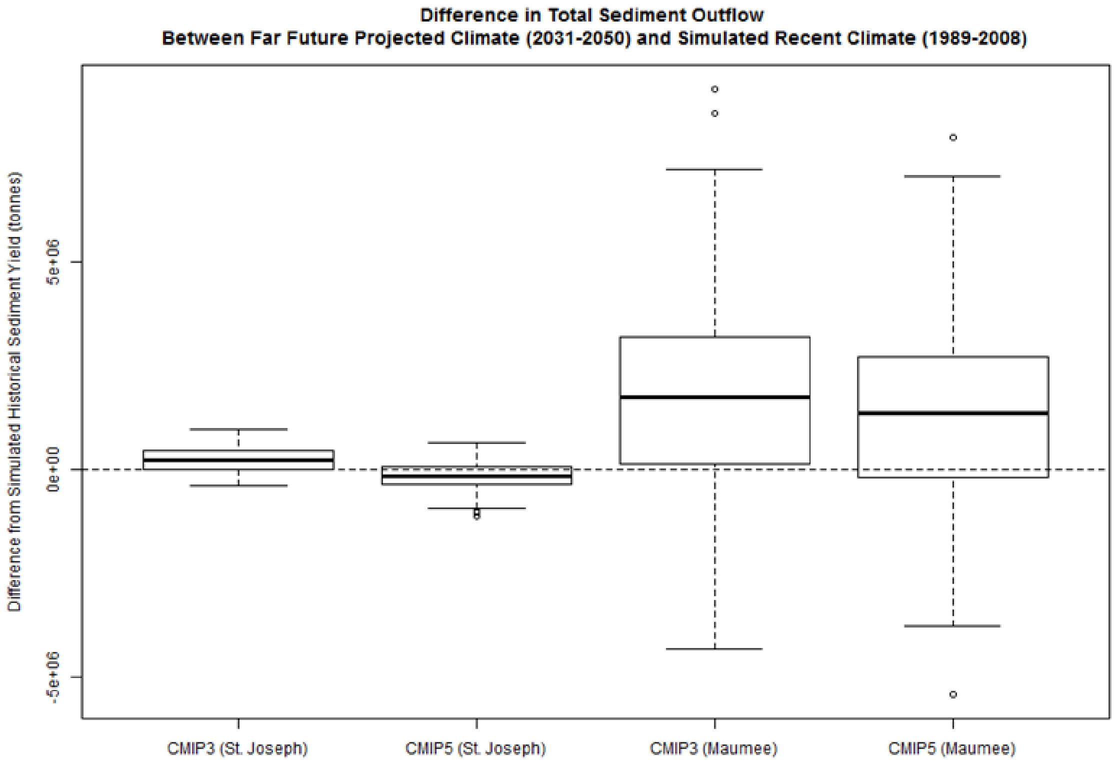
e. Sediment Delivery to Harbor. The sediment delivered to the harbor at the watershed outlet drives dredging needs. Figure 9 and Figure 10 show boxplots of the sediment delivered to the watershed outlet, and Table 6 summarizes these same results. Similar to the sediment yield results, the CMIP3 scenarios indicate small increases in the amount of sediment arriving at the harbor that grow in the far time period. The CMIP5 scenarios, however, project a decreasing trend for the St. Joseph River. Toledo Harbor, at the outlet of the Maumee, could see very similar amounts of sediment coming from upstream in the near term but increasing amounts later in time, according to the CMIP5 scenarios.

Case Study 6C Table 6
Modeled Sediment Delivery to the Watershed Outlet Relative to 1989–2008

			St. Joseph		Maumee	
		Years	**2011–2030**	**2031–2050**	**2011–2030**	**2031–2050**
Sediment Delivered to Outlet	CMIP3	Min	-24.8%	-23.9%	-24.3%	-16.7%
		Avg	+8.5%	+14.2%	+1.5%	+6.0%
		Max	+50.1%	+54.3%	+23.7%	+36.2%
	CMIP5	Min	-29.0%	-44.0%	-17.9%	-18.0%
		Avg	-3.4%	-6.1%	+0.3%	+4.9%
		Max	+58.2%	+41.3%	+26.6%	+37.0%



Case Study 6C Figure 9. Modeled change in sediment delivery to the harbors between present (1989–2008) and near future (2011–2030)



Case Study 6C Figure 10. Modeled change in sediment delivery to the harbors between present (1989–2008) and far future (2031–2050)

f. Dredging Cost. The best linear regression for dredging costs at the St. Joseph inner and outer harbors dredging sites was a relationship to the moving average of sediment outflow for the last two water years (for example, dredging costs for 2008 were most strongly correlated to sediment outflow in water years 2007 and 2008 (October 2006–September 2007 and October 2007–September 2008, respectively)). This relationship, shown in Equation 1, had an r^2 of 0.478.

$$Dredging\ Cost_{SJ} = -\$693{,}700 + \$14.30 * (Sediment\ Out_{WY} + Sediment\ Out_{WY-1})/2$$

Case Study 6C Equation 1

(1) Dredging costs for the Maumee were estimated separately for the Maumee River and Maumee Bay dredging sites and then added together. Dredging data for the Maumee Bay site correlated best with the previous calendar year's sediment outflow ($r^2 = 0.301$) and is shown as Equation 2. Dredging costs for the Maumee River site showed very poor correlation to all model outputs. The best correlation was with the sediment deposited in the downstream reach for the current water year (Equation 3), but this had an r^2 of only 0.147. Comparisons to the historical period of the climate change scenarios showed that all of these estimates under-predicted the actual dredging quantities by \$303,525 to \$496,632 per year, relative to an actual annual average dredging cost of \$2,718,783.

$$Dredging\ Cost_{MB} = -\$859{,}800 + \$1.26 * Sediment\ Out_{CY-1}$$ Case Study 6C Equation 2

$$Dredging\ Cost_{MR} = \$3{,}159{,}000 - \$16.88 * Sediment\ Deposition_{WY}$$ Case Study 6C Equation 3

(2) Dredging of navigation channels is affected by a number of factors in addition to incoming sediment, including longshore transport of sediment, lake levels, navigational considerations, and funding. Due to these uncertainties and the relatively weak correlations between model outputs and dredging costs, Table 7 and Table 8 lists both the costs estimated from the developed relationships and those estimated based on a conservative estimate that assumes a one to one relationship between dredging costs and sediment outflow. Generally, the CMIP5 scenarios produce slight decreases in future dredging at St. Joseph but slight increases at Toledo. The range of these estimates is significant, however, on the order of +/- $500,000 at St. Joseph and -$500,000 to +$1,000,000 at Toledo.

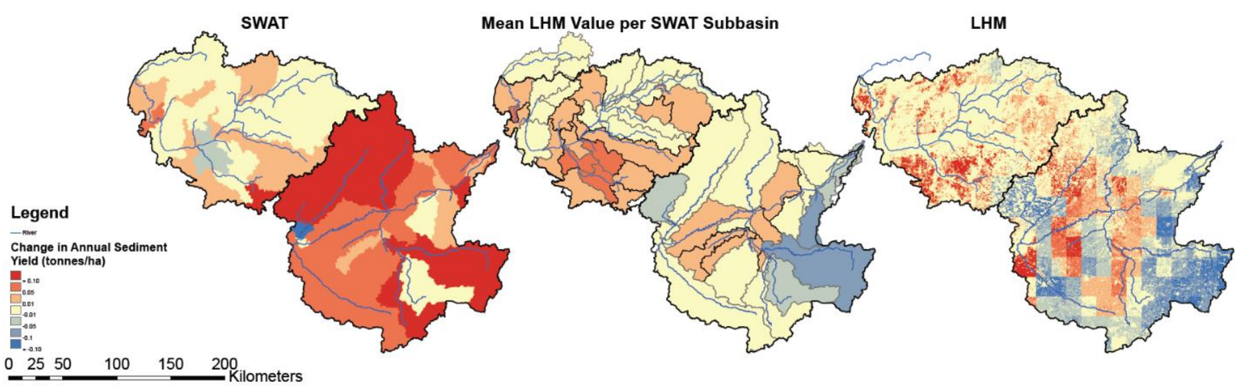
Case Study 6C Table 7
Forecasted Change in Average Annual USACE Dredging Costs at St. Joseph Relative to 1989–2008

			St. Joseph			
			Regression (See Case Study 6C Equation 1)		**1:1 Sediment Outflow (Based on 1989-2009)**	
		Years	**2011–2030**	**2031–2050**	**2011–2030**	**2031–2050**
Forecast Change in Dredging Costs	CMIP3	Min	-$320,591	-$247,983	-$128,317	-$123,660
		Avg	+$86,474	+$146,319	+$43,980	+$73,473
		Max	+$539,924	+$666,665	+$259,221	+$280,952
	CMIP5	Min	-$581,792	-$815,794	-$150,048	-$227,659
		Avg	-$64,892	-$131,701	-$17,592	-$31,562
		Max	+$466,208	+$438,786	+$301,131	+213,689

Case Study 6C Table 8
Forecasted Change in Average Annual USACE Dredging Costs at Toledo Relative to 1989–2008

			Maumee			
			Regression (See Case Study 6C Equations 2 and 3)		**1:1 Sediment Outflow (Based on 1990–2009)**	
		Years	**2011–2030**	**2031–2050**	**2011–2030**	**2031–2050**
Forecast Change in Dredging Costs	CMIP3	Min	-$84,047	-$52,972	-$660,664	-$454,037
		Avg	+$11,812	+$29,179	+$40,782	+$163,127
		Max	+$110,842	+$147,319	+$644,351	+$984,199
	CMIP5	Min	-$91,587	-$64,317	-$486,662	-$489,381
		Avg	+$12,235	+$23,127	+$8,156	+$133,220
		Max	+$108,554	+$129,019	+$723,196	+$1,005,950

g. Model Comparison. The effect of model choice can be seen in Figure 11, where the results from SWAT are compared to those from LHM. The results are not only different between the two models, but the differences appear to vary spatially. There are a number of potential reasons for these differences, including the spatial resolution (lumped hydrologic response units and sub-watersheds vs. independent grid cells), the temporal resolution (daily vs. hourly computational time steps), and the physical processes used in the models (SCS curve numbers vs. excess precipitation calculation), in addition to geological differences between the watersheds.



Case Study 6C Figure 11. Comparison of spatial results for the CMIP5 RCP 6.0 ensemble run.

Note: Results are displayed as the change in annual sediment yield (tons/ha) from the historical to the near future time period. The graphics show, from left to right, the SWAT output, the LHM outputs aggregated to the same sub-watersheds as SWAT, and the spatially distributed LHM output. The large rectangular blocks visible in the LHM output are an artifact of the North

American Land Data Assimilation System (NLDAS) grid used for downscaling of the input temperature and precipitation data.

h. Other concerns. This study did not address a number of additional ways that climate change may impact dredging requirements for Great Lakes harbors. The largest potential impact is a sustained or permanent change in Great Lakes water levels. A significant decrease in water levels would not only increase dredging requirements to maintain commercial navigation, but could create headcuts that propagate upstream through river systems, releasing additional sediment. This study also did not examine potential changes to farming practices or land use patterns that may be driven by climate change.

7. Lessons Learned.

Identified lessons learned include the following:

a. Both CMIP3 and CMIP5 data may produce simulations biased relative to the current climate for a given area.

b. Despite modeling adjacent watersheds of similar size, the results differed between the St. Joseph and Maumee. This indicates that results, even from nearby watersheds, should be extrapolated to other areas only with great caution.

c. There are large differences in the results of the CMIP3 scenarios and the newer CMIP5 scenarios, even when the climate model biases are accounted for by looking at relative changes. This implies that conclusions from earlier studies performed using CMIP3 should be revisited.

d. Different models may show qualitatively different results, suggesting that multiple models should be considered when looking at potential climate change impacts.

Case Study 6D
Lower Snake River Sediment Yield Associated with Climate Changes and Wildfires

1. Case Study.

This case study provides content condensed from studies performed by the Walla Walla District to evaluate wildfire effects on sediment yield to USACE reservoir projects on the Lower Snake River. The study was completed in 2014. The partial content included in the case study demonstrates several concepts, but does not include USACE study requirements.

2. Introduction.

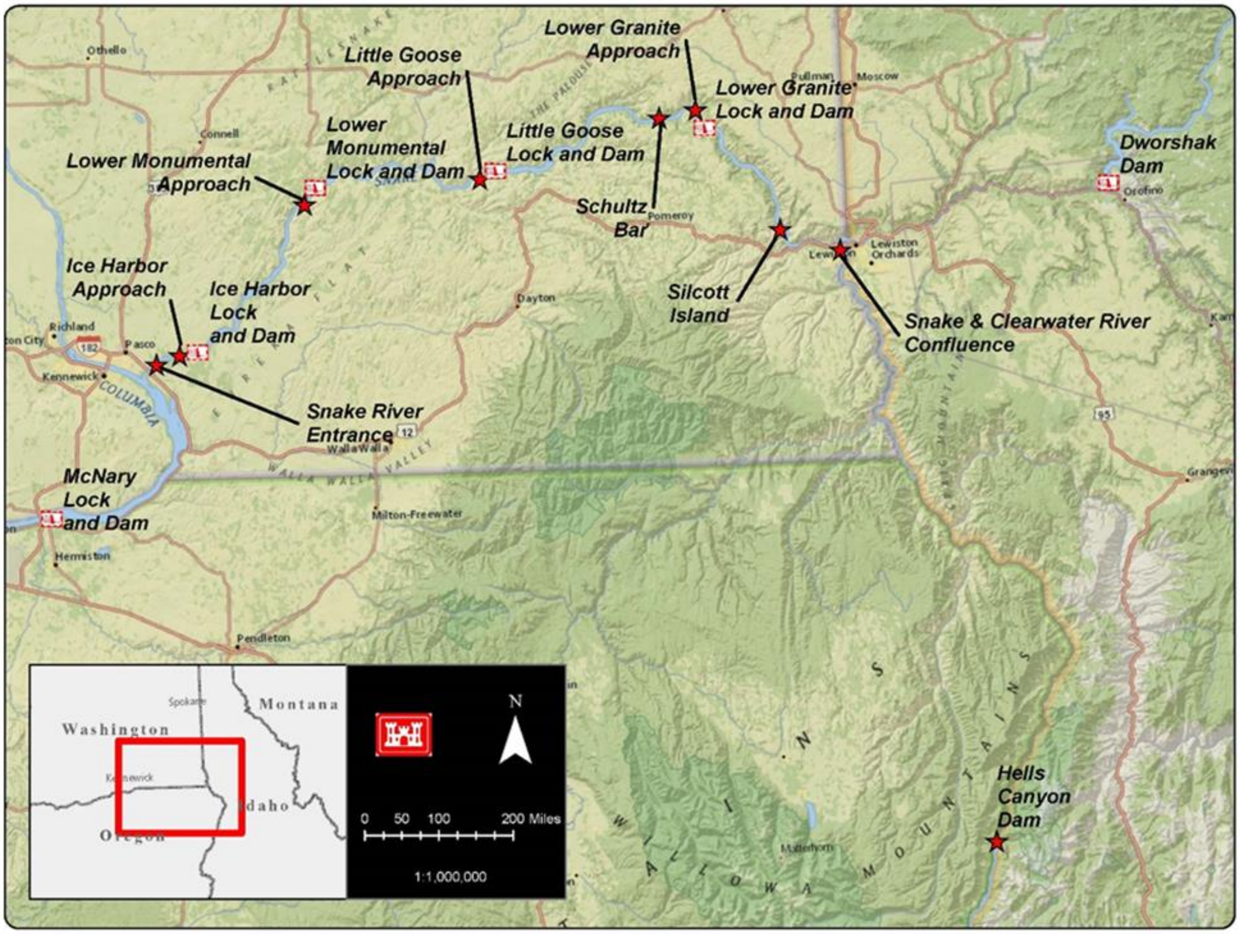
The Programmatic Sediment Management Plan (PSMP) was a multi-year Environmental Impact Study (EIS) to evaluate management associated with USACE's existing Lower Snake River Project (LSRP), a system of four dams (Ice Harbor, Lower Monumental, Little Goose, and Lower Granite) constructed between 1961 and 1975 on the Snake River in Washington State.

a. The sediment management area of the navigation system maintained by the Walla Walla District on the Lower Snake River includes this series of four dams and their associated locks and reservoirs, and extends along the Snake River from the confluence of the Columbia River to the upstream limits of Lower Granite Reservoir, including the lower portion of the Clearwater River in northern Idaho.

b. Lower Granite is the most upstream dam on the USACE Lower Snake River lock and dam system (Figure 1). Total watershed area upstream from Lower Granite Dam is 102,000 square miles, but only part of this area delivers sediment to the lower Snake River. Lower Granite Reservoir receives sediment from 27,000 square miles of forest, range, and agricultural land, bounded by the Hells Canyon Complex of dams on the Snake River and Dworshak Dam on the North Fork of the clearwater River. The size of the sediment delivery watershed of the lower Snake River reservoir system increases to 32,000 square miles at the mouth of the Snake River on the Columbia River.

c. Flood risk in the Lewiston, Idaho, levee system is a serious issue affected by the magnitude and rate of sediment accumulation. The Lewiston levees at the confluence of the Snake and Clearwater Rivers are part of the Lower Granite project. Without the levees, downtown Lewiston would be flooded by the backwater as the river approaches the pool upstream of Lower Granite Dam, which is located 32 miles downstream.

d. Approximately 80 million cubic yards (mcy) of sediment has accumulated in Lower Granite Reservoir since the reservoir was first filled in April 1975. Over a 37-year period, sediment from the Snake and Clearwater Rivers has progressively raised the predicted fixed-bed water surface elevations by about 4 feet at the confluence such that the risk of overtopping the Lewiston levees during extreme floods has increased. Figure 2 shows the change in estimated water surface elevation at the confluence for a standard project flood (SPF) discharge of 420 Kcfs over the period.

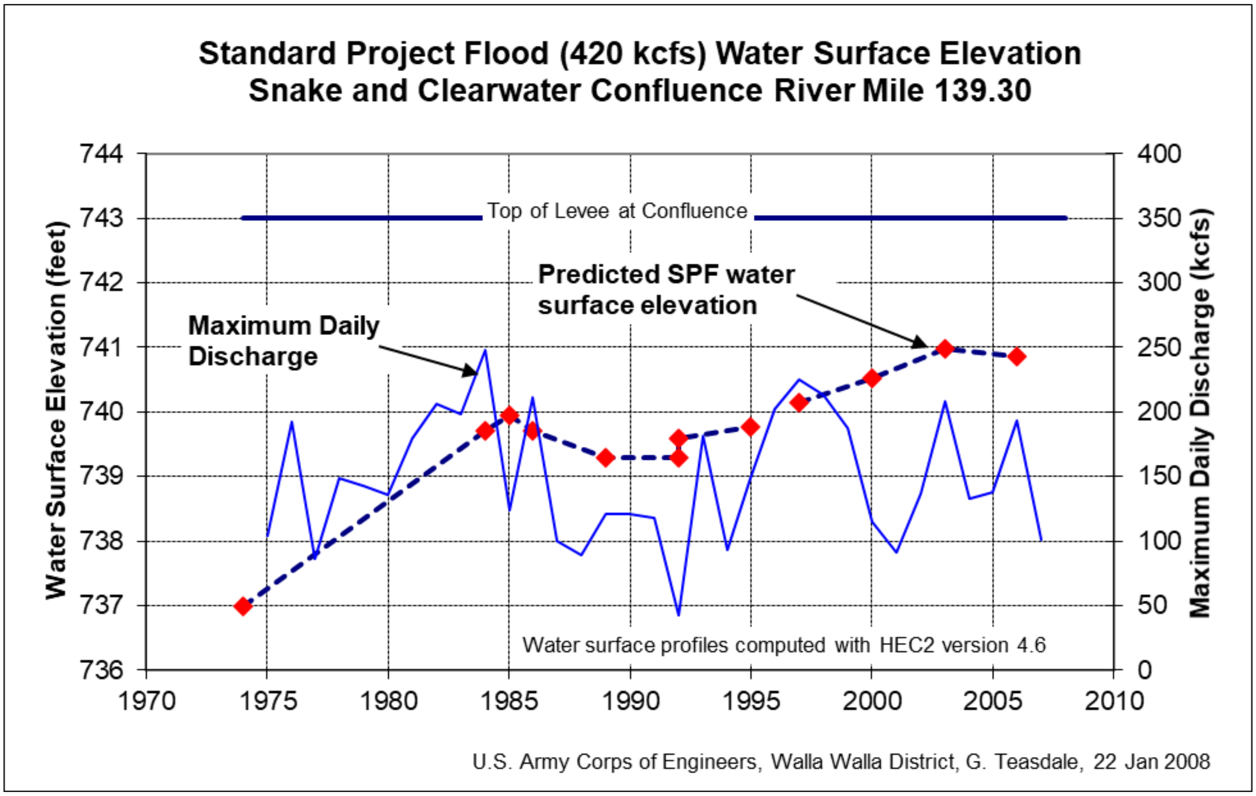


Case Study 6D Figure 1. Lower Snake River vicinity map

e. As part of the Environmental Impact Statement preparation for a PSMP, USACE-Northwestern Walla Walla (NWW) undertook extensive studies that included a sedimentation analysis for Lower Granite Reservoir and Lewiston Levee. The sedimentation analysis included studies of the watershed sediment yield in consideration of changes in meteorology and land cover with time.

f. Wildfire impacts are considered a primary driver of sediment yield in the western U.S. The potential for wildfires is expected to increase in response to climate change in the northwestern and southwestern states (U.S. Global Change Research Program 2014).

g. This case study presents only the portion dealing with climate change and wildfire impacts, extracted from Appendix F: Hydrology and Hydraulics, Lower Granite Reservoir Sedimentation Analysis, and Lewiston Levee Flood Risk Analysis (USACE-NWW 2014).



Case Study 6D Figure 2. Historic fixed-bed water surface elevation estimated at the confluence (USACE-NWW 2014)

3. Climate and Wildfire.

The analysis of trends from suspended and bedload measurements, performed primarily in collaboration with other agencies, suggests that sediment load in the Snake River may now be increasing. Characterization of sediment yield from forest land by U.S. Forest Service (USFS) scientists for the PSMP-EIS (Goode et al., 2010; Elliot 2013) and other recent evaluations (Morgan et al., 2008; Kirchner et al., 2001; Littell and Gworzdz 2011; Littell et al., 2009) indicated strong potential that future sediment yield will increase because of more extensive and frequent wildfires perhaps promoted by a warming climate.

a. Effect of Wildfire on Sediment Yield.

(1) The effect of wildfire on sediment yield in the Snake and Clearwater basins is evident. Descriptions of climate and the fire-affected landscape can be translated to sediment yield estimates by means of a watershed hydrologic model. Sediment yield hydrologic models are complex tools with many interdependent components to simulate hydrology, snowmelt and rainfall runoff, soil erosion, stream flow routing, groundwater interactions, and vegetation life cycles.

(2) While it was not realistic to expect precise quantification of the effect of climate variability and fire, reasonable quantitative estimates of watershed sediment yield are possible. (Miller et al., 2011; Bracmort et al., 2006). Extensive and frequent wildfires expose the watershed to erosion by intense storms. Widespread wildfire followed by an extreme regional storm would drastically increase short-term sediment yield and would likely disturb the drainage system.

(3) Cosmogenic analysis of sediment yield in the mountains of central Idaho by Kirchner et al. (2001) strongly suggests that such extreme events define the long-term sediment balance of the Lower Snake and Clearwater basins, and that current sediment yield rates, which have not been impacted by wildfires, are less than long-term rates by an order of magnitude.

(4) Synchronous events of extensive wildfire and extreme storms are difficult to predict, and the erosion that might occur in the aftermath is likely impossible to control with current resources and technology. The relationships between climate, wildfire, and sediment yield are therefore important to consider when planning a strategy to meet extreme sediment inflows and manage long-term sediment accumulation in Lower Granite Reservoir.

(5) While it is not realistic to expect precise quantification of the effect of climate variability and fire, estimates of watershed sediment yield can be obtained with appropriately formulated hydrologic models. Ideally, watershed sediment yield models should be calibrated with sediment load measurements. In the Lower Granite Reservoir watershed, reasonably comprehensive datasets are available that characterize the terrain, soil, hydrology, meteorology, climate, land cover, fire-affected area, and river channel system for a basin-scale sediment yield and routing model.

(6) Recent sediment load measurements in tributary rivers in the lower Snake and Clearwater Rivers provided the data to calibrate the sediment yield model at the scale of the major sub-basins. Estimates of storage depletion in the Lower Granite Reservoir from multiple surveys provided informative data on basin sediment yield (Table 1). Assuming a trap efficiency of 80% and an average sediment unit weight of 70 lb/ft^3, the 2.22 mcy/yr yields an average sediment yield of approximately 0.15 ton/ac/yr, which agrees well with the 0.19 ton/ac/yr from regression equations developed for measured sediment loads 2008 to 2011.

Case Study 6D Table 1
Summary of Sediment Accumulation in Lower Granite Reservoir

Lower Granite Cumulative Gross Sediment Accumulation 1974–2010									
	Sediment Volume (million cubic yards)								
Range Survey Period	**Snake Above Confluence[1]**	**Clearwater Above Confluence[2]**	**Snake Confluence to Silcott[3]**	**Snake Below Silcott[4]**	**Snake Below Confluence[5]**	**Dredge Volume**	**Period Total Volume**	**Average Annual Volume**	**Cumulative Total Volume**
1974–1995	0.50	0.51	7.50	32.91	40.41	2.060	43.48	2.07	43.48
1995–1997	-0.15	0.51	2.09	10.24	12.33	0.030	12.71	6.36	53.19
1997–2000	0.43	0.04	0.93	6.26	7.19	0.118	7.78	2.59	63.98
2000–2003	0.34	0.04	1.29	3.82	5.10	0.000	5.48	1.83	69.45
2003–2006	-0.17	-0.06	-0.44	3.15	2.71	0.553	3.03	1.01	72.49
2006–2009	0.10	0.02	0.51	3.35	3.86	0.000	3.98	1.33	76.47
2009–2010	0.26	-0.03	0.60	2.53	3.13	0.000	3.36	3.36	79.83
Total, mcy	1.30	1.03	12.48	62.26	74.74	2.761	79.83	–	–
Percent of total	1.6%	1.3%	15.6%	78.0%	93.6%	3.5%	100.0%	–	–
Average, mcy/yr	0.04	0.03	0.35	1.73	2.08	0.08	–	2.22	–

[1] Snake River mile 148.83–139.43, [2] Clearwater River mile 0.28–1.66

[3] Snake River mile 139.29–130.66, [4] Snake River mile 130.66–107.73, [5] Snake River mile 139.29–107.73

b. This study utilized USGS 30-meter resolution digital elevation data from the National Elevation Dataset (NED) (Gesch et al., 2009), land cover datasets from the Multi-Resolution Land Characterization (MRLC) consortium 2001 and 2006 National Land Cover Datasets (NLCD), the U.S. Department of Agriculture National Agricultural Statistics Service (NASS) Cropland Data Layer and land cover data from the U.S. Department of Interior and USDA Forest Service Landfire program. Landsat 5, Landsat 7, and Moderate Resolution Imaging Spectroradiometer (MODIS) satellite imagery was also examined to verify the occurrence and extent of recent wildfires. Surface soil parameters were derived from the USDA digital U.S. General Soil Map State Soil Geographic (STATSGO2).

c. Soil erosion by surface water and landslides in saturated soils are the dominant processes that deliver sediment to rivers and streams in the lower Snake and Clearwater River basins. The rates and frequency of soil delivery events strongly depend on the hydrology of the basin. Representative long-term meteorological and climate data was utilized to model and simulate sediment yield. Soil erosion by wind was assumed to be negligible and not considered in development of the sediment yield models.

d. Climate.

(1) Climate parameters for hydrologic modeling were derived by the USDA from long-term records from stations throughout the U.S. The SWAT model weather generator uses the climate parameters to simulate meteorological inputs for extended simulations (Neitch et al., 2005). In SWAT, actual weather data may be substituted for all or some of the meteorological parameters. In the initial work, the SWAT climate stations were used, however some climate

parameters were subsequently modified to better represent higher elevation conditions. Data on current climate conditions was also obtained from other published sources. Long-term climate averages for selected stations were obtained from the University of Idaho, College of Agriculture Bulletin 784 (Abramovich et al., 1998). Current normal (1971–2000) precipitation and temperature grids were acquired from the Parameter-elevation Regressions on Independent Slopes Model (PRISM) Climate Group at Oregon State University.

(2) Future climate conditions for the Snake and Clearwater River basins were derived from bias-corrected and downscaled World Climate Research Programme (WCRP) CMIP3 climate and hydrology projections that were jointly developed by multiple Federal agencies (USBR 2011).

(a) Climate projections for the sediment yield analysis were derived from outputs of the GFDL 2.1 model developed by the U.S. Dept. of Commerce, National Oceanic and Atmospheric Agency, and Geophysical Fluid Dynamics Laboratory (Delworth et al., 2006).

(b) The climate projections selected for the sediment yield analysis are for the Special Report on Emissions Scenarios (SRES) A2 scenario which represents higher emissions with more fragmented technological change and economic growth. Monthly grids of precipitation, minimum temperature, and maximum temperature for the years 2056 to 2065 were aggregated to produce a 10-year average of monthly and annual values that centered on year 2060.

(c) Climate projections continue to be refined as the result of additional research. Updated projections should be consulted before undertaking new studies.

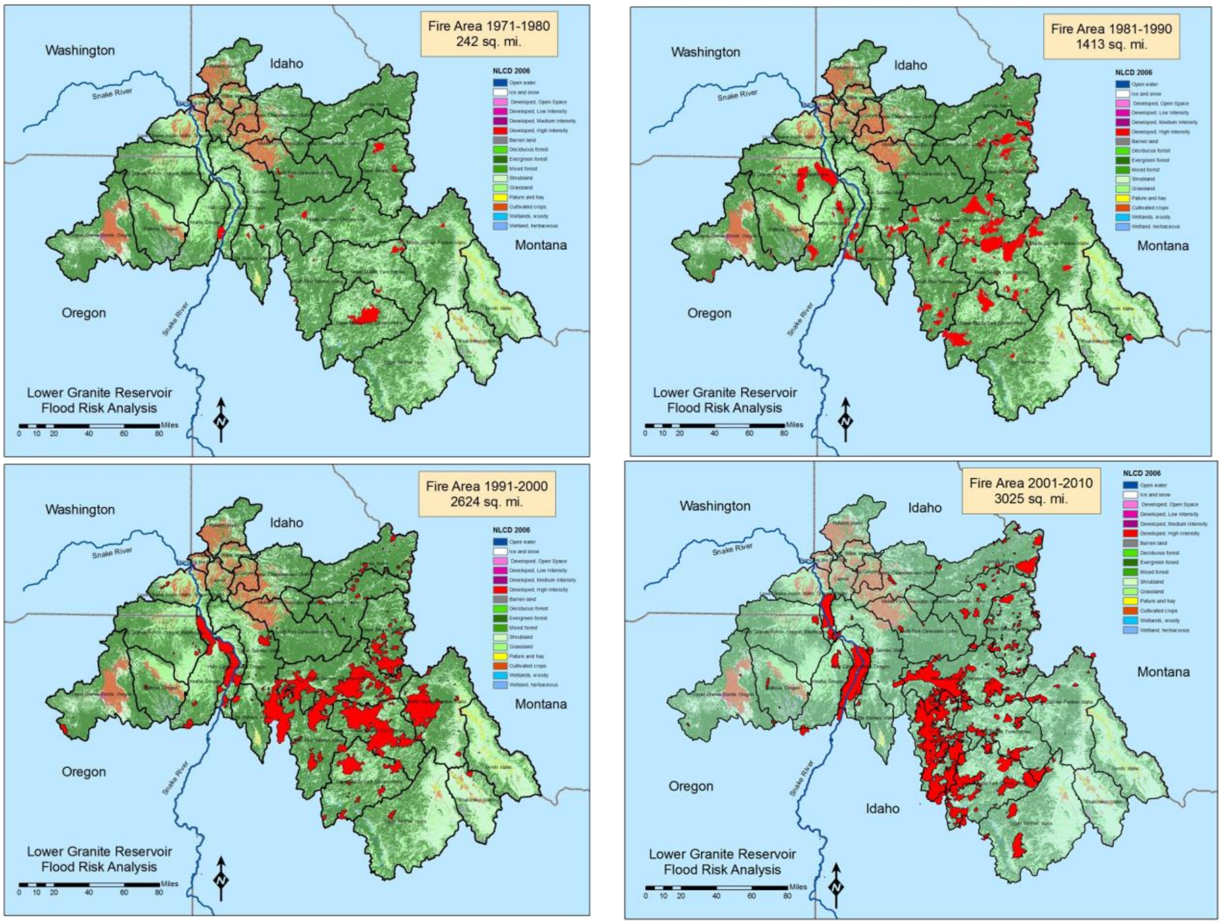
e. Wildfire Data and Evaluation.

(1) Wildfire increases the exposure of soils to rainfall and runoff and substantially increases the potential for erosion and delivery of sediment to the stream channel system (Goode et al., 2010; Elliot 2013). The extent and timing of large fires must be considered in developing a sediment yield model for the Lower Granite sediment yield watershed. Federal and state agencies record information about wildfire occurrence and extent. Annual fire data has been compiled since about 1916, and this extensive dataset has allowed analyses of longer term wildfire-climate relationships.

(2) Fire perimeter data is available from the Geospatial Multi-Agency Coordination Group (GeoMAC). The GeoMAC website provides access to GIS shape files of current fire locations and perimeters in the conterminous 48 states and Alaska, submitted by member organizations. Gibson (2006) describes the origin and characteristics of the historic fire perimeter data for the Northern Rocky Mountains dating back to 1889. Historic fire perimeters for the Grande Ronde basins are available from the USFS Region 656. Fire perimeter data is available directly from GeoMAC beginning in the year 2000.

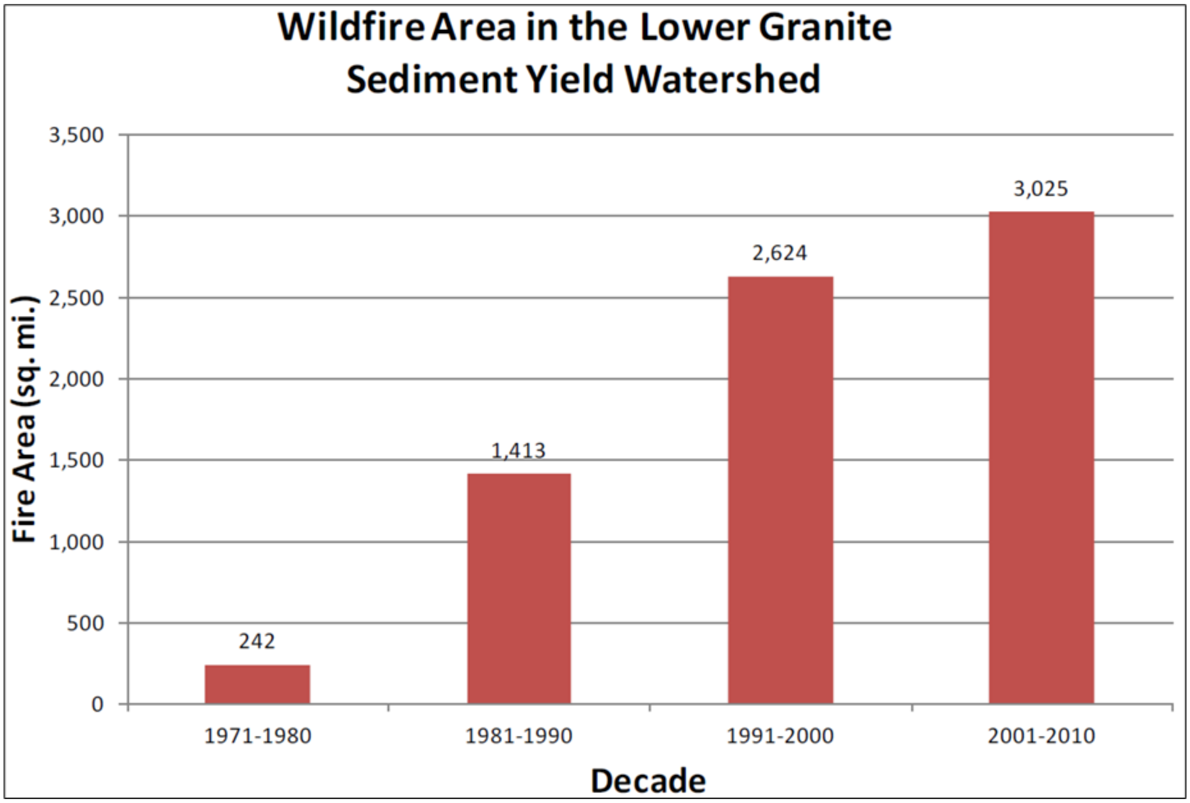
(3) Annual fire perimeter polygons for the Lower Granite sediment yield watershed were combined into decadal polygons (Figure 3). Fire perimeter polygons prior to 2000 were obtained

from the historic data assembled by Gibson and Morgan (2009) for the Northern Rocky region and the Blue Mountain fire history polygons for the Grande Ronde basin. Fire perimeter polygons for 2000 through 2010 were obtained from GeoMAC for all basins.



Case Study 6D Figure 3. Decadal wildfire areas, Lower Snake River drainage; top left = 242 square miles for 1971–1980; top right = 1,413 square miles for 1981–1990; bottom left = 2,624 square miles for 1991–2000; bottom Right = 3,025 square miles for 2001–2010. (USACE 2014)

(4) The data showed a striking increase in the extent of wildfire in the last two decades. During the decade 1971–1980, mapped fire area totaled only 242 sq. mi. in the basin. Conversely, in the 2001–2010 decade, mapped fire area increased to 3025 sq. mi. (Figure 4). Comparisons of contemporary fire perimeters with the very early fire data are limited to qualitative analysis because of the changing methods of delineating burn areas.

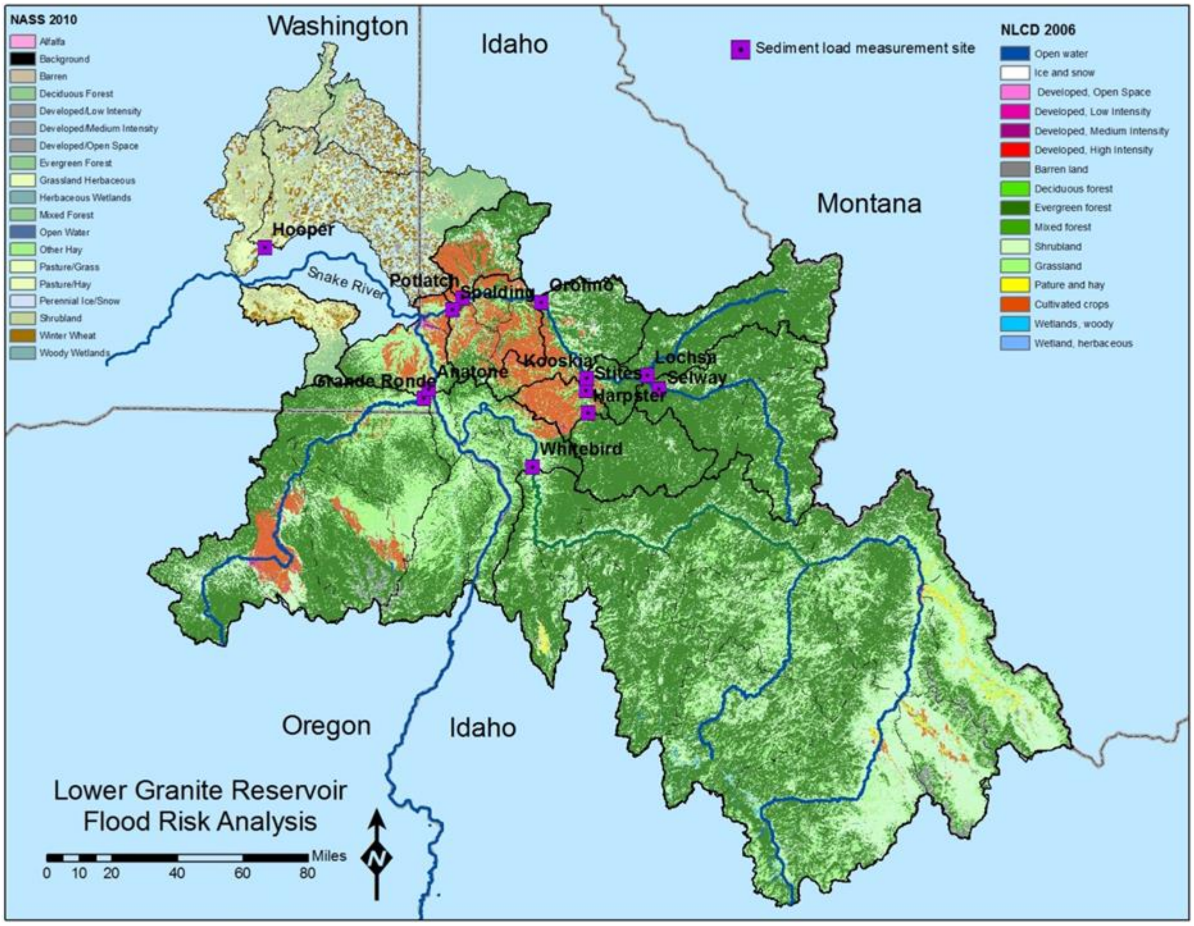


Case Study 6D Figure 4. Decadal wildfire areas, Snake River drainage (USACE-NWW 2014)

f. It is challenging to develop physically based sediment yield models for the many sub-basins of the 27,000 square mile sediment yield watershed of Lower Granite Reservoir. The task is more difficult when the overriding objective is to evaluate the effect of climate variability and fire on both short-term and long-term sediment yield. The semi-physically based SWAT model (Neitsch et al., 2011) was selected because its physical and biological functions are sensitive to the meteorological variables necessary to simulate a realistic response in long-term sediment yield under climate forcing. While fire-affected land cover types are not currently defined in the default SWAT database, existing land cover parameterizations can be adapted to simulate fire effects.

4. Sediment Modeling.

Sediment yield models were developed for the major sub-basins in the sediment delivery watershed for Lower Granite Reservoir and the Palouse River basin (Figure 5). To aid calibration, the outlets of the initial sub-basin models coincide with established sediment load measurement sites. The initial sub-basin models were developed as individual SWAT models. Sediment and flow routing in the main tributary rivers utilized HEC-RAS.



Case Study 6D Figure 5. Sub-basins of the Lower Granite basin sediment yield model (USACE-NWW 2014)

a. Catchments for the SWAT model of the Salmon River watershed above Whitebird (SRWB) were derived by semi-automatic analysis of the 30-meter DEM in the SWAT modeling system. Channels in the catchments were extracted from the DEM and thus only approximate the hydraulic properties of the actual stream system. Accurate representation of the channel system is less important when evaluating sediment yield over a period of several years and decades. Stream channels in the Lower Granite sediment yield watershed may temporarily store transient sediment, but are generally supply-limited so over a period of a few years have capacity to transport all sediment delivered from upland sources.

b. SRWB Sub-Basin Model.

(1) Climate and meteorology for initial simulations for the SRWB sub-basin model were derived from the climate stations included in the SWAT modeling system. There are 11 climate stations in the SRWB area. Meteorological variables in the SWAT data generally agreed well with climate summaries for Idaho (Abramovich et al., 1998). Runoff in the SWAT model was determined by the curve number method, and soil erosion is computed with the MUSLE (Williams 1975).

(2) Initial scenarios of sediment yield were simulated for a period of 30 years to estimate the long-term sediment yield of the sub-basins under constant land cover and climate scenarios. Simulated sub-basin period average yields of water and sediment were compared to measured values at the outlet of the sub-basin model. Accuracy objectives of plus or minus 20% were targeted for both water and sediment yield simulated by the initial sub-basin models.

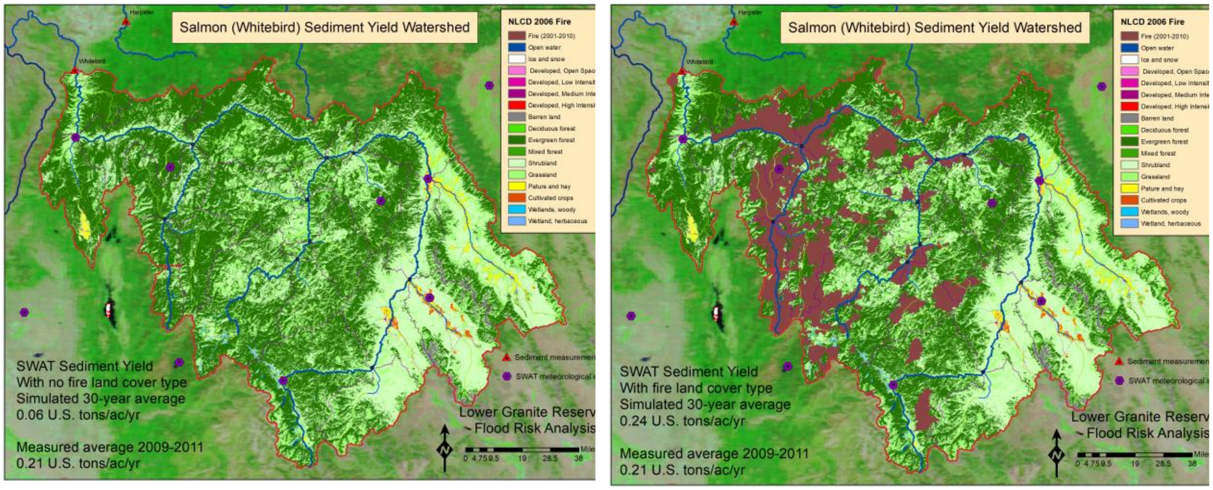
(3) The first 30-year simulation sub-basin produced a water yield that was 17% less than the average yield of 10.9 inches, as determined from the USGS streamgage on the Salmon River at Whitebird for the period 1980 to 2011. Because this was within the accuracy tolerance range, no further calibration adjustment was made to the meteorological parameters or the runoff curve numbers.

(4) Sediment yield for the SRWB was simulated with and without fire-affected area included in the land cover representation (Figure 6). Sediment yield for the first 30-year simulation averaged 0.06 ton/ac/yr, which is 73% less than the 0.21 ton/ac/yr measured at Whitebird for the period 2009–2011.

(5) The first simulation did not include any fire-affected areas during the simulation period, so the sediment yield was expected to be lower than that measured. Fire-affected area was included in a second 30-year simulation of sediment yield in the SRWB sub-basin.

(6) A grid of the fire perimeter area for the period 2001–2010 was merged with the NLCD 2006 grid to produce a composite land cover grid. A SWAT land use type for fire area in forestland and rangeland was defined based on the land use type for evergreen forest (SWAT land use code FRSE). The only change to the FRSE type was to increase the cover and management factor (USLE_C) from 0.001 to 0.01.

(7) With the included fire affected land cover, the simulated sediment yield for the SRWB sub-basin for the 30-year period was 0.24 ton/ac/yr, which is 12% more than the measured sediment yield.



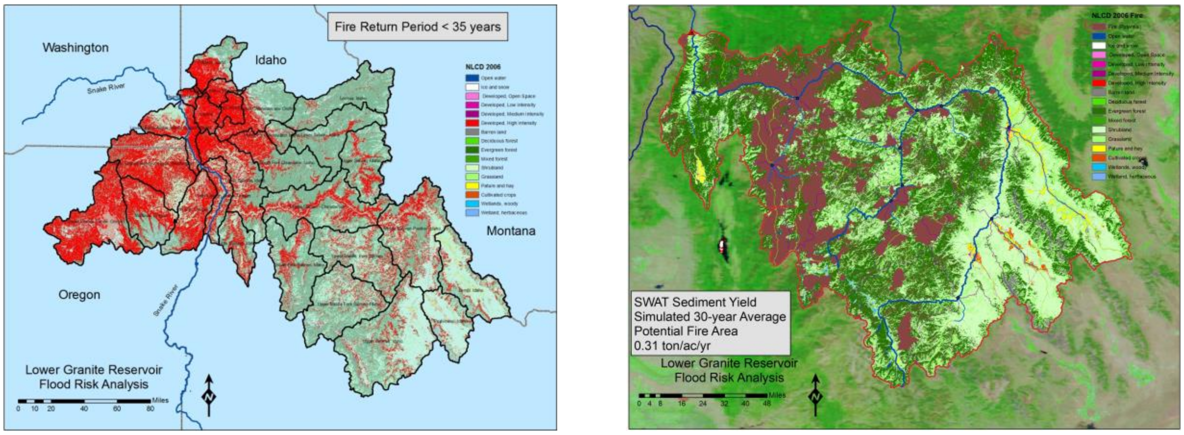
Case Study 6D Figure 6. Salmon River at Whitebird SWAT model catchments with (left) and without (right) wildfire areas (USACE-NWW 2014)

c. Sediment yield models were developed for other main sub-basins in the lower Snake River and Clearwater River basins. Model development procedures were similar to those described above for SRWB. The sediment yield models were calibrated to the measured sediment loads without having to make extraordinary adjustments to the SWAT meteorology and land use parameters. In general, when fire-affected area was included in the land cover parameterization, the SWAT sub-basin models simulated long-term sediment yield reasonably well with only small adjustments of land use parameters. The 20% accuracy objectives were met except for the Lochsa River and the Harpster and Sites sub-basin models for the South Fork of the Clearwater River (Table 2).

d. Evaluation of Fire-Affected Area. The developed SWAT model was used to evaluate potential sediment yield for variation in both the basin fire-affected area and climate change.

(1) Increased area of fire-affected land cover is expected to increase the sediment yield in the Salmon River basin. In the future, it is plausible that the fire-affected area could expand to the remaining forestland and shrubland that has moderate to high cover density and high relative likelihood of fire.

(2) Expected fire-affected areas were identified in Salmon River basin as grid cells that had greater than 50% cover for the forest and shrub class in Landfire existing vegetation canopy cover (EVC) grid, and had a greater than 50% relative likelihood of wildfire in the Parisien et al. (2012) full-model grid. With this criteria, the fire-affected area increased from 16% to 22% of the SRWB sub-basin and the modeled 30-year average sediment yield increased from 0.21 ton/ac/yr to 0.31 ton/ac/yr, a 48% increase (Figure 7).



Case Study 6D Figure 7. Salmon River at Whitebird; left: figure regime group with fire return period less than 35 years; right: SWAT model catchments with potential wildfire areas (USACE 2014)

(3) This simulation of the effect of increased fire-affected area on sediment yield is likely conservative. As vegetation recovers in older fire-affected areas, the exposure to surface erosion decreases. Wildfire will eventually affect all the areas identified, but the annual rate of increase in fire-affected area may be offset by the recovery of previously burned areas, so that annual sediment yield remains relatively constant.

(4) Studies have illustrated a variable rate of decline in the wildfire-affected sediment yield rate, with the initial 1 to 3 years following fire occurrence having the highest rate, followed by recovery to near normal sediment yield rates within 7 to 10 years (Robichaud et al., 2000). This benign projection seems less plausible considering the recent history of fire in the Lower Granite sediment yield watershed.

e. Evaluation of Climate Change Effects.

(1) The potential effect of climate change on sediment yield in the Lower Granite basin was characterized with the aid of the GFDL climate projections (Delworth 2006). Monthly grids of precipitation, minimum temperature, and maximum temperature for the years 2056 to 2065 were aggregated to produce a 10-year average of monthly and annual values that centered on year 2060. The average annual grids of current (1971–2000) normal precipitation and temperature were extracted from grids developed by the PRISM group for the U.S.

(2) Using GIS methods, the basin average annual precipitation and temperatures were derived for each of the hydrologic unit code (HUC) 8 sub-basins in the Lower Granite sediment yield watershed and for the main river basins. The derived temperature and precipitation values agree with recent descriptions of expected climate trends in the Pacific Northwest River Management Join Operating Committee (RMJOC 2011).

(3) As noted previously, a warming climate is expected to increase the extent of fire-affected areas. According to the GFDL model, total annual precipitation in the Lower Granite sediment yield watershed will not change substantially. Under the predicted warming trend, proportionally more precipitation will occur as rainfall on a land surface that is less protected by vegetative cover and snowpack, resulting in increased sediment yield. Snowpack and snowmelt regimes will change as a result of both climate warming and loss of vegetative cover.

(4) Simulation of these climatic changes, including the extent of fire-affected area, requires refined meteorological parameterization and more detailed sub-basin parameters than those in the initial sub-basin models discussed above to discern response of the SWAT model to fire effects. The reasonable agreement between the simulated and measured values of long-term historic sediment yield, however, showed that the SWAT model was capable of reproducing water and sediment yield with sufficient accuracy to characterize the response of the Lower Granite sediment yield basin to changes in fire-affected area and climate.

Case Study 6D Table 2
Summary of SWAT Sub-Basin Models in the Lower Snake and Clearwater River Basins

	Area	Measured Sediment Yield	Simulated Sediment Yield	Sim Sed Yield Error	Actual Water Yield	Simulated Water Yield Error	Forestland Fire Area (2001-2010)	Conservation Practice Adjustment for WWHR and SWHT[3]		
Watershed	**mi^2**	**U.S. tons ac^{-1} yr^{-1}**	**U.S. tons ac^{-1} yr^{-1}**		**in.**	**Percent**	**mi^2**	**CN 2**	**USLE P**	**Filter W (m)**
Asotin[1]	326	0.08	0.08	0%	4.0	16.3%	–	75	0.6	10
Palouse-Hooper	1940	0.42	0.45	7%	4.1	2.6%	–	–	0.7	2
Selway	1904	0.10	0.10	0%	26.3	-9.6%	185	–	–	–
Lochsa	1130	0.08	0.20	150%	33.0	-36.2%	115	–	–	–
Potlatch	593	0.25	0.23	-8%	8.85	17.9%	–	–	0.7	5
Salmon Whitebird	13387	0.21	0.24	14%	10.9	-17.2%	2105	–	–	–
South Fork Clearwater Harpster	870	0.09	0.06	-33%	–	–	–	–	–	–
South Fork Clearwater Sites	1163	0.08	0.12	50%	11.7	23.5%	–	70	0.6	5
Lapwai[2]	269	–	0.21	–	3.9	6.7%	–	–	0.7	0
Tucannon	504	–	0.13	–	4.6	-6.5%	–	–	–	–

[1] Asotin sediment measured as total suspended solids (TSS)

[2] Diversion for irrigation above gage.

[3] WWHR is winter wheat high-residue SWAT class, SWHT is spring wheat.

5. Summary.

The SWAT model appears to respond reasonably well in capturing the effects of burned area from a simple parameter change to account for the cover and management factor. Coupled with methods being developed to evaluate the spatial distribution of wildfire probability across the western U.S., such as those by Parisien et al. (2012), it may become possible to project watershed sediment yields more effectively for those most affected by fire. Other geospatial tools, such as the Automated Geospatial Watershed Assessment (AGWA) tool (Goodrich et al., 2005), can make the process even easier.

Case Study 7A
Sediment Impact Assessment and Stable Channel Design

1. Case Study.

This case study was condensed from content in Appendix G of Hydraulic Design of Stream Restoration Projects (Copeland et al., 2001). A similar presentation of this information was also included in the National Engineering Handbook, Part 654, Technical Supplement 13B (USDA 2007b). The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

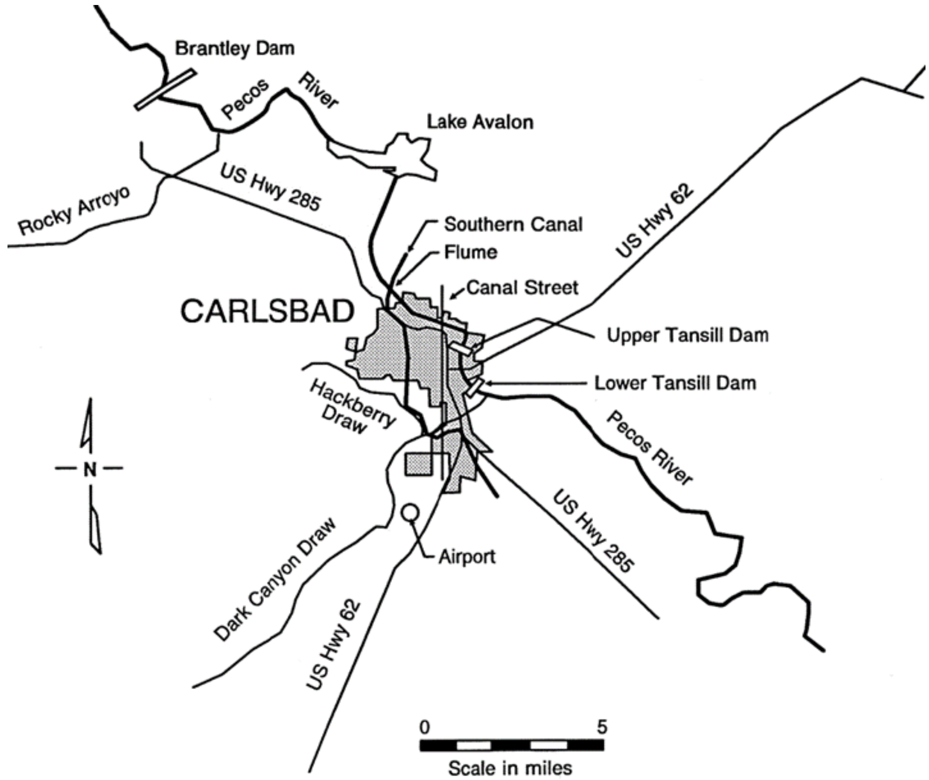
a. Study Background. A sediment impact assessment was conducted as part of the reconnaissance-level planning study for a flood damage reduction project for the City of Carlsbad, New Mexico (Copeland 1995). The assessment purpose was to identify the magnitude of possible sediment problems that might be associated with the proposed project. One potential source of flooding was Dark Canyon Draw, a tributary of the Pecos River (Figure 1). One of the flood damage reduction alternatives being considered was a bypass channel to divert Dark Canyon Draw around the City of Carlsbad. The proposed diversion would begin near the city airport and flow northeasterly to the Pecos River to a location about 5 miles downstream from the city.

b. Potential Concerns. Depending on the diversion channel design, several sedimentation and channel stability problems could occur. The sediment impact assessment was conducted to determine the magnitude of possible sediment degradation or aggradation problems and to obtain relatively stable dimensions for the diversion channel.

(1) If a threshold channel is constructed (a channel designed with little or no sediment transport potential), then bed material delivered from upstream would deposit at the diversion entrance. Sediment deposits would have to be removed periodically.

(2) Even if a channel is designed to carry the incoming sediment load, there will be a period of adjustment for the channel, as the bed and banks become established. Bed armoring may progress quickly or slowly, with extensive degradation, depending on the consistency of the material through which the diversion channel is cut and the sequence of annual runoff that occurs.

(3) If the diversion channel is too efficient in terms of sediment transport capacity, it could degrade and induce additional channel degradation upstream from the diversion location.



Case Study 7A Figure 1. Carlsbad and surrounding areas (Copeland et al., 2001)

3. Field Reconnaissance.

A preliminary assessment of channel stability and potential sediment impacts were determined during a two-day field reconnaissance. This brief reconnaissance provided insight for general observations related to channel stability.

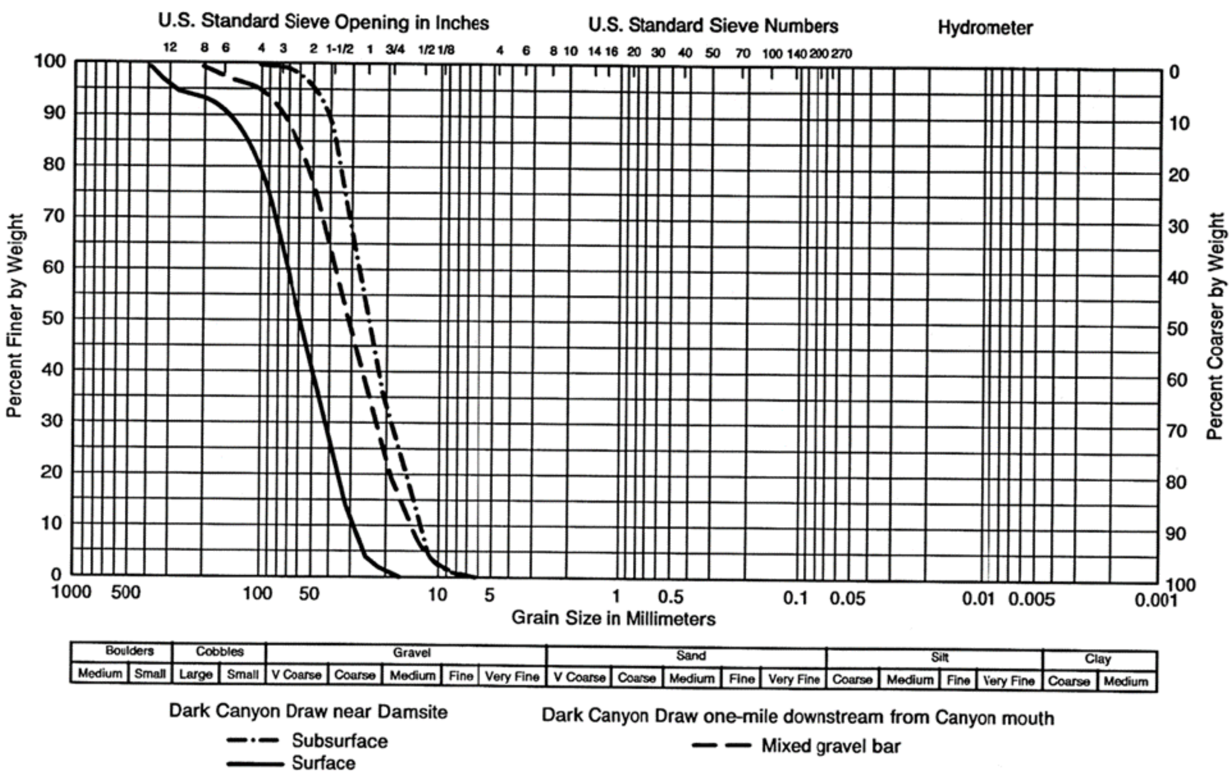
a. Dark Canyon Draw Characteristics. Dark Canyon Draw transitions from a wide, shallow alluvial channel, characteristic of southwestern United States alluvial fans, at its canyon mouth to an incised arroyo at its confluence with the Pecos River. At the time of the field reconnaissance, gravel mining was occurring in the lower reaches of Dark Canyon Draw between the Pecos River and the city airport, and appeared to have been occurring for many years. Due to the gravel mining, the channel had been both widened and deepened. The channel also showed signs of incision/degradation upstream from the airport.

b. Bed and Banks. The bed and banks of the incised channel were capable of supplying significant quantities of sediment to the stream. The bed surface of Dark Canyon Draw consisted primarily of coarse gravel and cobbles. Banks were generally composed of loose alluvial material ranging in size from clays and silts to boulders. The channel tended to migrate laterally, eroding banks and creating remnant gravel bars in former channels.

c. Armoring. Armoring was generally observed in the existing low flow channel. However, at high flows the channel migrated, mobilizing significant sediment from the gravel bars and from bank erosion.

d. Sampling. Bed material samples were collected during the field reconnaissance. Sample size class distributions were determined using the Wolman (1954) pebble count method. Due to the limited scope of the sediment impact assessment, samples were collected at only two sites (near the potential damsite at canyon and one mile downstream from the canyon mouth). Both surface and sub-surface samples were collected at the mouth of the canyon, which is several miles upstream from the proposed diversion channel.

e. Sampling Results. There was no coarse surface layer at the second site, which was located on a gravel bar, about 1 mile downstream from the canyon mouth. The thoroughly mixed bed form was an indication that active-layer mixing had occurred during the last flow events at this site. Median grain size ranged between 22 and 55 mm for all the samples. The gradation determined at the downstream site was selected as the representative gradation for the sediment impact assessment because it was characteristic of a fully mobile bed. Bed material gradations determined from these samples are shown in Figure 2.

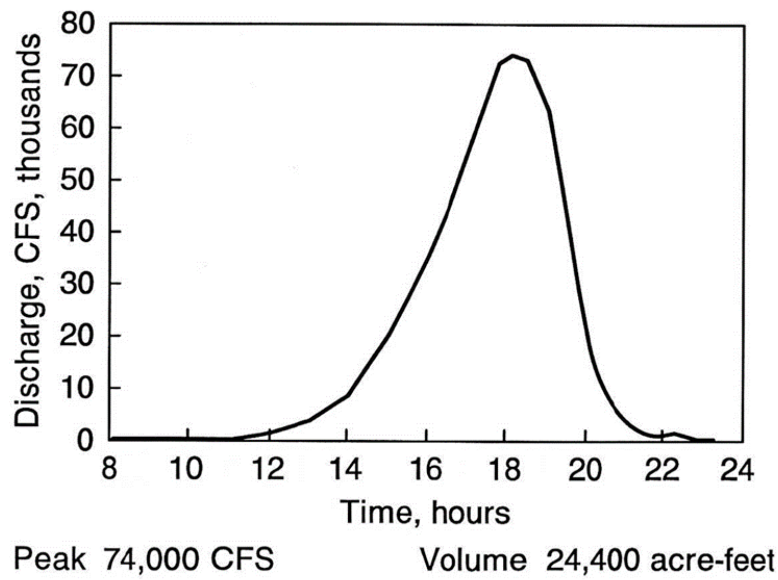


Case Study 7A Figure 2. Bed material gradations, Dark Canyon Draw (Copeland et al., 2001)

4. Hydrology.

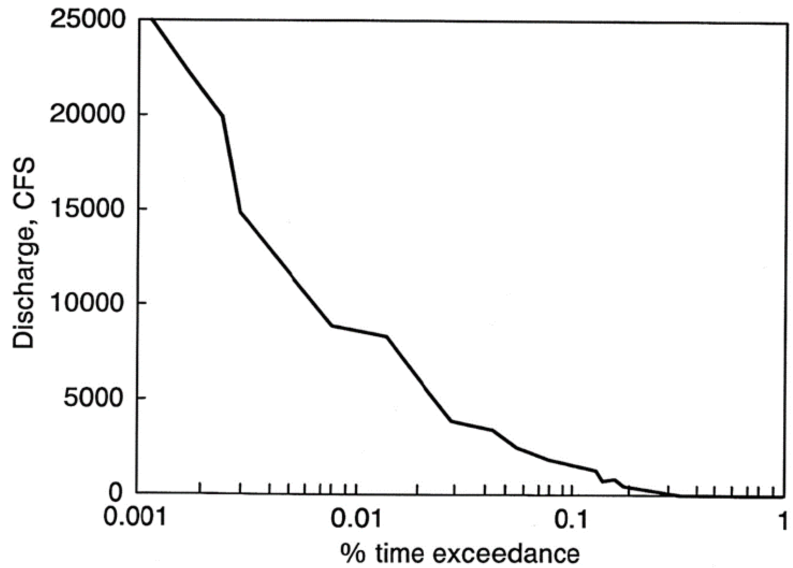
Hydrographs used in the sediment impact assessment were developed using the HEC-1 (USACE 1998) hydrograph package, the USACE predecessor to HEC-HMS (HEC 2012). These were used to calculate sediment yield for flood events.

a. Peak Flows. The peak discharge for the 1% exceedance flood was 74,000 cfs. The 10% chance exceedance hydrograph was assumed to have the same shape as the 1% chance exceedance flood; discharges on the hydrograph were calculated by multiplying the 1% chance exceedance hydrograph by the ratio of the peaks. The 10% chance exceedance peak discharge was 20,000 cfs. The 1% chance exceedance hydrograph for Dark Canyon Draw is shown in Figure 3.



Case Study 7A Figure 3. The 1% chance exceedance hydrograph, Dark Canyon Draw (Copeland et al., 2001)

b. Flow Duration. A flow-duration curve was developed from 18 years of USGS mean daily flow data from the Dark Canyon at Carlsbad gage. Durations of published peak flows greater than the maximum mean daily flow were added to the flow-duration data by assuming the historical flood hydrographs had shapes similar to the 1% chance exceedance hydrograph. The flow-duration curve is shown in Figure 4.



Case Study 7A Figure 4. Dark Canyon Draw flow duration (Copeland et al., 2001) (to convert cubic feet per second to cubic meters per second, multiply by 0.02831)

5. Hydraulic Parameters.

a. Reach Selection. A typical reach in the existing Dark Canyon Draw channel was selected from a HEC-2 (USACE 1990a) backwater model, the USACE predecessor to HEC-RAS (HEC 2016a). The typical reach chosen for this analysis was about 2 miles long and was located adjacent to the Carlsbad Airport. The reach was considered to be in a state of non-equilibrium due to its proximity to gravel mining operations.

b. Recommendation for Further Study. A reach farther upstream, less influenced by gravel mining operations, was preferred for determining long-term sediment yield. However, the existing backwater model did not extend any farther upstream. It was recommended that additional cross-section surveys upstream be obtained for more detailed sediment studies.

c. Hydraulic Computations. Water surface elevations and hydraulic variables were calculated using the HEC-2 model for a range of discharges. Average values for hydraulic variables were then determined using the reach-length weighted averaging procedure in SAM (Thomas et al., 2000).

6. Sediment Transport Rating Curve.

The bed material sediment yield of Dark Canyon Draw is important when considering sediment transport and channel stability questions.

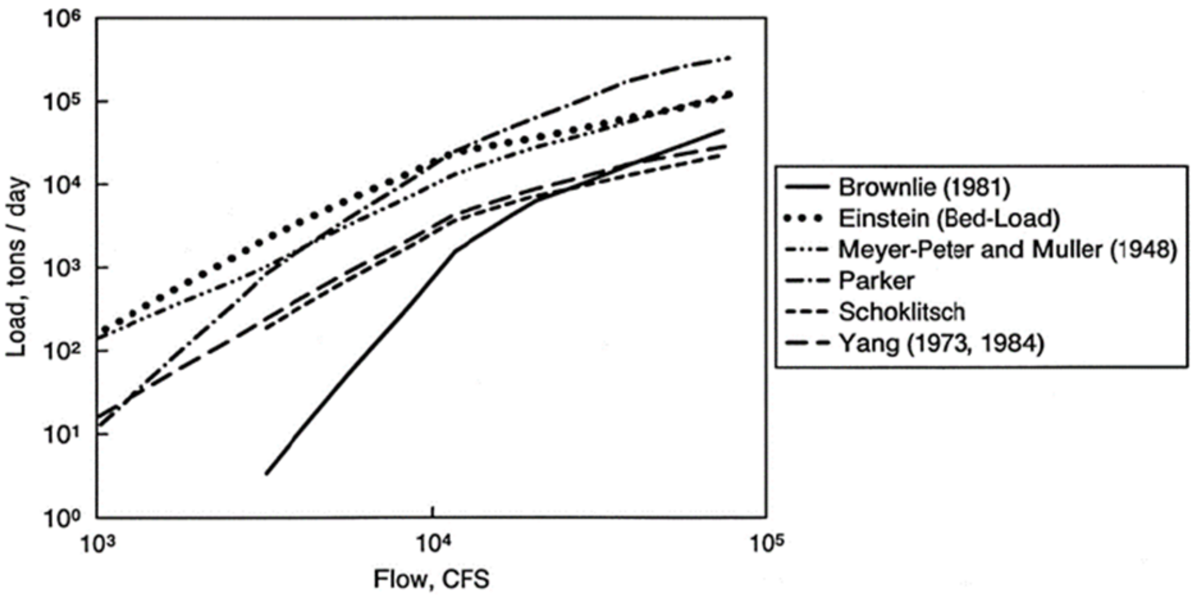
a. Bed Material Load. The bed material sediment load consists of the sediment sizes that exchange with the streambed as they are transported downstream. The bed-material yield is most likely to be relatively small compared to the total sediment yield because the bed of Dark Canyon Draw consists primarily of gravels and cobbles and a significant volume of material is not projected to be transported. The wash load component of the total sediment yield will be transported through the system to the Pecos River unless it is trapped by a reservoir or introduced into a ponded area.

b. Sediment Transport Equations. Sediment transport was calculated using several sediment-transport equations available in the SAM program. The equations chosen were those that included at least some data from gravel-bed rivers in their development.

(1) As can be seen from the sediment-discharge rating curves shown in Figure 5, there is a wide range in predicted sediment transport rates. There is no available data on Dark Canyon Draw to aid in the selection of a transport equation. However, the guidance program in SAM identified the North Saskatchewan and Elbow Rivers in Saskatchewan, Canada, as having similar median bed grain sizes, depths, velocities, and slopes as Dark Canyon Draw at high flow.

(2) The guidance program determined from the available set of equations in SAM that the Schoklitsch equation (Shulitis 1935) best reproduced measured data on the North Saskatchewan and Elbow Rivers. Based on the comparison of calculated sediment transport rating curves using different sediment transport functions shown in Figure 5, the Schoklitsch equation produces a relatively low sediment yield.

(3) To cover the uncertainty range in the calculated bed material sediment yield, two additional sediment transport equations were chosen to calculate yield. The Parker equation (Parker 1990) was used to represent a high sediment transport load, and the Einstein (1950) equation was chosen to represent an intermediate sediment transport load.



Case Study 7A Figure 5. Bed material sediment transport rating curves, Dark Canyon Draw (Copeland et al., 2001)

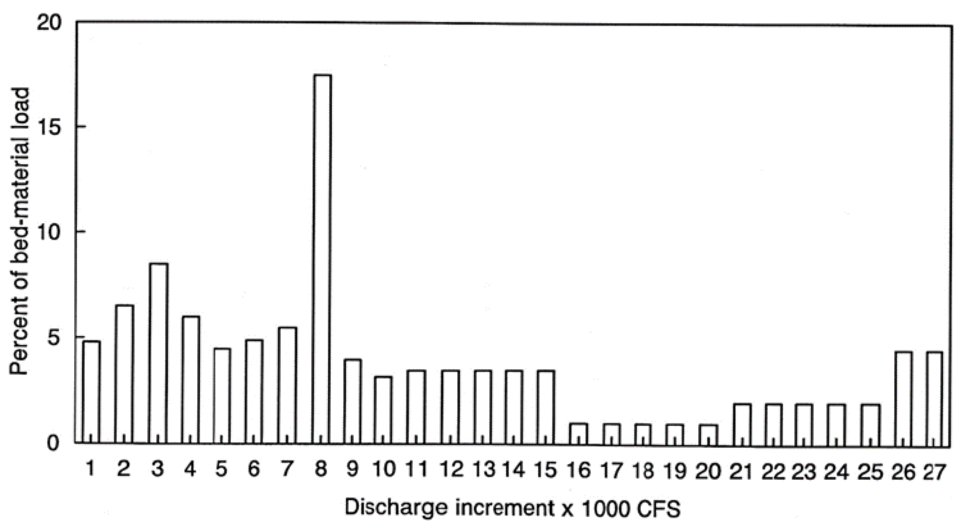
7. Diversion Channel Design.

The stable-channel analytical design method in SAM was used to size the low-flow channel. This method provides channel dimensions that will transport the incoming bed-material sediment load for a specified discharge.

a. Method. The method in SAM uses the Brownlie (1981) equation, presented in EM 1110-2-1601 (USACE 1994b) to calculate sediment transport and roughness on the channel bed. This equation was not developed for gravel-bed streams, and predicts lower sediment transport rates at lower discharges than other tested equations shown in Figure 5. This apparent deficiency in the sediment-transport equation is accounted for later by testing the resultant cross-section geometry using other transport equations.

b. Design Criteria. The criteria chosen for the diversion channel design were: (a) a composite-channel geometry with a low-flow channel designed to carry the effective discharge, and (b) the overbank designed using threshold criteria for the 1% chance exceedance flood. Assigned side slopes were 1V:3H, with a side slope Manning's roughness coefficient of 0.05.

c. Effective Discharge. The effective discharge is the discharge that transports the largest percentage of the bed material sediment load. This was determined by integrating the flow-duration curve for Dark Canyon Draw and a sediment-transport rating curve developed using the Einstein formula. A plot of percentage of bed-material load versus discharge increment is shown in Figure 6; an effective discharge of 8,500 cfs was indicated.



Case Study 7A Figure 6. Effective discharge, Dark Canyon Draw (Copeland et al., 2001)

d. Sediment Parameters.

(1) The inflowing sediment concentration was determined for the effective discharge from the sediment-transport rating curve developed for the typical reach of Dark Canyon Draw.

(2) The Brownlie sediment-transport equation was used for typical reach to be compatible with the calculations in the design reach.

(3) The bed material gradation in the diversion channel was assumed to be the same as in Dark Canyon Draw. This is a reasonable assumption for the long-term condition in the diversion channel. However, initial conditions should be determined with available site sampling data. The transition from initial to final conditions could be determined in future, more detailed studies using a numerical model such as HEC-6 (Thomas 2002) or HEC-RAS (HEC 2016a).

e. Slope Selection. Using the natural slope between the proposed Dark Canyon Draw diversion and the Pecos River, a unique solution for width and depth was determined for the effective discharge channel. The average slope between Dark Canyon Draw at the airport and the Pecos River is 0.0047. The ground slope is steeper at the airport and becomes very mild as it crosses the Pecos River floodplain. A more detailed analysis should include different channel geometries due to variation in slope.

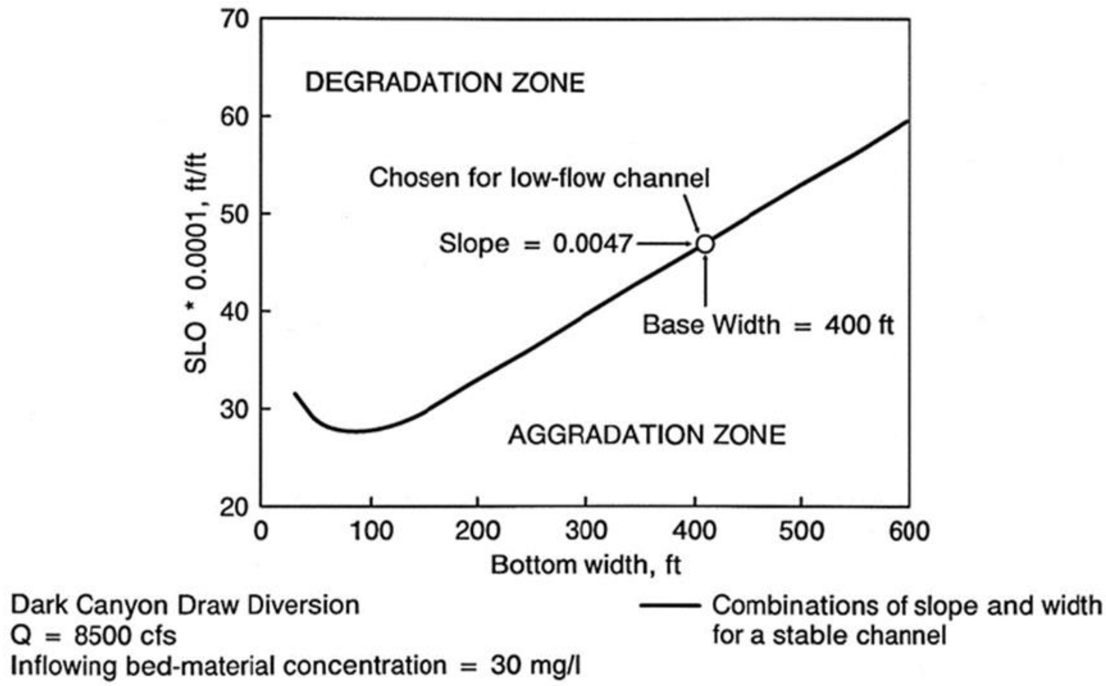
f. Stable-Channel Curve. The stable-channel curve for 8,500 cfs is shown in Figure 7. This curve is developed using different combinations of width and slope for the defined sediment inflow concentration, side slope, bank roughness coefficient, bed material d_{50}, bed material gradation coefficient, and water discharge.

(1) If sediment inflow is to be calculated, which is the recommended procedure in SAM, then additional data are required for the supply reach: base width, side slope, bank roughness coefficient, bed-material median grain size, geometric gradation coefficient, average slope, and discharge. It is important that the base width represent the total movable-bed width of the channel.

(2) The bank roughness should serve as a composite of all additional roughness factors, (such as channel irregularities, variations of channel cross-section shape, and the relative effect of obstructions, vegetation, and sinuosity). Only flow that is vertical above the bed is considered capable of transporting the bed material sediment load. Refer to the SAM (Thomas et al., 2002) and HEC-RAS (HEC 2016a) user manuals for additional information.

(3) The stable-channel curve (Figure 7) defines combinations of width and slope that provide for movement of the inflowing sediment load through the diversion channel. Constraints on this wide range of solutions may result from a maximum possible slope, or a width constraint due to right-of-way. Maximum allowable depth could also be a constraint. Depth is not plotted in Figure 7, but it is calculated for each slope and width combination determined.

(4) With constraints, the range of solutions is reduced. The average slope for the diversion channel, if no drop structures were employed, is 0.0047. With this slope, the stable-channel method suggests that a base width of about 400 feet is stable. The calculated depth was 3.5 feet.



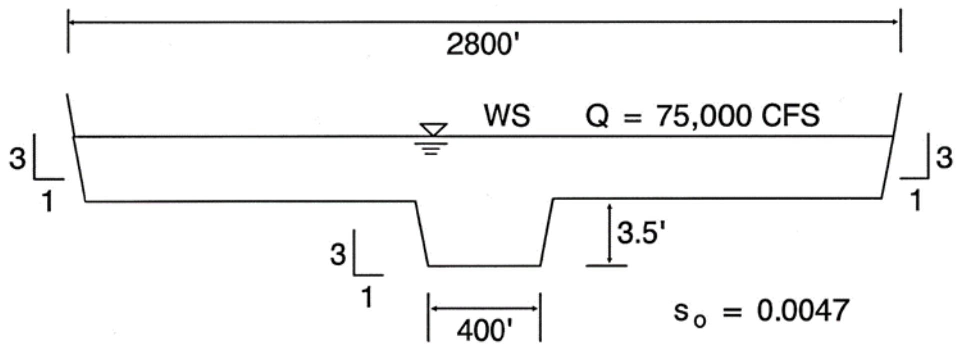
Case Study 7A Figure 7. Preliminary diversion channel design (Copeland et al., 2001)

(5) The width of the overbank portion of the channel was determined by trial and error using threshold velocity criteria. With a median bed material size of about 30 mm, chosen from the 22 mm to 55 mm range mentioned earlier, and a water depth of 5 feet, a threshold velocity up to 6 ft/s would be appropriate for channel stability considerations.

(6) Roughness on the overbank was calculated using the Brownlie roughness predictor (USACE 1995b).

(7) The total width of the overbank and channel was determined to be 2,800 feet. The details of the final geometry are shown in Figure 8.

(8) If the threshold velocity is exceeded, degradation can be expected. The extent of degradation can be estimated in a more detailed study using a numerical sediment transport model.



Case Study 7A Figure 8. Cross-section Dark Canyon Draw diversion channel (Copeland et al., 2001)

8. Sediment Budget.

The magnitude of potential aggradation or deposition problems in the Dark Canyon channel can be determined by calculating bed material sediment yield through a typical reach of the existing channel and comparing it to calculated sediment yield in the project reach.

a. Bed material sediment yield was calculated for the existing channel using the flow-duration sediment transport curve method and SAM. Sediment yields were calculated for the 1% and 10% chance exceedance floods using synthetic hydrographs and average annual conditions, using the flow-duration curve. Bed-material sediment yields calculated using three different sediment transport equations are listed in Table 1.

Case Study 7A Table 1
Calculated Bed-Material Sediment Yield Dark Canyon Draw (Copeland et al., 2001)

	1% exceedance flood		10% exceedance flood		Average Annual	
	m^3	yd^3	m^3	yd^3	m^3	yd^3
Schoklitsch	2,400	3,100	530	690	180	230
Einstein	11,300	14,800	3,300	4,300	1,300	1,700
Parker	27,700	36,200	4,100	5,400	1,100	1,500

Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m^3 (93 lbs/ft^3).

b. Sediment yield was determined in the diversion channel using the same procedure that was used to calculate sediment yield in the typical reach of the existing channel. Trap efficiency was then determined for flood hydrographs and for average annual conditions.

c. The potential for aggradation/degradation in the diversion channel for a 10% and 1% chance exceedance floods and for average annual conditions was determined using the sediment budget approach. Bed material sediment yield was calculated using three sediment transport equations and compared to the calculated bed material sediment yield in the existing Dark

Canyon Draw. Bed material sediment transport was assumed to occur only in the low flow channel in the diversion.

d. Calculated bed material sediment yield and its percentage of the total bed material yield calculated for Dark Canyon Draw is shown in Table 2, which indicates that deposition will occur in the diversion channel for all cases tested. For the 1% chance exceedance flood, between 34% and 38% of the inflowing bed material sediment load will deposit in the diversion channel. For the 10% chance exceedance flood, between 12% and 17% of the inflowing bed material load will deposit. For average annual conditions, between 6% and 18% of the inflowing sediment load will deposit.

e. A range of the deposition quantities can be determined from these calculations. Recall that the Schoklitsch equation produced sediment transport quantities closest to the measured data from a river with similar characteristics.

Case Study 7A Table 2
Calculated Bed-Material Sediment Yield Diversion Channel (Copeland et al., 2001)

Sediment Transport Function	1% Exceedance Flood			10% Exceedance Flood			Average Annual		
	m^3	yd^3	Percent of Inflow	m^3	yd^3	Percent of Inflow	m^3	yd^3	Percent of Inflow
Schoklitsch	1,600	2,050	66	450	590	86	150	190	82
Einstein	7,500	9,800	66	2,900	3,800	88	1,200	1,600	94
Parker	17,100	22,400	62	3,400	4,500	83	1,000	1,300	87

Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m^3 (93 lbs/ft^3).

f. At the next-level planning, it is necessary to evaluate the temporal development of the diversion channel using the HEC-6 (Thomas 2002) or the HEC-RAS (HEC 2016a) numerical sedimentation model. In this sediment impact assessment, the bed material gradation was assumed to be already developed. A more detailed study requires knowledge of the existing soil profile through which the channel will be cut. The armoring process needs to be simulated with a numerical model. In addition, the slope of the diversion channel will vary between the diversion point and the Pecos River. This requires a more detailed analysis of spatial variability in the sedimentation processes.

Case Study 7B
Sediment Impact Assessment of Navigation Projects for Apalachicola River, Florida

1. Case Study.

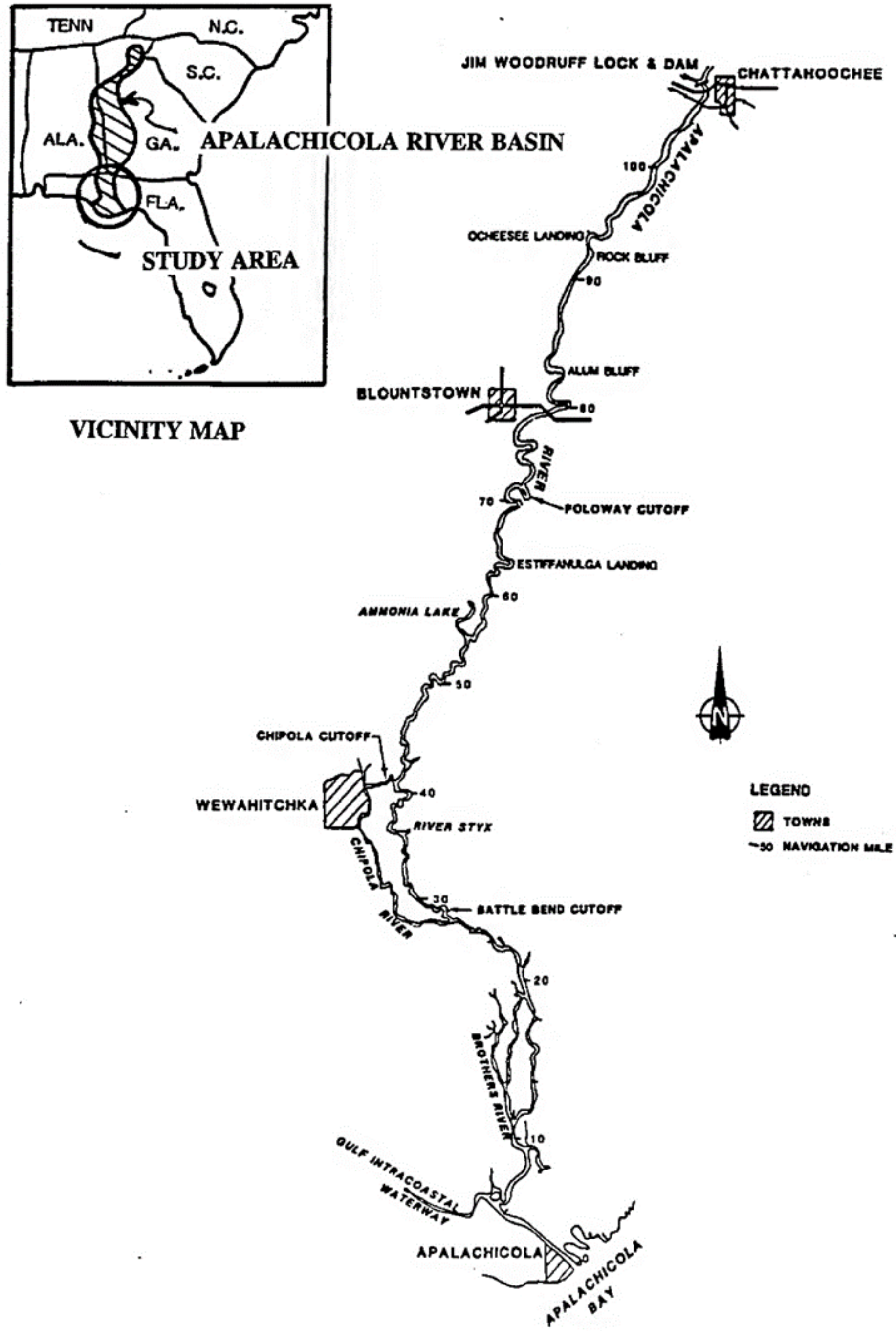
This case study documents a sediment impact assessment completed for the USACE Mobile District. This case study is condensed from the document "Sediment Impact Assessment for Navigation Channel Maintenance, Alabama River, Alabama, and Apalachicola River, Florida" (McComas and Copeland 1996). The partial content demonstrates several concepts but is not comprehensive of USACE study requirements. The case study refers to using models HEC-2 and SAMwin, which have been succeeded with models such as HEC-RAS and HEC-6. The case study illustrates a reconnaissance study detail level pertaining to navigation channel sedimentation concerns.

2. Purpose.

The Apalachicola River in Florida was facing proposed navigation channel design changes due to proposed changes in the minimum release from upstream dams. The purpose of the sediment impact assessment was to identify and roughly quantify the magnitude of sediment problems associated with alternative proposed navigation channel designs. The selected sediment budget approach is generally appropriate for a reconnaissance level planning study.

3. Introduction.

The relative magnitudes of potential dredging requirements for three proposed channel modification plans on the Apalachicola River were compared using a sediment budget approach. The study reach on the Apalachicola River extended between Apalachicola Bay and Jim Woodruff Lock and Dam (Figure 1). Sediment transport rating curves were calculated for each plan at five typical dredging reaches. Average annual flow duration curves were then numerically integrated with the sediment transport rating curves to calculate average annual sediment transport capacity for each plan in each of the designated reaches.



Case Study 7B Figure 1. Study location map

4. Hydraulic Model Parameters.

a. Three Channel Geometries. The channel geometries for the three alternative channel modifications were developed in the form of HEC-2 backwater models. The models extended from navigation mile (NM) 6.0, near Apalachicola Bay, to Jim Woodruff Lock and Dam at NM 106. The three models represent conditions where a nine-foot depth navigation channel is maintained for minimum flow discharges of 9,300; 11,000; and 13,000 cfs. The HEC-2 models did not include overbank geometry. The models included estimates for flow diversion percentages into bypass channels for discharges of 9,300; 11,000; and 13,000 cfs.

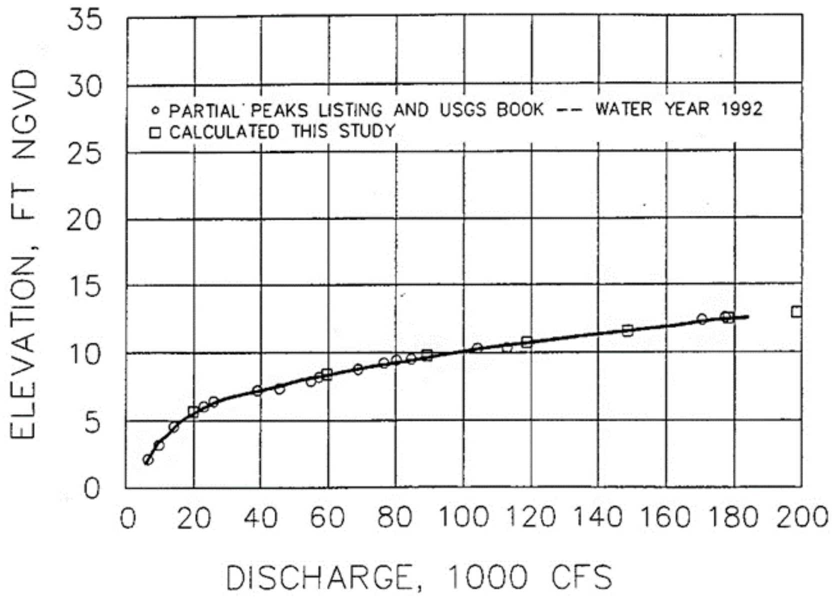
b. Model Flow Range. The discharge range in the original models had to be expanded to determine hydraulic parameters for the full range of the average annual flow duration curve (4,700–185,000 cfs). The model for the existing minimum flow channel of 9,300 cfs was used as the base model to estimate channel discharges for the sediment budget analysis. Roughness coefficients determined from previous studies were used in the backwater model. The downstream water surface elevation was assumed to be mean sea level for the full range of discharges.

c. Bypass Flow. Bypass flow diversion percentages determined in previous studies were used in this study.

(1) Initially, the percentages determined for the 13,000 cfs minimum flow channel were assumed for 13,000 cfs and all discharges greater than 13,000 cfs. The percentage determined for the 9,300 cfs minimum flow channel was used for 9,300 cfs and discharges less than 9,300 cfs.

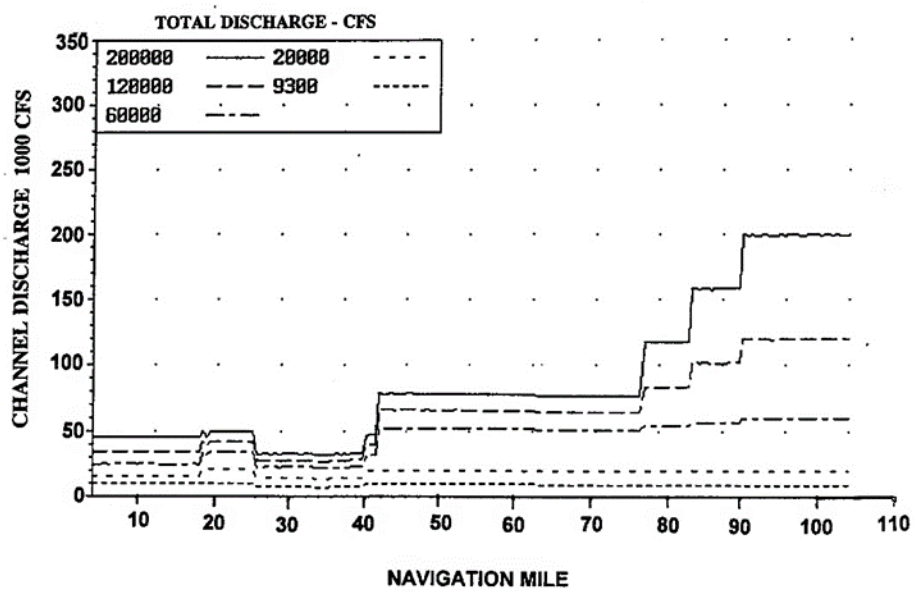
(2) At higher flows, a greater percentage of the total discharge is diverted onto the overbanks, and since the model geometry included only the main Apalachicola River channel, it was necessary to estimate the flow percentage in the overbanks over the full range of discharges.

(3) A trial and error procedure in which channel discharges were adjusted was used to develop calculated water-surface profiles that matched measured stage-discharge rating curves at four gages on the Apalachicola River. These were near Sumatra (NM 20.6), near Wewahitchka (NM 44.2), near Blountstown (NM 78), and at Chattahoochee (NM 106). These rating curves were based on measured stages at the gages and the total discharge at some upstream point where the total discharge could be determined. Figure 2 presents an example of rating curve model results.



Case Study 7B Figure 2. Comparison of measured and calculated stage-discharge rating curves, Apalachicola River near Sumatra

d. Discharge Distribution. The measured and calculated rating curves demonstrate that the HEC-2 model produces stages and discharges in the Apalachicola River main channel suitable for the scope of this evaluation, despite the limitations of the assumptions. The same longitudinal discharge distribution was used for all three minimum flow channel alternatives. Figure 3 presents the final model channel discharges used to reproduce the stage rating curves.



Case Study 7B Figure 3. Channel discharge used in the HEC-2 Model, Apalachicola River

5. Sediment Evaluation.

Five reaches were chosen as typical dredging reaches for the sediment budget evaluation. These are reaches where dredging has been required in the past. Dredging records between 1957 and 1994 were used in the selection process. Average hydraulic parameters for each reach were calculated from the HEC-2 output using the SAM hydraulic design package. The study reaches are listed in Table 1.

Case Study 7B Table 1
Study Reaches

Reach No.	Navigation River Miles
1	17.4–19.0
2	35.5–37.2
3	38.8–41.5
4	60.2–66.8
5	87.1–87.5

a. Bed Material. Bed material samples used to define the existing bed gradation were collected from the Apalachicola River in 1987 and 1991. The bed material is coarsest at the upstream end of the study, downstream from Jim Woodruff Dam, and generally becomes finer downstream towards Apalachicola Bay.

(1) The median grain size was 0.40 mm in the reach between NM 3.6 and the lower end of the Chipola Cutoff (Reach 1 and 2, Table 1); 0.45 mm in the reach between the upper end of the Chipola Cutoff and Blountstown (Reach 3 and 4, Table 1); and 0.80 mm in the reach between Blountstown and Chattahoochee (Reach 5, Table 1). In the two lower reaches, only about 5% of the bed was gravel, whereas in the upper reach about 30% of the bed was gravel.

(2) For the sediment budget analysis, an average of all bed samples was used to obtain an average gradation for the entire reach. This simplification is deemed appropriate for the comparative evaluation approach used in the sediment budget analysis. The median grain size was 0.48 mm.

b. Transport Function Selection. Available measured suspended-sediment data did not include sufficient particle size distributions, rendering the data inadequate for analysis of bed-material transport. Size distribution of suspended load would allow comparison to bed gradation to inform on the portion of suspended load that is composed of bed material.

(1) Sediment transport function applicability is defined by material size, Chapter 5 of this manual. Without adequate information on transport size, the applicability of sediment transport functions could not be demonstrated.

(2) In the absence of data, transport functions developed by Toffaleti and Yang, which have been demonstrated to be reliable for the lower reaches of the nearby Alabama River, were used for the Apalachicola River. In comparisons with measured data from the Alabama River, the Toffaleti function was found to slightly over-predict sediment transport rates, and the Yang function was found to slightly under-predict sediment transport rates. These functions provide a high and low estimate of sediment transport capacity for the sediment budget analysis.

c. Hydrology. The flow duration curve was developed from 20 years of data (1974 through 1993) from the Apalachicola River near Blountstown gage. Data from previous years were excluded because of dam construction in the watershed. West Point Lake, which became operative in 1974, is the last reservoir to significantly influence flow duration curves. Data was obtained from USGS published records. The maximum flow of record (between 1974 and 1993), which was 185,000 cfs, was added to the flow duration curve for the sediment transport capacity calculation and assigned a 0% exceedance. As a simplification appropriate for this study, the same flow duration curve was used for all three minimum flow channel alternatives.

d. Sediment Transport Capacity Results. Average annual sediment transport capacity was calculated using the SAMwin hydraulic design package for each channel modification plan for each of the five reaches. Calculated sediment transport capacity was determined using both the Toffaleti and Yang functions. The results are presented in Table 2.

Case Study 7B Table 2
Comparison of Calculated Annual Sediment Transport Capacity

		Navigation Channel Geometry				
		9,300 cfs Existing	**11,000 cfs**		**13,000 cfs**	
	Reach NM	**1,000 yds³/yr**	**1,000 yds³/yr**	**Percent of Capacity in 9,300 cfs Channel**	**1,000 yds³/yr**	**Percent of Capacity in 9,300 cfs Channel**
Toffaleti Function	17.4–19.0	691	723	104.6	748	108.4
	35.5–37.2	388	415	107.2	439	113.2
	38.8–41.5	543	545	100.4	545	100.5
	60.2–66.8	577	588	101.8	590	102.2
	87.1–87.5	629	661	105.0	695	110.5
Yang Function	17.4–19.0	446	471	105.5	495	110.9
	35.5–37.2	306	342	111.5	371	121.1
	38.8–41.5	545	546	100.3	548	100.6
	60.2–66.8	443	455	102.6	457	103.0
	87.1–87.5	359	379	105.7	403	112.3

(1) Dredging Implications from Sediment Transport Results. More dredging is required to maintain the 9-foot depth navigation channel in the lower minimum discharge channels. Lower minimum discharges require deeper dredged channels across crossings.

(a) Therefore, at high and normal discharges, when water surface elevation differences in the three channel alternatives are negligible, the deeper dredged channels have a larger channel cross-sectional area and a lower channel velocity.

(b) Therefore, it is expected that sediment transport capacity is smallest with the 9,300 cfs minimum flow channel and greatest with the 13,000 cfs minimum flow channel.

(c) The sediment budget analysis supports this anticipated trend in all reaches.

(d) The calculated increase in sediment transport capacity above the 9,300 cfs minimum flow alternative varied between 0.3% and 11.5% for the 11,000 cfs minimum flow channel, and between 0.5% and 21.1% for the 13,000 cfs channel. The average calculated increase in sediment transport capacity for the 11,000 cfs channel was 4.5%. The average calculated increase in sediment transport capacity for the 13,000 cfs channel was 8.3%.

(2) Variability in Transport Capacity. The variability in the calculated sediment transport capacity is attributed to the magnitude of dredging requirements in the individual reaches, and to other hydraulic factors that act to offset the effect of increased channel size. These include: (a) increased percentage of flow in the channel due to increased dredging depths and thus, increased sediment transport potential; and (b) increased slope due to lower downstream water surface elevations created by downstream channel deepening, and thus, increased sediment transport potential.

(3) Impact of Transport Function. Sediment transport quantities were higher when the Toffaleti function was used, but percentage differences were greater when the Yang function was used. This demonstrates the sensitivity of the calculation to the sediment transport function selection.

(4) Transport by Reach. The calculated sediment transport capacities are significantly different for the different reaches. This demonstrates the variability of sediment transport capacity through the 100-mile study reach. Aggradation or degradation in specific reaches of the river depends not only on localized hydraulic and sediment characteristics, but also on upstream and downstream conditions.

6. Conclusions.

Dredging activities at one location in a river system affect hydraulic conditions and sediment transport downstream from the dredging location, and, under some conditions, even upstream.

a. The sediment budget analysis indicated that differences in the relative sediment transport capacities for the various minimum low-flow channel alternatives on the Apalachicola River are small.

b. Minimum low-flow channels of 11,000 and 13,000 cfs were compared to the existing 9,300 cfs minimum low- flow channel. An average increase in sediment transport capacity of 4.5% was calculated for the 11,000 cfs channel. The average calculated increase in sediment transport capacity for the 13,000 cfs channel was 8.3%. It may be inferred that relative differences in sediment transport capacity can be used to assess relative differences in rates of deposition.

7. Recommendations.

Recommendations for future evaluations were developed based on study results as follows:

a. Sediment Budget Limitations. The sediment budget approach neglects system effects. A more detailed study that considers sediment continuity effects is recommended for the next level of planning study. This is accomplished using a detailed sedimentation model that allows for treatment of looped systems such as the Apalachicola River.

b. General Sediment Model. For the next level of study, a generalized sedimentation model such as HEC-RAS of the Apalachicola River between Apalachicola Bay and Jim Woodruff Lock and Dam are recommended to predict the effects of dredging at specific sites and the effects of any channel improvement structures on dredging. The model should include a refinement that includes the longitudinal variation in bed material gradation observed from the measured data.

c. Suspended-Sediment Measurement and Transport Functions. It is recommended that suspended sediment measurements be collected on the Apalachicola River at higher flows to confirm the sediment transport equations used in the study. The collected suspended-sediment samples should be analyzed to determine the particle size distribution of the measured load.

d. Study Hydrology. In this study, the flow duration curves were assumed to be the same for all minimum flow channels. In future, more rigorous studies, the hydrology should account for different annual hydrographs for each minimum flow channel alternative.

e. Flow Distribution. Flow distribution into the bypasses and overbanks is critical to sediment transport processes in the Apalachicola River. In this study, flow distribution assumptions were approximate and did not consider any differences in distribution for the alternative minimum flow channels.

(1) The next level of planning study should include a more detailed definition of flow distribution. This is best accomplished with a field data collection effort which establishes coincident discharges for all major tributaries and distributaries of the Apalachicola main channel.

(2) In addition, the geometric model of Apalachicola River should be extended to include overbank areas and the major cutoff channels.

Case Study 7C
Sediment Impact Assessments of In-Stream Gravel Mining Proposal for Animas River, La Plata County, Colorado

1. Case Study.

This case study provides content condensed from an analysis performed by the Sacramento District to evaluate potential sediment impacts from gravel mining. The analysis was performed to assist with evaluation of a permit application. The partial content demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Study Background.

The applicant is in the process of obtaining a new permit for conducting gravel mining from the Bar-D gravel pit in the Animas River at a location approximately 9.5 miles north of Durango, Colorado. The permit applicant has conducted mining operations at the site since the mid-1970s.

a. The most hydraulically significant aspect of the permit application is the selection of a “stable” profile at which gravel extraction can occur when bed elevations are higher, while providing a sufficient amount of material for harvesting to make the business profitable. Additionally, the proposed thalweg profile or “take line” should be selected so that the gravel mining operations do not negatively impact upstream and downstream properties or cause the river system to become less stable.

b. An additional concern is the impact of the selected thalweg profile on potential flooding of the Animas River on the development located upstream, along, and downstream of the mining operations. The FEMA floodplain mapping was developed in 1981. In 1990, two reaches were remapped, resulting in a lower base flood elevation as a direct result of gravel mining operations along the Animas River. The mining lowered the channel invert, increased the cross-sectional area, and increased the capacity of the channel.

c. The applicant has based selection of a “stable” profile on a second-order polynomial curve fit (also called Quadratic Equation Profile, or QEP) of the thalweg profile as surveyed in 2003/04. No additional physics-based analysis that would normally include hydraulic, sedimentation, and possibly geomorphic studies has been provided. There appears to be no consideration of the link between the QEP and the fluvial processes of the river.

d. It should be noted that the term “stable” is not the best word to describe the channel invert profile or take line through the gravel pit. A stable profile suggests a zone of the river in which there was neither significant erosion nor deposition. That is, the net cross-sectional area of the channel remains relatively constant through time, although there could be significant changes in the channel cross-sectional shape and/or channel alignment and planform. This situation is sometimes referred to as dynamic stability. Given that the applicant desires to maximize the deposition volume in the pit, it follows that the proposed river profile not be stable, but be depositional.

3. References and Information Sources.

This assessment was based on existing data. Both existing and historical data were collected from studies and reports. The information included sediment gradation data from two sources. Streamflow records were obtained via the Internet from the USGS. Aerial photography was obtained from a combination of USGS black-and-white digital orthophoto quadrangles (DOQs) via Terrain Navigator Pro and color photography dated 2003 and 2005 from the La Plata County GIS website. A previous USACE report titled “Flood Hazard Information, Animas River and Hermosa Creek, Hermosa, Colorado” (dated October 1977) played a significant role in the assessment. Recent cross-sectional survey data for the river and CAD-based drawings were used for cursory hydraulic and sedimentation analyses and to establish river stationing.

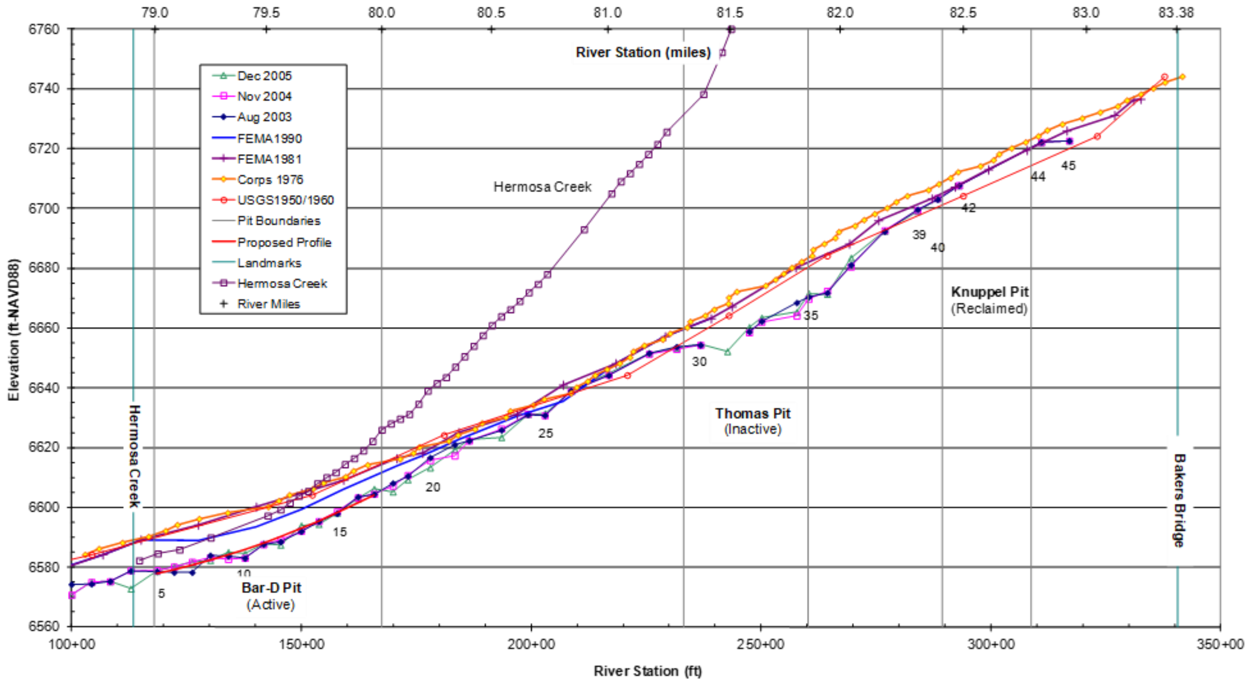
4. Review Existing Data.

This assessment began with a review of the existing data. A cursory assessment was made in which the sediment transport rates and historic erosion and deposition were estimated. The base condition and with-proposed project transport rates were then compared, from upstream to downstream, to assess the likely impacts. The computed data was then used to assess potential additional proposals which could produce an acceptable condition in the Animas River.

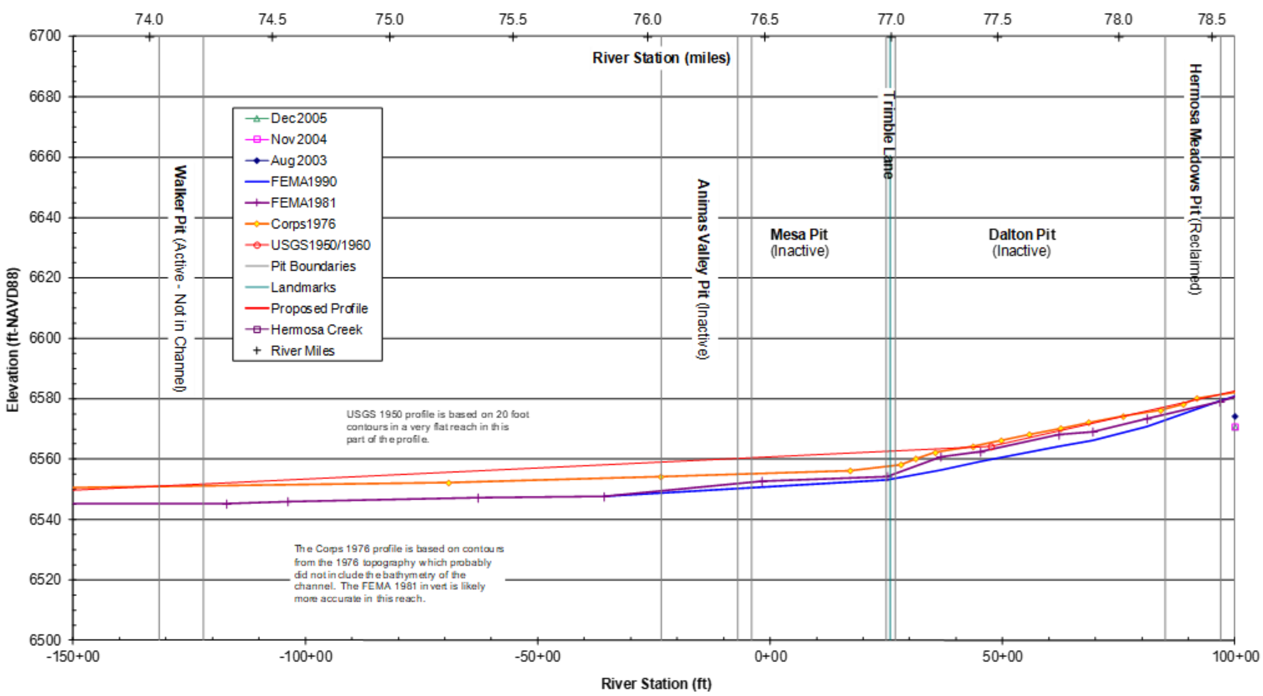
a. Sediment transport rates and average annual yields were estimated using the SAMwin program. The channel geometry was based on selected cross sections from the applicant’s surveyed cross sections. Sediment gradation data was based on available gradation information and flow duration curves were based on local streamgage data from the USGS.

b. Channel profiles of the Animas River are shown on Figures 1 and 2. The earliest profile was taken from the 1:24,000 USGS Hermosa and Durango West quadrangle maps. The topography for these maps consists of 40-foot contours supplemented by 20-foot contours. The topographic data was from aerial photos dated 1950, 1956, and 1960 for the Hermosa quadrangle, and from 1960 for the Durango West quadrangle. Channel profiles were also obtained from the USACE 1977 flood hazard report (2-foot contour mapping dated June 1976), the 1981 FEMA floodplain information report, and two reports by Goff Engineering and Surveying which were the basis of the modifications to the FEMA floodplain elevations in 1990.

c. Additionally, detailed cross-section geometry was obtained from the applicant at 44 cross sections. The surveyed cross sections are located along the river from a point about 1,900 feet downstream of the Bar-D Pit boundary to a point about 15,000 upstream of the Bar-D Pit boundary. The cross sections have apparently been surveyed several times in the recent past. Cross-section data included station and elevation data from January 2003, August 2003, March 2004, August 2004, September 2005, and December 2005. Survey data for each survey period was not available for every cross section.



Case Study 7C Figure 1. Animas River, channel profiles – upstream reach



Case Study 7C Figure 2. Animas River, channel profiles – downstream reach

d. Various studies have used different vertical datums. The USACE flood hazard report and the FEMA studies used the National Geodetic Vertical Datum (NGVD) datum of 1929. The more recent cross-section surveys use the NAVD of 1988. Datum differences were computed using the USACE "Corpscon for Windows" program, version 5.11.08, and 4.18 feet was added to elevations in NGVD 1929 to convert them to NAVD 1988.

e. On Figures 1 and 2, the channel profiles illustrate that the Animas River has a significant change in grade in the vicinity of the Bar-D Pit and the Hermosa Creek confluence. The nominal channel slope changes from about 0.0078 upstream to about 0.0003 downstream. Furthermore, the nominal slope of Hermosa Creek upstream of the confluence is about 0.0169. Thus, both the Animas River and Hermosa Creek transition from relatively high energy systems to a substantially lower energy system.

f. It is reasonable to expect that the river system tends to deposit significant amounts of sediment in the transition zone, since the downstream river could not efficiently transport the material delivered from upstream. In the study area, the Animas River confirms this supposition in that the river has extensive sediment deposits, and the fact that this reach of river has been the site of several gravel mining operations over time. Currently, all of the gravel mining pits are inactive except the Bar-D Pit.

g. Of special note are the wide variations in the channel profiles throughout the study area. In most cases, the current invert elevations are lower than the historic elevations, indicating that the channel has experienced wholesale degradation. Specific points of interest are listed below, from downstream to upstream.

(1) Downstream from the Bar-D Pit, channel thalweg profiles indicate degradation up to 4.2 feet between 1981 and 1990. Given that up to four gravel mining operations were located in the reach, this variation is not surprising. It is also conceivable that upstream mining also reduced the sediment inflow to this reach and may have contributed to the channel lowering.

(2) In the Bar-D Pit reach, the profiles show between 9.5 and 16 feet of lowering between 1976 and 2005. The lowering is most likely a direct result of the mining activities in the Bar-D Pit. Lowering of the channel immediately downstream of the Bar-D Pit ranges from about 9.8 to 16 feet, and is likely due to the decreased sediment transport out of the Bar-D Pit, coupled with the mining activities and channel lowering caused by other currently inactive downstream mining operations. As an aside, a comparison of the elevation data for the downstream portion of Hermosa Creek indicates that the creek has been lowered by up to 3 feet, between 1977 and 2005, which is likely a direct response to the base lowering of the Animas River.

(3) Upstream from the Bar-D Pit, up to the Thomas Pit, there has been a lowering of up to 9.8 feet in the downstream half of the reach since 1976. The upper half of this reach appears to be relatively unchanged when comparing the 1976 and 2005 profiles. The bulk of channel lowering in the lower end of the reach is most likely a result of headcutting from the lowered elevations in the Bar-D Pit.

(4) Farther upstream is a reach containing currently inactive gravel pits. One of these pits appears to have been refilled (Knuppel Pit), but the Thomas Pit shows up to 14 feet of channel lowering since 1981. Given the flat slope on the downstream end and the over-steepened slope of the upstream end of this sub-reach, it is expected that this sub-reach is unstable with deposition near the downstream end, and erosion, and possibly headcutting, on the upstream end. Furthermore, this condition will likely exist until the lowered channel fills in and the channel slope approximates that of the 1981 profile.

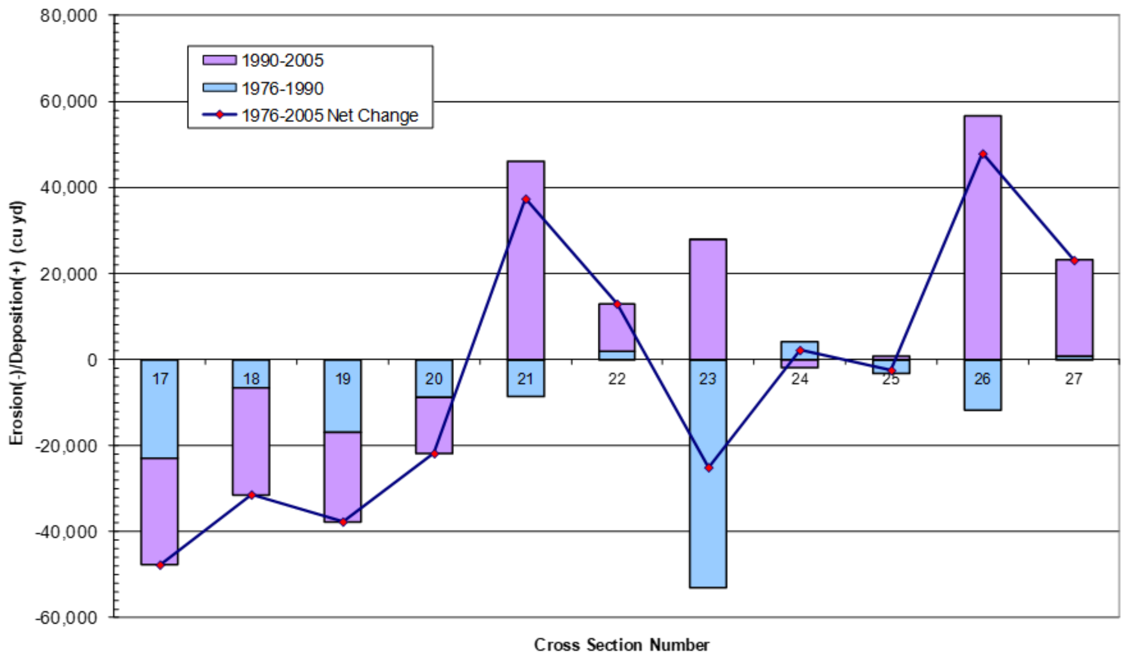
5. Determination of Erosion and Deposition Quantities.

Given the recent survey data for the 44 cross sections, a cursory estimate could be made of the volume of the erosion for the reach located between the Bar-D Pit and the Thomas Pit upstream.

a. Specifically, the volume of erosion was estimated by using the change in the cross-sectional area of the channel multiplied by the average length to compute volume (area x length). An average length was computed as the distance between the cross-section midpoints where each midpoint is located halfway between the adjacent cross sections. These results are shown on Figure 3 and indicate that consistent with the invert profiles of Figures 1 and 2, sections 18 through 20 have experienced significant erosion. Specific erosion volumes per section are listed in Table 1 below:

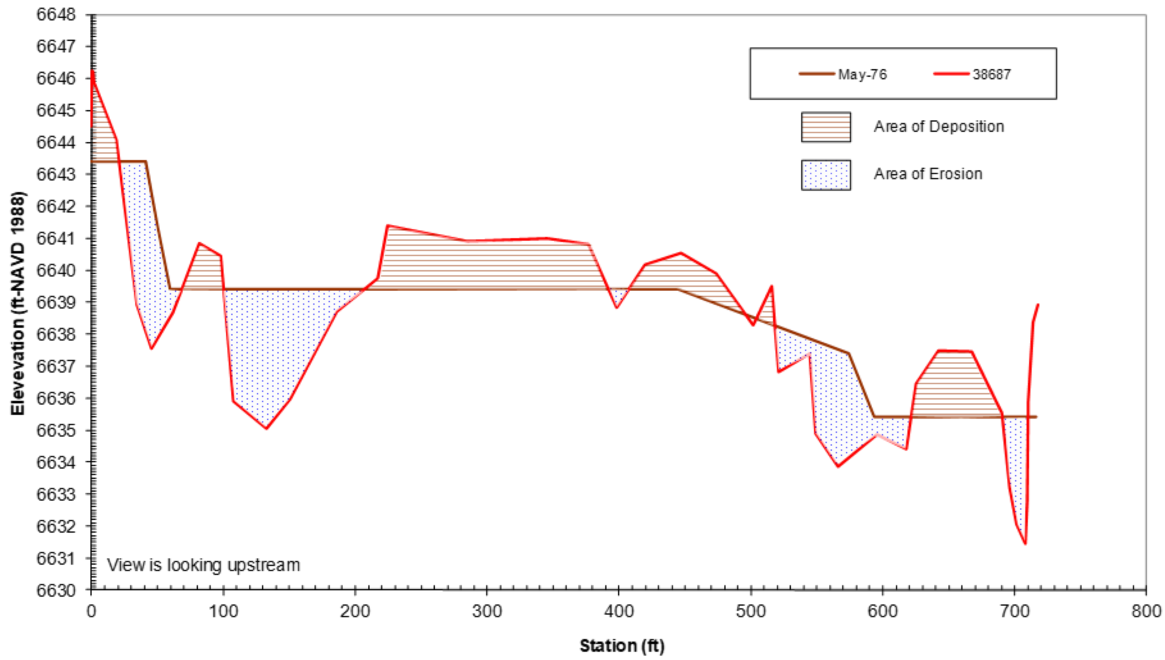
Case Study 7C Table 1
Volume of Erosion (yd^3)

Section	1976–1990	1990–2005	1976–2005
18	6,580	24,800	31,400
19	17,000	20,700	37,700
20	8,840	13,000	21,800



Case Study 7C Figure 3. Animas River, volumes of erosion and deposition at specific cross sections

b. This data illustrates the order of magnitude for the volume of material eroded just upstream of the Bar-D Pit. In addition, it is interesting to note that during the 1976–1990 period, the overall reach was primarily in an erosive mode. However, during the 1990–2005 period, sections 21 through 27 was in a net depositional mode. Comparison of the cross sections 24 and 25 shows that the thalweg elevations have been lowered but the channel mid-bars have aggraded. A sample of this trend is shown for section 25 on Figure 4.



Case Study 7C Figure 4. Animas River, aggradation and degradation at cross section 25

6. Determination of Flow Duration Data.

The distribution of flow in the Animas River represents the engine that drives sediment delivery to the subject reach. In the case of the Animas River, average daily flow values are sufficient for estimating the potential volume of sediment delivered on an average annual basis since about 90% of the volume of sediment is delivered to the study reach by flows less than about 5,000 cfs. In addition, although large flood flows have large sediment transport rates, the duration of these flood flows is minuscule when compared to the flows which occur day in and day out.

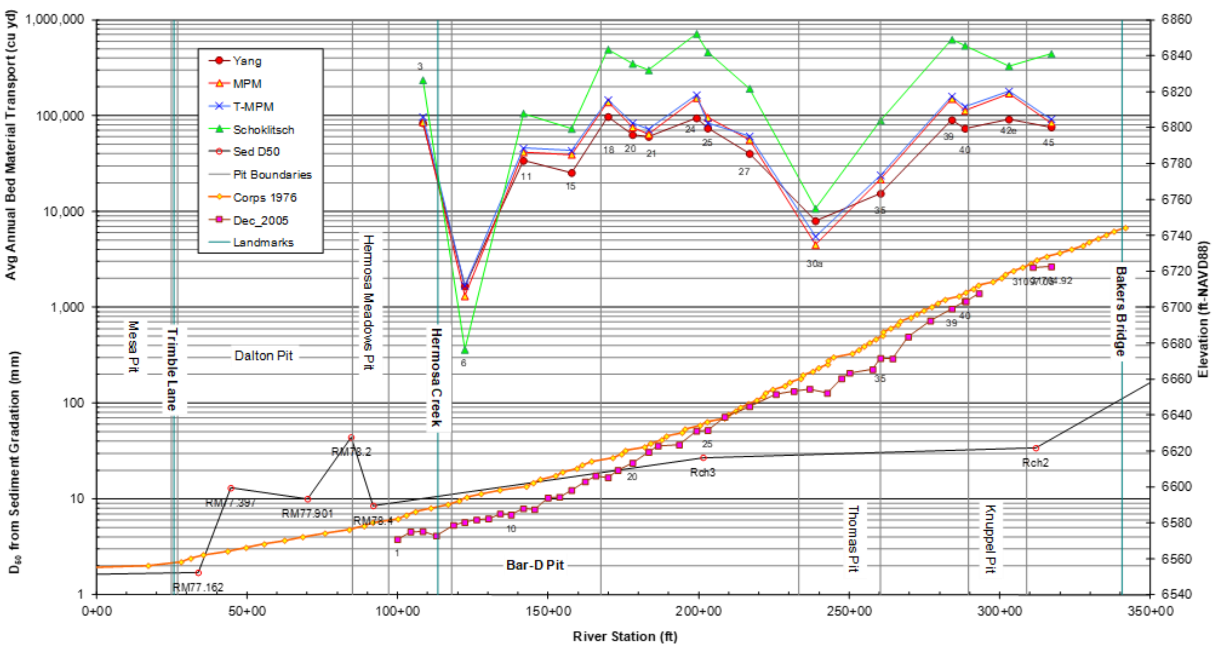
a. Daily flows were obtained from the USGS website (http://www.usgs.gov) for the Animas River at Durango (gage no. 9361500) and for Hermosa Creek at Hermosa (gage no. 9361000) for the period of record for each gage. At the Durango gage, the average annual water volume is about 585,000 acre-feet, which is similar to the 2004 water year, which had about 508,000 acre-feet. Conversely, 2005 was a high year with a volume of about 837,000 acre-feet.

b. Given that most of the study reach is the Animas River upstream of the Hermosa Creek confluence, and that the only local streamgage for the Animas River is located at Durango, which is downstream of the Hermosa Creek confluence, there was a need to generate flow data for the reach of the Animas River upstream of Hermosa Creek. In general, it was assumed that the flow upstream of Hermosa Creek was approximately equal to the flow at Durango minus the flow from Hermosa Creek.

c. Using a spreadsheet, the daily flows at the Durango gage were plotted against the "computed" flows for the Animas River upstream of Hermosa Creek for the period of record where the Hermosa Creek gage had data. This approach resulted in a linear curve fit to depict the relationship between flow at the Durango gage and flow on the Animas River upstream of Hermosa Creek. By a simple subtraction computation, the missing flows from the Hermosa Creek period of record were then estimated.

7. Sediment Gradations.

Sediment gradation data are a critical part of any sediment transport computation. Figure 5 shows the variation in the median grain size, d_{50}, versus the river station. As expected, the material transitions from coarse to fine from upstream to downstream. At the upstream end of the study reach, the material is in the cobble and boulder range, in the vicinity of the Bar-D Pit, the material consists of gravel and cobbles, and downstream of Trimble Lane, the material consists mostly of sand-sized particles.



Case Study 7C Figure 5. Animas River, average annual bed-material transport, channel profiles, and sediment d_{50}

a. Representative sediment gradations were developed from available data for the reaches upstream and downstream from Hermosa Creek. The representative gradations were adjusted by removing the finest 10% of the sample and normalizing the remainder of the data to represent 100% of the sample. This adjustment is performed to minimize errors in the transport computations due to the presence of material which is more likely part of the wash or suspended load and not representative of the bed-material load. Sediment gradation data was not available for Hermosa Creek. Therefore, it was assumed that the gradation is similar to the gradation in the Animas River near the confluence.

b. A more thorough sediment sampling program should be undertaken to better define the sediment gradations occurring in the Animas River and Hermosa Creek prior to a more detailed sediment investigation.

8. Sediment Transport.

To quantify the sediment transport processes in the Animas River, a cursory sediment transport and sediment yield analysis was conducted in support of this assessment. The analysis was conducted using the SAMwin Hydraulic Design Package.

a. In general, the first step in a SAMwin analysis is to generate the hydraulic parameters for a given cross-section geometry for a range of flows using the assumption of normal depth. The second step computes the sediment discharge rating curve for the discharges corresponding to the flows in step one. Finally, the sediment transport-based yield is computed by either integrating a flow duration curve or a hydrograph with the sediment discharge rating curve. The SAMwin package has several limitations and must be used carefully to generate meaningful results. It is assumed that a more detailed analysis using a sediment routing model (HEC-6, etc.) normally follows a SAMwin analysis to perform a more thorough analysis.

b. In the case of this assessment, one extra step was conducted during the SAMwin analysis to more accurately predict the slope of the energy grade line as opposed to using the invert slope. This step included using the 44 surveyed cross sections to create an HEC-RAS model. The HEC-RAS model was then enhanced to include additional interpolated cross sections, areas of ineffective flow, and correct channel reach lengths to account for the sinuosity of the main channel.

c. Given the uncertainty in predicting sediment transport rates, four sediment transport functions were used in the SAMwin computations. These functions included Yang's function, the Meyer-Peter-Muller (MPM) function, a combination of the Toffaleti and Meyer-Peter-Muller (T-MPM) functions, and the Schoklitsch function. These transport functions were selected because they tend to work well with channels that have a coarse bed material. The benefit of using multiple functions is that of conducting sensitivity tests, as well as acknowledging that the "true" answer is probably unknown and is better represented by a range of results.

d. A series of cross sections were then selected at which the sediment transport and yield computations were performed. These locations are shown on Figure 5, along the lines which depict the average annual bed material transport (sediment yield). Also shown on this attachment is the d_{50} of the bed material and the river profile for the 1976 and current (2005) conditions. The information on Figure 5 is presented on one plot to illustrate the relationship between the sediment transport, channel slope, and sediment size.

e. The average annual bed material transport results shown on Figure 5 are for the existing or current conditions. In general, sections 40 through 45 occur where the river has a single channel which may be considered the sediment delivery reach. That is, this reach appears to be capable of transporting the material from the upstream sediment supply reach without inducing significant deposition or aggradation.

f. The average annual transport volumes shown here probably indicate the long-term yield of bed material from the upper watershed and are in the range of 76,000 to 92,000 cu yd per year. This range excludes the results of the Schoklitsch function. During the assessment, it was observed that the results predicted using the Schoklitsch function tended to be unrealistically large and effectively acted as outlier data when compared to results for the other functions.

g. Sections 30a and 35 appear to be an area where the transport is reduced tenfold due to the flattened slope of the energy grade line. This location corresponds to the location of the inactive Thomas Pit and illustrates that this stretch of the river is acting as a sediment sink. This condition will probably exist until the river regains a profile similar to the 1981 profile shown in Figure 1.

h. Based on a rough calculation, it appears that approximately 275,000 cu yd of material may be required to backfill the river in the Thomas Pit area. Using the annual transport volumes for cross section 45 minus the corresponding volumes for section 30a, it would indicate that a period of 3.2 to 4.0 years, on average, may be required to backfill the Thomas Pit reach. Of course, this timeframe may be longer or shorter, depending on the actual runoff as well as the availability of sediment from the watershed.

i. In the sub-reach between sections 18 and 27, the annual sediment transport volumes appear to approximately match those of the sections above the Thomas Pit. Downstream of section 18 where the river enters the Bar-D Pit, the transport capacity decreases from approximately 100,000 down to 1,500 cu yd per year at section 6. This reduction in capacity shows that in the existing configuration (based on the December 2005 survey data), the Bar-D Pit traps almost the entire bed-material load of the river.

j. Downstream of the Bar-D Pit and the Hermosa Creek confluence, the sediment transport capacity of the river appears to equal that of the upstream delivery reaches. There are likely two reasons for the increase in transport capacity: (1) additional flow contributed from Hermosa Creek, and (2) the gradation of the material in this reach is much finer than that in the river upstream of the confluence. This result points to the need for a more detailed study that includes determination of sediment transport and reach continuity by size class.

k. To assess the sediment load from Hermosa Creek, a SAMwin analysis was conducted for one cross section, which was selected to be located sufficiently upstream to not be affected by channel-lowering of the Animas River. The 1976 topography was used for the channel geometry, and the flow duration curve for Hermosa Creek was used for the yield computations. The bed gradation in Hermosa Creek was assumed to be similar to the measured bed gradation in the Animas River near the confluence.

l. The results of this analysis, shown in Table 2, indicate a fairly wide range of potential sediment transport capacities for Hermosa Creek, depending on which Animas River bed gradation was assumed. The computed values, tabulated below, range from a low value of about 970 cu yd per year to a high of about 183,000 cu yd per year. Thus, while Hermosa Creek likely supplies a large sediment input to the Animas River sediment load during some periods, other supply sources are also important. A total sediment budget is required to identify all sediment inputs.

Case Study 7C Table 2
Average Annual Bed-Material Transport Capacity of Hermosa Creek (yd^3/yr)

Transport Function	Yang	MPM	T-MPM
Gradation RM 78.2	970	51,000	52,000
Gradation RM 78.4	168,000	143,000	183,000

9. Evaluation of Alternatives.

To assess the impacts of the proposed mining configuration, including the proposed QEP take line or invert profile, the HEC-RAS model was modified to include the proposed geometry.

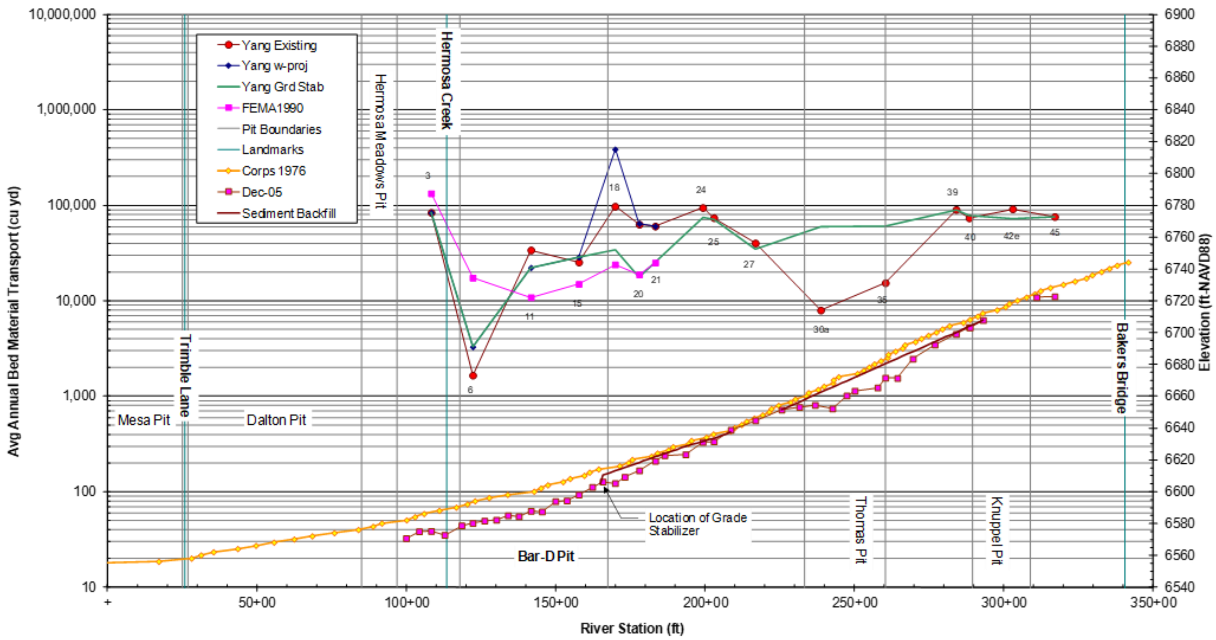
a. The most significant aspect of these profiles was the over-steepened slope of the water surfaces as the flow enters the Bar-D Pit excavation. This effect is due to a rapid expansion in the channel width from the irregular channel at section 18 to the wide, trapezoidal configuration of the extraction limits of section 17. For the existing conditions at section 18, the energy slopes ranged from 0.0083 to 0.0116, depending on the discharge, and increased to range between 0.0184 and 0.0455 for the proposed project conditions.

b. Evaluation of QEP Take Line. Average annual sediment transport capacities and average bed material yields corresponding to the proposed project conditions were calculated using the SAMwin package and are shown in Figure 6, along with existing conditions results. For clarity, results for only one sediment transport function are shown. Since the results of the transport capacities are similar for the with-project and existing conditions upstream of section 21, results evaluation should focus on the region between sections 3 and 21. For each of the four transport functions, the trends are similar and the results at specific locations are discussed below, comparing Yang Existing to Yang with-project (computed with the QEP geometry) as shown in Figure 6.

(1) There is an increase in transport capacity at section 6 resulting in potentially more material passing through the Bar-D Pit. This aspect may be an improvement over existing conditions because the sediment transport capacity exceeds the sediment transport supply in the downstream reach and this change may decrease the deficit in the annual sediment volume. Furthermore, a review of the detailed SAMwin computations indicates that only sand and small gravel-sized material would move at section 6.

(2) At section 18, there is also a large increase in transport capacity. This result was expected, given the large increases in the slope of the energy grade line. Therefore, the proposed configuration of the pit and associated sediment removal are predicted to exacerbate this

condition and induce additional erosion and incision to upstream reaches of the Animas River. The increased erosion condition will likely exist until the stream system reached a new “stable” profile, probably many years into the future. This profile consists of a deeper incised channel, with possibly more bank erosion as the bank heights are increased by incision to the point where the banks are over-steepened and fail.



Case Study 7C Figure 6. Animas River, with and without project average-annual bed-material transport using Yang’s Function

c. Alternative Concepts for River Stabilization. Given the above conclusion, the main problem with the proposed (and existing) configuration is the over-steepened portion of the river upstream of the Bar-D Pit. Thus, if the profile of the river is stabilized to preclude erosion or headcutting from moving upstream beyond the limits of the pit, an acceptable situation may be attained. Two potential approaches are discussed below and may or may not be acceptable for reasons other than channel stability (such as fishery and/or recreation issues).

(1) Grade Control Structure Concept 1. The first concept is to construct a grade control structure at the upstream end of the Bar-D Pit to prevent further erosion of the upstream river reaches.

(a) The stabilizer will return the upstream river grade to an approximation of the 1990 profile. Backfilling the river upstream of the Bar-D Pit will occur over time prior to transport of significant volumes of sediment to the Bar-D Pit.

(b) Average annual sediment transport capacities and average bed material yields corresponding to the grade control structure project conditions were calculated using the

SAMwin package. Modifications were made by adding grade control and an assumed backfill line upstream of the structure to the project condition. Results are shown in Figure 6 (Yang Grid Stabilizer line) to allow comparison to the existing conditions and proposed QEP take line (Yang with project) condition results that were previously discussed.

(c) The grade control structure concept may allow the applicant's proposed take line. The grade control structure location and the assumed backfill line upstream of the structure is also shown on the secondary elevation axis allowing comparison to USACE 1976 and 5 December invert profiles.

(d) In general, given a backfilled Thomas Pit sub-reach and a backfilled area immediately upstream of the Bar-D Pit, the sediment transport capacity appears to smoothly transition from the sediment delivery reach to the Bar-D Pit. Of significance is a decrease in the capacity for sections 18 to 24/25, where erosion and incision are currently occurring. In the pit, the transport capacity is a bit more than existing conditions. However, given the load from the sediment delivery reach, the pit will still trap a significant volume of the inflowing material (for example, about 95%). Also, of note is increased transport capacity through the Thomas Pit area at sections 30a and 35.

(2) Grade Control Structure Concept 2. The second concept for stabilization is to use the 1990 invert profile for the take line instead of the applicant proposed QEP take line. Results are shown in Figure 6 as the FEMA 1990 line.

(a) Although transport is raised at sections 3 and 6 for this alternative, transport is lowered at section 11, 15, and 18. In this area (sections 11 to 18), this approach stabilizes the river upstream of the Bar-D Pit by raising the base elevation of the river via deposition.

(b) If mining were limited to the 1990 invert profile in the pit, then over time, the over-steepened portions of the river will backfill and a channel invert similar to the 1990, and in some cases the 1981, profiles may be established. The 1990 profile was selected as a reasonable elevation to which the river could heal from the recent erosion and incision without inducing flooding beyond the limits of the 1990 base flood elevation.

(c) This concept may require a significant amount of backfilling by river flows prior to the start of mining activities. The sediment transport capacities corresponding to this approach are shown on Figure 6. Similar results to the concept 1 grade control approach, the backfilled Thomas Pit sub-reach (sections 30a and 35) will nicely convey the inflowing load towards the Bar-D Pit.

(d) Erosion and incision will be decreased immediately upstream of the Bar-D Pit at sections 18 to 24/25. In the pit, the transport capacity is a bit less than existing conditions for sections 11 and 15. However, at section 6, the capacity is significantly greater than existing conditions. Using section 11 as the limiting location for sediment transport, the pit could trap greater than 70% of the inflowing material load.

(e) A good example of the variation in the annual sediment discharge can be illustrated using data from recent years. Specifically, the water years 2002, 2004, and 2005 represent years with low, medium, and high runoff volumes, respectively, when compared to the long-term average. The flow duration curve for each year was developed for each of the three years and incorporated into the yield computations for cross section 45 for the existing condition. Table 3 summarizes the results.

Case Study 7C Table 3
Computed Bed-Material Transport at Cross Section 45 (yd^3/yr)

Water Year	Water Yield[7]	Yang	MPM	T-MPM
2002	173,000	11,000	12,700	14,200
2004	508,000	59,000	59,400	66,400
2005	837,000	117,000	139,000	150,000
Annual Average	489,000	75,800	84,300	92,100

d. Qualification of Results. This assessment is supported by a number of hydraulic and sediment transport computations conducted using the HEC-RAS and SAMwin packages. It is very important to understand the limitations of these tools and the data that went into them. Some of the most important items are discussed below.

(1) In general, a typical limitation of this type of assessment is the lack of calibration data for both the hydraulic and the sediment transport analyses. It is usually left to engineering experience and judgment to arrive at reasonable estimates and conclusions.

(2) The applicant-submitted cross-section data provided a fairly detailed geometry source. However, at some sections, the sections did not extend far enough to tie into high ground. Therefore, the hydraulic computations for the large discharges (such as 2,000, 5,000, and 10,000 cfs) at these points may be less accurate. Additionally, section 43 was never surveyed due to access problems. Unfortunately, this left a fairly long reach between sections 42 and 44 in which the geometry was assumed by interpolating between the two sections. This item may have induced some uncertainty in the hydraulic and sediment computations.

(3) The SAMwin package was developed for quick, reconnaissance-level analysis, and requires several assumptions for application. Some of the most obvious limitations include the lack of sediment routing, the inability to deal with armoring, and that the model does not update the geometry through time in response to deposition and erosion. As used in this assessment, these limitations result in cursory estimates of the sediment transport capacities of the river system.

(4) If more detailed information is required, then a sediment modeling effort (such as HEC-6T) should be performed to accurately determine sediment delivery through the river system, the time required for upstream backfilling, grain sorting, the areas where erosion will

continue to occur, and the likelihood of sediment deposits in the near- and long-term periods in the Bar-D Pit reach.

10. Conclusions and Recommendations.

Based on a review of the available permit materials, the proposed QEP take line determined as a second-order polynomial curve fit does not result in a “stable” river profile. Although the profile likely promotes deposition in the Bar-D Pit, the profile would continue to induce erosion and incision of the river bed upstream of the pit boundaries as discussed herein.

a. Based on a review of the river geometry upstream of the pit, it appears that 91,000 cu yd of material has been eroded from cross sections 18 through 20 between 1976 and 2005, with about 59,000 cu yd of the total value occurring between 1990 and 2005. It is likely that additional erosion will continue, given the over-steepened slope of the river between sections 18 and 26.

b. The sediment transport assessment discussed herein indicates that the river is capable of delivering about 76,000 to 92,000 cu yd per year of bed material in the vicinity of cross section 45. Currently, the lowered channel condition resulting from the inactive Thomas Pit limits downstream sediment transport to about 4,500 to 8,000 cu yd per year. As the lowered channel backfills over time, the transport rate may be increased possibly to as much as 60,000 to 128,000 cu yd per year (capable of transporting the material which is delivered from canyon sediment supply reach) in the vicinity of the Thomas Pit.

c. The reach between cross sections 18 and 23 is unstable with depositional and erosive trends changing during the 1976 and 2005 timeframe for sections 21 through 23. Sections 17 through 20 can easily transport more sediment than received from upstream and the net trend is that of erosion. It is imperative that the river be stabilized in this reach to preclude continued erosion, incision, and headcutting to upstream reaches.

d. Downstream of the Bar-D Pit, there is limited geometric data, except for cross sections 1 through 5. Nonetheless, using section 3, the transport capacity of the Animas River immediately downstream of Hermosa Creek is about 84,000 to 97,000 cu yd per year. Additionally, using readily available data, it appears that Hermosa Creek may be capable of supplying between 970 and 183,000 cu yd per year. There is uncertainty in these values, and additional analysis is required to fine tune the values for Hermosa Creek.

e. Whether there is a deficit in sediment supply downstream of the Hermosa Creek confluence is complicated.

(1) Given the sediment sizes and channel slope through and upstream of the Bar-D Pit, it is likely that fine sediments (such as sands and possibly fine gravels) are in suspension and are not included in the bed-material load transport computations discussed herein. The volume of this material is conceivably not limited by transport capacity, but is limited by the ability of the

watershed to deliver the material to the river system. However, downstream of the Hermosa Creek confluence, this fine material will likely constitute the bed material load.

(2) When combined with the load from Hermosa Creek and the bed-material load that is not trapped in the Bar-D Pit, sediment load quantity may be sufficient to satisfy the transport capacity of the river between the Hermosa Creek confluence and Trimble Lane. Downstream of the Trimble Lane bridge, the channel slope is excessively low, which leads to a relatively low bed material transport rate.

(3) Additional study is required to determine if this supposition is correct or if (1) too much sediment is trapped in the Bar-D Pit leading to additional erosion and instability in the reach between Hermosa Creek and Trimble Lane; or (2) conversely, Bar-D Pit trapping is satisfactory and additional transport to the reach downstream of Trimble Lane could lead to aggradation.

f. The sediment impact assessment identified additional data needs for a more detailed sediment study. These include:

(1) Channel cross-section surveys downstream from the Bar-D gravel should be collected to provide insight into the changes in the river since the 1976 and 1990 periods. Additionally, a few sections placed appropriately along Hermosa Creek could improve the estimate of sediment load from the creek.

(2) Additionally, it is understood that section 43 was never surveyed due to access problems. Cross-section data should be obtained somewhere in the vicinity of the proposed section 43 to reduce the uncertainty in the geometry of that reach.

(3) Hydraulic computations would be more accurate if future cross-section surveys extended a sufficient length such that the resulting cross sections would fully "contain" a discharge of at least 10,000 cfs. Some of the existing cross sections did not extend far enough to tie into high ground.

(4) Although the existing sediment gradation data was probably appropriate for the purpose for which it was collected, it was not complete from a sediment transport perspective. A thorough sediment sampling program throughout the Animas River study reach and in Hermosa Creek should be conducted for future sediment analyses. Such a program should consist of large sample sizes for the locations where cobble- and boulder-sized material are present and include running the material through grizzlies and/or sieves. At locations where the material may be in the range of three inches and smaller, sieve analyses may be sufficient. Gradations of both the armor and sub-surface layers should be collected.

(5) Two potential alternative concepts have been proposed which, if incorporated into the with-project conditions in one form or another, should allow continued mining of the riverbed. Either of the concepts will help stabilize the upstream river reaches by providing a stable base

elevation near the upstream boundary of the Bar-D Pit. The two proposals are not all-inclusive, and it is likely that other, possibly more creative, approaches exist to stabilize the Animas River.

Case Study 7D
A Sediment Budget Approach to Stable Channel Mining Levels Using Repeated Bathymetric Surveys

1. Case Study.

This case study provides content condensed from studies performed by the Kansas City District to evaluate the stable level of channel mining using bathymetric surveys. The partial content demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Background.

Commercial sand and gravel mining has been identified as a major cause of bed degradation on many rivers (USACE 1984; Collins and Dune 1989; Kandloff 1993) including the lower Missouri River (USACE 2009, USACE 2011b).

a. Numerical modeling is the standard of practice for establishing allowable mining rates and is necessary to determine the location, magnitude, or timing of impacts and to assess the interplay between potential causes. However, when sufficient bathymetric data are available, a sediment budget approach can be useful for a high-level analysis.

b. This case study documents a sediment-budget approach to determining the stable level of channel mining for the lower 500 miles of the Missouri River based on bed change and mining data from 1994 to 2014. This sediment budget is based on bed change calculated using historic bathymetric surveys, not sediment flux measurements.

3. Physical Principles.

For a river to be stable while being mined, the mining rate must equal the natural aggradation rate (ΔS_{nat}), which is the mass rate of sediment accumulation in the river bed with no mining. The natural aggradation rate is defined as sum of the mass of sediment entering the reach (Ms_{in}) minus the mass of sediment leaving the reach (Ms_{out}).

$$MINE_{stable} = \Delta S_{nat}$$ Case Study 7D Equation 1

$$\Delta S_{nat} = Ms_{in} - Ms_{out}$$ Case Study 7D Equation 2

The actual aggradation rate over any given time period (ΔS_{act}) includes the effects of channel mining ($MINE_{act}$).

$$\Delta S_{act} = Ms_{in} - Ms_{out} - MINE_{act}$$ Case Study 7D Equation 3

a. Assuming that the bed material transport flux into and out of the reach are independent of the channel mining (ignoring geomorphic feedbacks), Equation 3 rearranges to the following:

$$\Delta S_{act} = MINE_{stable} - MINE_{act} \qquad \text{Case Study 7D Equation 4}$$

$$MINE_{stable} = \Delta S_{act} + MINE_{act} \qquad \text{Case Study 7D Equation 5}$$

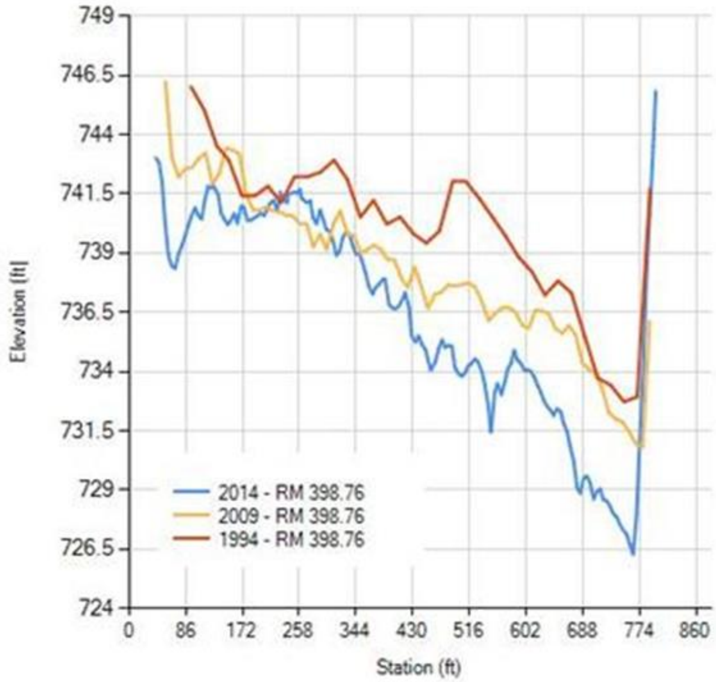
Equation 5 can be annualized to determine the annual stable rate of extraction:

$$MINE_{stable} = (\Delta S_{act} + MINE_{act})/n_{years} \qquad \text{Case Study 7D Equation 6}$$

b. Equation 6 requires the computation of the actual aggradation rate, which can be accomplished using bathymetric surveys from different years. The simplification that the rate of sediment transport is independent of the channel mining is reasonable when the boundaries of the reach in question extend beyond the zone of geomorphic influence of the mining operation, which could be true for an analysis of a very large reach. This type of analysis should not be used to assess the stability of a local reach in the vicinity of channel mining since the mining can significantly affect the local sediment transport rate.

4. Missouri River Stable Extraction Rate.

The actual aggradation rate was computed between 1994 and 2009, and 2009 and 2014 using cross-section data. Figure 1 provides an example of the cross-section data.



Case Study 7D Figure 1. Example of cross-section data (RM 398.76)

a. Table 1 indicates the number and average spacing of cross sections used in the analysis. For consistency, only cross sections present in both the start and the end year (1994 and 2009, or 2009 and 2014) and only the region of channel covered by both years were used in the computation of the aggradation rate. This cross-section analysis was facilitated using the XSViewer tool. This tool developed automate geomorphic analysis using river cross-section data (Shelley and Bailey 2017).

Case Study 7D Table 1
Actual Aggradation Rate for the Missouri River from RM 500 to 0

Time Period	# Cross-Sections Pairs	Average Spacing (ft)	ΔS_{act} (ft^3)[1]	ΔS_{act} (tons)[1]
1994 to 2009	1,812	1,439	-1,175,873,000	-57,048,000
2009 to 2014	5,263	500	-344,869,000	-16,731,000

[1] Negative numbers imply degradation. Values rounded to the nearest thousand.

The Missouri River bed degraded a total of 73,779,000 tons from 1994 to 2014. Over the same time period, 122,114,000 tons were mined from the river bed. By Equation 6:

$$MINE_{stable} = (-73{,}779{,}000\ tons + 122{,}114{,}000\ tons)/20\ years$$
$$MINE_{stable} = 2{,}417{,}000\ tons/year$$

Case Study 7D Equation 6

b. This analysis suggests that the stable extraction rate for the Missouri River from 1994 to 2014 was approximately 2.4 million tons/yr. The actual mining rate was 6.1 million tons/yr, which resulted in average bed degradation of 3.7 million tons/yr. The degradation was not evenly distributed. The most severe degradation in the 1994 to 2009 occurred at the locations of the most intense mining (USACE 2011b), particularly in Kansas City. The most severe degradation in the 2009 to 2014 time period occurred slightly upstream of the previous Kansas City degradation, which was likely the effect of the 2011 flood acting on the antecedent degradation (USACE 2015f).

5. Considerations/Limitations.

Whether or not continued mining at $MINE_{\text{stable}}$ rates would yield channel stability in the future depends on a myriad of factors, including how similar hydrologic and sediment loading in future years are to those used in the determination of $MINE_{\text{stable}}$. Also, this analysis does not address the local or reach effects that would be a function of the location and spatial concentration of the mining. An estimation of magnitude and location of bed change in specific reaches of a river necessitates numerical modeling. Given the uncertainties involved in any approach to determining the effects of channel mining, a robust monitoring program with adaptive management of mining levels is warranted.

6. Conclusion.

This case study provides a high-level analysis of the sustainable level of mining in a large, sand-bed river using volume change computed from repeat bathymetric surveys and historic mining rates.

Case Study 8A
Reservoir Area – Capacity Example Report

1. Case Study.

This case study demonstrates typical content of a reservoir area capacity report that is completed after a sediment survey at a USACE project. Note that this report does not include a detailed evaluation of past surveys or sediment depletion trends. That analysis is typically documented in an aggradation assessment or similar evaluation and reporting effort. The case study content illustrates concepts but is not comprehensive of the appropriate content for all USACE reservoir projects.

2. Project Background.

Salt Creek Dam No. 13 (Twin Lakes Reservoir) is located in Seward County in the southeastern part of the state of Nebraska. It is located on Middle Creek approximately 2 miles northwest of Pleasant Dale, Nebraska. The watershed drains 11 square miles. The rolled earth dam is 2,075 feet long and 58 feet high above the valley floor. Twin Lakes Reservoir, impounded by Salt Creek Dam No. 13, was filled in 1969 and is operated as part of the Salt Creek and Tributaries Flood Control project.

3. Summary.

a. This report contains Area and Capacity (A&C) results from a 2018 analysis using high-density hydrographic soundings obtained in 2018 combined with LiDAR data collected in 2009. This analysis involves a new analysis methodology designed to minimize errors in surface area and volume calculation through increased data resolution. All 2018 values are based on this analysis. A summary of the methodology can be found in Missouri River Basin (MRB) 40: "A Comparison of Survey Technologies and Their Effect on Reservoir Area-Capacity Estimates" (USACE 2021).

b. Capacity Depletion. Reservoir storage capacity changes between survey years are summarized in the tables and plots contained in this report. All elevations referenced are in the NGVD 29 vertical datum (U.S. Survey Feet). Summarizing capacity depletion from 1968 to 2018 at Twin Lakes Reservoir:

(1) Total storage (elev. 1,321.0–1,361.6) decreased from 11,278 acre-feet to 9,709 acre-feet. The reservoir has an average overall 0.27% annual loss rate and 86.1% of original capacity remaining.

(2) Storage loss by pool zone:

(a) Surcharge (elev. 1,355.0–1,361.6) storage decreased from 3,702 acre-feet to 3,464 acre-feet, with 93.6% of original capacity remaining.

(b) Flood control (1,341.0–1,355.0) storage decreased from 5,059 acre-feet to 4,581 acre-feet, with 90.6% of original capacity remaining.

(c) Multi-purpose (1,337.4–1,341.0) storage decreased from 791 acre-feet to 616 acre-feet, with 77.8% of original capacity remaining.

(d) Sediment (1,321.0–1,337.4) storage decreased from 1,727 acre-feet to 1,048 acre-feet, with 60.7% of original capacity remaining.

4. Methodology.

a. Field Survey. Field surveys were performed by an Omaha District survey crew and engineering analysis completed to update the Twin Lakes Reservoir surface area storage capacity tables. Previous survey data was collected in 1968, 1973, 1985 and 1994. The most recent survey of the reservoir was completed in 2018. The details of this survey are shown in Table 1.

Case Study 8A Table 1
Twin Lakes Reservoir – Metadata from 2018 Reservoir Survey

Survey Date(s)	April–June 2018
Surveyed By	USACE, Omaha District, River and Reservoir Engineering Section (Mr. Sam Sediment)
Equipment	Equipment utilized in the collection and processing of the data for this project included: • TRIMBLE R10 GPS Receivers • TRIMBLE TDL 450 H Radios • TRIMBLE Business Center • Hydrolite® Depth Sounder • ODOM CV100 Sonic Depth Sounder • ODOM Digibar Pro Sound Velocity Profiler • HYPACK 2017 Hydrographic Surveying Software • Boat, 14-ft, flat bottom • Boat, 20-ft, Semi-V flat bottom • Boat, 24-ft, Semi-V flat bottom
Horizontal Datum	Nebraska State-Plane Coordinate System, NAD83, Zone 2600
Vertical Datum	NAVD 1988 and converted to NGVD 1929 using USACE Corpscon 6.0.1 software
Units	U.S. survey feet
Accuracy	3rd order horizontal and vertical accuracy per EM 1110-2-1003

b. Capacity Computations. Newer survey methods and increased survey technology have led to different methods by which reservoir area and capacity analysis can be performed. Two different methods, the modified average end area (MAEA) and the GIS method, were used. Both field data collection and area and capacity calculations are tabulated in this report. Twin Lakes area and capacity computations used the MAEA in past efforts. GIS computation capabilities have the capability to use DEMs based on high-density data such as LiDAR and dense hydrographic data to compute reservoir capacity.

(1) MAEA Method. The MAEA computations have been automated using the Omaha Utilities Program (OUP). The MAEA method used traditional surveys along sediment range lines collected with single-beam bathymetry and topographic equipment. These traditional methods were repeated for this analysis. Hydrographic and land survey data were collected for all 14 of the previously established sediment range lines at Twin Lakes Reservoir. These 2018 sediment range survey data were processed using the MAEA method in OUP version 3.0. This method employs a ratio table to increase computed capacity accuracy when compared to a simple average end-area method. The ratio table is based on pre-construction reservoir area contour data.

(2) GIS Computation Using DEM. The GIS method uses the Reservoir Inundation Calculator (RIC), a routine in GIS software, with the Twin Lakes DEM to compute storage capacity by selectable elevation increment. Dense hydrographic surveys were completed on the reservoir in 2018. The hydrographic data were collected on 100-foot cross-section spacing with an average 10-foot spacing between points. The November 2009 LiDAR data, the best available LiDAR, was converted from NAVD 88 vertical datum to NGVD 29 to merge with the dense hydro data collected by the Omaha District. Both data sources were merged to create a single DEM for the reservoir area. The created DEM was processed using a triangle-volume difference algorithm in RIC to determine reservoir capacity.

(3) Method Used. The MAEA method has been shown to generate accurate results. However, this method is susceptible to variation at a single sediment range having an exaggerated influence on results. Since the LiDAR and dense hydrographic surveys used for the 2018 analysis provide detailed topographic data for the entire reservoir, the capacity estimate computed with the GIS method is likely more accurate than that obtained with the MAEA method. All area and capacity tables in this report are a result of this methodology.

5. Sediment Depletion Rate.

The sediment depletion rate is a useful metric that can be computed from capacity changes and indicates how quickly storage capacity is lost. Predictions of future benefit and storage capacity relies on accurate estimates of the sediment depletion rate.

a. Depletion Rate Variation. Depletion rate variation occurs due to several factors.

(1) Natural Variability. Sediment depletion varies between survey periods due to natural factors such as land use changes that affect sediment yield, variability in annual runoff volume,

precipitation intensity, and similar. As previously discussed, the computational methodology also affects results.

(2) Methodology. When switching data collection and analysis methodologies, computational differences can occur due to the change in methodology rather than an actual variation in the depletion rate. Therefore, it is recommended to compute capacity with both methods when a methodology change is made. This provides the ability to examine any shift in capacity that may be associated with the change in methodology.

(3) Accuracy. Data collection methods have evolved over time, and that has affected data accuracy due to changes in vertical point accuracy and point density.

(4) Localized Elevation Change. The MAEA method depletion that relies on average end area is accentuated when the original ratio table has diminished reliability. Depletion for each segment occurs when the bounding sediment range average end area changes. Therefore, minor localized sediment range elevation changes have magnified impacts on capacity. Examples of small-scale construction projects that affect capacity are land use site grading, environmental restoration, and road construction.

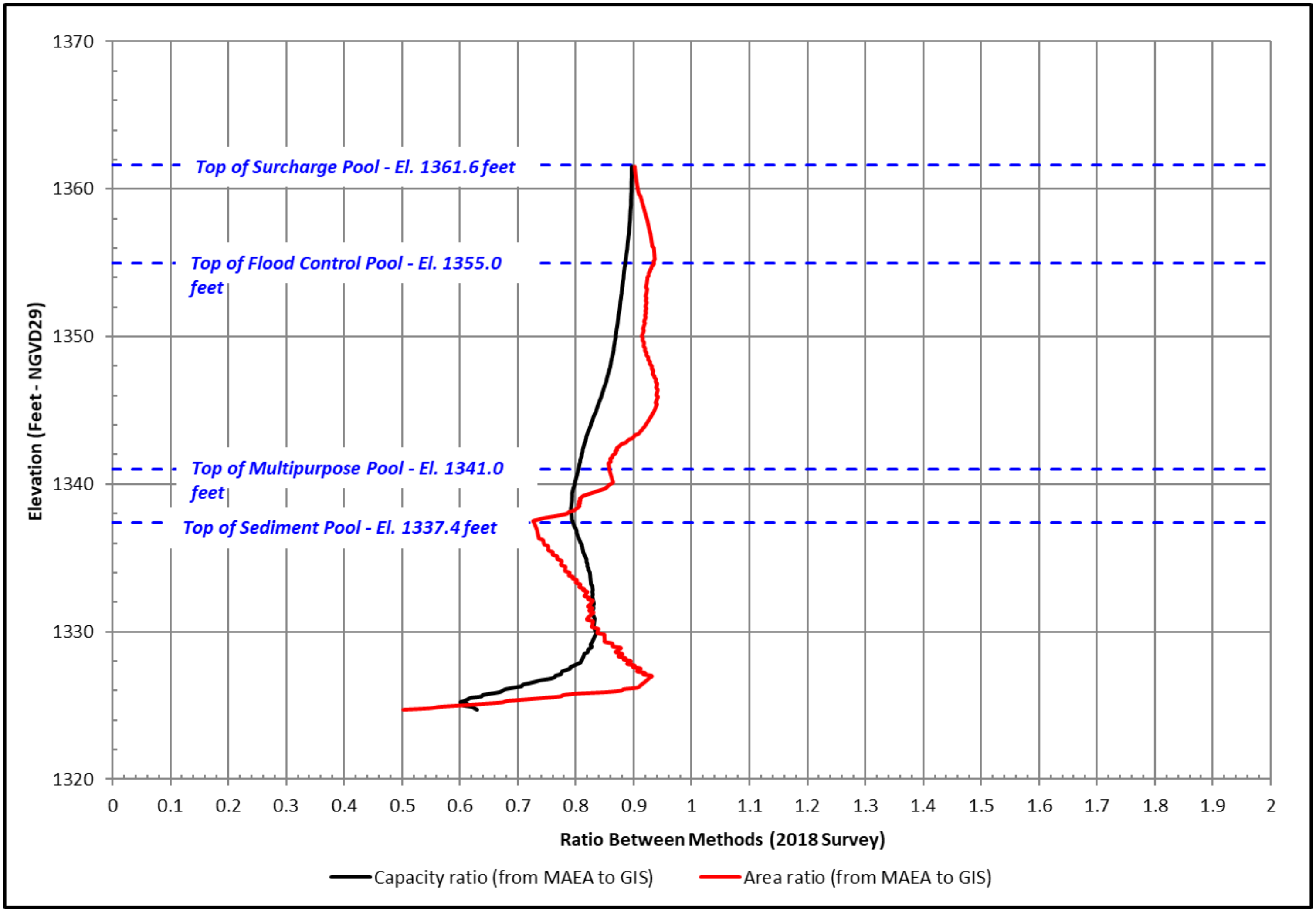
b. Trend Analysis Concerns. As a result of changing methodologies from MAEA to GIS, care must be taken when performing depletion trend analysis. A ratio that varies by elevation was derived to compare the capacity computed with the two methods using 2018 data. The difference between the two methods can be seen in Figure 1. The evaluation at Twin Lakes Reservoir is not constant and varies by elevation. A factor of 1.0 occurs if the capacity from both methods is equal. The variable ratio indicates that converting between the two methods is not possible. For that reason, estimating depletion rates at Twin Lakes Reservoir with the MAEA derived values is not recommended.

6. Twin Lakes Reservoir Summary Tables and Plots.

Table 2 shows a summary of capacity estimates and the variability between the different survey periods. While the GIS values are recommended for use, the MAEA values are provided for comparison and to give context to the historic survey values which used only MAEA.

a. Differences Between Methods. For the 2018 survey year, the GIS generated values are significantly less than those calculated with the MAEA method. Assuming this trend is consistent, it is likely that the historic MAEA values were also much greater than actual. Intermediate term depletion computations (for example, 1973 to 2018) using MAEA values likely under-predict actual depletion rates by an unknown factor. These values should be used only with caution.

b. Area and Capacity Curves and Tables. Figure 2 and Figure 3 show the capacity and area curves (respectively) for the current estimates, compared to several historical calculations. Figure 4 shows the percent change in capacity over time for the storage pools in the reservoir. Tables are provided in 0.1 foot increment for area (Table 3) and capacity (Table 4).

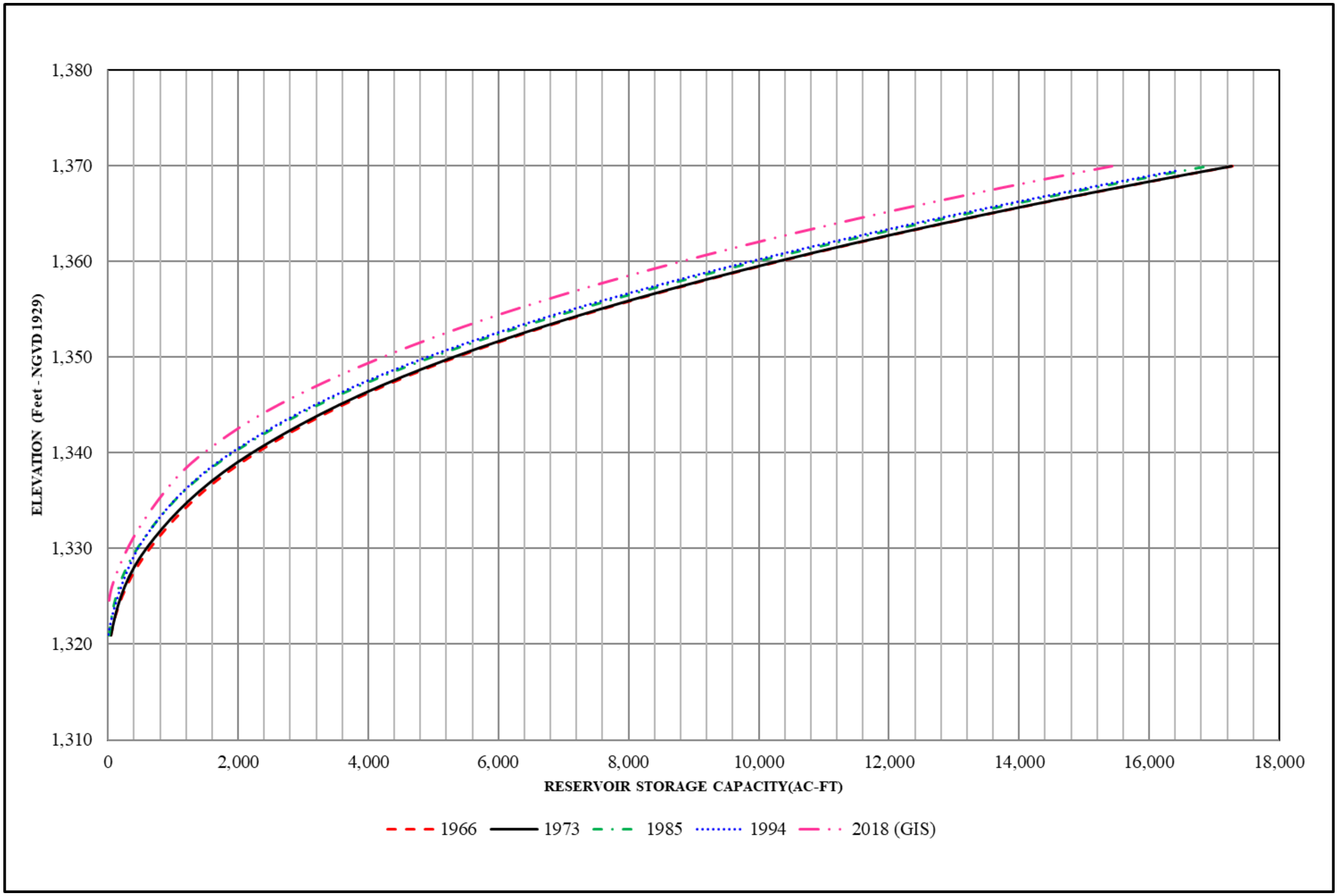


Case Study 8A Figure 1. Twin Lakes Reservoir comparison of capacity and area methods by pool elevation (2018 survey)

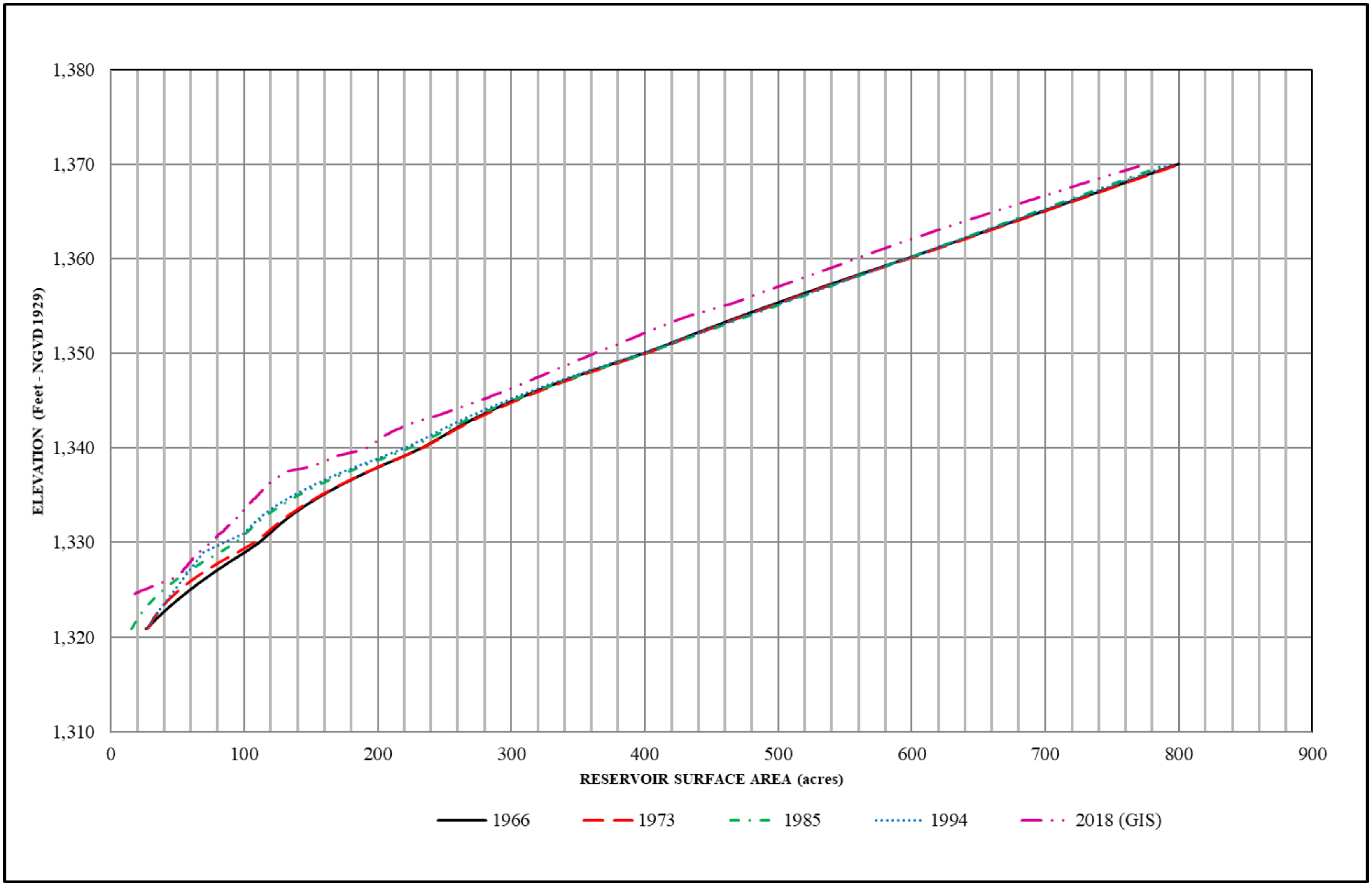
Case Study 8A Table 2
Salt Creek Dam No. 13 – Twin Lakes Reservoir Summary Tables

Summary of Reservoir Storage Capacity by Pool Elevation							
Top of Pool	**Elevation NGVD 29 (feet)**	**1966**	**1973**	**1985**	**1994**	**2018**	
						MAEA	**GIS[1]**
Max. Operating	1361.6	11,278	11,259	10,937	10,851	10,757	9,709
Percent Capacity Gained/Lost (since 1966)		–					-13.9%
Depletion Rate (ac-ft/yr)		–					30
Flood control	1355.0	7,577	7,545	7,220	7,139	6,999	6,245
Percent Capacity Gained/Lost (since 1966)		–					-17.6%
Depletion Rate (ac-ft/yr)		–					26
Multipurpose	1341.0	2,518	2,458	2,155	2,124	2,044	1,664
Percent Capacity Gained/Lost (since 1966)		–					-33.9%
Depletion Rate (ac-ft/yr)		–					16
Sediment	1337.4	1,727	1,666	1,404	1,387	1,300	1,048
Percent Capacity Gained/Lost (since 1966)		–					-39.3%
Depletion Rate (ac-ft/yr)		–					13
Summary of Reservoir Storage Capacity by Storage Zone							
Pool Zone	**Elevation NGVD 29 (feet)**	**1966**	**1973**	**1985**	**1994**	**2018**	
						MAEA	**GIS[1]**
Surcharge	1355.0–1361.6	3,702	3,714	3,717	3,711	3,759	3,464
Percent Capacity Gained/Lost (since 1966)		–					-6.42%
Depletion Rate (ac-ft/yr)		–					5
Flood Control	1341.0–1355.0	5,059	5,086	5,065	5,016	4,955	4,581
Percent Capacity Gained/Lost (since 1966)		–					-9.44%
Depletion Rate (ac-ft/yr)		–					9
Multi-purpose	1337.4–1341.0	791	792	751	736	744	616
Percent Capacity Gained/Lost (since 1966)		–					-22.16%
Depletion Rate (ac-ft/yr)		–					3
Sediment	1321.0–1337.4	1,727	1,666	1,404	1,387	1,300	1,048
Percent Capacity Gained/Lost (since 1966)		–					-39.30%
Depletion Rate (ac-ft/yr)		–					13
Total Storage	**1321.0–1361.6**	11,278	11,259	10,937	10,851	10,757	9,709
Percent Capacity Gained/Lost (since 1966)		–					-13.91%
Depletion Rate (ac-ft/yr)		–					30

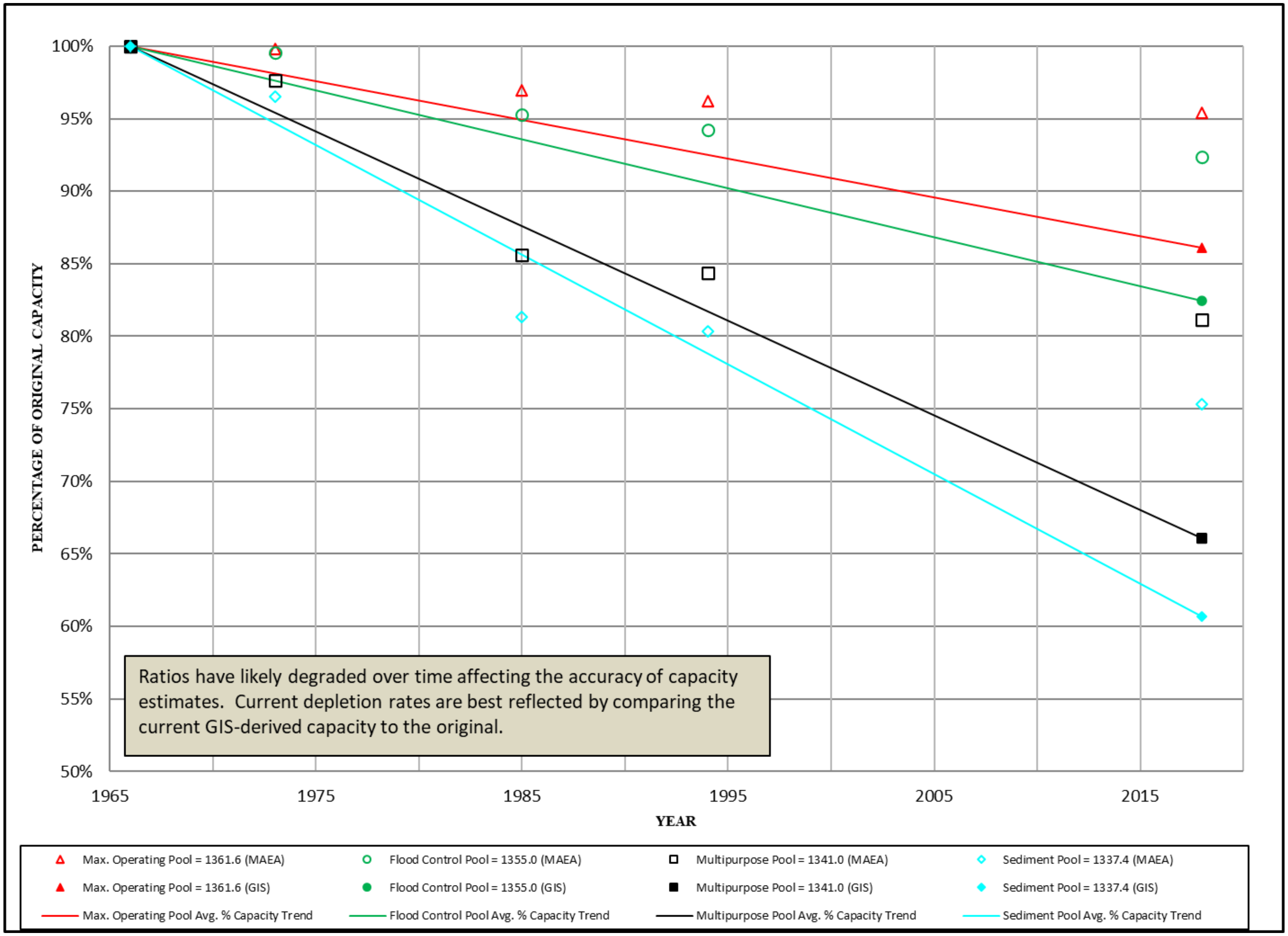
[1] GIS storage values are recommended for use. MAEA values provided for historic information only.



Case Study 8A Figure 2. Twin Lakes Reservoir capacity curves



Case Study 8A Figure 3. Twin Lakes Reservoir surface area curves



Case Study 8A Figure 4. Twin Lakes Reservoir changes in reservoir storage capacity

Case Study 8A Table 3
2018 Twin Lakes Area (Acres)

Elevation - NGVD 29	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1,321	0	0	0	0	0	0	0	0	0	0
1,322	0	0	0	0	1	2	2	3	3	4
1,323	5	5	6	7	8	8	9	10	11	11
1,324	12	13	14	15	15	16	18	19	21	22
1,325	24	26	28	29	31	33	35	36	38	41
1,326	43	44	46	47	48	49	50	51	52	53
1,327	54	54	55	55	56	57	57	58	58	59
1,328	60	60	61	61	62	63	63	64	65	66
1,329	66	67	68	68	69	70	71	72	73	73
1,330	74	75	76	76	77	78	79	80	80	81
1,331	82	83	84	85	85	86	87	87	88	89
1,332	90	91	91	92	92	93	94	95	95	96
1,333	96	97	98	98	99	100	100	101	101	102
1,334	103	103	104	105	106	106	107	108	109	109
1,335	110	111	111	112	113	113	114	115	116	116

Remainder of Table Removed for Brevity

Case Study 8A Table 4
2018 Twin Lakes Capacity (Acre-Feet)

Elevation - NGVD 29	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
1,321	0	0	0	0	0	0	0	0	0	0
1,322	0	0	0	0	0	0	0	1	1	1
1,323	2	2	3	3	4	5	6	7	8	9
1,324	10	11	13	14	16	17	19	21	23	25
1,325	27	30	32	35	38	41	45	48	52	56
1,326	60	64	69	74	78	83	88	93	99	104
1,327	109	115	120	125	131	137	142	148	154	160
1,328	166	172	178	184	190	196	203	209	215	222
1,329	229	235	242	249	256	263	270	277	284	291
1,330	299	306	314	321	329	337	345	353	361	369
1,331	377	385	393	402	410	419	428	436	445	454
1,332	463	472	481	490	499	509	518	527	537	546
1,333	556	566	575	585	595	605	615	625	635	645
1,334	656	666	676	687	697	708	719	729	740	751
1,335	762	773	784	795	806	818	829	841	852	864

Remainder of Table Removed for Brevity

Case Study 8B
Evaluation of Reservoir Capacity Using Two Methods at Bluestem Reservoir

1. Case Study.

This case study compares two reservoir capacity analysis methodologies. As survey technology evolves, and changes in data collection and analysis are inevitable, it is important to determine the capacity shifts associated with a change in methodology. Note that this report does not include evaluation of past surveys or sediment depletion trends. The case study content illustrates concepts but is not comprehensive of the appropriate content for all USACE reservoir projects.

2. Introduction.

Bluestem Lake, formed by Salt Creek Dam No. 4, is located on the North Tributary of Olive Branch Creek, a tributary of Salt Creek in Lancaster County, Nebraska, approximately 13 miles southwest of Lincoln. Construction was completed in 1963, and several storage capacity surveys have been performed. This report presents the comparisons between two survey styles, conventional and DEM from combining LiDAR and high-density bathymetry. Bluestem Lake and the sediment range section lines are shown in Figure 1.

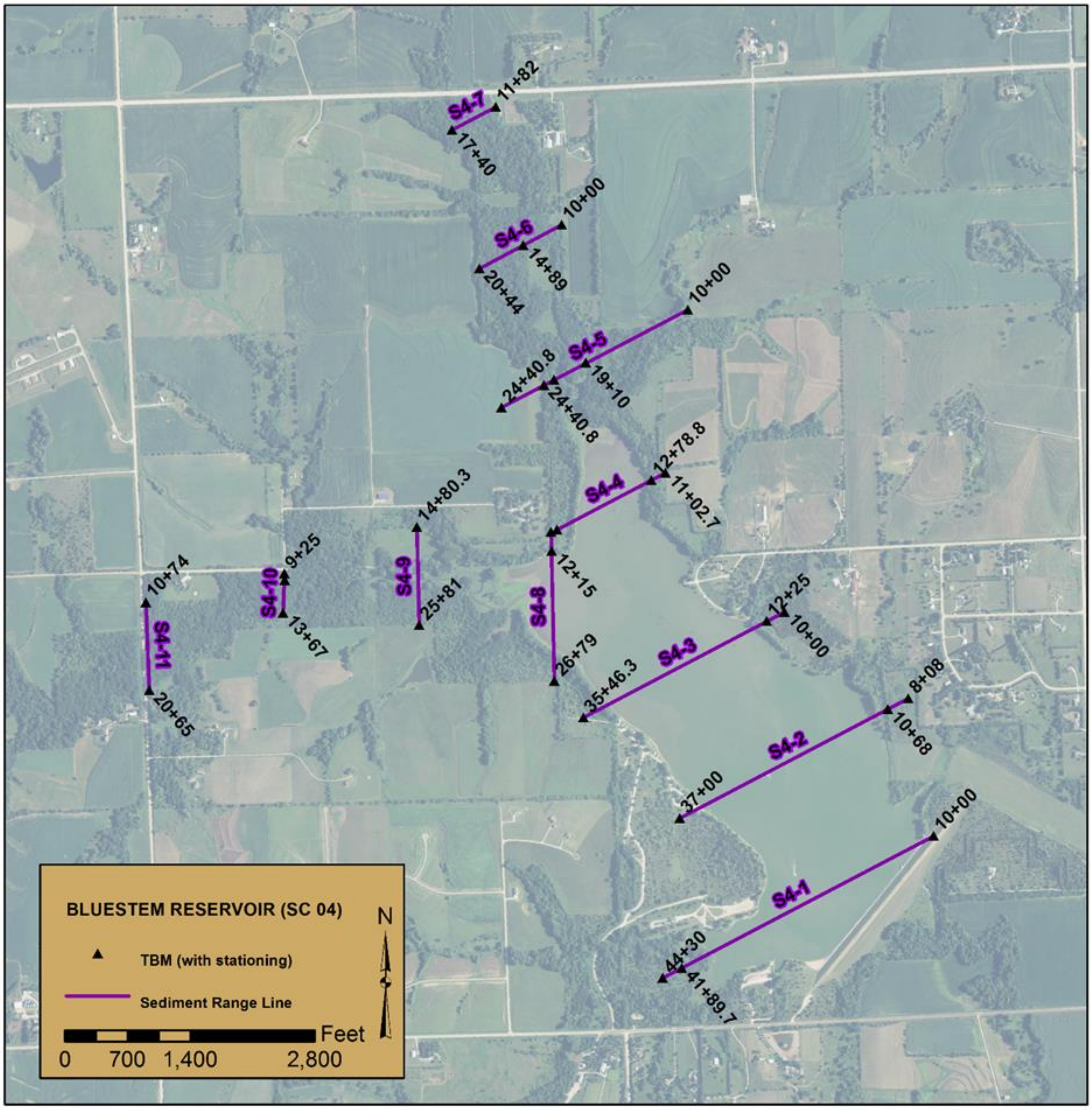
3. Capacity Methods.

Historically, reservoir storage volume calculations at Bluestem Lake were computed using the modified average end area method (USACE 1992) from sediment range surveys. When performing the latest capacity update, LiDAR data was available at no cost for Bluestem Lake. An evaluation was performed to compare reservoir capacity computed from the sediment range surveys to that determined using GIS software and the LiDAR surveys.

a. Sediment Range Methodology. The initial storage volume for Bluestem Lake was calculated from pre-construction contour mapping at 10-foot intervals. Sediment range lines were established at selected points with respect to the irregular reservoir boundary and topography, and also to reflect tributary sediment inflow locations.

(1) Volume between adjacent sediment range cross sections is computed using the standard average end area method. After computing storage volume from sediment range lines, volume capacity correction factor tables were developed to match sediment range computed volume to the pre-dam contour volume.

(2) The table of factors, which consists of a value by elevation, accounts for the non-uniformity of reservoir shape, slopes, etc., between the bounding sediment range locations. Using sediment range cross sections combined with the correction factor tables is referred to as the modified average end area method (MAEA). Incorporating the correction factor tables is a significant enhancement on accuracy compared to the standard average end area method.



Case Study 8B Figure 1. Bluestem Reservoir sediment range and temporary bench mark location map

(3) Computing volume change between surveys using the sediment range method has limitations that should be considered. Since the sediment range method uses only the established sediment range sections, any topographic changes that affect volume (dredging, gravel mining, landscaping, habitat creation, etc.) that occur between the sediment range sections are not represented in the new calculated volume.

(4) Small or micro-scale changes that affect only a small portion of the sediment range section are magnified by the distance between sections. For example, adding a walking trail or road with fill in a section would be applied to the entire reach. Minor changes to storage volume

that occur on a single sediment range, such an installing an elevated roadway or a gravel pit mine, will be over-emphasized in the new calculated storage volume.

(5) In many cases, the historic correction tables are no longer valid. The recommended practice is to plot all sediment range cross sections and compare to historic sections to verify areas of change.

b. GIS-Based Methodology. Using GIS to compute reservoir storage volume has become the preferred method for many USACE projects for many reasons, including:

(1) GIS systems interface well the remote sensing data collection systems and high-density data collected with current survey methods.

(2) GIS volume computations require large amounts of data to be collected and stored, something that present systems can easily handle.

(3) Aerial LiDAR data can be combined with dense bathymetric mapping to render a DEM which better represents the entire topography of the basin and yields a more accurate capacity.

(4) Reservoir LiDAR may be available from sources outside USACE which is a large cost savings. USACE topographic mapping standards must be followed in DEM creation.

4. Data Sources.

Sediment range surveys were collected in 2013. These conventional surveys consisted of both bathymetry and GPS land-based collection.

a. LiDAR. LiDAR for the dry land portion of the reservoir area was available at no cost from Lancaster County, Nebraska. The DEM was based on 4.5-foot posting intervals with a vertical accuracy of 0.3 feet that was collected in April 2010.

b. Hydrographic and Land Surveys. High-density hydrographic surveys were collected of the below-water area for combination with the LiDAR to create a combined DEM for the entire reservoir area. Land surveys were collected along each sediment range using standard GPS survey equipment.

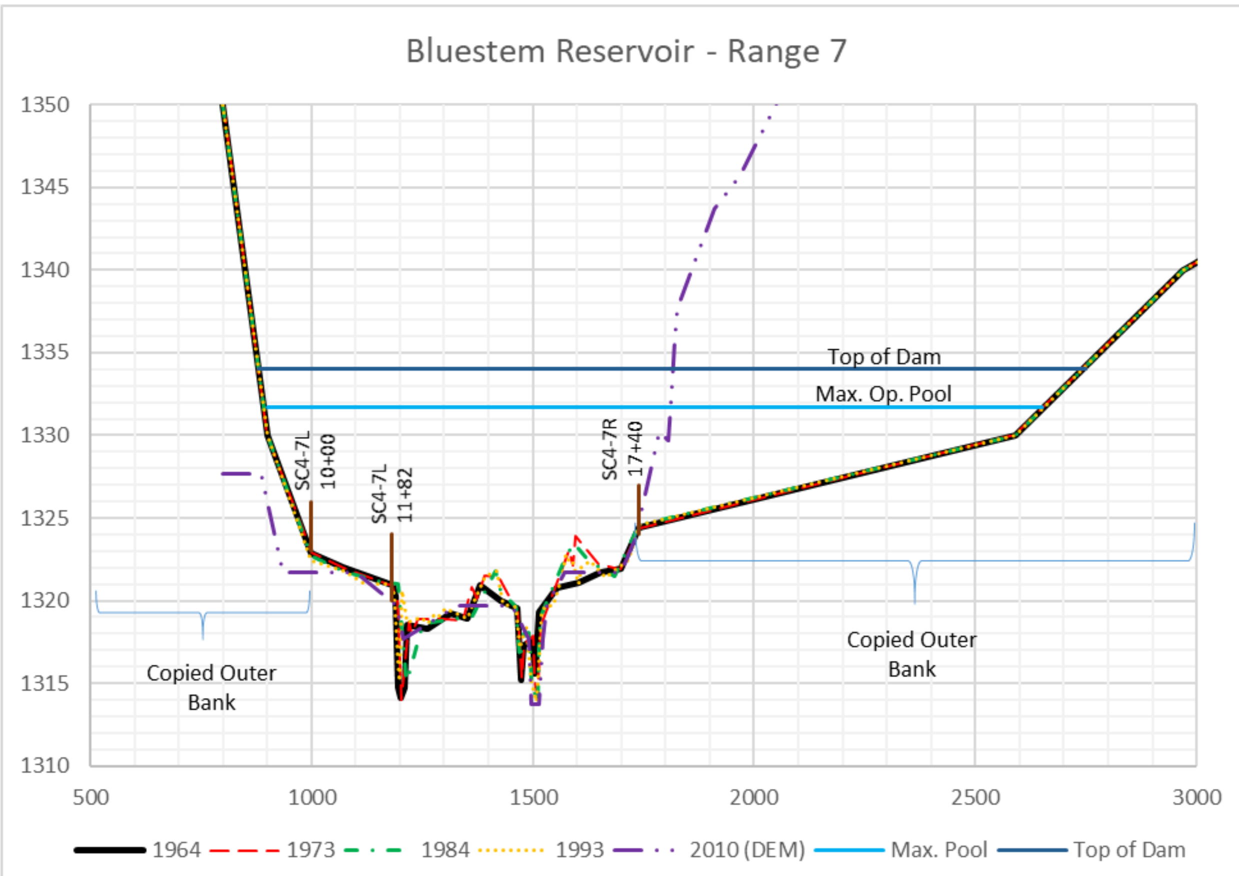
c. Additional Surveys to Aid Combining Data. Due to differing water levels at the times of the two surveys, additional topographic surveying had to be completed to fill in the gaps and make a seamless transition from the bathymetric data to the LiDAR data. This included walking the entire water’s edge perimeter of the reservoir and random points in the delta area between the pool and the incoming tributary.

5. Historic Sediment Range Data Issues.

When comparing to the DEM data, issues were noted with the historic sediment range data.

a. Sediment Range End Points. Historical sediment range cross-section data at Bluestem Reservoir contain data points further landward than the temporary bench mark (TBM) end points and have an unknown source. Throughout this report, these data points are referred to as outer bank segments/points. When performing sediment range surveys, established practice was to survey only to the first TBM points. The outer bank segment points were not surveyed, and historical data was simply copied from survey year to survey year as a cost savings measure under the assumption that these areas were mainly above the operating pool level and not subject to sediment deposition.

b. Comparison to DEM Data. Comparison to DEM data illustrated that in some cases, the historic sediment range back end points clearly do not accurately represent the current topography. Figure 2 shows one of the more extreme examples of the error in overbank topography. On sediment range 7, the stationing of the outward left TBM was historically at 10+00 and the right TBM is at 17+40. The 2010 (DEM) data series represents the overbank topography as derived from the LiDAR.



Case Study 8B Figure 2. Range 7 showing topographic discrepancy on the overbanks

c. Effect on Capacity and Depletion. The sediment range issues were reviewed to determine the potential effect on capacity and depletion. Unfortunately, the original topographic data and correction tables used to calculate the initial area and storage volume were not available. Therefore, the capacity and the development of the sediment range correction tables for use in the MAEA capacity computation could not be reviewed.

(1) Topographic Difference. Figure 2 illustrates that the sediment range and DEM derived section are quite different. It appears that data outside the bounding limits of the back TBMs are simply copies of the erroneous "outer bank" segment data from 1964.

(2) Capacity and Depletion Differences. While it is clear that the end point portion of several sediment range sections do not reflect actual elevations, the impact on actual reservoir capacity is undefined since it is possible that the correction table factors offset a portion of the erroneous end point data. Because of the copying of these outer bank segment data points from year to year, the depletion rate of the reservoir capacity has not historically been tracked by this data. However, variation in this area may actually be occurring due to sediment and erosion processes.

(3) Future Recommendation. Future use of the sediment range method in the future to determine reservoir capacity and depletion rate is not preferred due to the likely errors introduced by the outer bank point data.

6. Storage Capacity Computed from Surveyed Sediment Ranges.

Reservoir capacity was computed from sediment range surveys using the MAEA method. After sediment range field data collection, the survey data was reviewed and formatted. Capacity was calculated using the Omaha Utilities Program (OUP), which utilizes the MAEA capacity correction tables. Evaluation of capacity was performed with two variations of the sediment range data due to the historical data issues noted with a portion of the sediment range surveys. The two methods illustrate the effect of the erroneous historical data on the reservoir.

a. Replace End Points Method. The historic end points of the sediment range sections were replaced with topography derived from the LiDAR DEM collected in 2009. Using the DEM was necessary since the 2013 GPS land survey did not extend past the TBM back monuments.

b. Replace End Point Results. At the maximum operating pool elevation of 1,331.7 feet (NGVD 29), there is a 1.4% increase in capacity using the new combined LiDAR and sediment range sections. The table presents only the total capacity change by pool zone. Each individual segment, or the area between sediment range lines on which calculations are computed, can vary to a greater or lesser extent based on the similarities between the historic and the LiDAR section end point elevations. Table 1 presents the results of this calculation with the revised data set.

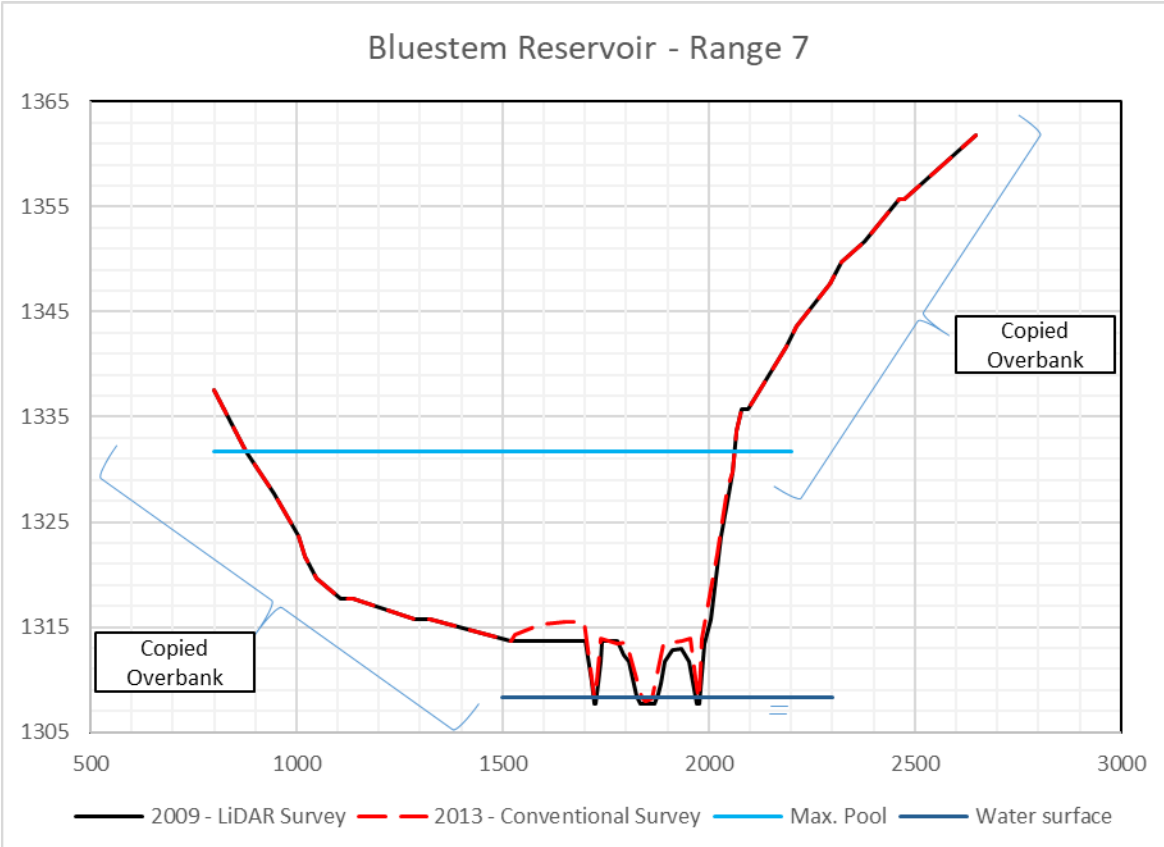
Case Study 8B Table 1
Bluestem Reservoir Capacity by Pool Level (Acre-Feet)

Pool Level	Elevation (NGVD 29)	MAEA – historic end areas	MAEA – LiDAR End Areas	Difference
Maximum Operating Pool	1,331.7	16,344	16,569	1.4%
Top of Flood Control	1,322.5	9,258	9,414	1.7%
Top of Conservation	1,307.4	2,343	2,337	-0.3%
Top of Sediment Pool	1,306.1	1,968	1,962	-0.3%

c. Extract Entire Sediment Range from DEM. A second comparison was performed between the storage capacity calculated from the conventional sediment range survey data vs. extracting the entire sediment range cross section from the DEM (LiDAR/bathymetry combination) using GIS software.

(1) LiDAR data does not always compare well to conventional survey data. There can be error associated with extraction of data from the DEM due to triangulation of the survey random data points and missing slope break points that are captured by the sediment range survey line. Within the land area portions of the section derived from LiDAR, trees and brush can hinder accurate collection in heavily vegetated areas. Cross-section ground surveys of the LiDAR collection area is recommended to provide spot checks and verify accuracy. Multiple USACE documents are available for guidance on survey data collection and DEM creation (USACE 2015d, 2013, 2011f, 2007).

(2) Figure 3 shows the comparison at a sediment range section between a conventional survey and a LiDAR/bathymetric survey. The two sections show quite a bit of variation. Sediment range at Bluestem Reservoir is characterized by heavy tree and brush cover that is concentrated near the normal reservoir pool level. In this example, a capacity calculation using the 2013 conventional survey data would yield a higher amount of deposition (aggradation) than the 2010 LiDAR data and a lower reservoir capacity.



Case Study 8B Figure 3. Comparison of LiDAR and conventional survey data

d. Sediment Range Methods Capacity Comparison. Table 2 presents the results of the storage capacity calculation using the two methods. At the maximum operating pool of elevation 1,331.7 feet, NGVD 1929, the capacity is 8.6% greater using the LiDAR extracted cross sections vs. the conventionally surveyed cross sections. Interestingly, the discrepancy between capacities at normal pool levels that are derived from the underwater portion of the survey are fairly low and are less than 1%. This is likely due to the lack of relief in the underwater topography and that most of the difference is between the land-based portions of the sediment range section.

Case Study 8B Table 2
Bluestem Reservoir Capacity by Pool Level (Acre-Feet)

Pool Zone	Elevation (NGVD 29)	MAEA – Conventional (w/LiDAR End Areas)	MAEA – LiDAR/ Bathymetric extracted cross section	Difference
Max. Operating Pool	1,331.7	16,569	17,989	8.6%
Top of Flood Control	1,322.5	9,414	10,217	8.5%
Top of Conservation	1,307.4	2,337	2,325	-0.5%
Top of Sediment Pool	1,306.1	1,962	1,944	-0.9%

7. GIS Volume Method.

Reservoir capacity was also evaluated using only GIS software. This method utilizes the DEM created from the LiDAR and bathymetric data.

a. Additional Bathymetric Surveys. For this method, additional hydrographic surveys were collected to provide a dense coverage of the pool area. Data was collected on a cross-section format with an approximate line spacing of 100 feet. Additional survey points were collected to define edge of water and structures within the pool. In areas of overlap between the hydrographic and LiDAR survey data sources, the hydrographic survey points were given preference when creating the DEM.

b. Capacity Computation. Once the combined DEM is created, reservoir capacity is computed using GIS software. The GIS method computes the volumetric difference between the DEM and a two-dimensional (2D) plane set at a constant elevation. The computation is repeated at all desired capacity elevations. Table 3 presents the volumes by pool level computed using the GIS method compared to the LiDAR extracted sediment range method. The results from the GIS method yield values 14% to 18% less than the results from the MAEA method. This large difference is concerning.

Case Study 8B Table 3
Bluestem Reservoir Capacity by Pool Level (Acre-Feet)

Pool	Elevation (NGVD 29)	MAEA – LiDAR/ Bathymetric extracted cross section	GIS – LiDAR/ Bathymetric 3D mesh	Difference
Maximum Operating Pool	1,331.7	17,989	14,709	-18.2%
Top of Flood Control	1,322.5	10,217	8,489	-16.9%
Top of Conservation	1,307.4	2,325	1,998	-14.1%
Top of Sediment Pool	1,306.1	1,944	1,661	-14.5%

8. Comparison of Results.

Results were evaluated to estimate change in capacity related to switching from the sediment range modified average end area to GIS computations with DEM data. Table 4 presents the results of the comparison of the two methods.

Case Study 8B Table 4
Bluestem Reservoir Capacities by Pool Level (Acre-Feet)

Pool Level	Elevation (NGVD 29)	MAEA – Conventional w/ LiDAR End Areas	GIS – LiDAR/ Bathymetric 3D mesh	Difference
Maximum Operating Pool	1,331.7	16,569	14,709	-11.2%
Top of Flood Control	1,322.5	9,414	8,489	-9.8%
Top of Conservation	1,307.4	2,337	1,998	-14.5%
Top of Sediment Pool	1,306.1	1,962	1,661	-15.3%

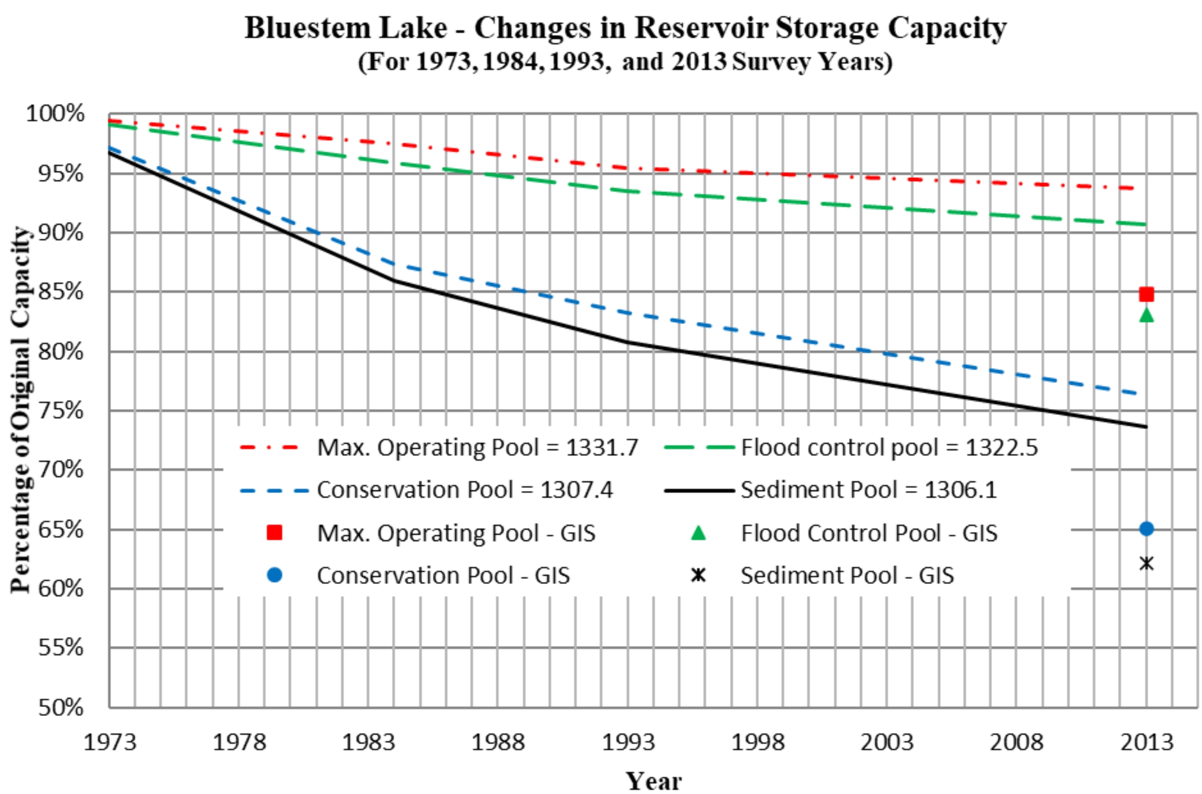
a. Evaluation of Differences between Methods. In the case of Bluestem Reservoir, the difference between methods is about a 10% to 15% decrease in reservoir storage capacity from the sediment range MAEA method to the LiDAR-based GIS method. The GIS method using a DEM that represents the complete reservoir topography of a reservoir basin is likely more accurate than the MAEA method, provided that the DEM data sources, both LiDAR and bathymetric, are sufficient quality. The MAEA method based on historic sediment range data also produces reasonably accurate results. Specific conclusions regarding Bluestem Reservoir are summarized as:

(1) The accuracy of the LiDAR-based DEM in the heavily vegetated areas near the normal pool level is questionable at Bluestem Reservoir.

(2) The seemingly wide variation between the GIS and MAEA methods is partially due to incomplete data in the DEM in the incoming tributaries.

(3) Calculated capacity below the normal pool had greater difference between the two methods. This is likely due to the DEM assembly. The density of bathymetric data is a large factor in the underwater capacity difference.

b. Impact on Depletion Rates. Computing depletion rates is a critical tool in reservoir management. Developing accurate depletion rates requires a consistent methodology for computing capacity. Figure 4 shows the depletion of capacity at Bluestem Reservoir and the discontinuity from prior years that used the MAEA method. The final data point in the trend, from the GIS survey, shows a drastic capacity change related to the change in capacity computation methodology.



Case Study 8B Figure 4. Depletion rate discontinuity

9. Summary and Recommendations. Historically, reservoir storage capacity computations at Bluestem Lake were computed using the modified average end area method from sediment range surveys. When performing the latest capacity update, a second method to compute capacity using LiDAR data was also used. An evaluation was performed to compare methodology and results from the two techniques. Significant conclusions are as follows:

a. The difference between the two computation methods was large, with the GIS method about 10% to 15% lower capacity than the MAEA method. Capacity difference between the two methods varied by elevation.

b. GIS capacity computations using a DEM of the entire reservoir storage area should provide better accuracy of current reservoir capacity than the sediment range MAEA method, provided that the DEM meets accuracy standards, that bathymetry is collected to USACE standards, and that there are no LiDAR accuracy limitations because of dense vegetation.

c. A disadvantage of the sediment range method is that topographic changes that occur only to a small area at the sediment range, such as that from roadway fill or gravel mining, are magnified by the average end area computation process.

d. Comparison between GPS-surveyed sediment range sections and LiDAR data illustrated a surprisingly wide variation. The greatest difference occurred in areas with heavy tree and brush cover near the normal reservoir pool level. When using LiDAR surveys, it is highly recommended to survey all or a portion of the sediment ranges with conventional survey methods to evaluate LiDAR DEM accuracy.

e. A full bathymetric survey of all incoming tributaries into Bluestem Lake was not completed. Conventional surveys in these areas that were obscured or below water in the LiDAR data set are difficult to collect due to shallow depths and vegetative cover. The amount of capacity in these channels is thought to be minor in relation to the entire capacity of the reservoir. However, this is one source of variation between the two methods.

f. A detailed cost comparison between the two methods was not performed. However, the field surveys and computations necessary for the MAEA sediment range are fairly quick on a small reservoir. In this case, the Bluestem Lake LiDAR data source had no cost. Note that the DEM-based computation method requires additional effort.

g. Due to the large difference between the capacities computed using the GIS and MAEA methods, computing a depletion rate cannot be performed with the GIS estimated capacity. In general, computing depletion rate trends cannot be accurately performed when the method used to compute storage capacity is changed.

h. Potentially, the capacity difference between the two methods could be used as an offset to establish capacity depletion trends with a future survey. However, using an offset should be performed at small elevation increments since the capacity difference is not constant. In addition, depletion trends also vary by elevation.

i. Due to the large difference between the two methods at Bluestem Lake, using both methods to compute capacity in the future is preferred to increase the accuracy of depletion trends.

j. When converting reservoir capacity estimates from a sediment range to GIS method, it is recommended to compute capacity with both the sediment range and GIS method to evaluate differences.

Case Study 8C
Effects of Climate Change on Reservoir Sedimentation at Garrison Dam

1. Case Study.

This case study provides content condensed from studies performed by the Omaha District to evaluate climate change effects on sediment yield to Garrison reservoir on the Missouri River. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

Changes in reservoir capacity due to sedimentation can greatly affect future reservoir operation and benefits. The Climate Change Associated Sediment Yield Impact Study was undertaken to evaluate how climate change will affect the future basin runoff, sedimentation rates, and operations of Garrison Dam. Figure 1 shows Garrison Dam and Lake Sakakawea.

Case Study 8C Figure 1. Garrison Dam
(Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations – Sedimentation Focus (USACE 2012a))

3. Project Overview.

This study was part of a larger interagency effort that included members of USACE, Bureau of Reclamation (BOR), U.S. Geologic Survey (USGS), and National Oceanic and Atmospheric Administration (NOAA). This study provided insight about the long-term availability of reservoir storage and future reservoir operation planning. This case study illustrates and condenses the methods and findings of the Climate Change Associated Sediment Yield Impact Study (USACE 2012a).

a. Garrison Dam. Traveling from upstream to downstream, Garrison Dam is the second of six mainstem dams on the Missouri River (after Fort Peck Dam in Montana). Garrison Dam impounds Lake Sakakawea, the largest USACE reservoir in the U.S. with a gross capacity of 23.8 million acre-feet, approximately one-third of the total Missouri River reservoir system storage. The lake is 178 miles long and 14 miles wide at its widest point.

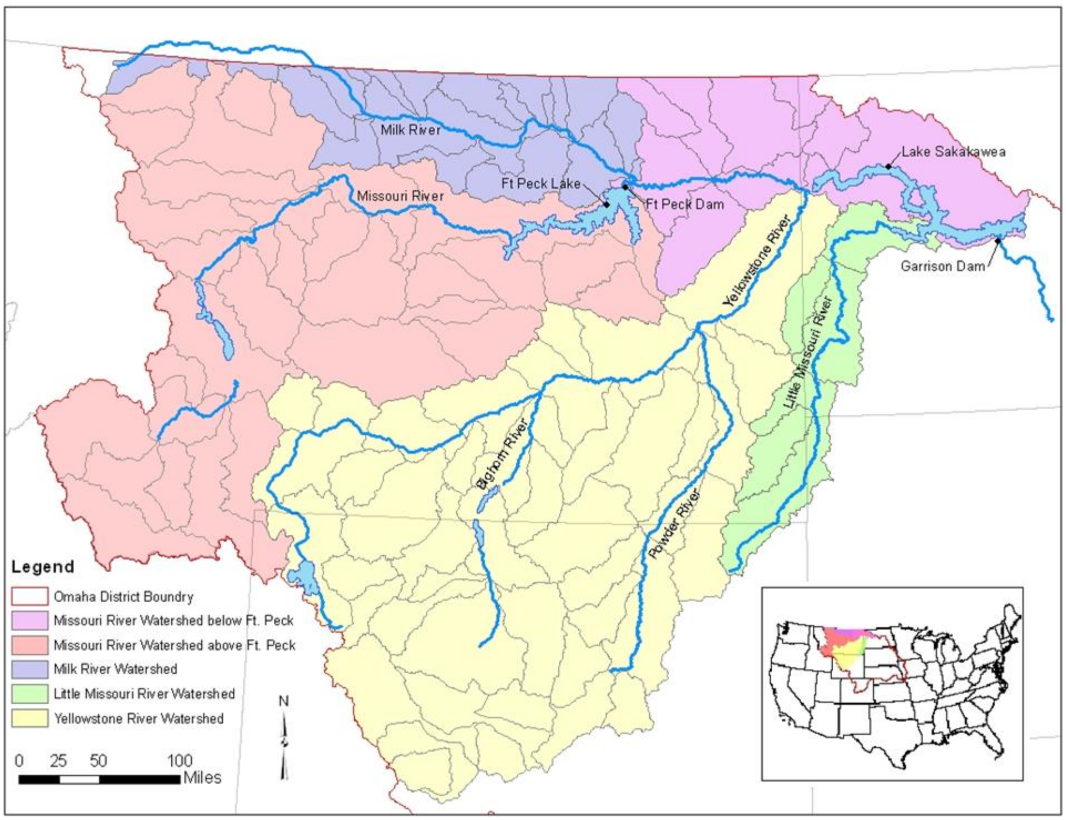
b. Basin. The basin draining into Garrison Dam is primarily in Montana, and includes parts of South Dakota, North Dakota, Wyoming, and Canada, covering approximately 181,400 square miles including the 57,500 square miles above Fort Peck Dam. The major tributaries to the Missouri River above Lake Sakakawea include the Milk River, the Yellowstone River, and the Little Missouri River. There are several other USACE and USBR impoundments in the basin, which are listed in Table 1. A schematic of the study area is shown in Figure 2.

c. Climate. The climate of the Upper Missouri River Basin ranges from semi-arid on the plains to sub-humid in the mountains. Temperature and precipitation can vary widely due to the vast scale of the basin, with a topography that rises to over 8,000 feet beginning at the headwaters of the Missouri River in western Montana and falls to approximately 1,800 feet as the Missouri River flows through North Dakota and crosses into South Dakota. Seasonal extreme temperatures in the basin range from -40 °F in the winter months to 110 °F in the summer. Average temperatures in January, the coldest month, are in the lower teens and in July, the hottest month, temperatures are typically near 70 °F.

d. Precipitation. Approximately 75% of the total annual precipitation falls between April and September. Most precipitation in the study area is dominated by brief, powerful summer thunderstorms that can produce precipitation intensities up to several inches per hour. Snow is the primary source of precipitation from November to March in the upper Missouri River Basin, but snow can fall as early as September and as late as May. Precipitation varies widely across the basin ranging from less than 6 inches in the plains to more than 40 inches in the high mountains, the drainage basin has an annual average rainfall of 15 inches.

Case Study 8C Table 1
Major Impoundments in the Garrison Dam Drainage Basin

Project Name	River	Location	Incremental Drainage Area (mi^2)	Regulated By
Garrison Dam	Missouri	Riverdale, ND	123,900	USACE
Fort Peck Dam	Missouri	Fort Peck, MT	57,500	USACE
Clark Canyon	Beaverhead	Dillon, MT	2,320	USBR
Canyon Ferry	Missouri	Helena, MT	13,580	USBR
Gibson	Sun	Augusta, MT	575	USBR
Tiber	Marias	Chester, MT	4,920	USBR
Fresno	Milk	Havre, MT	3,776	USBR
Bull Hook	Milk	Havre, MT	54	USACE
Buffalo Bill	Shoshone	Cody, WY	1,500	USBR
Boysen	Wind	Thermopolis, WY	7,710	USBR
Yellowtail	Bighorn	St. Xavier, MT	10,420	USBR



Case Study 8C Figure 2. Missouri River Drainage Basin upstream of Garrison Dam (Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations – Sedimentation Focus (USACE 2012a))

4. Study Overview.

The Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations (USACE 2012a) consisted of hydrologic and sedimentation investigations to evaluate future operations at Garrison Dam. This case study illustrates the methods and results of the sedimentation portion of the study.

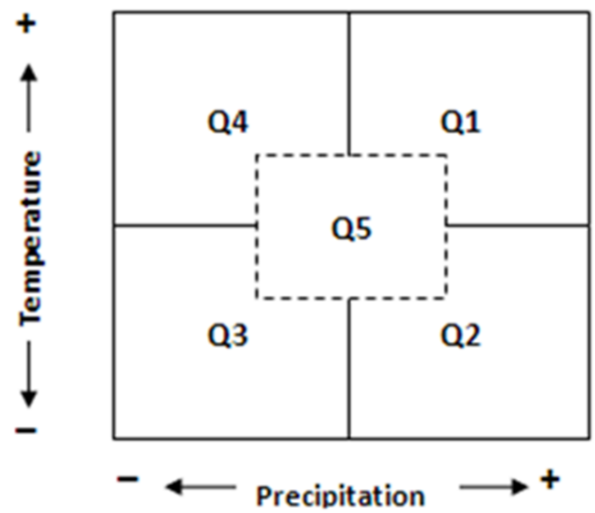
a. Effect on Operations. Future operations of Garrison Dam are influenced by changes in the timing and volume of inflows, in combination with reductions in storage capacity in Lake Sakakawea caused by the deposition of sediment. The study evaluated five different future climate scenarios in the upper Missouri River Basin: drier and cooler, drier and warmer, wetter and cooler, wetter and warmer, and a median precipitation and temperature condition. Flow timing and volume, sediment deposition rates in Lake Sakakawea, and associated future reservoir capacities were calculated for each climate scenario for a near future (2010 to 2039) and distant future (2040 to 2069) period.

b. Evaluation Limitations. While sediment supply and erosion are influenced by vegetation cover and land use, which could be affected by climate change, only changes in sedimentation rates driven by the timing and volume of inflows, not land conditions, were considered for the purposes of the Garrison basin scale part of this study. Thus, sediment discharge rating curves were developed based on past observations and coupled with the future projected inflow timing and volume of the climate scenarios to arrive at the predicted sedimentation rates and reservoir capacities.

c. Small Scale Detailed Evaluation on Little Missouri. A smaller scale but more detailed evaluation was performed on the Little Missouri River basin, a major tributary sub-basin of the watershed drained by Garrison Dam, in order to assess the impacts of climate change on sediment yield when precipitation is not evenly distributed, and variables including soil type, land use, slope, and vegetation are considered.

5. Future Climate Change Scenarios.

Five precipitation and temperature change scenarios were developed by the USBR from 112 downscaled climate projections using the Variable Infiltration Capacity (VIC) model. The climate scenarios used in this study represent conditions that are wetter and warmer (Q1), wetter and cooler (Q2), drier and cooler (Q3), drier and warmer (Q4), and the median change condition (Q5) as shown in Figure 3.



Case Study 8C Figure 3. Climate projection key

a. Two Future Periods. Each climate scenario contained two different periods: 2010 to 2039 (near future) and 2040 to 2069 (distant future). The future baseline scenario, which models the current climate trend continued into the future, was also evaluated to provide a comparison baseline.

b. Climate Projections. Detailed information about the development of the climate projections may be found in the report Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations (USACE 2012a).

6. Future Inflows Estimation.

a. The daily inflow rate into Lake Sakakawea for each of the climate scenarios were calculated based on the USBR's VIC model derived and bias-corrected runoff values. The VIC model used the Bias-Corrected Spatially Downscaled (BCSD) temperature and precipitation projections from an ensemble of 16 phase three CMIP3 models.

b. Reservoir Inflow Basins. The daily reservoir inflow provided by the VIC model is not separated by source (such as tributary contribution). Table 2 lists the major drainage basins and their contributions to Lake Sakakawea. Calculations of the Lake Sakakawea daily inflows for each of the five future climate projections are described in detail in the report "Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations" (USACE 2012a).

Case Study 8C Table 2
Average Annual Percent Contributions into Lake Sakakawea

USGS Streamgage	Incremental Garrison Drainage Basin above Gage[1] (mi^2)	Percent of Incremental Basin[2] (%)	Gaged Sediment Load Contribution[3] (%)	Gaged Inflow Contribution[3] (%)
Missouri River near Culbertson, MT	34,001 (91,557 total)	27%	18%	44%
Yellowstone River near Sidney, MT	68,392	55%	59%	54%
Little Missouri River near Watford City, ND	8,310	7%	23%	2%

[1] Area upstream of Fort Peck Dam (57,556 mi^2) not included in incremental Garrison drainage basin (123,844 mi^2 incremental, 181,400 mi^2 total).
[2] Remaining 11% is ungaged area located downstream of the gage stations on the reservoir.
[3] Percent contribution based on streamgage measurements only, not including ungaged contributions.

7. Sedimentation Estimation.

Sediment load measurements will often differ by orders of magnitude for any given discharge due to the physical complexity of sediment transport as well as variations in available sediment supply.

a. Seasonal Variation Rating Curves. In an effort to improve the sediment load to discharge rating curves, seasonal variations were examined, and seasonal rating curves were developed for the USGS gages near Lake Sakakawea shown in Table 3, Table 4, and Table 5. The amount and type of data was considered when selecting eligible gage stations. Two attempts were made to define an indicator of seasonal change: one based on water temperature and the other based on calendar dates. All-season rating curves were also developed, and a sensitivity analysis was performed comparing the long-range sediment accumulation predictions between the all-season and three-season rating curve sets.

b. Seasonal Definition by Water Temperature. In the attempt to define seasonal changes by water temperature, the suspended-sediment data was split into 5 °C bands and plotted against flow. A summary of the temperature bands and correlation coefficients at five sampling locations is listed in Table 3.

Case Study 8C Table 3
Comparison of Correlation Coefficients – Data Divided by Temperature

USGS Gage Station Location	Correlation Coefficient for Temperature Band (˚C)				
	0–5	**5–10**	**10–15**	**15–20**	**>20**
Missouri River near Landusky	0.4645	0.7235	0.7159	0.7822	0.4251
No. Data Pts	37	37	43	64	43
Missouri River near Culbertson	0.2706	0.8072	0.7509	0.7013	0.3578
No. Data Pts	55	37	47	54	20
Yellowstone River near Sidney	0.6929	0.6587	0.8232	0.8831	0.8481
No. Data Pts	105	52	72	119	81
Powder River near Locate	0.7890	0.8272	0.8836	0.8943	0.8593
No. Data Pts	121	40	50	60	48
Milk River at Nashua	0.8338	0.8864	0.8833	0.7760	0.8970
No. Data Pts	66	24	22	24	44

c. Seasonal Temperature Results. The attempt to correlate seasonal sediment load to discharge relationships based on temperature bands was unsuccessful and revealed that:

(1) Suspended-sediment correlation results were not conclusive and are limited by the measured data quality, data quantity, and the existence of other sediment yield factors that are not well correlated with water temperature.

(2) The winter season (less than 5 °C) generally had the poorest correlation.

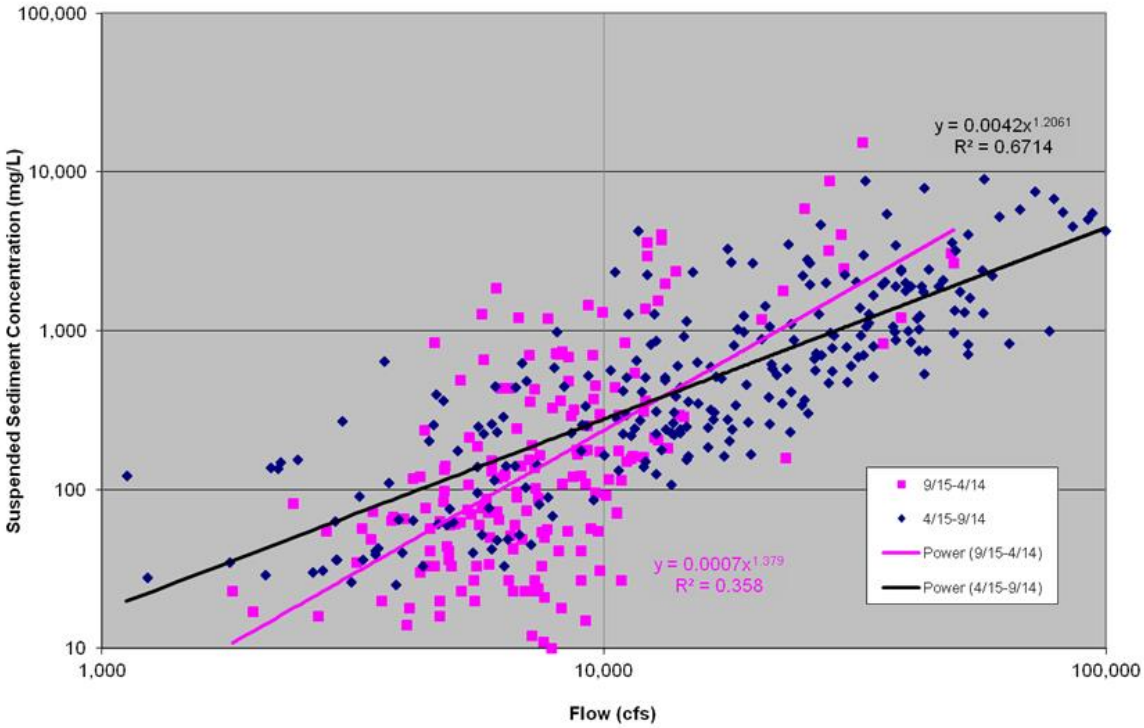
(3) The remainder of the temperature bands for the spring, summer, and fall seasons correlations varied for each site with no clear relations.

(4) The non-regulated river stations on the Yellowstone, Powder, and Milk Rivers had better correlation than the two gage stations on the regulated Missouri River. This was true for all temperature ranges except the two coolest bands for the Yellowstone River near Sidney.

d. Seasonal Definition by Calendar Date. Next, two-season and three-season sets of sediment load-discharge curves were developed to test seasonal relationships by calendar date. Additionally, the use of sediment concentration in milligrams per liter was compared with the use of sediment discharge in tons per day, which revealed that the use of sediment discharge produced some induced correlation.

(1) Two-Season Sediment Rating. In the two-season scheme, the year was split into two periods, a transition/warm season from 15 April through 14 September, and a cool season from 15 September through 14 April. The two-season rating curve plot for the Yellowstone River near

Sidney, Montana is shown in Figure 4. A summary of correlation coefficients for five sampling locations is listed in Table 4.

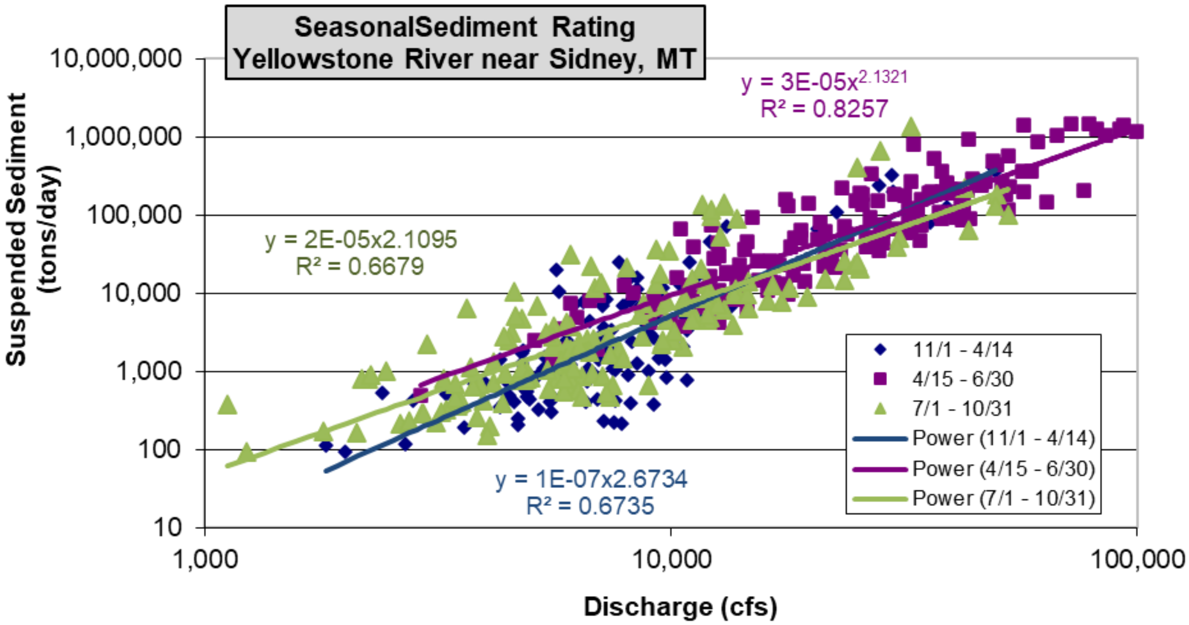


Case Study 8C Figure 4. Two-season suspended-sediment concentration rating curve for the Yellowstone River near Sidney

Case Study 8C Table 4
Comparison of Correlation Coefficients – Data Divided into Two Seasons

USGS Gage Station Location	Suspended-Sediment Discharge (tons/day)			Suspended-Sediment Concentration (mg/L)		
	All Season	Two Season		All Season	Two Season	
		9/15–4/14	4/15–9/14		9/15–4/15	4/15–9/15
Missouri River near Landusky	0.6544	0.6168	0.6448	0.4071	0.3386	0.3369
No. Data Pts	227	92	135	227	92	135
Missouri River near Culbertson	0.4998	0.3630	0.7005	0.1687	0.0868	0.3416
No. Data Pts	213	96	117	213	96	117
Yellowstone River near Sidney	0.8165	0.6432	0.8724	0.5973	0.3580	0.6714
No. Data Pts	431	179	252	432	180	252
Powder River near Locate	0.8630	0.0170	0.8978	0.5292	0.4337	0.5791
No. Data Pts	322	173	149	323	173	151
Milk River at Nashua	0.8378	0.8385	0.8887	0.3917	0.4621	0.4274
No. Data Pts	187	80	105	187	105	82

(2) Three-Season Sediment Rating. In the three-season scheme, seasons were defined as a winter season from 1 November through 14 April, a spring rise season from 15 April through 30 June, and a summer/fall season from 1 July through 31 October. The three-season rating curve plot for the Yellowstone River near Sidney, Montana is shown in Figure 5. A summary of the three-season correlations is listed in Table 5.



Case Study 8C Figure 5. Three-season sediment rating curve for the Yellowstone River

Case Study 8C Table 5
Comparison of Correlation Coefficients – Data Divided into Three Seasons

USGS Gage Station Location	Correlation for Suspended Sediment Discharge (tons/day)			
	All-Season	Three Season		
		11/1–4/14	4/15–6/30	7/1–10/31
Missouri River near Landusky	0.6544	0.5998	0.7024	0.4940
No. Data Pts	227	54	84	89
Missouri River near Culbertson	0.4998	0.2386	0.7568	0.7220
No. Data Pts	213	60	68	85
Yellowstone River near Sidney	0.8165	0.6735	0.8257	0.6679
No. Data Pts	431	120	160	151
Powder River near Locate	0.8630	0.8042	0.9369	0.7699
No. Data Pts	322	138	87	97
Milk River at Nashua	0.8378	0.8638	0.9027	0.7846
No. Data Pts	187	72	44	71
Yellowstone River at Billings	0.8222	0.2910	0.7459	0.7415
No. Data Pts	186	72	53	62
Missouri River at Toston	0.7852	0.7946	0.7949	0.7791
No. Data Pts	201	82	53	66

e. Results of Seasonal Correlation Evaluations. For further details on the seasonal rating curves estimation, see Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations – Sedimentation Focus (USACE 2012a). Significant results are summarized as:

(1) Seasonal shifts in the sediment-flow relationship are apparent in the USGS gage data.

(2) Although water temperature change occurs seasonally, efforts to determine a seasonal sediment-flow relationship based on water temperature changes had limited success.

(3) Suspended-sediment discharge correlation coefficients were higher than the suspended-sediment concentration coefficients. Therefore, suspended-sediment-to-discharge correlation analysis likely has some induced correlation.

(4) Of all the methods evaluated, the best correlation for suspended-sediment concentration and streamflow was determined with three seasons defined as 1 November to 14 April for the low flows of winter; 15 April to 30 June for the increasing flows from snow melt

and spring rains; and 1 July to 31 October for the decreasing flows due to warmer temperatures and less precipitation.

(5) For all the analyses methods, the non-regulated river stations on the Yellowstone, Powder, and Milk Rivers had better correlation coefficients than the two gage stations on the regulated Missouri River.

(6) Three season correlation coefficient was the highest for the 15 April through 30 June period at all locations. This also corresponds with the highest flow and sediment load period.

f. Sediment Load Analysis. Long-term sediment load trends were found to have experienced little change based on USACE suspended sediment records from 1937 to 1974 and collected USGS data. Therefore, trend analysis was performed assuming a continuing steady relationship between sediment load and discharge.

(1) Unmeasured Load Estimate. Unmeasured sediment load was estimated with a value of 12% to approximate the total accumulation in Lake Sakakawea based on past surveys, and further verified with the Bureau of Reclamation's Automated Modified Einstein Procedure (BORAMEP) program. For details on long-term trends and the BORAMEP analysis, see Climate Change Associated Sediment Yield Impact Study: Garrison Dam Specific Sediment and Operation Evaluations – Sedimentation Focus (USACE 2012a).

(2) Total Load Estimate. The total sediment load estimates for Lake Sakakawea were determined using the projected inflows and the all-season sediment rating curves for the USGS streamgages nearest to the reservoir shown in Table 6.

Case Study 8C Table 6
USGS Steam Gages Representative of Lake Sakakawea Inflows

USGS Gage Name (ID Number)	Contributing Drainage Area above Gage (mi^2)	Period of Record Discharge	Period of Record Suspended Sediment
Missouri River near Culbertson, MT (06185500)	91,557	1941–Present	1971–1976
Yellowstone River near Sidney, MT (06329500)	68,392	1910–Present	1971–2010
Little Missouri River near Watford City, ND (06337000)	8,310	1934–Present	1947–1976

(3) Yellowstone River. The Yellowstone River represents 54% of the inflow, and the remaining gaged and ungaged drainage areas represent the other 46% of the projected inflow. The sediment load contribution of the Yellowstone River is described by Equation 8C-1, and the sediment load contribution from the remaining area is described by Equation 8C-2. Equation 8C-

2 was back calculated to approximate the observed sediment accumulation values from the three most recent area-capacity surveys from 1969, 1979, and 1988 after accounting for the estimated Yellowstone River sediment load. Both equations include an additional 12% to account for the unmeasured load.

$$Yellowstone\ River\ Sediment\ Load = 1.12 * (0.000002 * Inflow^{2.3803}) \qquad \text{Case Study 8C Equation 1}$$

$$Remaining\ Area\ Sediment\ Load = 1.12 * (0.0176 * Inflow^{1.6338}) \qquad \text{Case Study 8C Equation 2}$$

(4) Sediment Volume. The resulting sediment load values from Equations 8C-1 and 8C-2 are combined and converted from a sediment load in tons to a sediment volume in acre-feet using a sediment density of 107.4 pounds per cubic foot (45 pcf dry density), as determined by a series of density probe surveys conducted between 1959 and 1972 (USACE 1975).

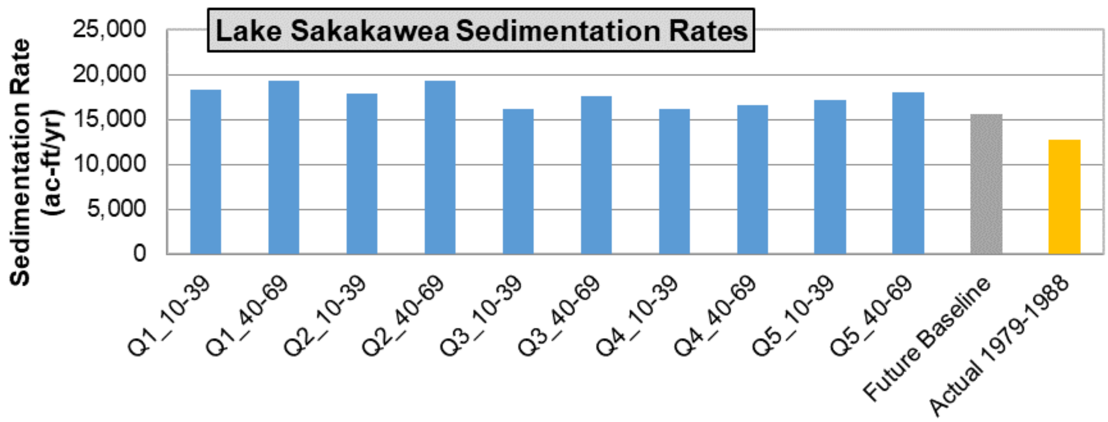
(5) Projected Rates for Future Climate Scenarios. Projected sediment accumulations and sedimentation rates for the future climate scenarios are listed in Table 7. The sediment accumulation in the second column of Table 7 is the sum of sediment accumulation since dam closure in 1967 and the future projected sediment accumulation. The sediment accumulation rate for each climate projection is displayed in Figure 6. The sedimentation rates may be compared to the future baseline which is the computed sediment rate based on observed inflow data applied to the sediment equations for the study period.

Case Study 8C Table 7
Total Sediment Accumulation in Lake Sakakawea for Each Climate Projection Based on All-Season Sediment Load Estimation

Climate Projection	Sediment Accumulation (ac-ft)[1]	Sediment Accumulation Rate (ac-ft/yr)	Sediment Accumulation Rate (million tons/yr)	Percent Difference from Future Baseline
Warmer and Wetter (Q1) 2010–2039	788,345	18,334	42.9	17.4%
Warmer and Wetter (Q1) 2040–2069	829,265	19,285	45.1	23.4%
Cooler and Wetter (Q2) 2010–2039	769,957	17,906	41.9	14.6%
Cooler and Wetter (Q2) 2040–2069	831,140	19,329	45.2	23.7%
Cooler and Drier (Q3) 2010–2039	696,303	16,193	37.9	3.7%
Cooler and Drier (Q3) 2040–2069	755,047	17,559	41.1	12.4%
Warmer and Drier (Q4) 2010–2039	699,427	16,266	38.0	5%
Warmer and Drier (Q4) 2040–2069	713,682	16,597	38.8	6.2%
Median Condition (Q5) 2010–2039	738,628	17,177	40.2	10.0%
Median Condition (Q5) 2040–2069	777,059	18,071	42.3	15.7%
Future Baseline[2]	671,746	15,622	36.5	–

[1] Sediment accumulation is calculated using the Equations 1 and 2 for the 43-year study period.

[2] The Future Baseline is the computed sediment rate based on observed inflow data applied to the sediment equations for the study period.

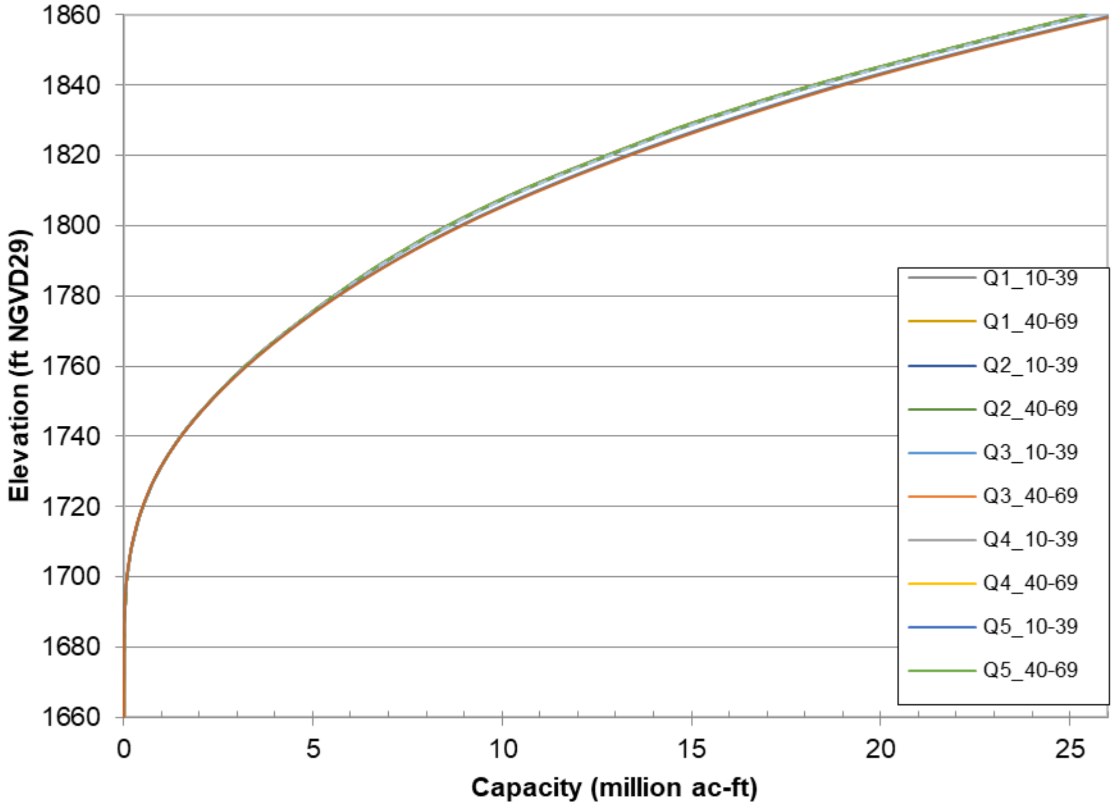


Case Study 8C Figure 6. Sediment accumulation rate for each climate projection based on all-season sediment load estimation
Climate projection key: Q1-warmer and wetter, Q2-cooler and wetter, Q3-cooler and drier, Q4-warmer and drier, Q5-median precipitation and temperature condition

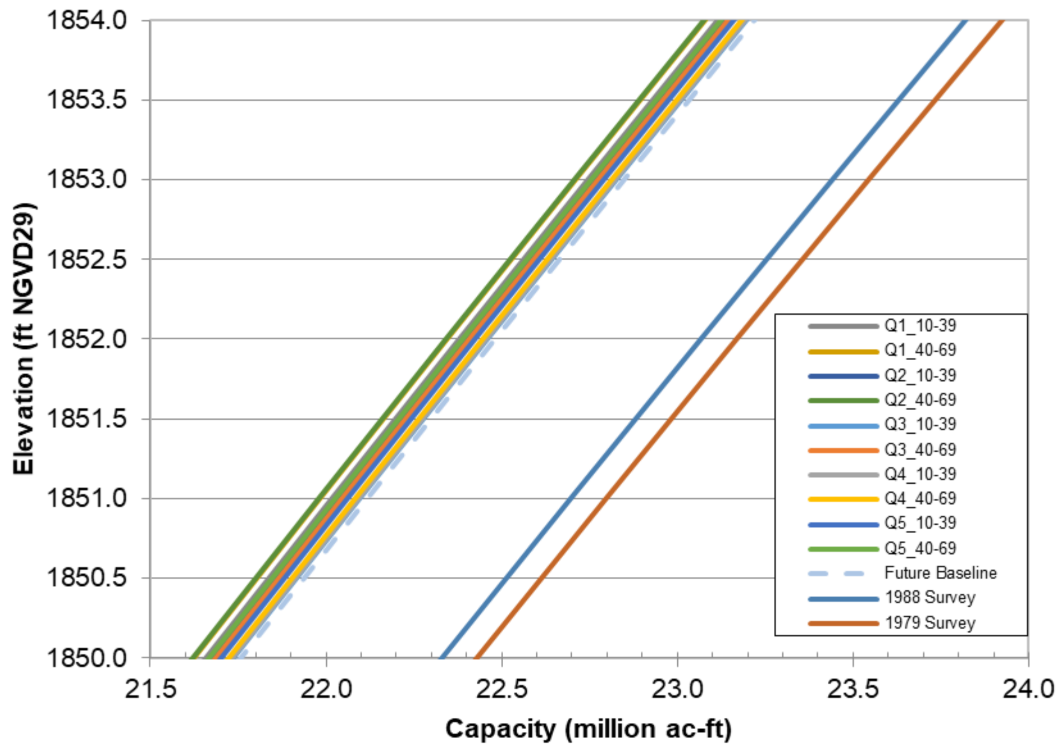
8. Reservoir Capacity Evaluation.

After determining the daily sediment load and total sediment accumulation for each climate projection, the corresponding change in reservoir capacity was also determined. Similar to current, future trap efficiency of Lake Sakakawea was assumed to be 100% as Garrison Dam effectively acts as a total sediment trap.

a. Capacity Curve Update. Sediment was distributed by elevation to create new capacity curves by using the distribution of sediment accumulation which occurred between the 1979 and 1988 surveys. The 1988 survey was used as the baseline reservoir capacity to which the predicted change in capacity was applied. Figure 7 displays the baseline and future area capacity curves for Lake Sakakawea, and Figure 8 displays the area capacity curves for the Exclusive Flood Control Pool between elevation 1,850 feet and 1,854 feet NGVD 29 to illustrate changes in this critical zone. The updated reservoir capacity curves were used in the hydrologic analysis to assess possible impacts to dam operations (USACE 2012a).

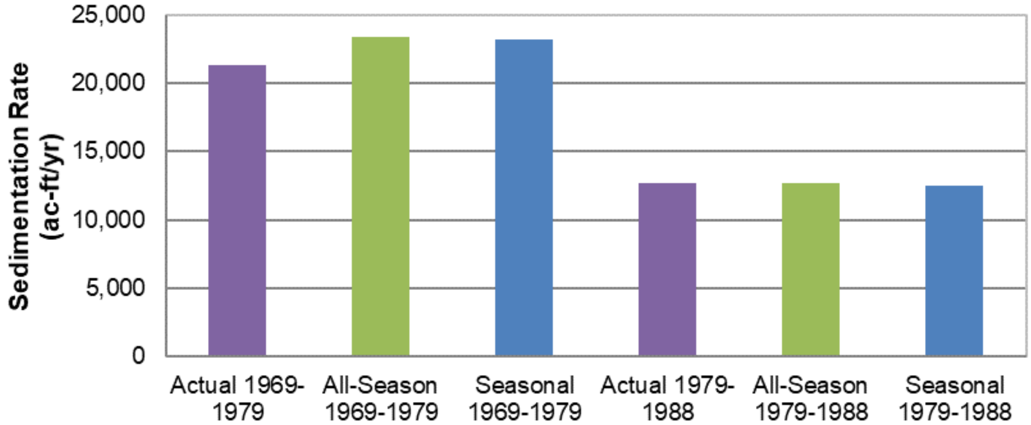


Case Study 8C Figure 7. Lake Sakakawea capacity curves based on the sediment equations Climate projection key: Q1-warmer and wetter, Q2-cooler and wetter, Q3-cooler and drier, Q4-warmer and drier, Q5-median precipitation and temperature condition



Case Study 8C Figure 8. Lake Sakakawea capacity curves representing the exclusive flood control pool
Climate projection key: Q1-warmer and wetter, Q2-cooler and wetter, Q3-cooler and drier, Q4-warmer and drier, Q5-median precipitation and temperature condition

b. Sensitivity Analysis. Differences in performance between the all-season and seasonal equation sets were assessed by comparing the historical sediment accumulations measured by range-line surveys and sediment accumulations calculated by the equations sets for the same time period. Comparisons were made for the 1969–1979 and 1979–1988 survey periods. The comparison between sedimentation rates is illustrated in Figure 9 and listed in Table 8. Reservoir capacity loss between seasonal and all-season equations for two of the future climate projects Q2 (cooler and wetter) and Q3 (cooler and drier) is listed in Table 9. When predicting future reservoir capacity, there was less than 1% difference between scenarios, indicating a low relative sensitivity for sediment prediction.



Case Study 8C Figure 9. Observed and calculated sedimentation rates

Case Study 8C Table 8
Comparison of the Observed and Calculated Sediment Accumulation Rates

Survey Period	Surveyed Accumulation Rate[1] (ac-ft/yr)	All-Season Accumulation Rate[2] (ac-ft/yr)	All-Season Percent Difference	Seasonal Accumulation Rate[3] (ac-ft/yr)	Seasonal Percent Difference
1969–1979	21,328	23,341	9.4%	23,156	8.6%
1979–1988	12,705	12,721	0.13%	12,514	-1.5%

[1] Surveyed data based on range line survey data.
[2] All-Season values calculated using Equation 1 and Equation 2 and the historic reservoir inflow.
[3] Seasonal values calculated using Equation 3 to Equation 8 and the historic reservoir inflow.

Case Study 8C Table 9
Comparing the Results of Calculating Reservoir Capacity Loss

Climate Prediction	Sediment Accumulation All-Season (ac-ft)	Sediment Accumulation Seasonal (ac-ft)	Percent Difference
Q2_40-69	831,140	827,960	0.38%
Q3_10-39	696,303	695,090	0.17%
Future Baseline	671,746	666,353	0.80%

c. Results of Streamgage Data Analysis. Global climate change will affect the volume and timing of runoff and sediment load into reservoirs. The Garrison basin wide streamgage analysis of the effects of climate change indicate that:

(1) All climate change scenarios resulted in an increase in sediment loading and inflows due to changes in the timing, precipitation, and snowmelt.

(2) Climate adjusted runoff and sediment yield will increase the rate of storage depletion in the Garrison Reservoir.

(3) Seasonal shifts in the sediment discharge relationship are apparent in USGS gage data, however, long-term sediment accumulation projections are not sensitive to these seasonal differences.

(4) The impacts from changing sedimentation rates on flood storage capacity and regulation would be minor for this large mainstem reservoir given its geologic and geomorphic conditions, though the hydrologic changes could potentially be significant.

(5) Precipitation is more influential than temperature. Considering only precipitation by subtracting the sedimentation rates of projection Q1 (warm and wet) from Q4 (warm and dry), and the rates of projection Q2 (cool and wet) from Q3 (cool and dry), an annual average of 4.8 million tons of additional sediment enters the reservoir in wet scenarios than in dry. When considering only temperature, using the same technique, an average annual decrease of 0.3 million tons of sediment enters the reservoir in warmer scenarios than in cooler.

9. Little Missouri River Sub-Basin Study.

The initial evaluation illustrated the potential climate change impacts for the entire Garrison Dam drainage basin based on projected reservoir inflows and sediment-flow relationships. A smaller sub-basin-scale evaluation was performed on the Little Missouri River to assess the impacts of climate change on sediment yield when the precipitation is not evenly distributed, and variables including soil type, land use, slope, and vegetation are considered.

a. Drainage Basin. The Little Missouri River drains approximately 7% of the incremental Garrison Dam drainage basin. It enters Lake Sakakawea above Watford City, North Dakota, well into the length of the lake. The river is highly turbid with much of its sediment supply coming from eroding stream banks and the terrace faces of the badlands topography in its watershed. The gaged sediment contribution of the Little Missouri is slightly higher than the Missouri River even though the contributing sediment area of the Little Missouri is 8,310 mi^2 compared to the Missouri River's 34,000 mi^2.

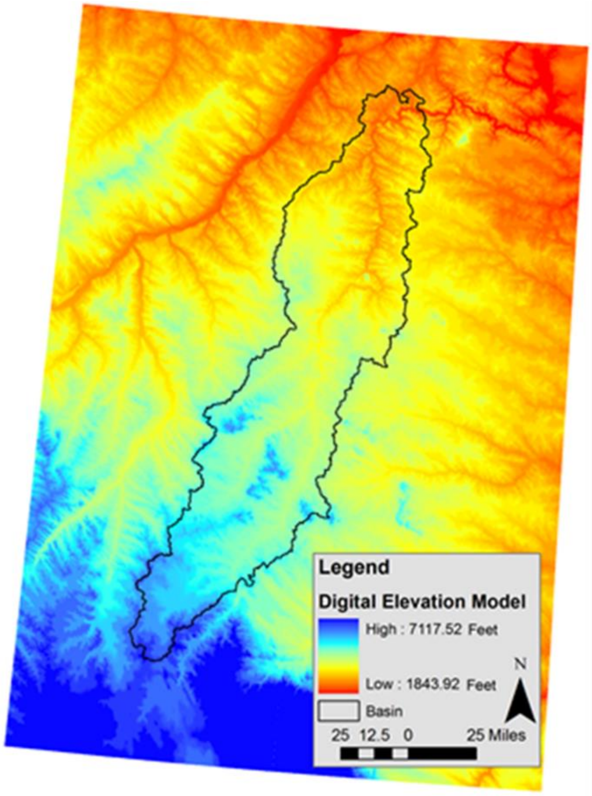
b. Climate Evaluation Method. The climate effects on the Little Missouri River sediment yield drainage basin were evaluated using the Soil and Water Assessment Tool (SWAT). SWAT is a physically based continuous simulation watershed scale model that incorporates varying soil, land use, and management practices found in the watershed.

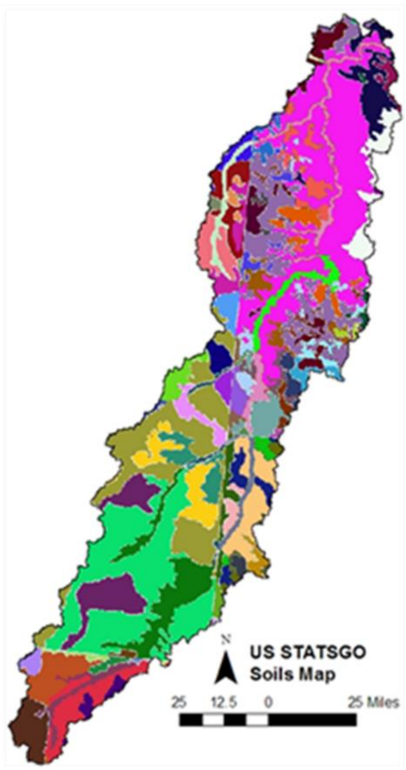
(1) SWAT requires specific information for weather, topography, and vegetation. ArcSWAT, version 2009.93.7b, is an ArcGIS extension. ArcSWAT provides a graphical user interface for the SWAT model and allows for the use of DEMs and land use and soil maps to model the watershed. ArcSWAT divides the watershed into sub-basins and hydraulic response units (HRUs) where areas of similar soil, slope, and land use are grouped together within each sub-basin for calculations. Several of the hydrologic processes can be modeled using different algorithms within ArcSWAT, the selected methods are listed in Table 10 (modified from Hotchkiss et al., 2000). The DEM, and the layout of sub-basins and HRUs are illustrated in Figure 10.

(2) The large size of the Garrison Dam drainage basin and the considerable SWAT model data requirements limited the practical basin areal extent. Therefore, the Little Missouri River basin was selected for SWAT modeling due to the size of the watershed and the data availability. Note that ArcSWAT uses the metric system for all input and output and the units differ in this analysis from the rest of the study.

Case Study 8C Table 10
Hydrologic Processes Used in the Little Missouri ArcSWAT Model

Process	Algorithm
Precipitation	Observations or First-Order Markov Chain Model
Surface Runoff	SCS Curve Number Method
Channel Routing	Variable Storage Coefficient Method
Snowmelt	Mean Air and Snowpack Temperature, Melting Rate, and Areal Snow Coverage
Lateral Sub-surface Flow	Kinematic Storage Model
Groundwater Flow	Baseflow Period, Groundwater Storage, and Re-evaporation
Transmission Losses	Lane's Method
Evapotranspiration	Hargreaves Method
Sediment Yield	Modified Universal Soil Loss Equation (MUSLE)





Top Left: The Little Missouri River Basin DEM.

Top Right: The Watershed Subbasin Delineation with the Weather Station Locations.

Bottom Left: The US STATSGO Soils Maps

Case Study 8C Figure 10. Maps of the Little Missouri River

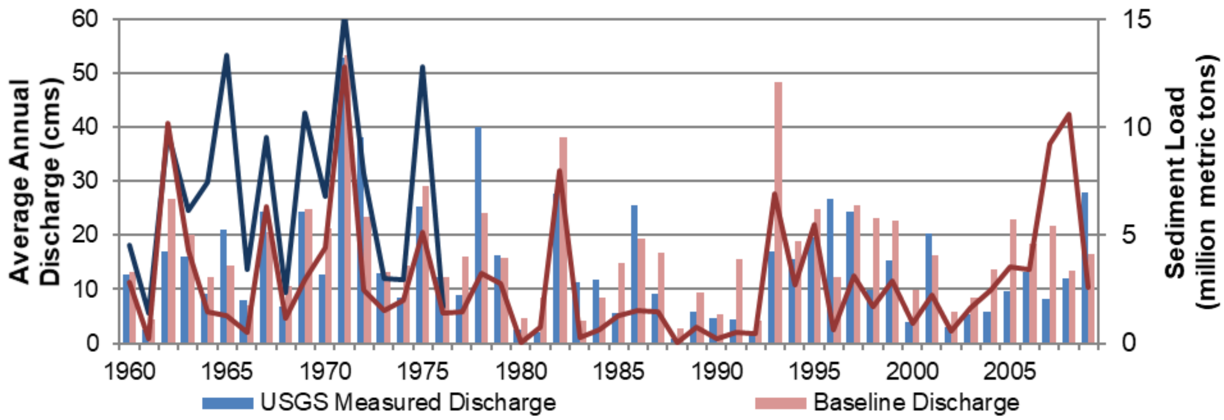
c. Model Calibration. Limited calibration was performed using the available historic data. The ArcSWAT model was limited in its ability to accurately reproduce USGS observed data. Model limitations likely result from a combination of factors including the sparse precipitation and temperature data (such as lacking isolated thunderstorms), data breaks which required generation from regional averages, imitated sediment yield data for the basin, and the unique nature of the Little Missouri River region which is strongly influenced by spring snowmelt and the Badlands geology.

d. Calibration Results. The ArcSWAT model created a flashier system with significantly higher peaks of shorter duration than the actual system. The snowmelt peak flows were consistently underestimated while the summer flow was overestimated. The ArcSWAT model underestimates the annual sediment load by approximately half of the expected value. The average annual and monthly discharges and sediment loads measured by the USGS streamgage are compared to the baseline model output in Table 11 and Figure 11.

Case Study 8C Table 11
Comparison of the Baseline ArcSWAT Model and Observed Mean Monthly Discharge and Sediment

	USGS Measured Data[1]		ArcSWAT Baseline Model		% Difference	
Month	**Discharge, cms**	**Sediment Load, metric tons**	**Discharge, cms**	**Sediment Load, metric tons**	**Discharge**	**Sediment Load**
1	0.37	2,109	2.05	6,681	458%	217%
2	6.83	41,659	10.04	66,463	47%	60%
3	50.69	773,388	24.46	335,848	-52%	-57%
4	41.06	1,132,186	19.13	202,314	-53%	-82%
5	21.64	1,743,638	34.33	813,604	59%	-53%
6	29.74	1,766,318	54.49	1,107,657	83%	-37%
7	13.96	517,467	24.27	172,928	74%	-67%
8	5.92	343,103	8.16	36,343	38%	-89%
9	4.39	100,427	9.43	48,026	115%	-52%
10	4.45	382,476	11.68	82,226	163%	-79%
11	1.87	9,172	4.95	23,898	165%	161%
12	0.57	366	1.66	7,274	193%	1,890%

[1] Based on USGS reported statistics of monthly mean data at Watford City, North Dakota

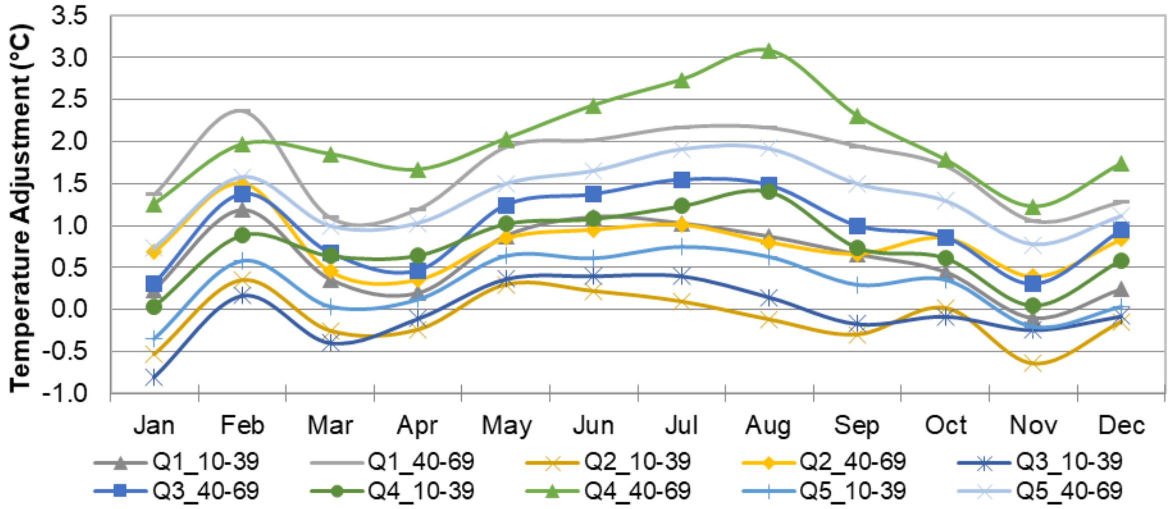


Case Study 8C Figure 11. Average annual discharge and total sediment load; observed and baseline model data. The bars represent water discharge while the lines represent sediment discharge; note daily suspended-sediment data at the Watford City streamgage were collected only up to 1976.

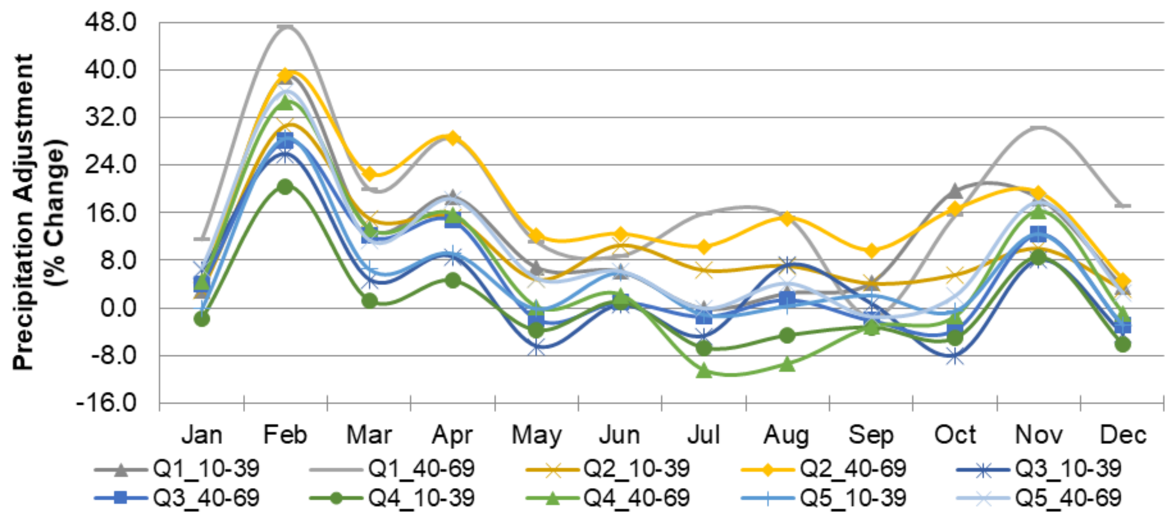
e. Evaluation of Climate Scenarios. Although the baseline ArcSWAT model does not reflect the observed conditions of the Little Missouri River basin well, the baseline model may be used to identify the effects of the projected climate changes in a generic watershed by comparing the baseline and climate change scenarios.

(1) Modeling the Climate Scenarios with ArcSWAT. Historic weather data from six climate stations were downloaded from the National Climatic Data Center (NCDC). Maximum and minimum daily temperature and daily precipitation data for the period of 1 January 1950 to 31 December 2009 were downloaded. The NCDC ground weather stations Watford City 14S cooperative station identification (COOPID 329246), Medora (COOPID 325813), Marmarth (COOPID 325575), Ekalaka (COOPID 242689), Albion 1 N (COOPID 240088), and Hulett (COOPID 484760) were selected to represent the watershed. To fill in any missing data, the built-in ArcSWAT weather generator was used to produce appropriate values for solar radiation, wind speed, and relative humidity, along with any missing data in the rainfall and temperature datasets according to regional weather trends.

(2) Adjusting Precipitation and Temperatures. The ArcSWAT model allows users to set adjustment factors for precipitation and temperature data to aid in climate change studies. The precipitation data is adjusted by month using a specified percent change; for example, setting the precipitation factor as 10 for January will make rainfall equal to 110% of the original daily value. Temperature can also be adjusted by degrees Celsius for each month. To simulate the five future climate scenarios, the adjustment factors shown in Figure 12 and Figure 13 were applied to monthly temperature and rainfall data, respectively.

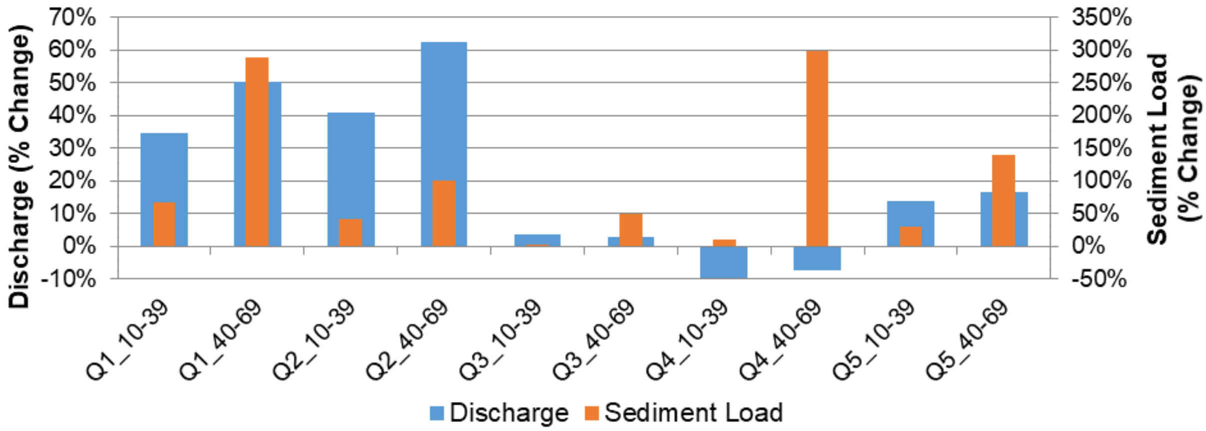


Case Study 8C Figure 12. Temperature adjustment factors for the ArcSWAT model Climate projection key: Q1-warmer and wetter, Q2-cooler and wetter, Q3-cooler and drier, Q4-warmer and drier, Q5-median precipitation and temperature condition



Case Study 8C Figure 13. Precipitation adjustment factors for the ArcSWAT model Climate projection key: Q1-warmer and wetter, Q2-cooler and wetter, Q3-cooler and drier, Q4-warmer and drier, Q5-median precipitation and temperature condition

f. ArcSWAT Model Results. The ArcSWAT model was run for each climate scenario and a baseline scenario modeled using historic NCDC precipitation and temperature data. Insight regarding sediment yield from the Little Missouri River basin may be used to assess the general effect of climate change on the sedimentation rates in Lake Sakakawea. The percent difference between the model baseline and each of the climate-projected average annual discharges and sediment loads are displayed in Figure 14.



Case Study 8C Figure 14. Percent difference between the baseline and projected average annual discharge and sediment load Climate projection key: Q1-warmer and wetter, Q2-cooler and wetter, Q3-cooler and drier, Q4-warmer and drier, Q5-median precipitation and temperature condition

g. Summary. Observations from the comparison of the ArcSWAT results include:

(1) The distant future models for the period 2040 to 2069 have a similar trend when compared to the near future models for the 2010 to 2039 period, with a more exaggerated result as expected with the continued changes from current conditions.

(2) Projections with increased expected precipitation (Q1 and Q2) produced the highest discharges.

(3) All scenarios show an increased sediment load entering the reservoir regardless of the fact that not all projections had an increased inflow.

(4) As expected, the Q4 (warmer and drier) scenario produced the lowest discharge. However, the sediment load increased significantly, 300%, from the baseline condition.

(5) Increased sediment yield is not only a function of increased precipitation, but the condition of the soils (for example, soil moisture) and land cover (such as vegetation), both of which are strongly influenced by seasonal climate conditions, are also important factors.

(6) Seasonal timing shifts affect the sedimentation rates. The timing of precipitation, which influences precipitation type and intensity, is a critical factor in addition to the precipitation quantity.

10. Lessons Learned.

The lessons learned through the performance and results of this study include:

a. Even climate scenarios with less precipitation can result in increased reservoir sediment load inflows due to changes in timing. This finding was evident in the streamgage-based analysis as well as in the results of the ArcSWAT model.

b. The ArcSWAT sediment yield analysis reinforced the findings of the sediment rating curve-based flow and sediment load analyses. All climate projections will increase the sedimentation rate in Lake Sakakawea.

c. Sediment impact evaluations based on storage change and streamgage data do not reflect runoff timing and vegetation change, which can affect basin sediment yield.

d. Reservoir sedimentation rates are typically highest following closure of the dam and decrease over time until they level out. The sedimentation rates in Lake Sakakawea, as measured by the area-capacity surveys, continue to decrease, making the assignment of the current sedimentation rate difficult.

e. Further study could be performed focusing on climate-induced vegetation changes in a basin and its influence on sediment yield.

Case Study 8D
Reservoir Sustainability: Evaluation of Climate Change Impacts to Reservoir Management Operations at Coralville Dam, Iowa

1. Case Study.

This case study provides content condensed from studies performed by the Rock Island District to evaluate climate change effects on reservoir operation at Coralville Dam, Iowa. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

This study evaluates the use of future climate projections to assess the potential impacts of climate change on the operation of a USACE multi-purpose reservoir in east central Iowa. The Coralville Reservoir, on the Iowa River just above Iowa City has been in operation since 1958. The two largest floods during the period of operation have occurred in the last 25 years, with the largest occurring during the Midwest Flood of 2008.

a. Climate conditions in the Iowa River basin have changed significantly since the reservoir was placed into operation. Analysis of historical precipitation and flow data demonstrate increased reservoir inflow volumes compared to pre-project conditions upon which the project was originally designed. Observed changes in reservoir inflow have resulted in periodic modifications to the water control plan; however, the threat of continued climate change in the future and the uncertainty associated with those changes has the potential to result in increased future risks to meeting project purposes.

b. Using a calibrated hydrologic model of the Iowa River basin and dynamically downscaled climate data, the risk to the reservoir system associated with future climate scenarios was analyzed. Reservoir operations for a number of future climate scenarios were simulated in order to test the robustness of the reservoir system to potential climate change effects and to identify potential adaptation strategies.

c. The study concludes that the numerous limitations associated with climate and hydrologic modeling makes it difficult to fully assess the risks for a project due to climate change using modeling tools alone. A project-based resilience-robustness approach that considers the vulnerabilities of the project to changes in climate, such as the approach by Brown et al. (2011), gives a better picture of the climatic risk for a project. Specific to reservoir management, this study concludes that long-term reservoir planning is not as valuable a tool in meeting the missions of a reservoir as is short-term weather forecasting and a framework that allows for real-time, risk-based, decision making for reservoir operations.

3. Background.

a. Study Area. Coralville Dam (Figure 1) is a 1,400-foot-long, 100-foot-high rolled earthfill dam impounding Coralville Reservoir on the Iowa River, located 83.3 miles above its confluence with the Mississippi River and 5 miles above Iowa City. There are 3,115 mi^2 of mainly row-cropped agricultural land draining into the Iowa River above the dam. An additional 9,400 mi^2 of uncontrolled drainage (below Coralville Reservoir) flows from the Iowa-Cedar watershed to the Mississippi River.

(1) The primary authorized project purpose is flood risk management for areas below the lake on the Iowa and Upper Mississippi Rivers. Other congressionally authorized purposes include low-flow augmentation, fish and wildlife management, and recreation. Construction on the dam began in July 1949 but was delayed by the Korean Conflict. The reservoir began operation in September 1958.

Case Study 8D Figure 1. Coralville Dam during Midwest Flood of 2008

(2) The reservoir is regulated by a gated conduit outlet with a discharge capacity of 20,000 cfs at full flood control pool (712 feet NGVD). At pool elevations above full flood control pool, the emergency spillway is activated, and uncontrolled release begins. The 500-foot-long uncontrolled concrete chute spillway has a discharge capacity of 244,000 cfs. The spillway has been activated twice in the history of the project, once each during the 1993 and 2008 floods.

(3) During normal (non-flood or drought) operations the reservoir is regulated to maintain a seasonal conservation pool elevation (see Table 1). During flood operations, the release schedule for the reservoir changes based on forecasted pool elevations (storage utilized) and downstream constraints to control flooding. When the pool elevation is forecast to exceed elevation 707 feet (NGVD), major flood operations are initiated, and flows are regulated to maximize use of the remaining storage.

(4) During non-major flood operations, maximum releases are controlled by downstream constraints, including seasonal limitations due to agricultural production and river stage control points on the Iowa River (at Lone Tree and Wapello, Iowa) and the Mississippi River (at Burlington, Iowa). Additionally, releases are temporarily reduced to manage flash flood flows at Iowa City.

(5) When reservoir inflows fall below the required minimum conservation release, the reservoir's drought contingency plan is activated, providing for low-flow augmentation of releases. During this time, the highest priority is given to meeting downstream water supply requirements.

Case Study 8D Table 1
Coralville Lake Seasonal Conservation Pool Elevations

Date	Regulation (Elevation ft NGVD)	Action Purpose
15 Feb–20 Mar	Lower from 683 to 679	Increase storage for spring snowmelt
20 Mar–20 May	Hold elevation 679	Duration of spring snowmelt period
20 May–15 Sept	Hold 683	Storage for low-flow augmentation
15 Sept–15 Dec	Hold 683–686	Increase in lake area for migratory waterfowl
15 Dec–15 Feb	Hold 683	Storage for low-flow augmentation

b. Current Climate. Iowa City, just downstream of Coralville Dam, has a mean annual temperature of 50 °F and averages about 34.9 inches of precipitation per year (cumulative data since 1893 from Iowa Environmental Mesonet AgClimate data). Near the headwaters of the Iowa River basin is Northwood, Iowa, which has a mean annual temperature of 44 °F and averages about 32.2 inches of rainfall per year. The climate across the basin is generally homogeneous as it lacks significant topography to affect precipitation and temperature patterns. The basin has a humid continental climate, which is characterized by large seasonal temperature differences including hot, humid summers and cold, sometimes frigid, winters.

(1) Average annual temperature, total annual precipitation, and the number of days per year with precipitation have increased in Iowa from the late 19th to the early 21st century. At the Iowa City gage within the Iowa River basin, these trends are statistically significant at 95% confidence. Since 1893, mean annual temperature has been rising at an average rate of 0.32 °F per decade at Iowa City. Prior to 1960, only 6 years out of 67 (9%) measured a mean annual temperature at or over 52 °F, but 1960 and later, 24 of 52 years (46%) have met or exceeded that threshold.

(2) In Iowa, the biggest changes in temperature are due to wintertime and nighttime temperature increases. There are more frost-free days per year (about five more at the start of the 21st century than in the mid-20th century, and about eight to nine more than at the beginning of the 20th century). Warmer temperatures increase the length of the growing season, due to fewer days of frost. There is also earlier seasonal snowmelt, and lakes and streams remain frozen for less time. There has been a decrease in the number of extreme high temperature events (days above 100 °F). Increased summer precipitation and soil moisture have suppressed surface heating and reduced daytime summer maximum daytime temperatures.

(3) From the report by the Iowa Climate Change Impacts Committee (2010, p. 9): "If Iowa were to experience a severe drought, as has occurred frequently in the past, the slow and steady rise in statewide annual mean temperature, now masked in summer by moist surface

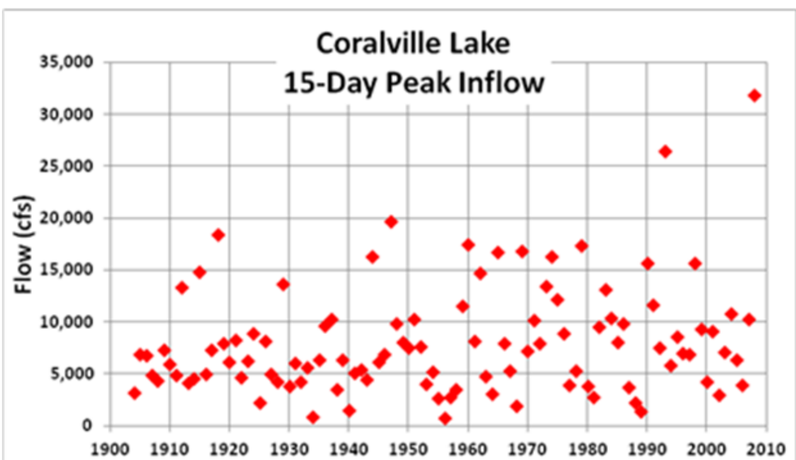
conditions, could lead to an abrupt switch to extreme summer heat comparable to the summers of 1983 or 1988."

(4) On average, annual total precipitation has been rising by 0.43 inches per decade. There has been an increase in year-to-year variation in annual total precipitation as well, with an increase in 30-year coefficient of variation (CV) in annual precipitation from around 0.11 to 0.17 in the early 20th century to around 0.19 to 0.24 in the early 21st century.

(5) On average, there has been one more rainy day per year every 6.4 years. While currently there are not as many rainy days as the late 1940s, total annual precipitation has increased steadily, which is due to a combination of more rainy days and increased frequency of moderate to intense rainfall.

(6) Streamflow is largely driven by rainfall, although for any one event, antecedent conditions play an important part in runoff-generating processes. Over time there has been an increase in average annual streamflow volume on the Iowa River as well as an increase in annual peak discharge. The 15-day peak discharge past Marengo is an important inflow metric for operations at Coralville Reservoir on the Iowa River, and its trend is shown in Figure 2. There is a clear increase in the average annual 15-day maximum flow, as well as an increase in the interannual variation for that parameter.

c. Current Problem/Concern. Historical Iowa River flows into Coralville Lake show an increase in the mean and variance of annual 15-day peak discharge between the design period (pre-reservoir streamflow records) and the period over which the reservoir has been in operation (1959 to present).



Case Study 8D Figure 2. 15-day peak discharge, Iowa River at Marengo

(1) Of particular significance are the floods of 1993 and 2008, both of which exceeded the largest historical event upon which the original water control plan was developed. The largest historical floods, available in the record at the time of project design, were predominately spring snowmelt (or rain on snow) events, whereas the record flooding in 1993 and 2008 resulted from persistent late spring and summer thunderstorms occurring over a heavily saturated watershed.

Increased total precipitation has led to higher soil moisture content, which has runoff implications both through affecting antecedent soil conditions preventing infiltration, and an increase in the installation of agricultural tile drains.

(2) The floods of 1993 and 2008, coupled with significant flooding in 2010, raised questions regarding the operation of Coralville Reservoir and (from the public's perspective) whether the reservoir was giving adequate weight to the risk of urban flooding from a major discharge event versus favoring protection of downstream agricultural areas during minor discharge events. Public and community interest led Iowa's Governor to formally request that USACE conduct a re-evaluation of the water regulation procedures at each of the four large flood risk management reservoirs operated by USACE in Iowa.

(3) Uncertainty in future climate conditions has the potential to be a major risk driver in the evaluation of alternative water management strategies to better manage future flood risk in the Iowa River Basin.

4. Purpose and Scope.

The study is concentrated on the following central question: "How do we incorporate climate change considerations into reservoir operating policies that will be robust and adaptive, incorporating the interest of long-term management of risks into project operating purposes?"

a. Previous Studies. This pilot study is the first attempt at evaluating the potential effects of climate change on the operation of Coralville Lake or on hydrology in the Iowa River basin. However, other studies have been completed that evaluate the regulation plan for the lake in response to past floods. In 1997, a Section 216 (Review of Completed Works) study was completed for Coralville Lake and its regulation plan. Several alternative initiatives were proposed to enhance benefits at the lake, but none garnered Federal interest.

b. Methodology and Approach.

(1) The study utilized the following approach:

(a) Investigate original design assumptions for the dam and determine which metrics are sensitive to climate change.

- Evaluate changes in meteorology from historical to potential future.

- Examine possible bias or error in Global Circulation Models (GCM)/Regional Climate Models (RCM) results.

(b) Obtain downscaled climate data for the Iowa River basin, the area of interest.

(c) Run observed meteorology and downscaled climate scenarios through a calibrated hydrologic model to obtain flow information at critical locations for a variety of scenarios.

(d) Use post-processing tools to learn more about the effects of changes in climate and hydrology.

- Reservoir sedimentation model: How is storage in the reservoir changing due to sedimentation?

- Flow routing model: How are the operational conditions for the dam changing?

- Reservoir operations model: How much influence does operation have on the possible changes at the reservoir?

(2) The first step in evaluating the potential impacts of climate change for the reservoir was to understand the design parameters and assumptions upon which the original project design and water control plan were based. Using the design documentation and regulation manuals for the project, critical design parameters and assumptions were tabulated (see Table 2). These parameters serve as guidance on whether or not the project is currently functioning as intended, and if these assumptions might be violated in the future due to climate change. Tools were developed to answer the question of whether or not these parameters might be sensitive to changes in climate in the future.

Case Study 8D Table 2
Design Parameter Matrix

Design Parameter	Original Design Assumption	Observation During Operations
Frequency of uncontrolled release over emergency spillway	Uncontrolled release would occur about once in 30 years	2 spillway events since 1958 (~54 years, about 27-year average interval)
Sedimentation/loss of storage space in reservoir	Loss of storage would occur at a rate of about 750–1,200 ac-ft/yr	Average yearly loss of approximately 1,700 ac-ft
Timing/mechanism of annual flood flows	Heaviest floods would occur due to spring snowmelt and flood magnitude would be related to amount of snowpack	Largest floods occurred during the late spring or early summer due to persistent and intense thunderstorm events (1993, 2008)
Spillway design flood/dam safety	The dam was designed with freeboard above a probable maximum flood computed from the transposition of a historical storm during worst case operational conditions, with a peak inflow of 326,000 cfs (top of dam elevation NGVD 743)	Dam has never been overtopped; max pool elevation ~717’ (~26’ freeboard)
Conservation pool storage volume	Maintain minimum discharge of 150 cfs at Iowa City and Lone Tree from 07/01 to 02/28 (243 days) with strong drought conditions; equating to a volume of 17,000 ac-ft	Due to sedimentation, the elevation of the conservation pool has been increased in order to maintain design volume

(3) Climate change is highly visible in its impacts on hydrology. Changing climate conditions affect the water balance by directly changing the amount of evapotranspiration and precipitation, and timing and type of precipitation that occurs. In order to assess these impacts quantitatively, the climate simulations were coupled with a hydrologic model of the study area.

c. Hydrologic Model. The hydrologic analysis was performed using a quasi-distributed continuous hydrologic model, SWAT (Neitsch et al., 2011). It was forced using observed meteorological data and RCM-downscaled results from GCMs. No land use change scenarios were tested for the future cases. The minimum inputs to run SWAT include a digital elevation model, land use/land cover, soil type and meteorology. Model input sources are listed in Table 3.

Case Study 8D Table 3
Iowa River SWAT Model Input Sources

Input	Source
Land use/land cover	NLCD 2006 (MRLC)
DEM	1 Arc second NED (~30 meter resolution)
Soil coverage	STATSGO data for the United States included with SWAT model

(1) Meteorology inputs for the model came from a variety of sources in order to have a long enough record of all required forcing variables to calibrate the model. Observed meteorology was necessary to calibrate the model to observed streamflow. Once the model was calibrated to match historical rainfall-runoff responses, the model was run with downscaled climate data to evaluate the effect of climate change on hydrology. USDA-ARS SWAT format meteorological data were used in calibration and observed meteorology runs. The data provided from this source were daily maximum and minimum temperature and daily total precipitation. These data span January 1950–October 2009. Relative humidity, solar radiation, and wind speed, in addition to temperature and precipitation, were from Iowa Environmental Mesonet data available over the time period January 1998–December/2010.

(2) The model was first calibrated for daily discharge at the Marengo, Iowa gage using historical observed meteorological data. Observed flow and meteorological data are at a daily timestep, and thus the model was run at a daily timestep. Calibration results are summarized in Table 4.

Case Study 8D Table 4
SWAT Model Calibration Results

Event	Location	Nash-Sutcliffe	Volume Error	R^2
Calibration (1999–2001)	Marengo	0.85	+5.7%	0.87
Validation (2006–2008)	Marengo	0.80	-7.9%	0.84
Validation (2003–2005)	Marengo	0.64	-0.51%	0.75

(3) While achieving relatively good scores on the selected calibration metrics shown in Table 4, one significant weakness of the model is in estimating the highest peak flow values. The model was unable to capture the most extreme flows and the relatively large variance in observed daily streamflow. Baseflow recession and the timing of peak flows were generally well matched to observed hydrographs; however, the volume error grew with overestimation of baseflow contribution and underestimation of the most extreme peak flows. Additionally, some peak flow events were missed within the simulations (and some existed in model results without corresponding observed peaks) because of the coverage of precipitation gages.

(4) The daily discharge simulated by SWAT was used in three post-processing routines to gain information about dam sedimentation and reservoir operations.

d. Sediment Accumulation in Reservoir. Although SWAT has sediment modeling methods included in the model (based on the universal soil loss equation), sparse information for calibration and other factors made it difficult to set up and calibrate the model for sedimentation.

(1) An alternative, approximate approach was favored in order to estimate sedimentation rates in the reservoir. A power law relationship between sediment discharge and streamflow modified from USBR (1987) was established using observations at the Marshalltown gage, upstream of Marengo on the Iowa River. The Marshalltown gage recorded sediment loading for a short period (less than 10 years). The curve was applied to discharges at Marengo to compute a total sediment inflow to Coralville Lake.

(2) A sediment trap efficiency for the dam based on the reservoir capacity and the inflow (Brune 1953, Dendy 1974) was applied to the Coralville inflow hydrograph to compute the amount of sediment accumulating in the reservoir. The results of this method, when compared to historical sediment survey results, are acceptable for computing an estimate of annual average sediment accumulation.

e. Flow Routing (Inflow-Pool Elevation-Release Rate Computation). An Excel spreadsheet was created that routes reservoir inflow based on the water control plan in the current regulation manual (January 2001 revision).

(1) The model first attempts to discharge enough storage to achieve the seasonal conservation pool elevation, based on the pool elevation of the previous timestep and the inflow to the reservoir. The formal rules for maximum release are checked, including seasonal rules for maximum release (growing vs. non-growing season) and flow at control points downstream on the Iowa River. The action is first checked if informal rules regarding changes in pool elevation and release rates are being broken, but major flood and drought conditions override any informal rules.

(2) The model gives good results for events where reservoir regulation stayed true to the manual. Some aspects of the water control plan occur variably from year to year based on communication with project stakeholders. The spring drawdown and the fall pool raise are variable, so the model acted on the middle date of the available range of dates in the regulation manual. In other historical cases, the reservoir was operated under a temporary deviation to store more water and avoid downstream flooding. Additionally, the model could not account for the downstream flow constraints on the Mississippi River, where river stages may dictate a short-term (7-day) reduction in releases from Coralville to reduce peak Mississippi River flooding.

f. Climate Change Scenarios. The climate data used for the evaluations in this study came from the North American Regional Climate Change Assessment Program (NARCCAP) dataset (Mearns et al., 2007, updated 2011). The data were processed and exported in SWAT format by Dr Christopher Anderson of Iowa State University.

(1) Emissions Scenario. The greenhouse gas emissions scenario used to force the GCMs in the NARCCAP datasets is the A2 scenario. The A2 emissions scenario is a high-emission Special Report on Emissions Scenarios (SRES) (Nakicenovic and Swart 2000) greenhouse gas

(GHG) scenario family. It projects vastly increased GHG emissions throughout the 21st century, fueled by continuously increasing human population, an economic policy focus (as opposed to an environmental focus), and independent, regionally focused nations.

(2) A2 Scenario Selection. Although the A2 scenario (along with the A1FI and A1B scenarios) is near the highest projected rate of GHG emissions for the early 21st century (according to the SRES), there is evidence that global GHG emissions exceed those scenarios thus far this century (Raupach et al., 2007). The emissions scenario makes up the foundational assumption about the rest of the future climate simulations. It is the driving force behind the GCM simulation and has the greatest influence on the resulting simulations. For this study, A2 was a reasonable "worst case" assumption available at the time.

(3) Global Climate Models. A GCM is a model that simulates Earth systems, generally the coupled oceanic-atmospheric processes (AOGCM) that most characterizes climate. The coupled circulation models for atmosphere, land, ocean, ice, etc. are referred to as Global Climate Models.

(4) In this study, two GCMs were used for projections, the coupled global climate model (CGCM) and Community Climate System Model (CCSM) models. CGCM is the Meteorological Service of Canada of Environment Canada coupled atmosphere-ocean climate model from the Canadian Centre for Climate Modelling and Analysis (CCCma) Climate Research Branch. CCSM is the National Center for Atmospheric Research (NCAR) coupled climate model that incorporates four separate climatological models for atmosphere, ice, land, and ocean. The version used for the runs in this study is CCSM3, which have since been superseded by CCSM4 as part of the Community Earth System Model.

(5) GCMs are generally run at a coarse scale spatially (on the order of 2° to 5° resolution) and temporally (monthly) because of computational limitations. These results are not as useful on a local scale, especially for investigations of climate change impacts on regional or local hydrology, so a method to disaggregate these results needs to be used. Thus, the GCM results are downscaled to a finer resolution, in the case of this study ~50 km resolution with a daily timestep.

(6) Downscaling Method. The downscaling method in use for the NARCCAP data is dynamic downscaling (not a delta or statistical downscaling method). Here, RCMs are forced by the GCMs to produce finer scale results. RCMs are higher resolution numerical weather prediction models that are nested within a GCM, so that the GCM acts as a boundary condition over a focused area. This allows a higher resolution simulation of local weather process that are often of most interest in understanding regional climate.

(7) For the NARCCAP data, RCM runs are also forced with National Center for Environmental Prediction (NCEP) re-analysis data for atmospheric conditions for the late 20th century, which give an estimate of the best simulation that each RCM can produce. The re-analysis data have the same fluxes and states that GCMs would produce but are based on data assimilation and atmospheric modeling over the 20th century. The data incorporates observed historical data to make a best-estimate simulation of atmospheric conditions. Thus, the NCEP re-

analysis data are a good proxy for actual atmospheric conditions that can be used to force the RCM, which, in turn, gives a better estimate of the performance of the RCM over the particular application area.

(8) The RCM runs can also produce time series of other fluxes and states (other than temperature and precipitation) that are of interest for modeling. For example, the SWAT model also needs solar radiation, wind speed, and humidity (dew point or relative humidity) data, which are readily available outputs from many RCMs. The regional climate models used for downscaling the GCM outputs in this report are the Weather Research and Forecasting (WRFG) model (developed by NCAR) and the Canadian Regional Climate Model (CRCM) model (developed at the Université du Québec in Montreal).

(9) WRFG uses the Grell parameterization scheme (superseding the WRFP, PNNL scheme).

(10) Downscaled climate simulation results are gridded, so for the purpose of hydrologic modeling, the centers of the RCM grid cells were used as gage stations. Because different RCMs have different grid schemes, the number of gages used to cover the basin varied between RCMs but there were generally at least six gages over the basin. The RCM grids are at about 50 km resolution. All six forcing variables (T_{max}, T_{min}, P, relative humidity (RH), R_s, W) were read by the model from the downscaled RCM data.

(11) It is important to keep in mind that the resulting downscaled climate datasets are highly experimental and come with their own major limitations and caveats. This study attempted to investigate the utility of these downscaled data as applied to the Coralville project.

5. Physical System/Climate Findings.

a. Climate Data and Observed Meteorology. The initial analysis of the downscaled climate outputs revealed some shortcomings in the regional climate model representation of local meteorology. Using the RCM-downscaled NCEP re-analysis data, the precipitation results were compared to observed precipitation using long-term averages. As the re-analysis data acts as a proxy for observed data in place of a GCM, this analysis demonstrates the RCM's ability to generate local meteorology.

(1) WRFG generally reproduced how precipitation occurs in the study region—the temporal distribution throughout the year was accurate, and it produced storm events consistent with those in the region. It was, however, very dry compared to observation, being low by about 7 inches of rain per year while producing about the same number of rain events (see Table 5). It appears that the model reduces the amount of moderate precipitation events that occur, resulting in frequent very light or heavy events, with few events of a more moderate intensity.

(2) CRCM performed poorly at simulating local meteorology. CRCM precipitation results were more like Seattle, with most rain coming early in the year and the annual total precipitation coming as a result of a large number of small precipitation events. Intense events were very infrequent, and the annual maximum precipitation was close to constant between years of

simulation. CRCM split the precipitation over about 200 days of precipitation a year, where 100–120 is a more reasonable number. The total water balance for CRCM was much closer than WRFG, being slightly wet by about 1 inch per year on average. Brochu and Laprise (2007) similarly documented the observed precipitation biases of the CRCM model over the Mississippi River basin and show a wet bias, as well as a misdistribution of rainfall toward the earlier part of the year.

Case Study 8D Table 5
Comparison of Annual Rainfall Statistics for RCMs Forced with Re-Analysis Data

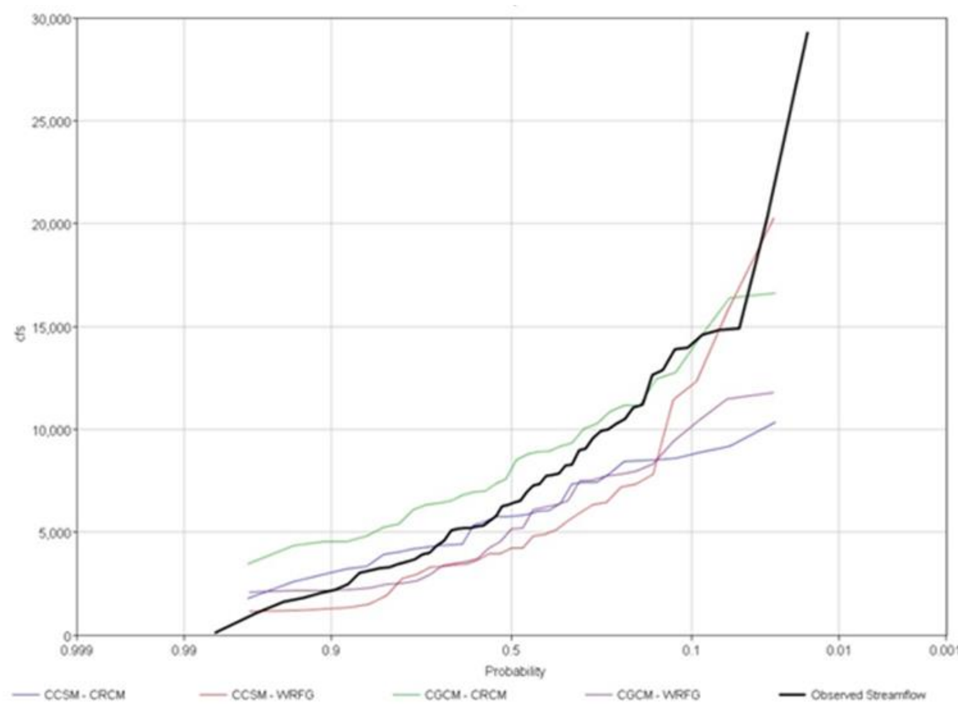
	Average Rainy Days Per Year	Annual Average Precipitation	Average Date of 50% Rainfall Accumulation
Observed	109	32.1 in	7/9
WRFG-NCEP	108	25.7 in	7/7
CRCM-NCEP	199	33.1 in	6/24

b. Future Climate Scenarios. In general, the shift from an RCM-GCM pair from historical emissions to future emissions scenario was not producing changes in extreme precipitation consistent with expectations of climate change in the Midwest. This is likely due to a combination of factors, namely the limitation of the RCMs noted above, as well as the short simulation periods. It is unreasonable to expect to sample events with average recurrence intervals longer than 50 or 100 years in a 25 to 30-year sample. The resulting data are heavily sampled out of the middle of the distribution of results, which results in very few extreme scenarios (flood or drought) that are of most concern.

(1) The underlying biases in the RCMs heavily influence the output results. The WRFG-downscaled GCM results reflect the overall dryness of WRFG, and CRCM-downscaled results have the above noted wet bias and temporal misdistribution of precipitation. Overall, the performance of WRFG was limited only by the dry bias; however, CRCM was producing results wholly inappropriate for the region.

(2) The additional limitation of the hydrologic model in simulating the highest peak events meant that climate data representing the middle of the distribution of data was being processed by a model that under-predicted variance and extremes, resulting in rather average-looking flows. This limits the ability to test the operation of the reservoir under events of the most interest (extreme flood and drought).

(3) Figure 3 shows the flow-frequency curves for 15-day peak flows for the four future scenarios when compared to observed streamflow. The reduction in variance in the streamflow results creates the reduced frequency of events observed on the tails of the inflow frequency curves. The reduction in variance is due to the forcing climate data and the spatial and temporal resolution of the data used in the hydrologic model.



Case Study 8D Figure 3. Annual maximum 15-day average flow past Marengo, observed and future model projections

(4) Table 6 summarizes the output from the sediment post-processing (annual average sedimentation rate) and the reservoir routing post-processing (amount of time in flood, amount of time in drought, number of spillway events). Spillway events are classified as any event where water goes over the spillway, even if this amount is trivial. (This designation has the habit of including some events where the elevation of the pool would likely be very close to going over without any flow being passed by the spillway.)

Case Study 8D Table 6
Post-Processed Hydrologic Model Results

RCM	Forcing	Time Period	Years	Average Daily Discharge (cfs)	Average Sed Rate (ac-ft/yr)	% Major Flood	% Drought	Spillway Events	Years With Major Flood
Observed Operations		09/17/1958–12/31/2010	52.3	2055	~1200			2	
Observed Meteorology		01/01/1999–10/30/2009	10.8	2171	1350	0.76%	0.00%	1	1
CRCM	NCEP	01/01/1981–11/30/2003	22.9	2641	1561	2.77%	0.00%	0	5
CRCM	CCSM	01/01/1969–11/16/1999	30.9	1825	909	0.24%	0.00%	0	2
CRCM	CCSM	01/01/2039–11/16/2070	31.9	1856	947	0.00%	0.01%	0	0
CRCM	CGCM	01/01/1969–11/16/1999	30.9	2737	1887	1.15%	0.02%	1	11
CRCM	CGCM	01/01/2039–11/16/2070	31.9	2700	1745	1.83%	0.03%	1	12
WRFG	NCEP	01/01/1981–12/25/2004	24.0	1318	596	0.45%	0.00%	0	2
WRFG	CCSM	01/01/1969–11/16/1999	30.9	1289	663	0.55%	0.00%	0	4
WRFG	CCSM	01/01/2039–11/16/2070	31.9	1282	711	0.34%	0.05%	1	4
WRFG	CGCM	01/01/1969–11/16/1999	30.9	1146	455	0.00%	0.03%	0	0
WRFG	CGCM	01/01/2039–11/16/2069	31.9	1813	991	1.26%	0.00%	0	5

(5) The resulting simulations did not point toward one clear consensus for the future of inflows to Coralville Lake. When examining the difference between the mid-21st century and 20th century simulations for an RCM-GCM pair shown in Table 7, there is no clear picture of the future for the system. The results for the same GCM but different RCM agreed somewhat; the CGCM results forecast an increased flood risk (increase in percent of time in major flood, and total years entering major flood operations) while the CCSM results show a slight decrease in time in major flood but also an additional spillway event.

Case Study 8D Table 7
Changes in Hydrologic Modeling Results Due to GCM-RCM Pair

Model pair	Mean discharge	% Major flood	% Drought	Spillway events	Years with major flood
CRCM-CCSM	+31 cfs	-0.24%	+0.01%	NC*	-2
CRCM-CGCM	-37 cfs	+0.68%	+0.01%	NC	+1
WRFG-CCSM	-7 cfs	-0.21%	+0.05%	+1	NC
WRFG-CGCM	+667 cfs	+1.26%	-0.03%	NC	+5

NC = No Change

(6) Considering the result of the simulations without accounting for the limitations in the data and the hydrologic model, it appears that modifications to the regulation plan would be sufficient to handle projected climate change. This is not a prudent lesson to take from the study, as the limitations associated with the climate data and the hydrologic model drive the overall results so much that there is little with which to test adaptation strategies. Those limitations suggest a cautionary approach is very important.

6. Effect on Results by Methodology.

The methodology used in the study was based largely on what was viewed as a traditional type of climate change impact analysis for hydrology, in which downscaled climate data were run through a calibrated hydrologic model for a watershed.

a. These runs were done under existing basin conditions, and the resulting climate change scenario results were compared to historical runs and observed hydrology in order to assess the impacts that climate change could potentially have on the hydrology of a watershed. The resulting climate change scenario runs were not as useful for testing the reservoir system's response as initially hoped.

b. No emergent processes were observed in the climate change simulations. The streamflow results show about what is expected in terms of increased winter rainfall resulting in streamflow and reduced spring snowmelt floods. Snowmelt flooding, which dominated the early period of record, has become less prevalent in the Iowa River basin, with the largest floods on record (1993 and 2008) resulting from later spring and early summer rains. The simulated increase in flow due to spring and summer storms is consistent with observations during the operational period of the reservoir.

7. Implications for Future Reservoir Management.

a. Large Flood Operations. The current water control plan for Coralville Lake is similar to other reservoir projects in the Rock Island District in that the release schedule limits downstream flows to safe discharges (no or minimal damage with limits tied to seasonal agricultural production) until such time that a significant portion of the flood control storage has been utilized. At this point, releases are quickly ramped up to reduce the likelihood of higher, uncontrolled releases that would result when the unregulated spillway is overtopped. The major flood release schedule contained in the current water control plan is based upon an optimization of available reservoir storage to the largest flood that had occurred prior to construction of Coralville Dam.

(1) As observed during the 1993 and 2008 major flood events, flood volumes in excess of the historically observed maximum can and will occur again in the future. The current water control plan, which emphasized optimization of flood volumes to historic events, does not necessarily optimize flood risk reduction during future major floods. In evaluating future climate change scenarios, it was anticipated that additional major flood events would be represented in the model simulations to evaluate alternative water control plans that would improve the risk performance of the reservoir across a wide range of large flood events. As discussed above, the future climate scenarios evaluated failed to produce events at the extremes of the inflow volume-duration-frequency distribution. As a result, the mid-century future climate scenarios evaluated do not provide a basis for defining a new optimized release schedule for future major flood events.

(2) The inability of the future climate scenarios to provide such a basis points to the importance of short-term climate forecasts and the need to develop tools capable of informing water managers with risk-based decision criteria to evaluate operational scenarios during major flood events. The required decision support system needs to be capable of incorporating modern forecast information into a risk-based decision tool. Such a system requires a clear set of risk-based criteria, consistent with project authorities, upon which water management decisions will ultimately be made. Tools capable of incorporating the hydrologic, hydraulic, economic, and public health and safety factors into the decision process are also required. USACE proposed CWMS National Implementation Plan would substantially develop many of these critical tools.

b. Drought/Low Flow Augmentation.

(1) Consistent with the major flood operations discussion, the future climate scenarios evaluated failed to produce events at the extremes of the inflow volume-duration-frequency distribution such that a range of severe drought conditions could be evaluated to identify improvements to the water control plan and improve the robustness of the project to meet future drought conditions. Historically, the greatest threat to meeting future conservation needs has been sedimentation of the reservoir.

(2) The future climate scenarios indicate (with one exception) that sedimentation rates are likely to increase over historical rates consistent with projected increases in precipitation and stream flow. Historically, the Rock Island District has conducted pool raises to offset anticipated sedimentation. The Rock Island District also periodically conducts surveys to re-evaluate reservoir storage. Increases in future sedimentation will likely force decisions regarding future conservation pool raises (and the corresponding reduction in available flood storage) earlier than anticipated based on historical sedimentation rates.

c. Dam Safety. Increases in temperatures and precipitation patterns from future climate change has the potential to increase maximum probable extreme event precipitation. This has major implications for dam safety if climate change results in increased probable maximum precipitation estimates. Due to the extreme nature of these design events (having an expected recurrence of approximately once every 10,000 to 100,000 years), it was expected that the 30-year blocks of future climate information do not support a direct analysis of climate change on the adequacy of the project's spillway design flood to meet future climate conditions. Continued monitoring of the trends in extreme precipitation is critical in order to detect changes in the intensity and frequency of heavy rainfall events.

8. Lessons Learned.

a. Lessons Related to the Physical System and Climate.

(1) The dynamically downscaled NARCCAP dataset was limited in its representation of hydrologic extremes (major flood or drought). This may be due to sampling error (limitation of using 30-year blocks of future climate data to evaluate extreme events having a frequency of significantly greater than once every 30 years), or limitations in the datasets resulting from climate model biases that under-represent precipitation variability. The expectation of this kind of climatic shift comes from literature and observed changes in the Midwest; however, the NARCCAP dataset was found to be insufficient to test these shifts in the system.

(2) It was observed that regional climate models may not adequately represent the local meteorology. The WRFG model performed better in terms of timing and frequency of precipitation, but overall, the results were biased on the dry side. The results from CRCM were not at all similar to local weather. Screening the RCMs prior to use would have helped guide dataset selection and allowed the team to use RCMs more "in tune" with local meteorology if they were available.

b. Lessons Related to the Methodology and Process Used.

(1) The original plan for the study was based on the expectation of greater precipitation and corresponding greater future flood risk (assumed direction of change). Consequently, the analysis was designed to answer questions specifically related to the expected outcome. The questions regarding climate vulnerabilities were too specific, and that more broad questions about these vulnerabilities are warranted. Asking, "How do I deal with greater and more frequent extremes?" is too specific and is biased by expectations about what the climate data will indicate. A broader question to ask is, "What vulnerabilities exist with my project related to future climate variability and how can those vulnerabilities be managed?"

(2) Understanding the limitations and biases of downscaled climate data would have changed the path of our study. In addition to broadening the questions that are asked of the climate data, the approach to analyzing the climate data would be determined best by first understanding the project's sensitivity and vulnerability to climatic variation and then formulating alternatives to reduce the climate sensitivity (increase robustness) of the project.

(3) Hydrologic models as tools for assessing climate change impacts have significant weaknesses, even if calibrated to the system being analyzed. The inability for the hydrologic model used in this study to simulate high peak flows made even the largest precipitation events result in moderate or moderately high flows. However, a hydrologic model calibrated to

simulating peak discharge events will not be able to capture long-term flow parameters important for other reservoir management considerations, such as sedimentation and drought.

(4) The inability of the dynamically downscaled climate data to provide a basis for developing regulation procedures to reduce risk in future major flood events emphasizes the importance of short-term climate forecasts and the need to develop tools capable of informing water managers with risk-based decision criteria to evaluate operational scenarios during an event. While this implies a level of flexibility in future water management operations that traditionally has not been built into water control plans, any such implementation needs to clearly establish the criteria by which water management decisions will be made. This is consistent with the current USACE national effort to fully develop and deploy the CWMS National Implementation Plan.

Case Study 8E
Debris Basin Design Example, F-1 Debris Basin, Las Vegas Wash and Tributary Project

1. Case Study.

The following case study information provides an example of design performed by the Los Angeles District for the F-1 debris basin in the Las Vegas, Nevada, vicinity. The design follows guidance prepared in 2000 by Los Angeles District (USACE 2000a). Although some design principles have since changed, the case study provides valuable information regarding typical debris basin design procedures and issues. However, the case study is not comprehensive of USACE study requirements. Information provided in this case study is highly condensed from the Los Angeles District design documentation.

2. Debris Basim Design Methodology.

Debris basin volume was calculated using the Los Angeles Debris Estimation Method (USACE 2000a). The study presents a method to estimate unit debris yield values for “n-year” flood events for the design and analysis of debris-catching structures in coastal Southern California watersheds, considering the coincident frequency of wildfire and flood magnitude.

a. Overview. Flood history in Southern California clearly demonstrates the debris yield hazard as one associated with singular storm events. Normal maintenance practice is to excavate immediately following a major flood event to regain storage capacity before subsequent storms occur. Such maintenance practice is essential to keeping construction costs at affordable levels and minimizing environmental effects associated with structure size.

(1) Equations were developed to estimate unit debris yield from coastal Southern California watersheds on a single-event basis. The estimation method is based on multiple linear regression analysis between measured unit debris yield and a set of physiographic, hydrologic, and/or meteorologic parameters found to influence the process of debris yield from these watersheds. The analysis is an update for the method previously developed by Tatum in 1963 using additional data (USACE 2000a).

(2) Additional methodology notes to consider prior to application are as follows:

(a) Factors such as the potential mobility of debris in storage within the watershed are difficult to quantify. Debris yield estimates are expected to vary somewhat from recorded data, depending on the stage of a given cut-and-fill cycle, the time elapsed since the last major storm occurred, and other factors. Only by application of the on-site Adjustment-Transposition (A-T) Factor can certain unquantifiable parameters be evaluated and included in the analysis of an individual watershed.

(b) The predictive yield equations presented herein were derived from recorded data for basins and dams located at the mouths of canyons. For locations that are downstream from canyon mouths, it is prudent to evaluate the sediment transport capability of the stream in the reach immediately upstream from the site to ensure that the stream is capable of transporting the estimated debris quantity. If not, then consideration should be given to using the transport capacity of the design flood as the adopted basin inflow debris estimate.

(c) Recognition of the role that human interference plays in the increase or decrease in debris yield rates is important. Destruction of channel or hillslope vegetation, grazing, homesite construction, road building, and other factors may have a substantial effect on erosion from a given watershed. On-site evaluation of these impacts, and the geomorphology and soils of the watershed, should routinely supplement application of the recommended regression equations.

(d) The procedure does not address the hazard associated with major landslides, nor those associated with overland mudflows.

(e) Applications outside the San Gabriel Mountains should be conducted with caution and should include full investigation of all available local information and thorough field inspection.

b. Data Collection and Debris Volume Predictive Equation Development. Predictive equations were developed from debris yield data obtained from a number of agencies in Southern California.

(1) The interval of debris volume measurements taken by agencies varied and were dependent on the noticeable reduction in reservoir or debris basin capacity. Measurements are taken more frequently following storm periods which yield large amounts of debris. Data interval between debris surveys varied from weeks to years.

(2) A total of 187 observations from seven watersheds were used to develop the debris volume equations. Some discontinuity exists between equations at the drainage area size juncture. When dealing with borderline cases, such as a watershed of 3.0 mi^2 in size for which both precipitation and runoff data exist, it is advised that debris yield be calculated using both equations, and the higher of the two results should be used. If recorded runoff data is used, care must be used to ensure that the runoff data is of high quality, and that the adopted peak unit runoff values are not the result of “debris flow” or landslide heightening of the recorded flow. Predictive equations are as follows:

$$\log DY = C_q \log Q + C_{rr} \log RR + C_{ff} FF + C_{da} \log DA$$

Case Study 8E
Equation 1

where:

DY = Unit Debris Volume Yield (yd^3/mi^2)
Q = Unit Peak Discharge (cfs/mi^2)
RR = Relief Ratio (ft/mi), watershed slope to debris basin, ratio of elevation difference/stream length from the basin peak to the debris basin site
FF = Fire Factor (dimensionless)
DA = Drainage area of watershed (acres)

(3) Coefficients C_q, C_{rr}, C_{da}, C_{ff} as shown in Table 1.

Case Study 8E Table 1
Coefficients for Use with Debris Volume Predictive Equation

Basin Size (sq mile range)	Coefficients			
	C_q	C_{rr}	C_{da}	C_{ff}
3 to 10	0.85	0.53	0.04	0.22
10 to 25	0.88	0.48	0.06	0.2
25 to 50	0.94	0.32	0.14	0.17
50 to 200	1.02	0.23	0.16	0.13

Debris Volume Estimate of Study Area: $DV = DY * DA * A/T$ Case Study 8E Equation 2

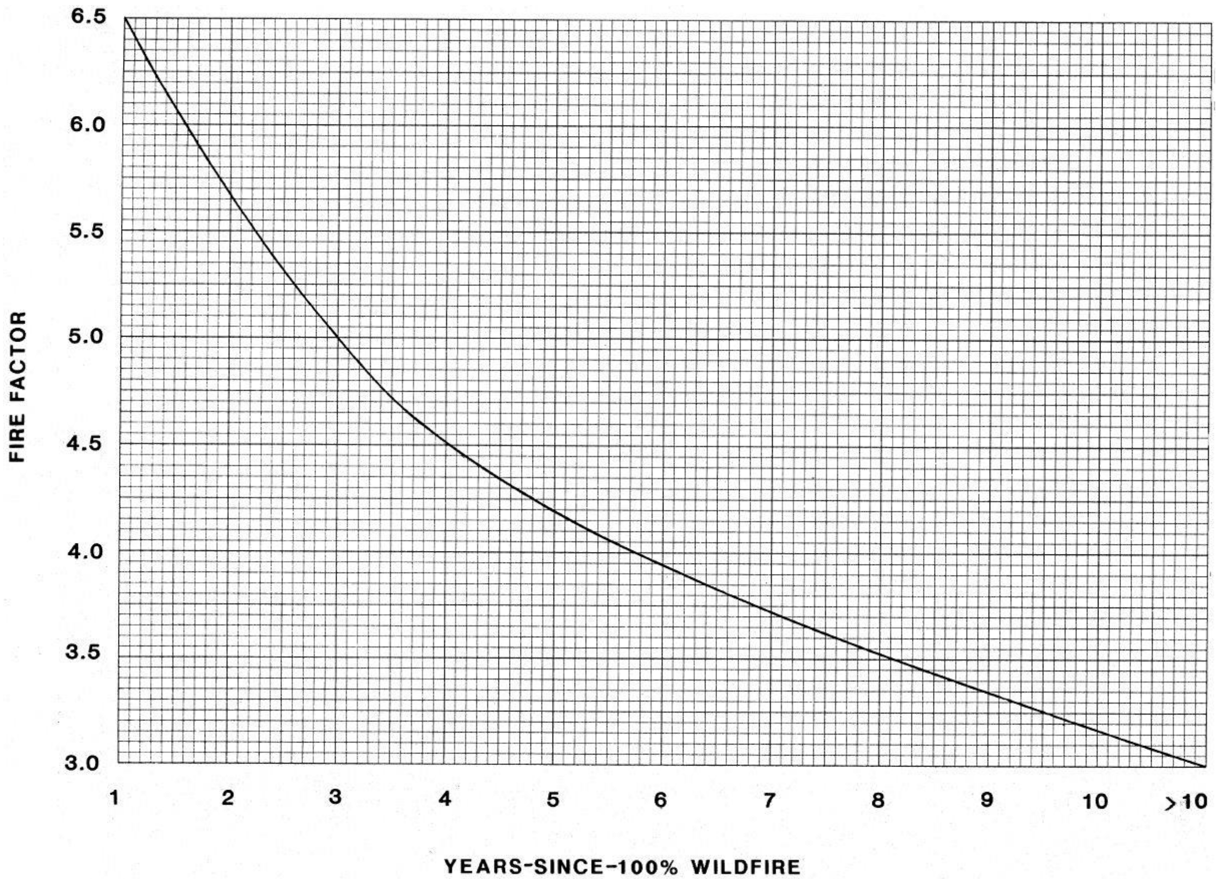
where:

DV = Debris volume (cubic yards)
A/T = Adjustment/Transposition Factor (dimensionless) which is based on quantitative assessment of the studies geomorphic, hydrologic, and meteorological characteristics to evaluate debris yield potential

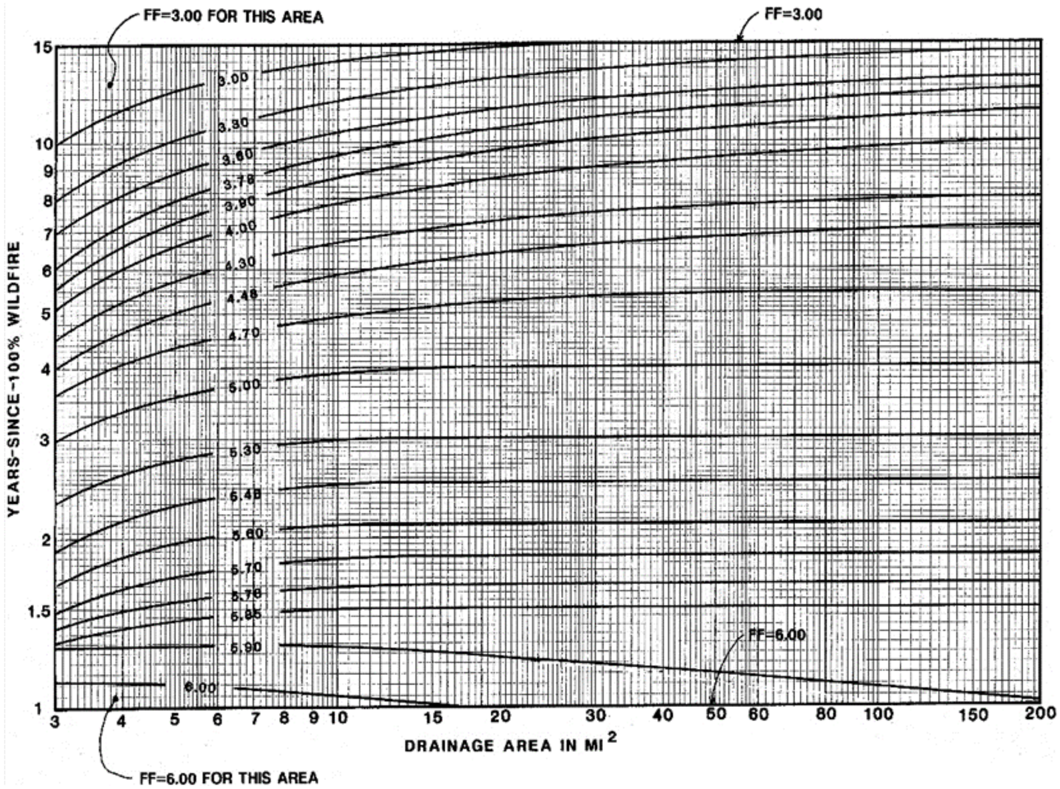
c. Fire Factor. "Fire Factor," as used in the methodology, is the name given to the relationship between debris yield and the time after burn for a given drainage basin. It is a dimensionless parameter that relates the relative increase in debris yield caused by wildfires.

(1) The occurrence of wildfire plays a significant role in the quantity of debris produced by any particular watershed. The Fire Factors developed displayed a high correlation to debris yield for watersheds in the San Gabriel Mountains. The study determined Fire Factor curves that relates the Fire Factor to the Years-Since-100% Wildfire occurrence and the drainage area of the watershed. This approach might be used after a wildfire occurrence to estimate the size of emergency debris basins or estimate sediment volumes for transport analysis.

(2) When wildfire plays an insignificant role in the production of debris yield, such as in desert watersheds where vegetal cover is minimal, the Fire Factor is fixed at a "normal" or unburned value of 3.0. Results from the study for Fire Factor are shown in Figure 1 for watersheds with a drainage area of 0.1 to 3.0 mi^2 and Figure 2 for watersheds with a drainage are of 3.0 to 200 mi^2.



Case Study 8E Figure 1. Fire Factor curve for watersheds 0.1 to 3.0 mi² (USACE 2000a)

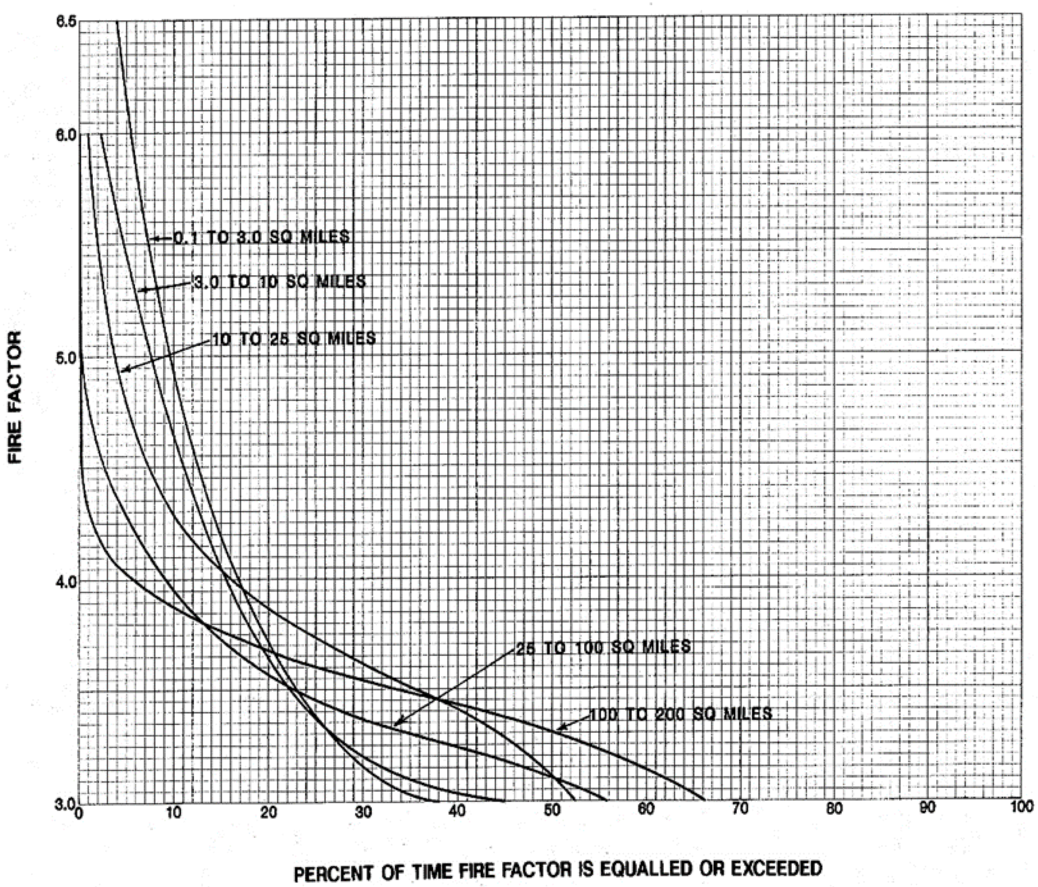


Case Study 8E Figure 2. Fire Factor curve for watersheds 3.0 to 200 mi² (USACE 2000a)

d. Fire Factor Without Fire History. In the absence of any fire history, Fire Factors and the percent of time the Fire Factor is equaled or exceeded can be obtained from the generalized fire duration curves.

(1) These curves were developed from a number of coastal Southern California watersheds ranging from Santa Barbara County to Orange County and are not meant to be used outside the principal area of application. The curves were developed by placing the watersheds into size groups (0.1–3.0 mi², 3.0–10 mi², 10–25 mi², 25–100 mi², 100–200 mi²), determining the fire history for each watershed, calculating the Fire Factors for each year in the fire history, and computing a ratio of the number of occurrences for Fire Factors to the total number of years.

(2) Discontinuities occur at the drainage area range transitions. The curves shown in Figure 3 should be used with caution in understanding data source and the limitations of applicability outside the area.



Case Study 8E Figure 3. Generalized fire duration curves for drainage areas 0.1 to 200 mi^2 (USACE 2000a)

e. Adjustment-Transposition Factor. The A-T Factor was developed to account for the difference in geomorphology between the subject watershed and the original watersheds from which the regression equations were generated.

(1) This factor considers the surficial geology, soils, and hillslope and channel morphology. Because there are few debris yield measurements available on an event basis for debris retention structures in low erosion areas, the A-T Factor was developed using readily available average annual sediment yield data. Although this factor is subjective in both development and application, there was no practical alternative that permitted quantification of these variables.

(2) Watersheds of the San Gabriel Mountains from which the regression equations were developed have an A-T Factor of 1.0. Watersheds in areas with higher debris potential would have an A-T Factor greater than 1.0, while areas of lesser debris yield capacity would have an A-T Factor less than 1.0. Four techniques are presented to calculate an A-T Factor for a specific watershed (USACE 2000a). The four methods consist of:

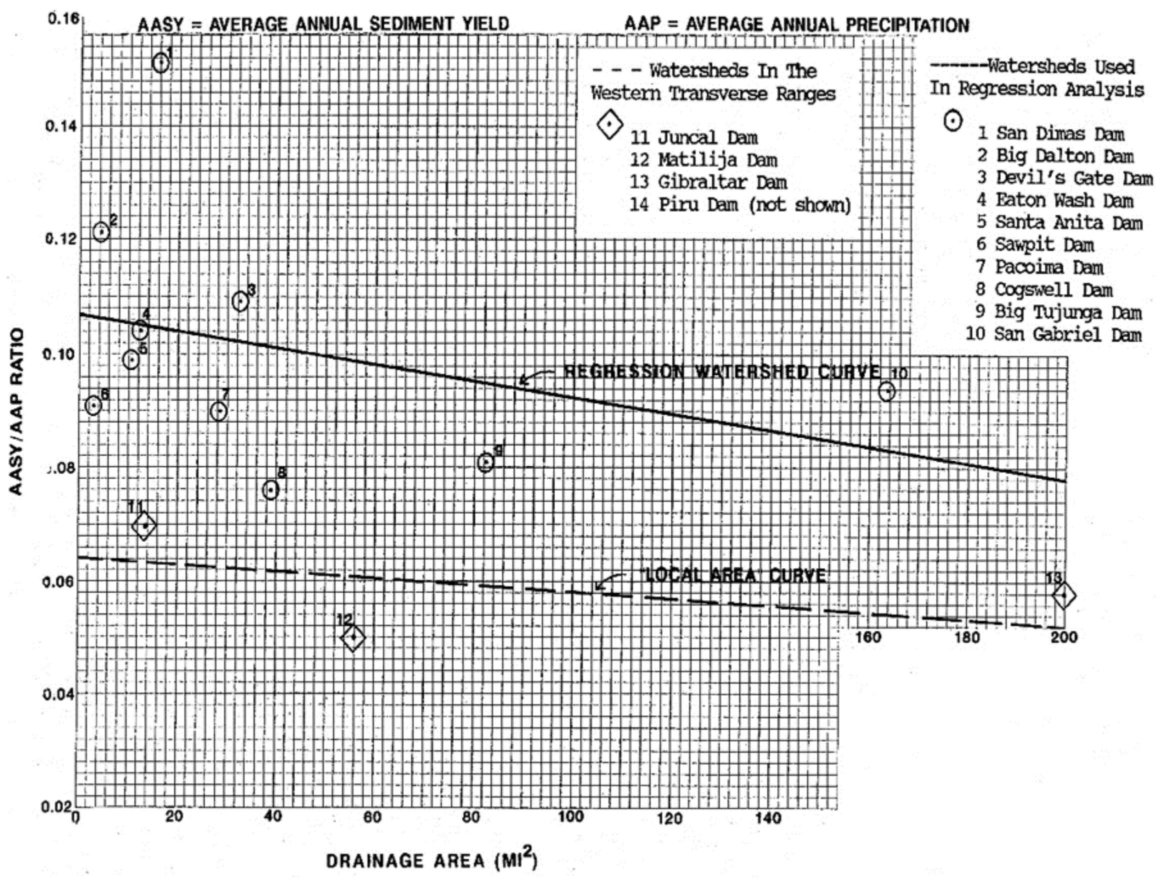
(a) Sediment/debris record for study watershed with single-event debris yield values.

(b) Sediment/debris record for study watershed contains periodic survey results only.

(c) No sediment/debris records available for study watershed, nearby watersheds have periodic surveys.

(d) No records available for study watershed or nearby watershed.

(3) Detailed explanation of information for each of the four methods is presented (USACE 2000a). An example of method 2, deriving a regression equation ratio for an equivalent watershed, is shown on Figure 4 as the regression watershed curve 10. An example of method 3, deriving a local area curve from nearby watersheds, is also shown on Figure 4. This figure provides measured average annual sediment yield data for specific sites using the ratio of the average annual sediment (AASY) to the average annual precipitation (AAP).



Case Study 8E Figure 4. AASY/AAP ratios for drainage areas 0 to 200 mi² with regression watershed curve (method 2) and local area curve (method 3) (USACE 2000a)

f. Project Frequency Analysis. The regression equations include two determined variables (drainage area and relief ratio) and two estimated variables (discharge or precipitation and Fire Factor).

(1) The magnitudes of discharge (or precipitation) and the fire condition are associated with an exceedance probability and because the two are independent of each other, any combination of the two can occur. Combination of these factors results in the determination of debris yield for exceedance frequency that can be used in basin design (USACE 2000a). These relationships reflect the debris yield for the watershed for a range of unit discharge values for each interval of the years since wildfire occurrence.

(2) The debris yield is calculated using the appropriate basin equation for the range of unit discharges using the Fire Factors associated with each Year-Since-100% Wildfire value. Table 2 provides an example of the debris response frequency relationship. This analysis can be combined with project risk to determine appropriate debris yields to use with project design.

Case Study 8E Table 2
Example of Debris Response with Years Since 100% Wildfire

Years-Since-100% Wildfire (Bi)	Unit Discharge (ft³/s/mi²)								
	Q = 1,489	Q = 1,000	Q = 719	Q = 499	Q = 288	Q = 176	Q = 96	Q = 29	Q = 2.2
	Debris yield (yd³/mi²) (A)								
1	185,417	127,559	93,585	66,325	39,612	24,877	14,025	4,515	405
2	161,677	111,227	81,603	57,833	34,540	21,692	12,230	3,937	353
— Add additional years to define relationship									
15	57,299	39,419	28,920	20,496	12,241	7,688	4,334	1,395	125

g. Risk Application. The regression equations presented provide debris yield estimates that should be considered as “expected debris yield” under a given set of conditions. Therefore, debris yield estimates used for design should include measures of confidence or associated risk. The debris method provides the standard deviation (SD) of the estimate for the various drainage basin sizes to vary from 0.242 log units and 0.484 log units (2 SD) above or below the estimate. Refer to the debris yield guidance document (USACE 2000a) for additional information regarding confidence limits.

3. F-1 Debris Basin Design Application.

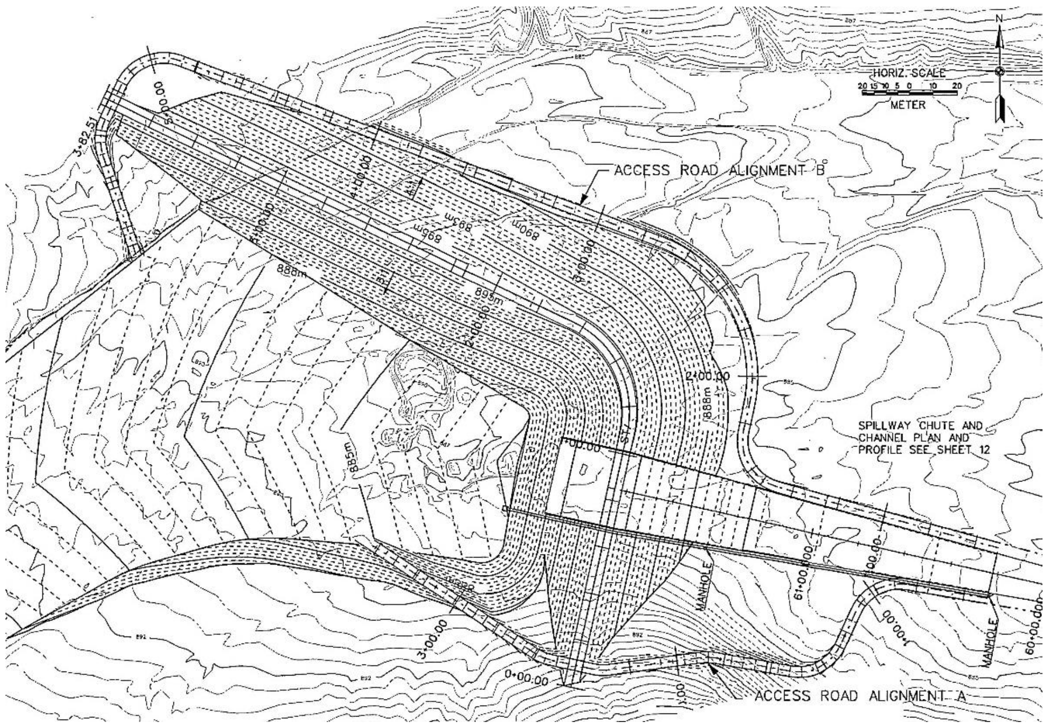
The F-1 debris basin was built as part of a flood risk management project that is described in the Design Documentation Report for the F-1 Channel and Debris Basin, Las Vegas Wash and Tributaries (Tropicana and Flamingo Washes), Las Vegas, Nevada (USACE 2003a).

a. Introduction. The project was designed to address the flooding threat posed to the Las Vegas community by the Tropicana and Flamingo Washes. The plan includes a comprehensive system of detention basins, debris basins, collector channels, and primary channels designed to provide protection for the 1% annual chance exceedance event (ACE, 100-year) to the affected areas.

(1) A component of this plan, the F-1 debris basin, was designed to capture sediment from the upstream watershed before it can enter the downstream channel. The basin intercepts sediment volume only and is not intended to reduce the peak downstream flow. The debris basin includes a dam embankment and excavation and grading within the basin. Debris basin F-1 construction was completed in 2004.

(2) The F-1 Watershed, a nomenclature carried from the overall project watershed delineations, is located east from the steep gradient Blue Mountain Hills. Outflow from the debris basin enters the downstream F-1 channel, which flows east into the Flamingo detention basin. The contributing watershed area to the debris basin is 2,963 acres (4.6 square miles). A plan view of the basin is shown in Figure 5 and primary detention basin features are illustrated in Figure 6.

Case Study 8E Figure 5. F-1 debris basin plan view (courtesy Google Earth)



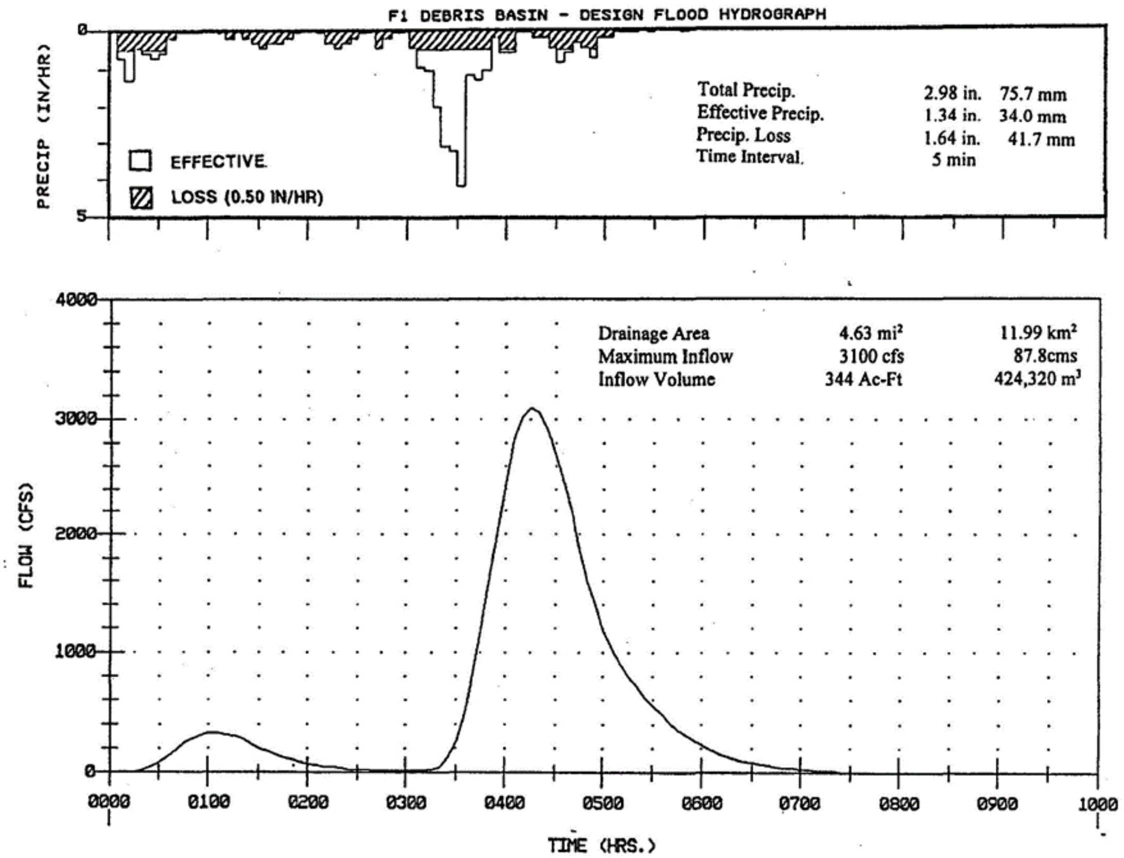
Case Study 8E Figure 6. Debris basin F-1 features (USACE 2003a)

b. Hydrology. Flood events in Clark County, Nevada have most commonly been generated by short-duration, high-intensity summer thunderstorms occurring in the mountainous and alluvial fan regions of the watershed.

(1) Precipitation depth was based on NOAA Atlas 2 (Miller et al., 1973) data set with a transposition factor of 1.19 to account for local inaccuracies driven by unique orographic effects on the area in the NOAA 2 estimates. The resulting 100-year precipitation is 2.98 inches for a 6-hour duration. The Probable Maximum Precipitation Value of 13.30 inches for 6 hours was based on the Hydrometeorological Report No. 49, “Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages” (NWS 1977).

(2) A constant loss rate of 0.5 in/hr was used to generate an effective runoff volume for the 100-year event and 0.25 in/hr for the Probable Maximum Flood (PMF) due to an expected antecedent rainfall. Rainfall transformation into runoff was determined using SCS unit hydrographs provided by the Clark County Region Flood Control District and a calculated short basin lag time of 0.78 hours.

(3) The 100-year inflow into the basin is calculated 3,100 ft^3/s with an inflow volume of 344 acre-feet. The PMF peak inflow is calculated as 22,000 ft^3/s with an inflow volume of 2,914 acre-feet. The 1% ACE inflow design hydrograph is illustrated in Figure 7.



Case Study 8E Figure 7. F-1 debris basin inflow design hydrograph (USACE 2003a)

c. Debris Volume Computation. The debris volume was estimated using the regression equation for a small watershed (USACE 2000a) with a drainage area between 3 and 10 mi^2, computed as:

$$\log DY = 0.85 \log Q + 0.53 \log RR + 0.22FF + 0.04 \log DA$$

$$\text{Debris Volume Estimate of Study Area: } DV = DY * DA * A/T$$

Case Study 8E Equation 2

where:

DY = Debris Volume Yield (yd^3/mi^2)
Q = 674, Unit Peak Discharge (cfs/mi^2)
RR = 462, Relief Ratio (ft/mi), watershed slope to debris basin, ratio of elevation difference/stream length from the basin peak to the debris basin site
FF = 3, Fire Factor (dimensionless), set at 3.0 for no fire and lack of vegetal cover
DA = 2963, Drainage area of watershed (acres)

A/T = 0.4, A-T Factor (dimensionless) which is based on quantitative assessment of the studies geomorphic, hydrologic, and meteorological characteristics to evaluate debris yield potential

DV = Debris volume (cubic yards)

d. Estimated Debris Volume. The calculated 100-year debris yield, DY, is 39,323 yd^3/mi^2 using the units for which the regression equation was developed. Applying the adjustment/transposition factor of 0.4 with the drainage basin area DA of 4.63 mi^2 (2,963 acres), results in a debris volume (DV) of 72,821 cubic yards, which equates to 45.5 acre-feet. The average annual sediment debris yield was estimated as 1/15 of the 1% ACE debris yield or 4,855 cubic yards (3.1 acre-feet). No data was available to calibrate at the time of design.

e. Sizing Basin F-1 Debris Storage Volume. As previously described, the F-1 debris basin was initially designed to store a 1% ACE single event sediment volume of 73,000 cubic yards (45.5 acre-feet).

(1) Additional storage was included for an antecedent sediment volume of five annual events equivalent to 25,000 cubic yards (15.5 acre-feet). This resulted in a total capacity of 98,400 cubic yards (61 acre-feet).

(2) Deposition slope was assumed to extend upstream from the spillway crest as ½ the natural channel slope. The height of the spillway was established in conjunction with upstream excavation.

(3) Due to unavailability of a suitable temporary disposal site used to clean the basin, the storage capacity was increased to two times the 1% ACE single event volume, for a total of 146,800 cubic yards (91 acre-feet). Increasing the basin storage capacity to provide two times the 1% ACE single event volume was greater than the risk-adjusted volume.

f. Service Spillway. The service spillway is a converging spillway chute 36 feet wide which reduces to 13 feet wide within 230 feet following a transition ratio of 20:1. Flow over the converging chute spillway was modeled with Los Angeles County’s Water Surface Pressure Gradient (WSPG) program which determines non-uniform steady-flow profiles. A sub-critical approach apron was included upstream of the chute that varied from 100 feet to 36 feet wide over a horizontal distance of 64 feet for a 1:1 transition ratio.

g. Outlet Works. The low level outlet works consist of a 3.5 foot by 3.5 foot box riser with ¼ foot diameter polyvinyl chloride (PVC) openings spaced 1 foot horizontally and 2 feet vertically, with each side of the box having openings staggered every foot.

(1) The tower has a base intake elevation of 2,893.7 feet, with the top of the tower at an elevation 0.5 feet above the service spillway crest with a grate on top of the tower.

(2) Capacity of the outlet works at the design event peak is 110 ft^3/s, which results in an estimated basin drain time of 8.5 hours. Stage-discharge relationship for the outlet tower was based on the orifice equation to each opening with head differential between the outside and

inside of the tower. The outlet conduit is a 3 foot by 3 foot box culvert cast in place with a 0.005 slope, a 205 foot length, and a design head of 27.9 feet.

h. Emergency Spillway. Unlike the standard Los Angeles District debris basin design criteria, which passes both the 1% ACE and PMF events, the F-1 basin, due to costs, was initially designed to pass the 1% ACE event with 2 feet of freeboard. A 588-foot-wide roller-compacted concrete (RCC) emergency spillway was added to pass the PMF event. The RCC overflow spillway was designed as a broad crested weir with a weir coefficient of 2.65. The emergency spillway crest elevation of 2,922.9 feet is 1.8 feet above the 1% ACE peak stage of 2,921.1 feet. The PMF peak flow of 22,000 cfs has a peak stage of 2,927.9 feet, which is 6.8 feet above the 1% ACE peak.

i. Embankment. The final embankment elevation for the basin was 2,933 feet, which is the PMF flood passed with 5 feet of freeboard, resulting in the maximum height of the embankment approximately 26 feet above the existing grade. The embankment crest is 19.7 feet wide with a 2.0% cross-slope downward toward the basin. The upstream slope is 3:1 and the downstream slope is 3:1, with a milder 10:1 slope at some locations to balance earthwork and allow for vegetation. The slope of the basin floor is 3.8%. The embankment upstream slope protection is RCC.

4. Operation and Maintenance.

The local sponsor would accomplish the operation, maintenance, periodic inspections, repair, replacement, and rehabilitation of the flood control features. Guidance was provided to the local sponsor for debris basin maintenance.

a. Access. Maintenance and inspection of the flood control facilities will be performed with vehicular equipment operating (1) on maintenance roads provided on each side of the channels and accessing the debris basin, (2) within the debris basins, and (3) on the channel invert, accessible by ramps. The maintenance roads allow circulation of trucks removing debris from the basins. For example, empty trucks may approach the debris basin on the south side of the channel, and full trucks may leave the basin on the north side of the channel.

b. Debris Removal. Debris stored in the basin is usually removed via excavation after a major storm event. Debris stored in the basin after any single flood event should not exceed 25% of the basin capacity. As debris storage is event-based, there is potentially a time period before deposition within the basin impairs debris storage function. However, desert events are historically episodic in nature, and major events may require multiple debris removal actions in any given year.

Case Study 8F
Spillway Erosion Analysis, Yatesville Dam

1. Case Study.

This case study was condensed from a spillway erosion analysis that was performed by USACE-ERDC for the Huntington District in 2010. Project information was compiled from multiple data sources within Huntsville District. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

The integrity of Yatesville Spillway was analyzed using the Water Resource Site Analysis Computer Program (SITES) (version of 2005) spillway analysis model. Consult the USDA-ARS software web page for current software (https://www.ars.usda.gov/research/software/). Five different hydraulic loadings were considered for the analyses as percentages of the PMF.

a. The study area consists of sedimentary rocks from the Middle Pennsylvanian Breathitt Group, which is composed mostly of interbedded and consolidated sandstones, shales, and coals. Considering the uncertainty of the behavior of the geologic material in resisting erosion, three sets of erosion indices were selected: lower, average, and upper bound values.

b. The analyses showed that by using the average value of material properties, the Yatesville Spillway is predicted to breach at reservoir elevation of 663.8 feet (47% PMF hydraulic loading or 1,400 years return period). Using the lower bound material properties, the spillway is predicted to breach at reservoir elevation of 657.0 feet (27.5% PMF hydraulic loading or 575 years return period). SITES calculation estimated the volume amount of erosion, which can be useful for estimating sediment transport at the downstream of the dam.

c. Several recommendations are suggested for unlined spillway remediation. The recommendations are divided into three main categories: lessening the hydraulic loading, improving the material resistance against erosion, and a combination of both categories.

3. Yatesville Project.

a. The Yatesville Dam and Spillway are located in Lawrence County, in the northeast part of the state of Kentucky, on Yatesville Dam Road. The area is approximately five miles west of Louisa, Kentucky.

b. Spillway. The Yatesville Spillway sits on a narrow ridge 2,300 feet east of the dam on the Blaine Creek Valley, a tributary of the Big Sandy River. The spillway consists of a 110-foot-wide chute, and has a crest elevation of 645 feet with a 216 foot-long access bridge. The spillway has a peak discharge of 63,000 cfs and a maximum velocity of 32 fps during a spillway design flood. The projected pool record of 644.7 was set in March of 1997 (USACE 2008b).

(1) The spillway cut-slopes (mostly shale) exhibited more deterioration than was expected during the design stages. The spillway sidewalls were lined with concrete under a construction contract in 1991 (USACE 2008b). Also, as part of this contract, a concrete sill was constructed downstream of a previous sill to prevent erosion during spillway discharge.

(2) It should be noted that the severe weathering of the shales during design of the spillway caused the over-excavation of the sidewalls and floor. This affected the concrete thickness of the sidewalls and the over-excavated rock on the spillway floor was replaced with compacted clay. As mentioned above, to prevent erosion damage, the spillway has two concrete sills. The upstream sill (Sill 1), which is 3 feet deep, was built during construction in 1988, and the downstream sill (Sill 2), which is 13 feet deep, was built in 1991 (Figure 1). Sill 1 and Sill 2 are 5 feet wide and located at approximately Station 4+00 and Station 5+40, respectively (USACE 2008b).

Case Study 8F Figure 1. Aerial view of Yatesville Dam and Spillway (googlemaps.com)

4. Site Geology.

The Yatesville Spillway area corresponds to the Adams Quadrangle. The area belongs to the Upper Pottsville and Lower Allegany Series, part of the Pennsylvanian Breathitt Group, which is the predominant formation exposed through the drainage basin. It is composed mostly of interbedded and consolidated sandstones, shales, and coals (USACE 1990c). Figure 2 gives a general overview of the geology of the area.

a. Gravel, sand, silt, and clay have been deposited along the larger streams and rivers over the last one million years, forming unconsolidated Quaternary floodplain deposits (Qal).

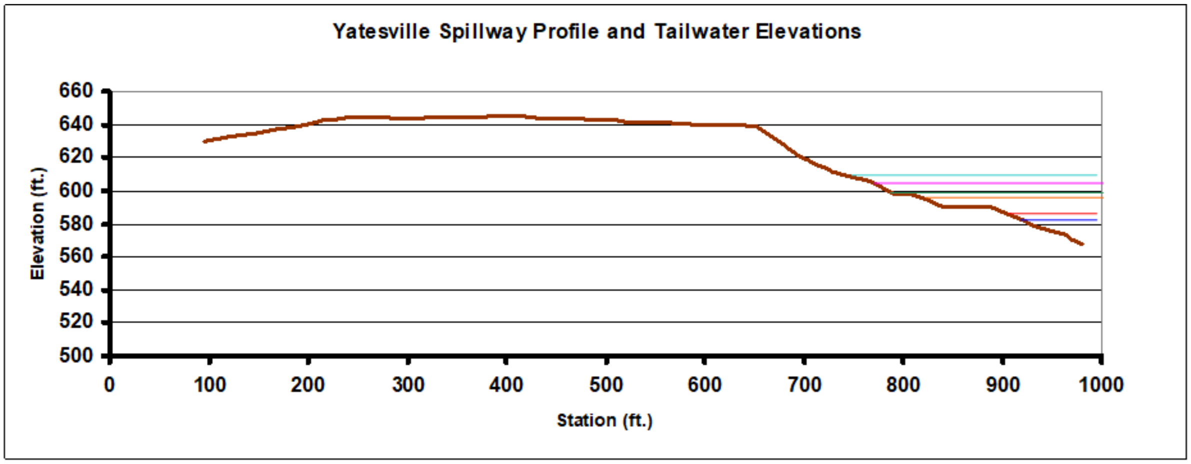
b. The Princess Formation (Ppr) is predominant in the spillway area, although most of it was removed during excavation. This formation is composed of siltstones, shales, and coals.

c. The Conemaugh Formation (Pc) seen in Figure 2 consists of sandstones, siltstones, limestones, shales, and coal beds. This formation overlays the Breathitt Group and is exposed only at higher elevations.

Case Study 8F Figure 2. Geologic map of the study area, 1:24K (Kentucky Geological Survey)

5. Spillway Geometry.

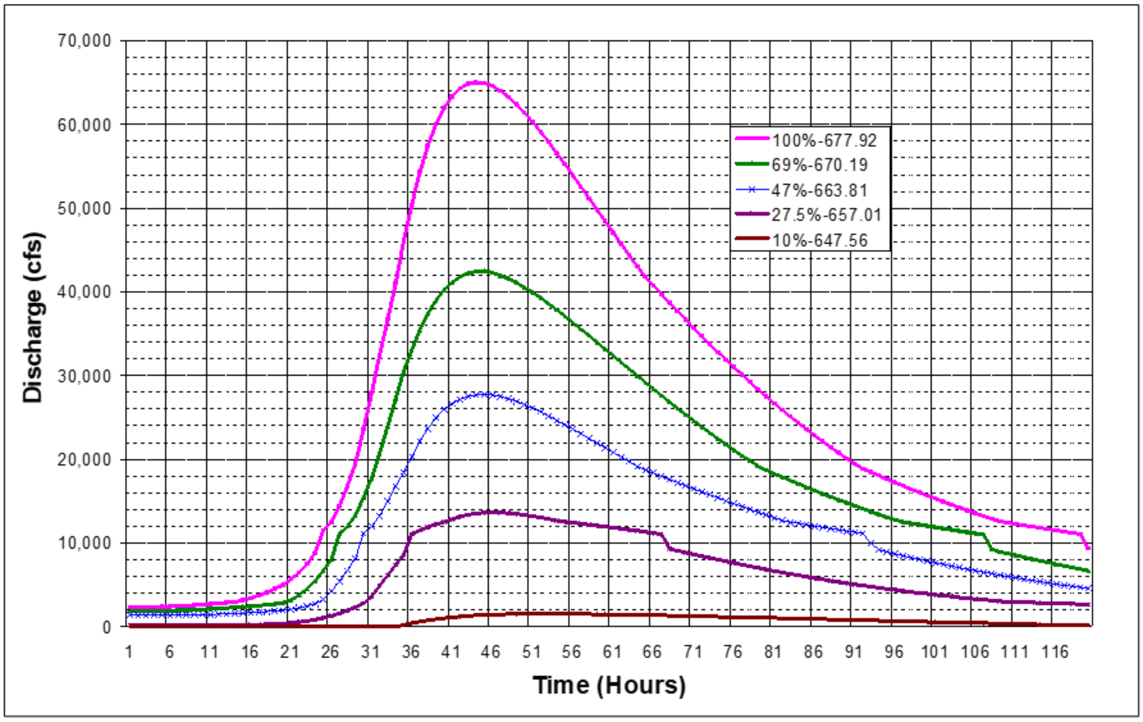
The width of Yatesville Spillway is 110 feet with a maximum elevation of 645 feet based on available plans furnished by the Huntsville District. Figure 3 shows the Yatesville Spillway longitudinal section along the spillway channel. The relatively horizontal section of the spillway has a slope of approximately 1.4° and is about 425 feet long from Sta. 225 to Sta. 650. This section is followed by a steep section with approximately 14.6° slope from Sta. 650 to Sta. 950.



Case Study 8F Figure 3. Yatesville Spillway longitudinal section with tailwater for different hydraulic loadings

6. Spillway Hydrographs.

Huntington District calculated the hydrograph of Yatesville Spillway outflow for several reservoir elevations. Five reservoir elevations are chosen to cover the elevation between spillway crest elevation (645 feet) and reservoir elevation at PMF (677.9 feet). These elevations and hydrographs represent 10%, 27.5%, 47%, 69%, and 100% PMF or 120, 575, 1,400, 3,500, and 9,500 years return periods, respectively, as shown in Figure 4 and Table 1. For the spillway analysis, the valley floor of the spillway is represented by the tailwater elevation of each hydrograph.



Case Study 8F Figure 4. The calculated hydrographs: 10%, 27%, 47%, 69%, and 100% PMF

Case Study 8F Table 1
Summary of Tailwater Elevations

Case	% PMF	Return Period (Years)	Maximum Discharge (cfs)	Reservoir Elevation (ft)	Tailwater Elevation (ft)
1	10	120	1,538	647.56	585.7
2	27.5	575	13,659	657.01	594.5
3	47	1,400	27,740	663.81	599.9
4	69	3,500	42,489	670.19	604.1
5	100	9,500	65,032	677.92	609.1

7. Determination of Material Properties in Resisting Erosion.

a. Headcut Erodibility Index. The headcut erodibility index of a material K_h, represents the ability of a geologic material to resist erosion. In 1982, Kirsten (1982) proposed a classification system for the excavation of natural materials. The index uses four geologic material properties for describing the level of excavation efforts: strength of material, block size number, joint strength, and joint orientation. In 1997, the USDA (USDA 1997, 2014) started using the excavation classification system as an erodibility index and it was incorporated into the

SITES model to predict the progress of erosion in spillways. The detailed procedure for determining the value of K_h can be found in USDA publication (USDA, NRCS 2001).

(1) The general form of the erodibility index, K_h, is:

$$K_h = M_s * K_b * K_d * J_s \quad \text{Case Study 8F Equation 1}$$

where:

M_s = material strength number
K_b = block or particle size number
K_d = discontinuity or inter-particle bond shear strength number
J_s = relative ground structure number

(2) Where data permitted, an upper, average, and lower boundary were determined to be used in the erosion analysis. This allowed stretching of the results in both directions instead of having just one scenario in the analysis process.

(3) The material strength, M_s, was calculated by conducting the unconfined compressive strength test of the 17 rock samples. These samples were selected from C 09-1 and C 09-2 cores and are considered representative of the analyzed material. Testing was done according to the ASTM Standard D 7012-14 test methods. The test results are summarized in Table 2.

(4) In general, the M_s value of medium to hard rock is the same as the unconfined compression strength in megapascals (MPa). However, this was not the case for the shale layer. Based on field and core data observations, the shale is considered to be a moderately soft rock, which, according to USDA guidance (USDA, NRCS 2001), its unconfined compressive strength values range between 5 and 12.5 MPa.

(5) The block or particle size, K_b, is the average size of the individual material block unit estimated by the spacing of discontinuities in the rock mass. It is determined using the relation:

$$K_b = \frac{RQD}{J_n} \quad \text{Case Study 8F Equation 2}$$

(6) Due to the rock quality designation (RQD) sensitivity to the length of the core run, it is recommended that its calculation be based on the actual run length used in the field (Deere and Deere 1989). The RQD was obtained directly from the core logs and their values range in quality from very poor for some shale and coal layers to excellent for sandstone layers. It is worthwhile to point out that for the most part, the lower RQD values in the sandstone are due to interbedded layers of weaker material or material loss.

(7) J_n is the joint set number, which is a representative factor that accounts for the relative occurrence of different joint sets. Based on available data, the rock mass at the spillway is

considered to have two joint sets plus random joints. One joint set is the bedding plane, and the other set is oriented N45°W. This setting corresponds to a factor of 2.24.

(8) The next component of the equation, K_d, represents the discontinuity shear strength number in a rock mass. It has been determined (Barton et al. 1974) that the shear strength of a rock discontinuity is directly proportional to the degree of roughness of the opposing faces and inversely proportional to the degree of alteration. This relationship is expressed as:

$$K_d = \frac{J_r}{J_a} \qquad \text{Case Study 8F Equation 3}$$

(9) The joint roughness, J_r, represents the degree of roughness of joint surfaces of a rock discontinuity. The condition of the joint surface at the site ranges from smooth planar to rough and irregular planar, which corresponds to a range of 1.0 and 1.5.

(10) The joint alteration number, J_a, is a factor that describes the degree of alteration of the joint walls, which is a function of the filling material, aperture width, and weathering state of the joint face. It was identified that the joint alteration varies from being tightly healed with hard, non-soft, impermeable mineral filling (0.75) to clean, open joints with relatively fresh walls only and no infilling (1.0), with an aperture width of less than 1.0 mm.

(11) The last component in the calculation of the headcut erodibility index is the J_s, which is the relative ground structure number. The relative ground structure number depends on apparent dip, ratio of joint spacing, and flow direction. According to geologic data from 1990, the spillway bedrock is oriented N40°W, with the least favorable joint set dipping 45°E in the direction of the flow (See Figure 7). Since in this case, the spillway flow is in the same direction as the apparent dip, the effective dip can be calculated using:

$$q = a - \alpha \qquad \text{Case Study 8F Equation 4}$$

where:

q = effective dip (degrees)
α = slope of spillway channel (degrees)

(12) For an apparent dip of 45° and a spillway slope of 1.4°, by using the above equation, the effective dip is determined to be 43.6°. According to joints mapped during the spillway construction, the joint ratio spacing ranges from 1:1 to 1:6 which gives a J_s value of 0.42.

(13) A more detailed calculation of material erodibility index can be found in the USDA guide for the headcut erodibility index (USDA, NRCS 2001).

b. Particle Diameter. As part of the material properties of the model, the particle diameter is needed to describe the material's resistance to erosion. For rock, the material diameter (in meters) was calculated using the equation (USDA, NRCS 2001):

$$D = \frac{10}{(105 - RQD)} \qquad \text{Case Study 8F Equation 5}$$

Case Study 8F Table 2
Summary of Parameters to Determine the Headcut Erodibility Index

Headcut Erodibility Index Data Calculation				
	Parameter	**Low**	**Average**	**High**
Shale	M_S (MPa)	5.00	8.80	12.50
	RQD	22.00	75.70	100.00
	J_r	1.00	1.25	1.50
	J_a	0.75	0.88	1.00
	J_n	2.24		
	J_s	0.42		
	K_h	37.59	124.40	309.70
Homewood Sandstone (HWSS)	M_S (MPa)	27.70	33.60	47.10
	RQD	22.00	75.70	100.00
	J_r	1.00	1.25	1.50
	J_a	0.75	0.88	1.00
	J_n	2.24		
	J_s	0.42		
	K_h	114.00	677.30	1,764.50
Coalburg Sandstone (CB1-SS)	M_S (MPa)	29.40	33.10	36.00
	RQD	46.00	68.10	86.00
	J_r	1.00	1.25	1.50
	J_a	0.75	0.88	1.00
	J_n	2.24		
	J_s	0.42		
	K_h	253.10	600.10	1,161.20
Coalburg Sandstone (CB2-SS)	M_S (MPa)	33.20	37.40	45.50
	RQD	21.00	76.90	99.00
	J_r	1.00	1.25	1.50
	J_a	0.75	0.88	1.00
	J_n	2.24		
	J_s	0.42		
	K_h	131.1	771.8	1,689.3

8. Analysis Using SITES (Later become WinDAM B).

The SITES program uses a semi-empirical approach using unlined spillway erosion case histories collected by USDA since 1988. Later on, the SITES program was evolved to a more comprehensive software, Microsoft **Win**dows-based **D**am **A**nalysis **M**odules software (WinDAM) B, which analyzes dams and spillways simultaneously and now is available as a public domain software.

a. The erodibility of the geologic materials is assessed through the threshold line developed from graphs of stream power versus material erodibility index. The stream power per unit area is defined as:

$$P = \gamma q\, S_f \qquad \text{Case Study 8F Equation 6}$$

where:

γ = unit weight of water
q = unit discharge
S_f = energy slope

(1) For overall headcut conditions, the slope of the energy grade line is not well defined and the energy dissipation rate per unit of headcut width is given by:

$$P = \gamma q H_e \qquad \text{Case Study 8F Equation 7}$$

where H_e is the drop in the energy grade line over a distance associated with the plunging action of the flow over the headcut.

(2) SITES simplifies this relation by assuming that the changing in specific energy of the flow from upstream to downstream of the headcut is small compared to the height of the headcut. The power per unit width may then be represented by the product of unit discharge and headcut height, qH.

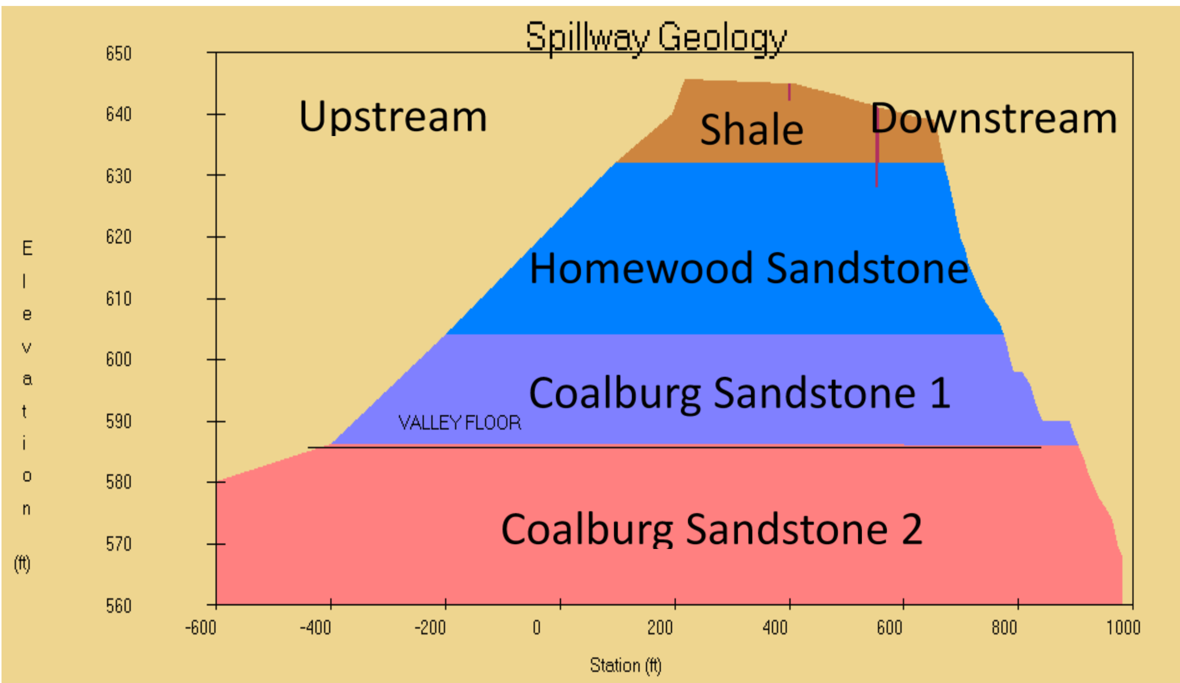
b. SITES Input Considerations. Input to SITES includes information about the structure being analyzed and information required to generate the inflow hydrograph function that represents the rate at which water flows into or out of the reservoir created by the structure. Execution of SITES generates several outflow hydrographs and other structural information, including predicted spillway erosion.

(1) Several considerations were made during the analysis. Since the dip angle in general is quite gentle, 40 feet in a mile, and considering that there is some variability of this dip angle, the geologic units' layers are assumed to be horizontal. With these assumptions, the spillway longitudinal section is considered relatively uniform across the 110-foot spillway width.

(2) The SITES program stops calculating the spillway erosion processes at the spillway crest, which is defined as two consecutive stations which have the highest and the same

elevations. According to the actual spillway geometry, the actual spillway crest is at the Station 400. Considering that the spillway erosion process will continue up to the reservoir, the longitudinal spillway geometry was modified slightly for having crest at the most upstream channel at the station about 225.

c. Spillway Section. Figure 5 below shows the ideal geological cross section along the spillway channel for the analysis. This cross section is considered the most critical section through the steep valley, about the middle of the downstream channel. The valley floors are defined as the elevation in the downstream at which the erosion process will end. Considering the erosion advancement will discontinue at about tailwater elevation, the valley floors are assumed to be the same as the tailwater elevation.



Case Study 8F Figure 5. Ideal geologic cross section along the spillway channel

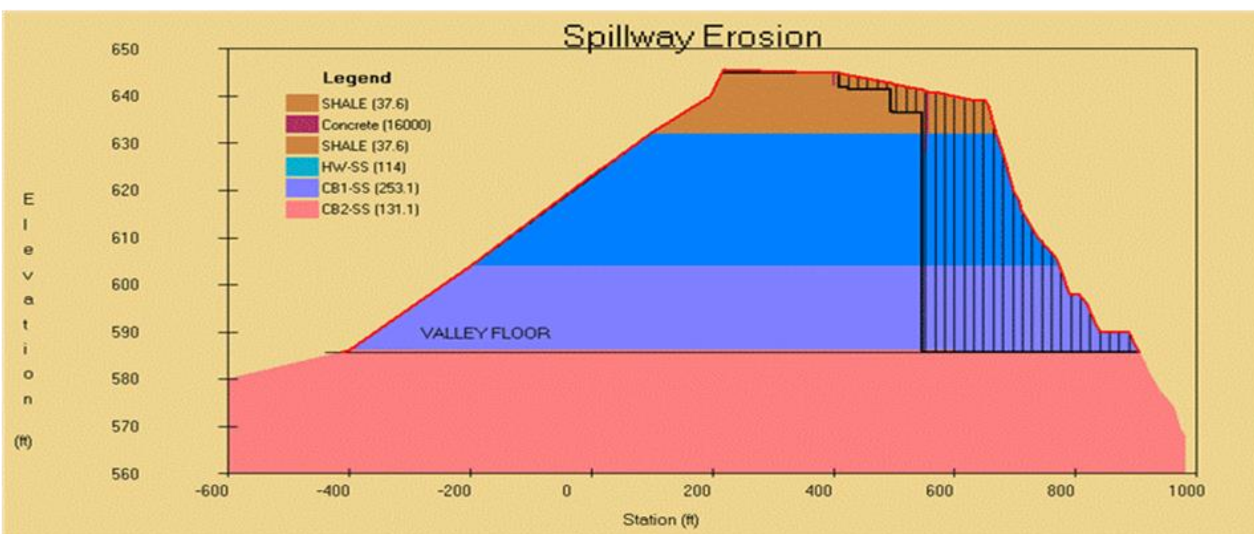
d. Spillway Channel Materials. As it was mentioned in the earlier section, the analyses consider the variability of geologic material across the spillway channel. Three values of Erosion Indices for each geologic unit are considered during the analyses: lower bound, average, and upper bound values. The values of the material erosion index are summarized in Table 3.

Case Study 8F Table 3
Summary of Material Erosion Indices

	Geologic Unit	Material Erosion Indices		
		Lower Bound	Average	Upper Bound
1	Shale	37.59	124.40	309.70
2	Homewood Sandstone	114.00	677.30	1764.50
3	Coalburg Sandstone 1	253.10	600.10	1161.20
4	Coalburg Sandstone 2	131.10	771.80	1689.30
5	Concrete Sill	16,000	16,000	16,000

e. Sites Analysis Results. The examples of SITES analysis are shown in this section for 10% of PMF loading (reservoir elevation of 647.7 feet) for lower bound, average, and upper bound values of the geologic materials.

(1) Figure 6 shows the result of the SITES analysis of the Yatesville Spillway with the lower bound values of material erosion indices. About 37.45% of the spillway is eroded. Mostly the erosion occurs at the downstream part of the channel. Under this loading condition, the analysis shows that the concrete sills help the shale layer from headcut advance erosion from the upper layer erosion. However, the shale unit upstream from the concrete sills has the potential of erosion. The shale unit downstream of the concrete sill is predicted to be eroded.

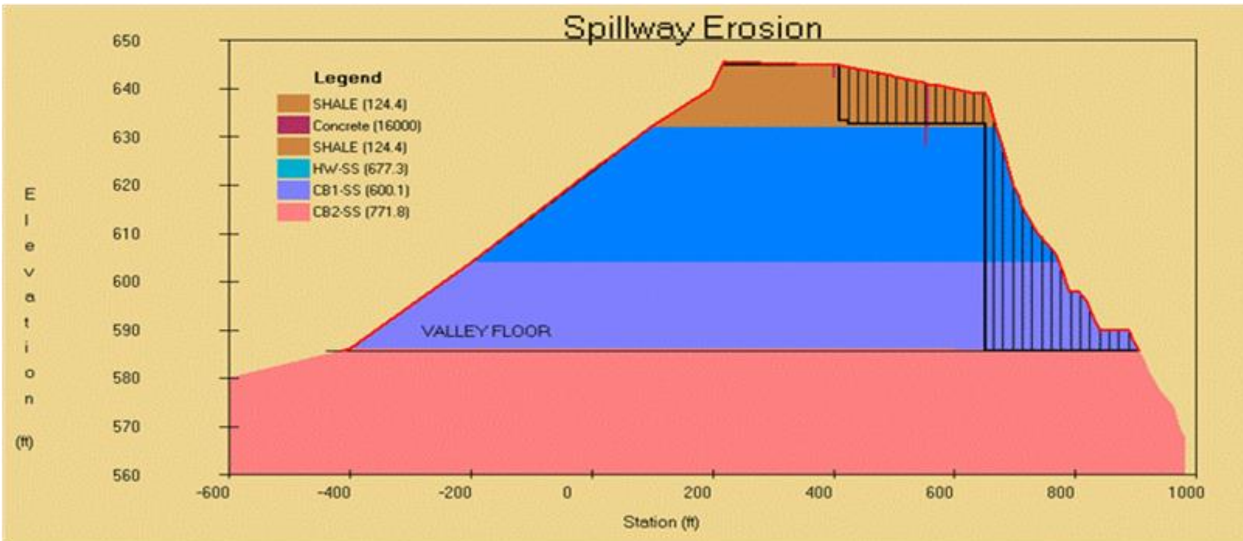


Case Study 8F Figure 6. SITES analysis output for Yatesville Spillway with lower bound of material erosion indices under 10% of PMF (120 years return period)

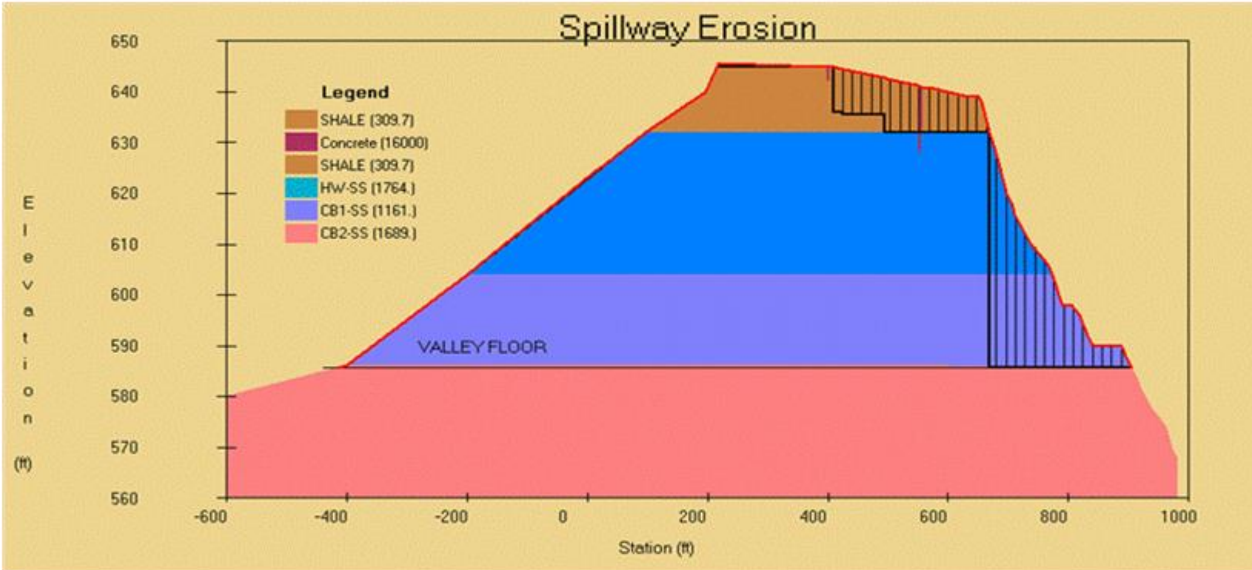
(2) Figure 7 shows the result of the SITES analysis of the spillway with the average values of material erosion indices. About 23.97% of the spillway is eroded. Mostly the erosion happens at the downstream area of the channel. The shale unit downstream of the concrete sill is eroded.

(3) Figure 8 shows the result of analysis of the spillway with the upper bound values of erosion indices. About 20.90% of the spillway eroded. The erosion occurs at the Shale and Coalburg Sandstone layer. The Homewood Sandstone resists the hydraulic attack quite well. Similar to the lower bound and average values, the shale unit downstream of the concrete sill is eroded.

(4) The complete results for 10%, 27.5%, 47%, 69%, and 100% are summarized in Table 4. The estimated the volume amount of erosion from SITES analysis can be useful for predicting the amount of sediment can be transported to the downstream of the dam during flood.



Case Study 8F Figure 7. SITES analysis output for Yatesville Spillway with average quality of material erosion indices under 10% of PMF (120 years return period)



Case Study 8F Figure 8. SITES analysis output for Yatesville Spillway with upper bound quality of material erosion indices under 10% of PMF (120 years return period)

Case Study 8F Table 4
Summary of SITES Analyses

Material Indices	**Hydraulic Loading, Years of Return Period**				
	120 years (10% PMF or Res. El. 647.6 ft)	**575 years (27.5% PMF or Res. El. 657.0 ft)**	**1,400 years (47% PMF or Res. El. 663.8 ft)**	**3,500 years (69% PMF or Res. El. 670.2 ft)**	**9,500 years (100% PMF or Res. El. 677.9 ft)**
Lower Bound	37.5% Eroded	Breached	Breached	Breached	Breached
Average	23.97% Eroded	75.8% Eroded	Breached	Breached	Breached
Upper Bound	20.9% Eroded	23.5% Eroded	66.1% Eroded	Breached	Breached

9. Yatesville Spillway Remediation.

Referring to this threshold methodology, two major components need to be addressed: hydraulic attack from hydraulic loading, and the behavior of geologic material in resisting erosion. One possible remediation method is to lessen the hydraulic attack by widening the spillway channel. The other possible method is to improve the geologic layer resistance to erosion. An additional possibility is to combine the two methods. Table 5 shows several options for mitigation methods.

Case Study 8F Table 5
Spillway Erosion Mitigation Methods

Category	Method	Area
Lessening Hydraulic Attack	Widening the spillway.	Spillway
	Deepening the middle part of spillway channel	Spillway
	Tunnel under the spillway channel	Spillway
	Additional Auxiliary Spillway	Southeast of the existing spillway
	Additional Auxiliary Spillway	Concrete spillway chute next to the main dam
	Blanket the slope part with a concrete slab with energy dissipation system to reduce the velocity	Steep slope of the downstream spillway
Improving Erosion Resistance	Blanket the spillway channel (including the downstream slope) with some type of surface protection (roller compacted concrete (RCC), Gabion)	Spillway
Combine methods	Deepening the middle part of spillway channel, and put blanket (RCC, Gabion etc.) on the spillway channel.	Spillway

10. Conclusions and Recommendations.

Significant conclusions and recommendations are as follows:

a. Assuming the average value of geologic material engineering properties against erosion, Yatesville Spillway will breach at reservoir elevation of 663.8 feet, or 47% PMF hydraulic loading (1,400 years return period).

b. Considering the average value of geologic material engineering properties against erosion, and due to 27.5% PMF hydraulic loading (reservoir elevation of 657.0 feet, or 575 years return period), Yatesville Spillway is predicted to have severe damage. About 78% of the spillway will be eroded.

c. Assuming the lower bound value of geologic material engineering properties against erosion, Yatesville Spillway will breach at a reservoir elevation of 657.0 feet, or 27.5% PMF hydraulic loading (575 years return period).

d. The volume of eroded spillway during flood can be used for predicting the amount of sediments transported to downstream Yatesville Dam.

e. Several recommendations for improving Yatesville Spillway's condition are summarized in Table 5. The recommendations consist of three categories: (a) lessening the hydraulic attack, (b) improving erosion resistance, and (c) combining categories (a) and (b).

f. Perform detailed engineering analyses for the mitigation of Yatesville Spillway deficiencies.

Case Study 8G
Spillway Erosion Analysis at Dahla Dam

1. Case Study.

The following case study information provides an example of spillway erosion analysis that was performed by the Omaha District during a dam raise feasibility study for Dahla Dam, located on the Archana River, Afghanistan. The condensed case study presents an application of challenges faced when evaluating spillway erodibility with limited data. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction – Project Dahla Dam, Afghanistan.

Dahla Dam, located on the Archana River, is approximately 34 km northeast of Kandahar, Afghanistan. Dam closure was completed in 1952. The dam is an earth fill structure, 50 meter high with a crest length of about 540 meter. Storage volume has been compromised by sediment deposition with an impact to water delivery.

a. Normal annual operation consists of unregulated spillway releases with the peak flow in March or April, followed by releases through the outlet works to meet irrigation season demand. Normal operation results in draining the pool on a nearly annual basis in the fall. Flood control is only an incidental benefit with the operation focus on water supply. There are two open-channel spillways cut through saddles in the reservoir perimeter, one 1.5 km and the other 2.2 km northwest of the main embankment, as shown in Figure 1.

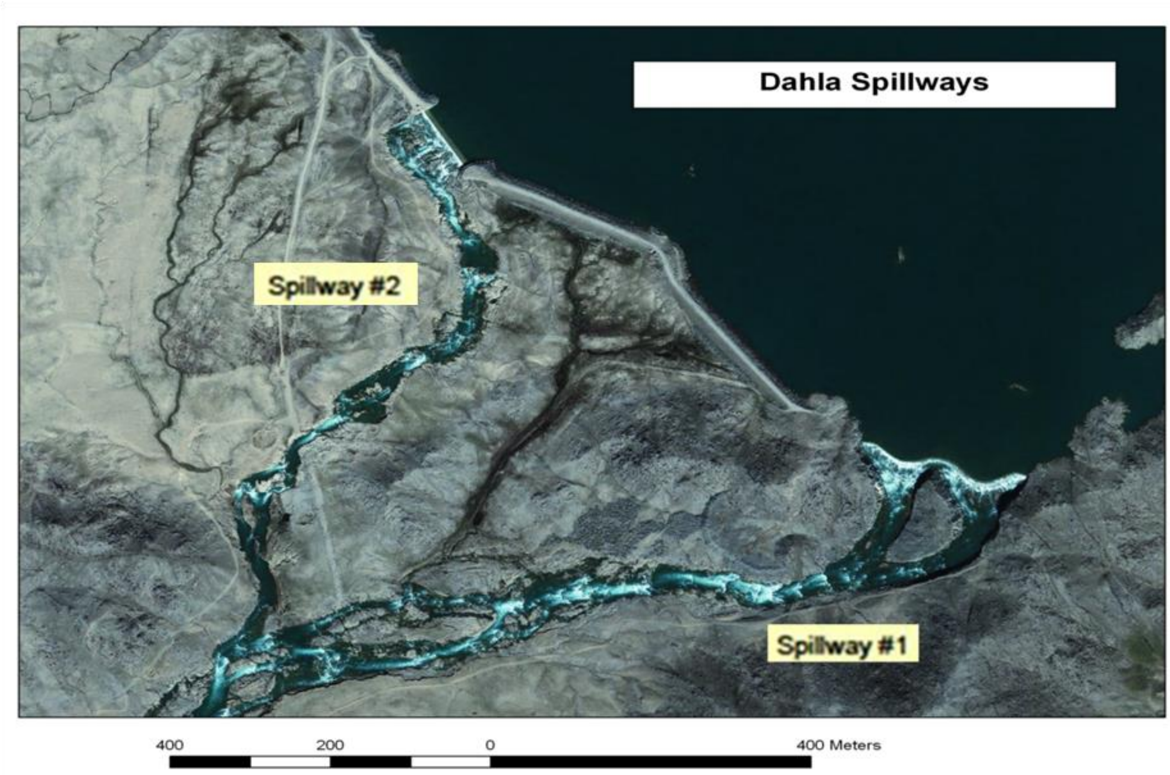
b. As part of the feasibility study, an evaluation was performed to investigate stability concerns. The study was conducted to evaluate potential changes as a result of the dam raise alternatives that were examined. The study included an HEC-RAS evaluation to establish spillway hydraulic flow parameters and a WinDAM B analysis to evaluate spillway headcut potential.

c. Each spillway has an ungated concrete weir, one 240 meter long and the other 100 meter long. They both discharge into existing natural bedrock channels that enter the Arghandab River some distance downstream of the dam. The existing spillways are open channels cut through natural material, lower elevation saddles in the reservoir perimeter located about 1.5 and 2.2 km from the main dam. The granite in the channel and at the crest is generally soft, weathered, and seamy rock. A close up view of both spillways is shown in Figure 2.

d. EM 1110-2-1602 (USACE 1980) provides guidance for energy dissipation and downstream channel protection for outlet works to prevent channel degradation, and also applies to channels downstream from spillways. The downstream rock channels have stability concerns for extreme flow events and show signs of headcutting.

e. USACE performance design objectives often include provisions for spillway channel stability measures for project operation during normal events and evaluation of performance during design flow events. An analysis was performed to examine stability of the existing spillways for current and future design flows.

Case Study 8G Figure 1. Existing conditions, spillway channel and dam location



Case Study 8G Figure 2. Spillway channels plan aerial photo

3. Spillway Existing Condition Assessment.

The existing project has two ungated spillways. Both spillways include an upstream concrete control weir, a short apron, and discharge into a steeply sloping downstream channel. Existing condition assessment results of the spillway discharge channels are summarized as follows:

a. The spillway channels are unlined native material referred to as granite in a 1950s-era report. Information regarding rock lining quality and durability is not available. Text report statements from the mid-1960s indicated some initial scouring occurred in each spillway channel followed by relative stability. No survey information or other assessment of spillway stability is available. Due to site access and security issues, there are no means of acquiring additional data for any future design phase.

b. Spillway 1, the larger of the two spillways that conveys nearly 60% of the flow, includes several elevation steps in the LiDAR data that appear to be headcuts with a maximum height of about 5 to 6 meters. The headcuts are located 100, 600, and 900 meters downstream of the crest.

c. Spillway 2 includes a headcut of about 4 meters in height located about 400 meters downstream of the spillway crest.

d. No information is available regarding headcut stability or migration rate.

e. The existing spillways have variable slope according to the original topography. Both spillway channels are steep with sloping sections of 5% to 10% maximum grade.

f. Active spillway headcuts are illustrated in Figure 3 and Figure 4.

Case Study 8G Figure 3. Spillway 1 upper reach in 2011

Case Study 8G Figure 4. Spillway headcut during low flow, circa March 2011 at unknown location

4. Spillway Proposed Plan and Operation.

The proposed project will result in a dam raise accompanied by a spillway raise of 5 to 8 meters. The current plan is to raise the spillways with an RCC structure. The RCC spillway control structure will extend upstream and include energy dissipation downstream of the crest using a short energy dissipation basin. The energy dissipation structure will be designed as a minimum, to match existing flow parameters at the basin exit.

a. The design flow event has a peak combined spillway channel flow of approximately 5,000 cms. While streamgages were not operated since 1980, the combined spillway estimated record peak flow that has occurred since project construction, derived from the gage record from 1952 to 1979 and project modeling, is about 760 cms.

b. Spillway flow duration for the design event is short, only 5 days above 100 cms. For the existing condition, the spillways flow nearly every year with flow duration usually in the range of 10 to 15 days. Maximum flow duration is estimated at 30 to 40 days.

5. Spillway Hydraulic Model.

In order to develop spillway rating curves and design the new spillway crest, an HEC-RAS model was constructed of each spillway. The HEC in Davis, California, developed HEC-RAS to calculate water surface profiles for uniform, steady-state flow using the standard step method and for unsteady one-dimensional flow simulation. Further analysis of the HEC-RAS computation model was examined for parameters such as flow velocity, flow depth, Froude number, and top width. A summary of model results is as follows:

a. HEC-RAS model results indicate extreme energy and turbulence in the spillway channels for both the historic peak and the spillway design event.

b. HEC-RAS model evaluation indicates flow velocities for the historic record peak flow (760 cms) in the range of 7 to 8 m/s and for the spillway design flow peak (5,000 cms) in the range of 10 to 12 m/s. Computed Froude numbers exceed 3 and indicate the formation of multiple hydraulic jumps within each spillway is likely for most flow rates.

c. Flow velocities are extreme for the design event. Spillway 1 velocities vary in the range of 6 to 10 m/s with a few locations greater than 12 m/s. The velocities in Spillway 2 are similar.

d. Computed spillway design event Froude numbers within Spillway 1 are in the 2 to 2.5 range with a peak near 4.5. Spillway 2 has slightly lower values in the range of 1.5 to 2 with a peak near 3.5.

e. The top width for Spillway 1 is confined in the lower reaches in the range of 100 to 200 meters for the design flow event. Spillway 2 has a slightly greater top width relative to flow, in the range of 120 to 150 meters in the lower reaches. Top width for both spillways expands at the upper end to the proposed spillway crest width.

6. WinDAM B Stability Model.

Due to the high flow parameters computed with the HEC-RAS model as previously presented, a spillway channel stability assessment was performed at Dahla Dam using the limited data available. When collecting input data and evaluating analysis capability, it quickly became apparent that result accuracy would be limited due to the lack of information. Therefore, the analysis was simplified considering only Spillway 1, using the worst-case spillway design flow hydrograph and including the existing headcuts that occur in the near vicinity and downstream of the crest in the invert profile.

a. The stability evaluation was performed with WinDAM B Integrated Development Environment (Alpha Version 2011), developed by USDA and USACE in cooperation with Kansas State University (KSU). WinDAM was developed by the USDA Agricultural Research Service in cooperation with the USDA Natural Resources Conservation Service and Kansas State University. The software is developed to route an externally developed hydrograph through a reservoir that has multiple spillways and the potential to overtop the dam. Consult the USDA-ARS software web page for current software (https://www.ars.usda.gov/research/software/).

(1) The WinDAM B software is used to determine the outflow hydrograph from a reservoir and to evaluate the potential for the dam to accept the resulting level of overtopping without failure. If a breach analysis is performed, surface protection and embankment material are evaluated against any overtopping flow to determine headcut development and progression, and resulting hydrograph.

(2) Available data was collected to provide input to the WinDAM B program. As stated above, WinDAM B has many capabilities. However, for this simplified analysis, only the Spillway Erosion Headcut Model portion of the program was utilized.

b. Headcut Model. In WinDAM B, Phase 3 of the earthen auxiliary spillway erosion process is the deepening and upstream advance of a vertical, or near vertical, headcut.

(1) Headcut advance rate is computed using a threshold rate model based on flow energy expressed in the form of the product of headcut height and unit width. WinDAM B provides the flexibility of two models, the USDA and the USACE model, for description of the relation of this energy to the rate of headcut advance.

(2) The USDA model is used in the full SITES program and described in the NRCS National Engineering Handbook (NEH) Part 628, Chapter 51 (USDA 1997, 2014). The relations used to compute the headcut erodibility index for the material are based primarily on analysis of spillways on USDA assisted flood control reservoirs.

(3) The USACE model uses simplified relations of the same general form, but with the coefficients and exponents specified by the user. This option allows experience from similar spillways to be incorporated directly into the analyses. For the Dahla Dam analysis, the USDA model option was specified.

c. Model Input Parameters. To simplify WinDAM B model input, only the headcut advance option was used with a reduced number of input parameters. WinDAM B uses English units, while the Dahla project was performed in metric units. Applicable conversions were performed for program data entry. The critical input parameters used for the WinDAM B program headcut analysis are summarized as follows:

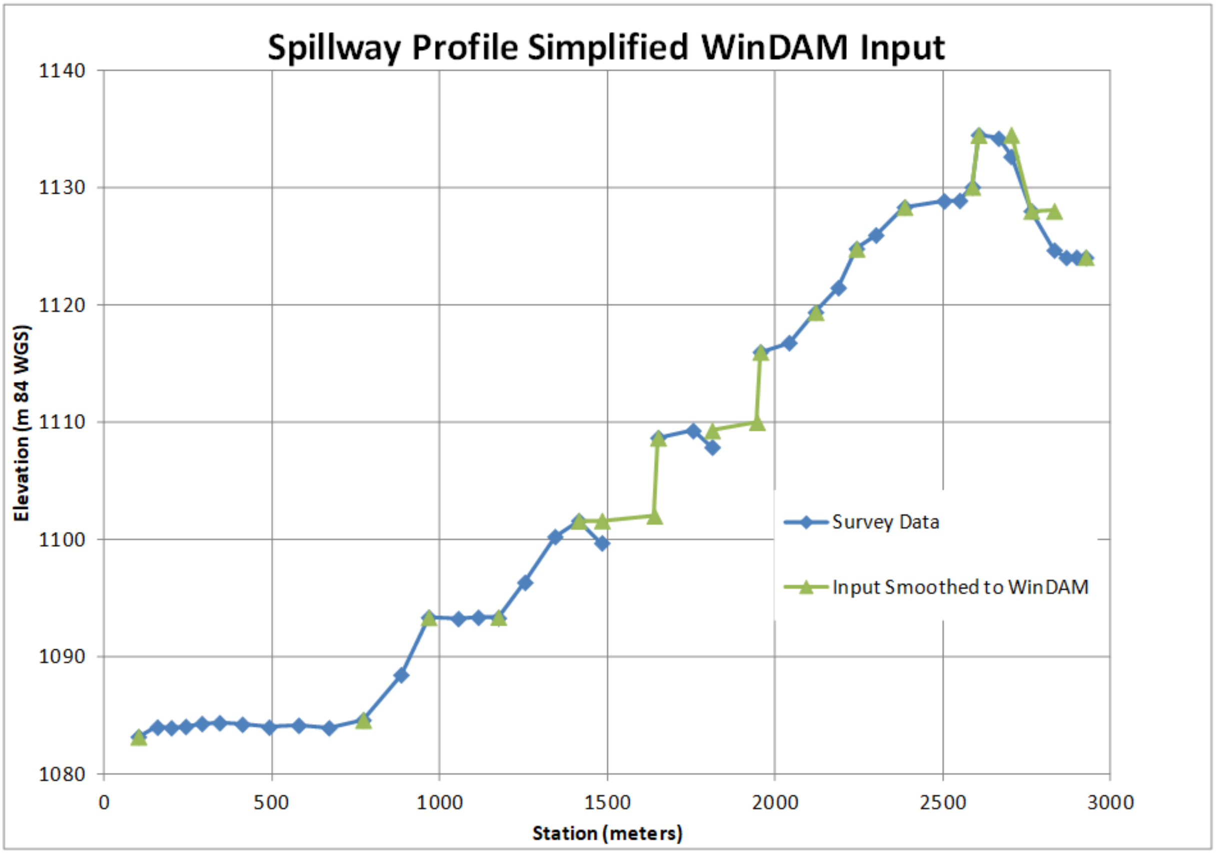
(1) Profile. The program uses an invert profile of the spillway. The survey profile input to the analysis was simplified to remove the adverse slope sections as shown in Figure 5.

(2) Spillway Material Properties – Headcut Index. The most critical program input parameter is the headcut erodibility index of the spillway material. The headcut erodibility index is a measure of the strength of the material and its resistance to headcut advance. The index takes the general form (USDA 2001):

$$K_h = M_s * K_b * K_d * J_s \qquad \text{Case Study 8G Equation 1}$$

where:

M_s = material strength number of the earth material
K_b = block or particle size number
K_d = discontinuity or interparticle bond shear strength number
J_s = relative ground structure number



Case Study 8G Figure 5. WinDAM B simplified spillway profile

(a) Values for the headcut erodibility index typically range from 0.01 for sand to greater than 10,000 for massive rock. The index may be approximated by comparing with materials for which the index has previously been determined. Materials for which values are listed in the WinDAM B Users Guide (USDA 2007c) and reference material (USDA 2001) include shales and sandstones with a value range from 20 to around 500. Other materials with more erosion resistance are much greater. For the purposes of the analysis, a value of 2,000 was used for the base condition to represent the site spillway channel rock. In the sensitivity analysis, a maximum value of 10,000 was used.

(b) Subsequent to the analysis, a site-specific analysis was conducted according to the National Engineering Handbook, United States Department of Agriculture, Ch. 52, Field Procedures Guide for the Headcut Erodibility Index, Part 628 Dams (USDA 2001). The standard quantifies the hydraulic erodibility of geologic material under various hydraulic conditions. The survey was conducted on exposed bedrock and fill material found on the spillway surface. Site parameters were tabulated by a geotechnical engineer for rock type, rock strength, rock weathering, number of joints, joint roughness, joint alteration, joint width, joint dip direction, and water infiltration.

(c) The field investigation suggests that both spillways are underlain by fresh to locally moderately weathered granitic bedrock. The bedrock is usually jointed in one direction and in both spillways, the joint width is between 2 and 5 mm that were filled with calcite.

(d) Based on the field data and methodology (USDA 2001), the headcut erodibility index (K_h) was estimated to be on the order of 9,000, indicating strong resistance to erosion and weathering under hydraulic and hydrodynamic forces.

(3) Channel Geometry. A representative bottom width was used to approximate the compound channel section. The trapezoidal channel was developed by using a flow area of 270 square meters with a flow depth of 6.5 meters. This would convert to a bottom width of 26 meters using a side slope of 2.4H on 1V. The value of 26 meters was used for the base condition with a maximum value of 200 meters used in the sensitivity analysis.

(4) Representative Diameter. The user must also specify the representative diameter (inches) for the material being described. The diameter being sought is the diameter representative of the "particle" being detached during erosion. For soil materials, use D_{75} (diameter for which 75% of the material is finer, by weight). For rock materials, use the cube root of the volume of the representative "particle." For the analysis, the base value of 0.23 meter (9 inch) was used with a maximum of 0.6 meter (24 inch) also evaluated.

(5) Spillway Design Flow Hydrograph and Peak. For the base condition, the design spillway flow hydrograph was used. Refer to the hydrology analysis for a discussion of the spillway routing used to determine the design condition spillway outflow hydrograph and presentation. For the sensitivity analysis, the combined peak flow of 5,000 cms was reduced to the historic peak combined flow value of 760 cms. In Spillway 1, the design flow hydrograph, with a peak of 3,000 cms flow (105,000 cfs), has a duration of approximately 30 hours during which flow is in excess of the historic peak flow of 760 cms. The peak flow rate occurs 27 hours after spillway flows initiate.

(6) Valley Floor Elevation. The elevation of the valley floor is used in auxiliary spillway integrity computations as the elevation below which the spillway will not erode because of downstream control. Since no information is available regarding this feature, the specified elevation was entered to be 100 meters below the spillway crest in order to allow the maximum depth headcut to form during program analysis. This variable was not changed in the sensitivity analysis.

7. Stability Results and Recommendations.

Several different combinations were evaluated as a sensitivity analysis to determine which of the input options were critical. The results of the base condition model and the sensitivity analysis using different input values are shown in Table 1.

Case Study 8G Table 1
WinDAM B Analysis Summary

Results					Input Parameters			
Cond.	Breach Time (hrs)	Breach Height (m)[1]	Headcut Dist. to Crest (m) Start	Finish	Head-Cut Index	Bot Width (m)	Rock Size (m)	Flow Peak[2]
Base	**30.5**	**Full**	**94**	**Breach**	**2000**	**26**	**0.23**	**Design**
Option 1	28	Full	94	Breach	500	26	0.23	Design
Option 2	36	Full	94	Breach	2,000	46	0.23	Design
Option 3	36.5	Full	94	Breach	10,000	26	0.23	Design
Option 4	34	Full	94	Breach	2,000	26	0.60	Design
Option 5	–	Full	94	66	2,000	200	0.23	Design
Option 6	–	Full	94	92	10,000	200	0.60	Design
Option 7	–	Full	94	94	2,000	26	0.23	Historic
Option 8	–	30	1050	1050	10,000	200	0.60	Historic

[1] Breach height refers to the maximum computed headcut height in meters. A designation of full refers to the breach height equaling the valley depth, an indicator of non-stability.

[2] Flow peak refers to the spillway design event peak flow condition, 5,000 cms combined, and the historic condition with an approximate peak of 760 cms combined.

a. Results of the stability analysis are summarized as:

(1) A spillway stability analysis using the program WinDAM to evaluate headcut indicated severe stability concerns. Model results indicate that the spillway would likely erode from the existing headcut all the way to the spillway crest structure in a single spillway design flow event.

(2) The hydraulic design of the project condition spillway is described in the hydraulics section of the design analysis. Regarding stability, the spillway project condition design was performed to generate similar flow parameters (energy grade, velocity, and depth) exiting the spillway crest control structure. As a result, stability conditions within the natural material spillway channels downstream of the structure should be similar.

(3) Modeling also indicates headcut advance for lesser events in the spillway channels.

(4) The sensitivity analysis indicated that for the design flow hydrograph, the predicted breach outcome is relatively independent of rock material and channel width. This indicates that the flow energy is excessive for almost any native material type.

(5) Although a breach is not indicated for all outcomes in the sensitivity analysis, the breach height reaches the full valley depth, in excess of 100 meters. This excessive height indicates stability is unlikely.

(6) However, the model also produces questionable results indicating concerns with model accuracy and limits for this application. For nearly all input values, the model indicates full erosion depth to the elevation specified as the valley floor.

(7) The sensitivity analysis indicates that assumptions regarding rock strength input to the model are not a significant factor in the model estimated outcome.

(8) In contrast to the model-predicted outcome is the operation history indicating spillway crest non-failure since dam closure. This indicates that model results may over-predict headcut advancement and are conservative.

b. Recommendation for a Spillway Monitoring Plan. The WinDAM B analysis indicated high potential for spillway headcut advancement. Analysis accuracy is limited by available information. Results are also offset by historical experience that the existing spillway channel is fairly stable.

(1) To assess future risk, development of a detailed spillway monitoring plan is recommended. The monitoring plan should be structured to periodically survey both spillway channels to determine headcut migration rate. The plan should also include a geotechnical investigation to check for spillway material variation and possible impact on headcut height and migration rate. The outcome of the monitoring plan will provide a basis for determining the risk of spillway headcut to current operations and a method to determine prudent actions for limiting spillway erosion if necessary.

(2) The spillway monitoring plan should include:

(a) A survey of each of the existing spillway channels, including installation of section monitoring markers and benchmarks to allow for repeat surveys with unskilled labor.

(b) A geotechnical evaluation of spillway lining material at the headcut locations and upstream to the spillway crest.

(c) Spillway level monitoring to allow correlation of flow with observed headcut migration.

(3) The monitoring and evaluation product would provide a basis for spillway headcut risk to operations. Prudent action could be taken based on the results of the monitoring.

c. Spillway Stability Summary. Dahla Dam is an earth embankment situated on the Arghandab River northeast and upstream of Kandahar City. An evaluation was performed of Dahla Dam reservoir to investigate stability concerns. The study was conducted to support the dam raise feasibility evaluation. The study included an HEC-RAS evaluation to establish spillway hydraulic flow parameters and a WinDAM B analysis to evaluate spillway headcut potential.

(1) The existing condition of the spillways exhibit signs of erosion and the formation of a headcut. The HEC-RAS analysis results show extreme flow velocities and excess energy for the spillway design event. The WinDAM B analysis determined that headcut advancement through the spillway crest is likely for the design event.

(2) Standard USACE design using EM-1110-2-1602 (USACE 1980) typically provides for extreme event spillway channel stability through provisions such as a lined spillway channel, a single or series of cutoff stabilization structures, or some other type of project feature to provide for energy dissipation and long-term stability. At Dahla Dam, no improvements to the existing spillway channel stability provided by the native rock material are proposed. This proposal is based on the following factors:

(a) The proposed dam and spillway raise project will not significantly alter the spillway flow frequency or flow duration. Initially, the additional storage will provide a slight reduction in spillway peak flow and duration. This reduction is eliminated over time as sediment deposition reduces storage.

(b) Unknowns regarding the spillway channel rock material are many. Consequently, spillway channel erosion modeling accuracy is suspect. The dam has operated in the current condition for about 60 years without spillway downstream-channel failure. Site visit discussions with the local dam tender indicated the headcuts have not moved significantly for some time.

(c) The spillway design was performed to ensure that flow parameters entering the natural spillway channel from the crest control structure are similar between the existing and proposed condition. If spillway channel headcut migration were to fail or severely damage the spillway crest control structure, the failure is not catastrophic and does not jeopardize the main dam embankment. The consequences of spillway crest lowering and pool drawdown, while potentially significant, are not dam embankment failure risk issues.

(d) In a worst-case failure scenario of the spillway crest control structure, it is reasonable to project that this failure would occur over time through multiple events due to a headcut progression. Flow release through the spillway will likely increase if the effective crest elevation is lowered by the headcut. However, the development of this condition is not projected to occur in a rapid breach manner. The downstream dam outflow hydrograph would not be significantly altered from the without-failure condition and would not significantly contribute to altered public safety or a warning time reduction.

(e) The consequence of a worst case failure would require spillway crest repair to resume optimum water supply operation. However, if repair finances from the country of Afghanistan or other international aid organizations were not available, public safety would not be in jeopardy or imminently threatened due to the spillway condition within the native rock.

(f) The design analysis will describe the existing spillway condition and a proposed monitoring plan to identify risk and current headcut movement rate. The monitoring and evaluation product would provide a basis for reviewing spillway channel stabilization measures. Although future maintenance and monitoring of the project is suspect, the plan would identify prudent action to be taken based on the results of the monitoring.

Case Study 8H
Wilson Lake Spillway Erodibility Assessment

1. Case Study.

This case study provides content condensed from studies performed by the Kansas City District to evaluate spillway erodibility at Lake Wilson. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

A spillway erodibility assessment was performed for the Wilson Lake, Kansas, emergency spillway under the Inflow Design Flood (IDF) and under the 1/25,000 flood. The spillway was analyzed using the approach set forth in Villanueva and Wibowo (2008).

a. Villanueva and Wibowo (2008) is a logistic regression approach based on output from the SITES model (USDA 1997, 2014) using 275 combinations of the following input parameters: spillway length, spillway slope, unit discharge, flow duration, and erodibility coefficient. These five parameters define the spillway geometry, hydrology, and geology (resistance to erosion). The model augments this parametric data set with data from 90 USDA and USACE case histories. Consult the USDA-ARS software web page for current software (https://www.ars.usda.gov/research/software/).

b. The output from these methods is a probability that spillway erosion falls within given thresholds for damage and the probability that the dam is breached due to spillway erosion (Villanueva and Wibowo 2008).

3. Spillway Geometric Description.

The spillway has a design bottom width of 450 feet with average side slopes of approximately 3.5:1. Based on a 2010 survey, the spillway length is approximately 1,044 feet with a 2% slope. Figure 1 provides a spillway aerial view.

Case Study 8H Figure 1. Aerial view of Wilson Lake spillway (Source: ESRI)

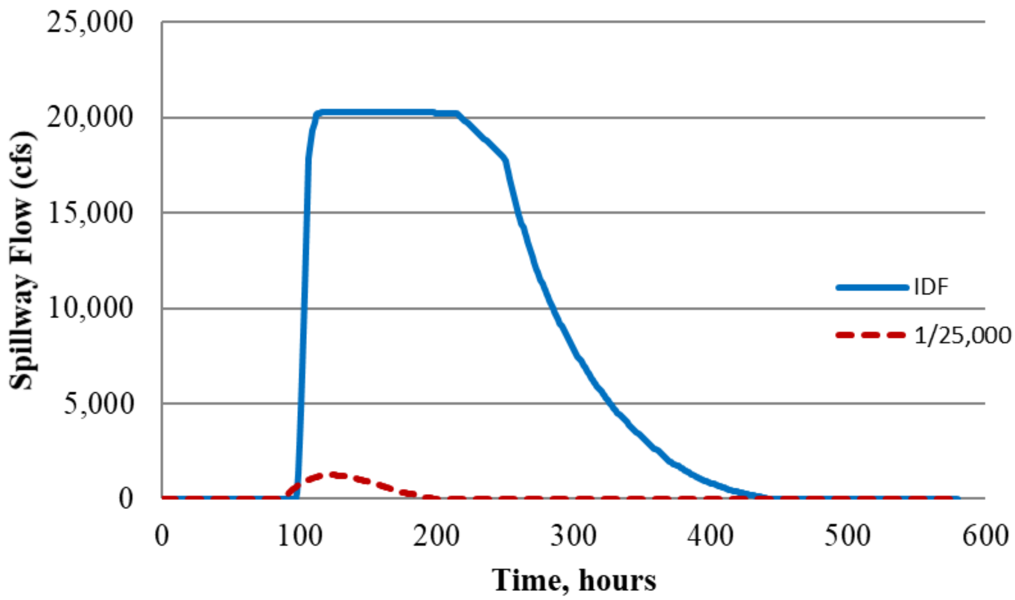
4. Hydrologic Description.

The IDF available for the study (Figure 2) was used for the spillway flows to assess erodibility. The IDF hydrograph was produced by subtracting the flow through the outlet works (7,160 cfs) from the IDF and the 1/25,000 flood, as computed for the 2014 Periodic Assessment.

a. In the IDF, the spillway flow briefly peaks at 20,304 cfs. For this erosion analysis, a flow of 20,000 cfs was used, which has an exceedance duration of 107 hours.

b. Combinations from the IDF hydrograph given in Figure 2 with lower flow but longer duration were also evaluated but found to yield lower probabilities of damage and failure.

c. For the 1/25,000 flood (flood with an annual exceedance probability of 0.00004), a flow of 1,100 cfs with a duration of 28 hours yields the highest probability of damage and failure.



Case Study 8H Figure 2. Spillway hydrograph under the IDF

5. Spillway Geologic Description.

Spillway geologic condition was assessed for determination of erodibility.

a. Villanueva and Wibowo (2008) use the erodibility index, proposed by Moore et al. (1994), to quantify the geologic susceptibility to erosion and headcutting. K_h is based on several geologic parameters including material strength number, block or particle size number, discontinuity or inter-particle bond shear stress, and relative ground structure number. These values can be estimated based on field observations, though laboratory testing is preferred.

b. Professional geologist Steve Jirousek and engineer John Shelley conducted a site visit to develop erodibility index (K_h) values for rock materials in the Wilson Dam spillway. Ten field points were examined: four surficial soil deposits, two shallow hand-dug (2 feet or less) test pits, one outcrop above the spillway floor (north spillway slope), and three small natural outcrops. An example of one such outcrop is given in Figure 3. Boring logs from the initial spillway construction were also examined.

Case Study 8H Figure 3. A natural outcrop used to guide the estimation of K_h

c. The outcrop (Figure 3) is approximately 995 feet downstream of the spillway cut but is at an elevation (1,567 feet) that may be indicative of the underlying spillway material. K_h for this material is estimated at 63.

d. Based on the limited observable outcrops, estimates for K_h range from 1 to 63 for rock material at elevations from 1,545 to 1,581. Based on boring logs, K_h ranges from 1 to 72 for rock material at elevations from 1,507 to 1,552. For this analysis, a K_h range of 1 to 63 was used because it corresponds to the first 700 feet of spillway. These are the K_h values for the rock material, not the soil material which would likely erode away very quickly during a significant spillway discharge.

e. Documentation from the original design of the spillway states that because of the nature of the shales and poorly cemented stand-stones, they can be expected to offer only slight resistance to erosion by the rare spillway discharges.

f. Site observations and the estimated K_h values support this statement. A more thorough description of the site visit, the factors comprising the K_h estimates, and the assumptions used to fill in data gaps are found in the memo "Preliminary Estimate of Erodibility Index (K_h) Values for Materials in the Wilson Dam Spillway, Kansas" (USACE-Northwestern Division Kansas City District (NWK), September 2014).

6. Probability of Damage and Failure.

A simplified relationship was used to assess the spillway damage risk. Villanueva and Wibowo (2008) give the probability that a spillway sustains greater than 70% damage by the following equations:

$$S_j = -2.640(LogK_H) + 5.469Log(q) + 1.435Log(D) + 0.305(S) - 0.987Log(L)$$

$$Probability\ Damage\ Exceeds\ 70\% = 1 - \frac{1}{1 + \exp(S_j - 6.035)}$$

Case Study 8H
Equation 1

where:

S_j = an intermediate variable
K_h = the erodibility coefficient
q = the unit discharge (cfs/ft)
D = duration of flow, hours
S = spillway slope, degrees
L = spillway length, ft

a. The pertinent parameters for Wilson Lake spillway are given in Table 1.

Case Study 8H Table 1
Spillway Parameters

	IDF	**1/25,000 Flood**
q (cfs/ft)	44.4	2.4
D (hours)	107	28
S (degrees)	2.0	2.0
L (ft)	1,044	1,044
K_h	1 to 63	1 to 63

b. For K_h values that range from 63 to 1, the probability of damage exceeding 70% ranges from 23% to 97% for the IDF and from 0.01% to 1.5% for the 1/25,000 flood. These probabilities apply to the rock material only. Significant soil material in the spillway would be eroded by almost any spillway discharge.

c. The probability of a breach occurring are given in the following equations, as presented in Wibowo (2009):

$$S_j = -0.684(LogK_H) + 2.838Log(q) + 0.863Log(D) + 0.273(S) - 0.235Log(L)$$

$$Probability\ Breach = 1 - \frac{1}{1 + \exp(S_j - 8.125)}$$

Case Study 8H
Equation 2

d. Results. For K_h values that range from 63 to 1, the probability of breach ranges from 4% to 14% for the IDF and from 0.08% to 0.26% for the 1/25,000 flood.

7. Conclusions and Limitations.

The methods used for this analysis simplify complex three-dimensional (3D) flow processes and geologic parameters that vary over time and space into regression equations using five bulk parameters: unit discharge, flow duration, length, slope, and erodibility index.

a. These simplifications could lead to results that under- or over-predict the probability of spillway erosion damage or failure. Furthermore, the erodibility coefficients were based on limited field observations and borings.

b. A practical way to improve the spillway erosion prediction at Wilson Lake would be to develop site-specific empirical equations following a lesser magnitude spillway erosion event, as was done for nearby Tuttle Creek Lake (Perlea et al., 2010).

c. The analysis documented in this memo indicates that the IDF has a 23% to 97% probability of severely damaging the spillway and a 4% to 14% chance of breaching the reservoir.

d. The 1/25,000 flood has a 0.01% to 1.5% probability of severely damaging the spillway and a 0.08% to 0.26% chance of breaching the reservoir.

e. Multiplying these probabilities by the hydrologic probabilities provides a probability of failure by spillway erosion for the dam.

Case Study 9A
Adaptive Hydraulics with Sediment Application

1. Case Study.

This case study provides content condensed from studies performed by the St. Paul District for a 2D modeling application on the Missouri River in the Omaha District. The partial content of the case study demonstrates several concepts but is not comprehensive of USACE study requirements.

2. Introduction.

a. Location. Lower Decatur Bend is located on the Missouri River near Decatur, Nebraska. The river reach running from river mile 685.8 to 686.9 was modeled using the Adaptive Hydraulics (AdH) model to produce velocity, depth, and bed displacement. AdH is a 2D model with sediment transport capability developed by ERDC (USACE 2017a). The AdH software produces 2D hydrodynamic calculations across a computational mesh.

b. Study Objective. The AdH model evaluation was performed to evaluate existing conditions and three alternatives. Design objectives are to maintain all authorized purposes, limit further flood damage risk, and sustain the existing shallow water habitat (SWH) to the extent practical. The evaluation of alternatives focuses on identifying the effects of structure placement to both maintain the navigation channel and optimize the creation of SWH. All modeling was conducted in SI (metric) units to facilitate the use of sediment transport in the modeling. The horizontal coordinate system was Universal Transverse Mercator (UTM) 15 with the National Geodetic Vertical Datum (NGVD) 1929 vertical datum.

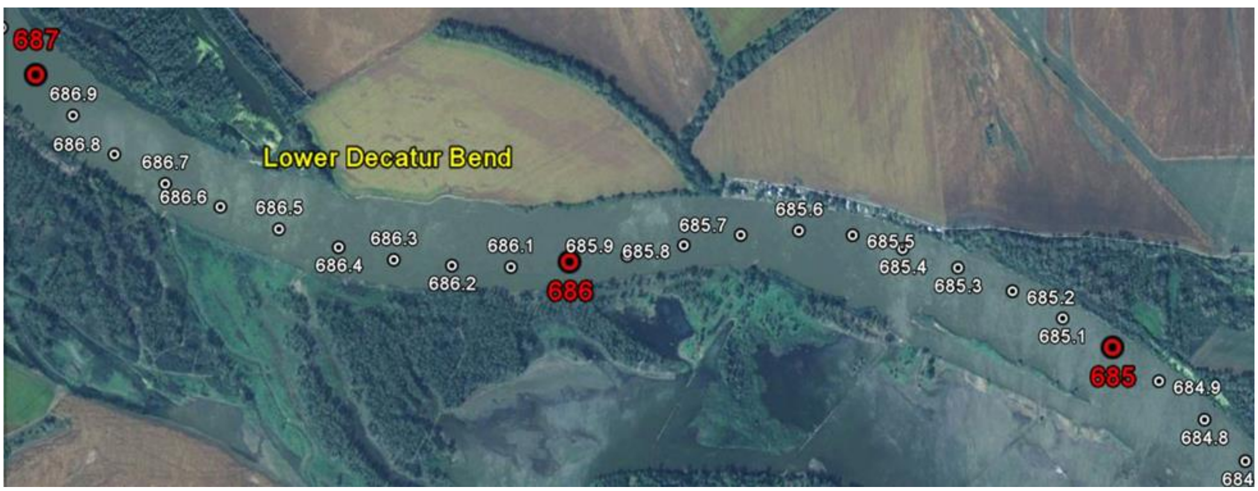
3. 2011 Flood Damage and Repairs.

The flood of 2011 caused severe erosion of the river bank at the location of the Lower Decatur Revetment Lowering Project in Burt County, Nebraska (RM 686-686.7) that was originally constructed under USACE's Section 1135 authority in 2008. This erosion resulted in over-widening of the river channel, which led to the shoaling in the navigation channel.

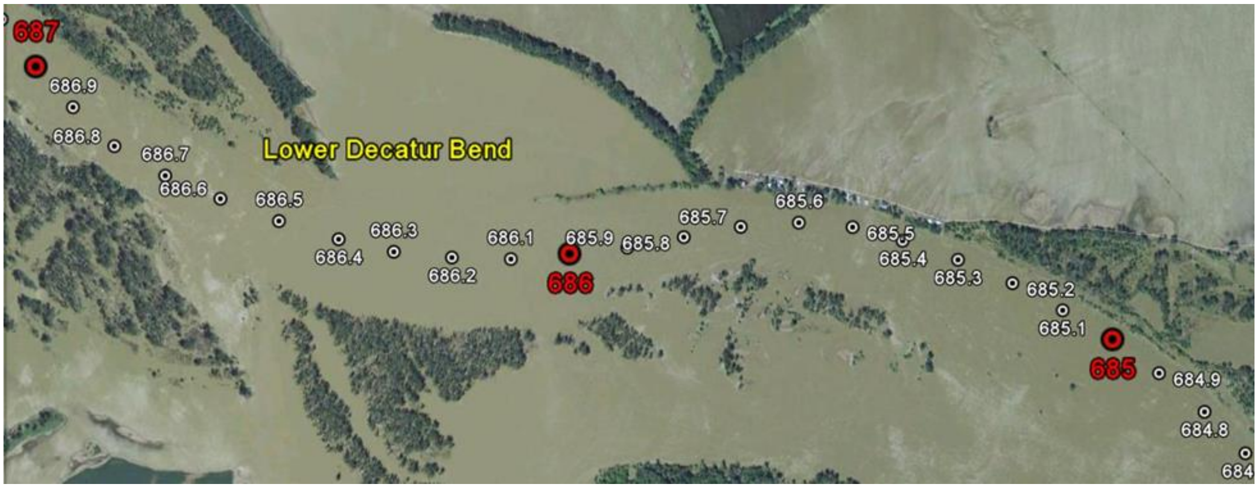
a. Navigation Channel Shoal and Project Vicinity. Figure 1 illustrates the navigation channel shoal (shown as an exposed bar due to low winter flow), the original revetment line, and the eroded bank area. Figure 2, Figure 3, and Figure 4 show a large-scale plan view of the Lower Decatur Bend vicinity condition for the before, during, and after 2011 flood conditions.

b. Repairs. To restore the desired depths in the navigation channel while still maintaining the shallow water habitat that exists in the area where the original revetment lowering project was constructed, USACE identified a series of repair actions in early 2012, consisting of existing navigation structure maintenance and the addition of new structures. The repairs actions were evaluated with a 2D model.

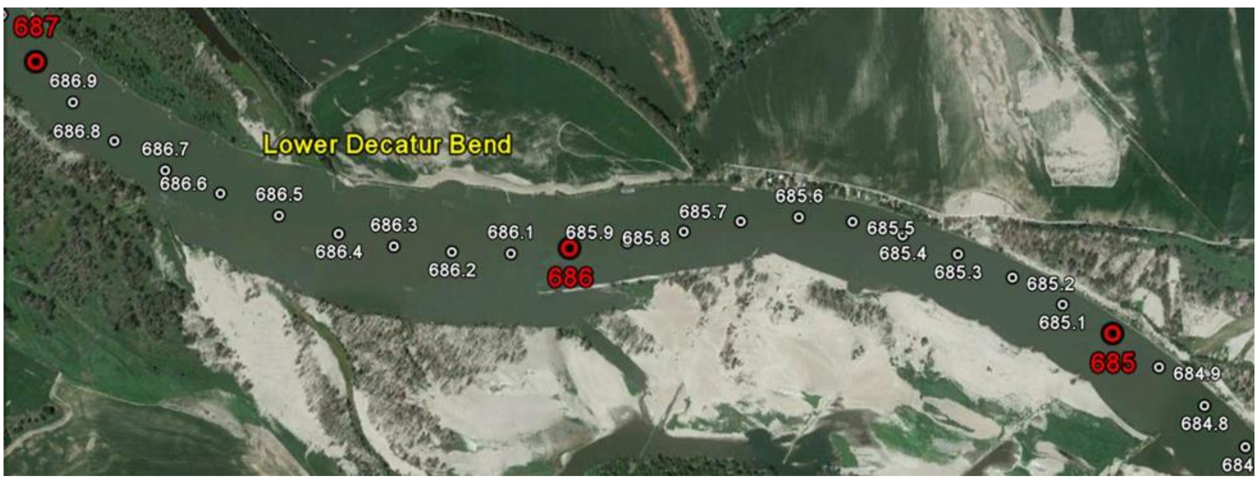
Case Study 9A Figure 1. Lower Decatur Bend, January 2012



Case Study 9A Figure 2. Lower Decatur Bend before 2011 flow event



Case Study 9A Figure 3. Lower Decatur Bend during 2011 flood event



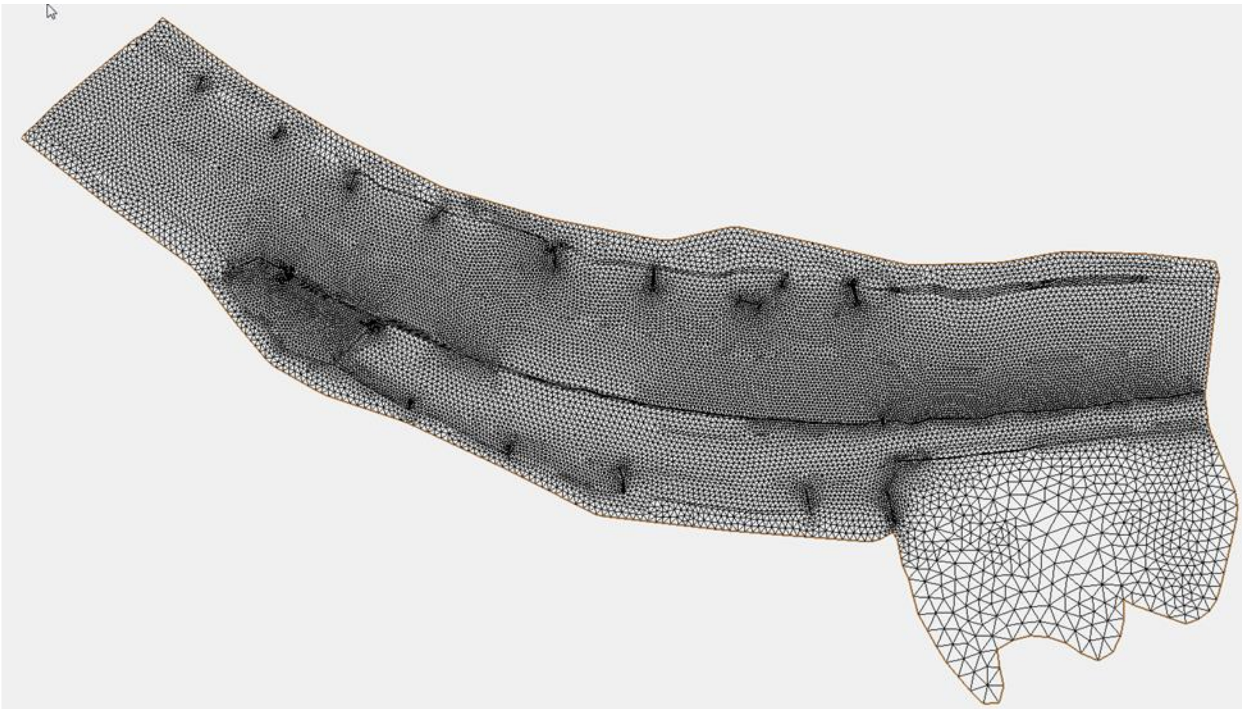
Case Study 9A Figure 4. Lower Decatur Bend after 2011 flood event

4. 2D Model Construction.

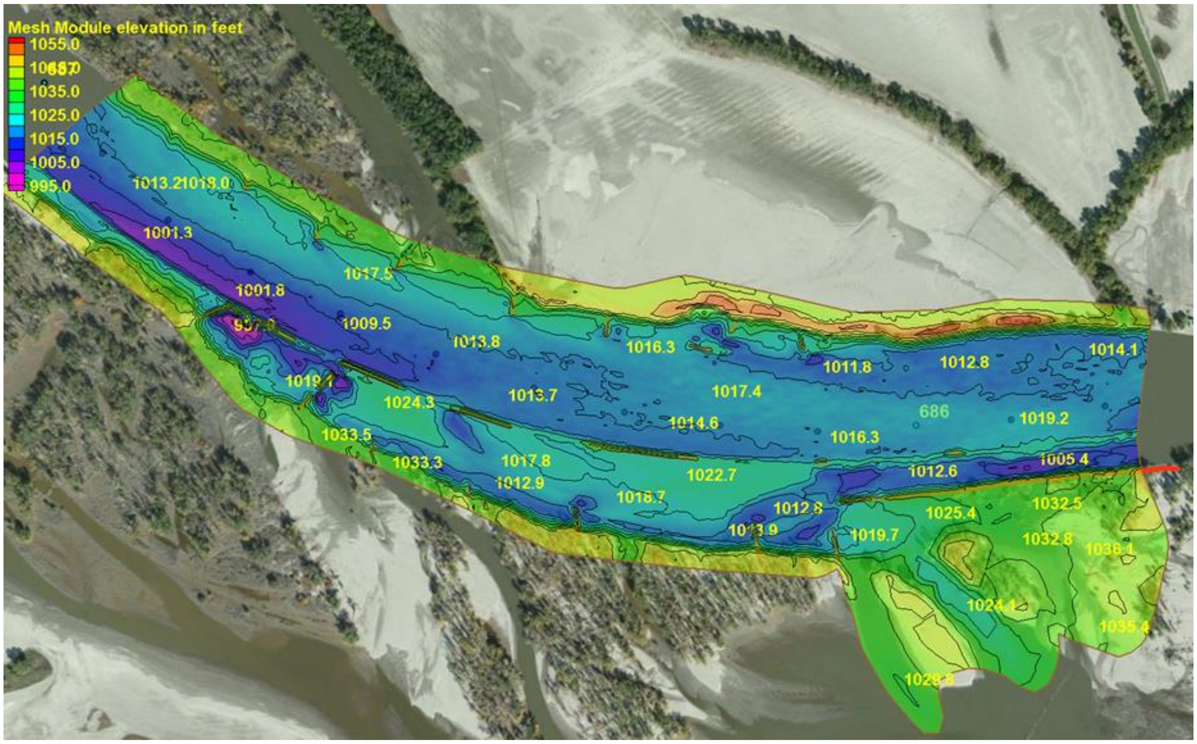
a. General. The model geometry was created using the Surface-water Modeling System (SMS) Version 11.1, which was developed by Aquiver. SMS is a pre- and post-processor for AdH that allows for computational mesh construction and the evaluation of model results. The multi-processor version of AdH (Version 4.4 (6 March 2014)) was used in the modeling.

b. Bathymetric and Topographic Data Sources. Hydrographic soundings were used to establish most of the underwater portions of the baseline condition model geometry. Additional point surveys were taken along the revetment crests and other important structure features. LiDAR data was used to provide ground elevations where hydrographic or surveys were not available. The SMS version 11.1 was used to automatically generate the mesh elevations from these datasets.

c. Mesh Creation. Manual revisions were necessary along the main revetment alignments to create model crest elevations that reflected those in the surveys. The final mesh has approximately 19,000 nodes and 37,000 elements. Figure 5 illustrates the computational mesh. Figure 6 illustrates elevations in the mesh computational area. Visible in Figure 6 are the raised portion of the revetment, navigation channel dike structures on the opposite bank, and the new bank stability structures that were added in the eroded bank area.



Case Study 9A Figure 5. Lower Decatur Bend computational mesh

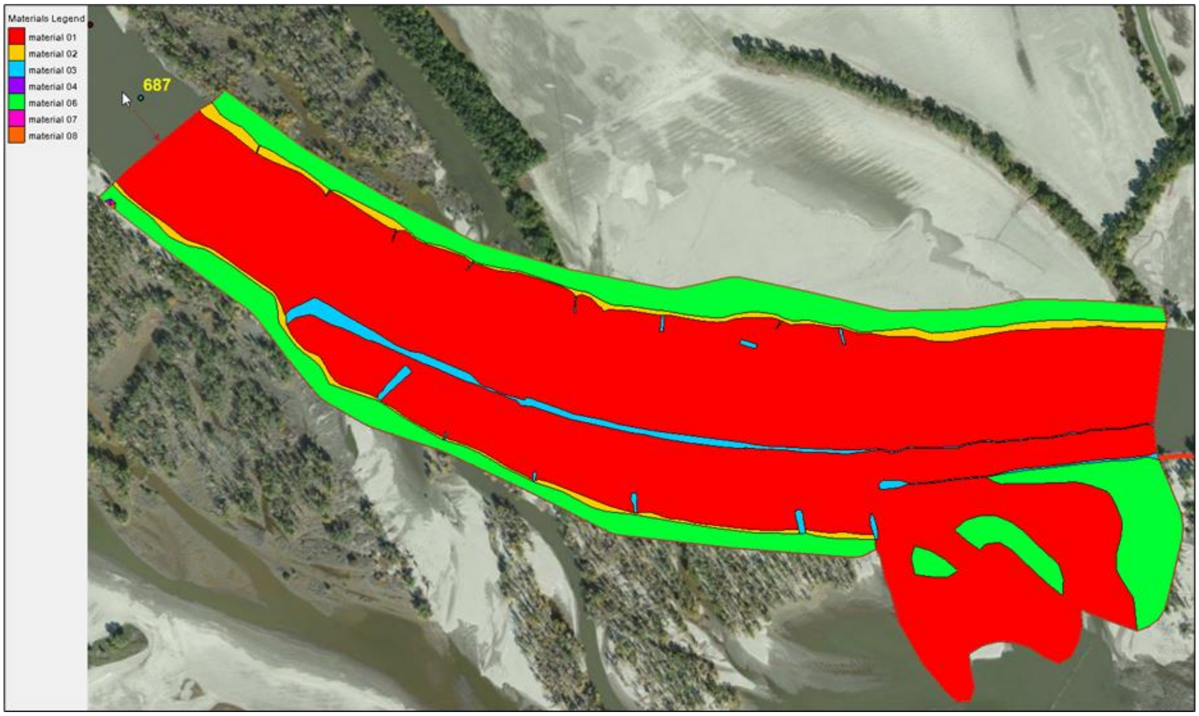


Case Study 9A Figure 6. Elevations within computational area

d. Bed Roughness. Roughness values were assigned for different areas of the site as shown in Table 1. Figure 7 illustrates the location of the material types assigned within the mesh. Several other material types were established during initial mesh construction. These were combined to simplify the calibration process.

Case Study 9A Table 1
Material Types and Roughness

Material Type	Roughness (Color in Figure)
Main channel	0.027 (red material #1)
Main channel banks	0.035 (gold material #2)
Existing navigation channel structures	0.042 (Blue material #3)
Non-channel overbank areas	0.032 (green material #6)



Case Study 9A Figure 7. Assigned material types

e. Computation Settings. AdH computation settings were determined for several critical parameters. The eddy viscosity function used a coefficient value of 0.5. For the element wetting and drying limit, the final values used were at or near to 0.6 meters. Stream vorticity was included in the model runs with and without sediment transport with a molecular diffusion rate of 5. For mesh refinement, an error tolerance value of 5 and one level of refinement were sufficient to produce good results. Computation settings presented in this section are incomplete and provides limited information applicable only to this model. The AdH User's Manual provides a discussion of parameter considerations critical to sediment transport analysis (USACE 2017a).

5. Model Parameters.

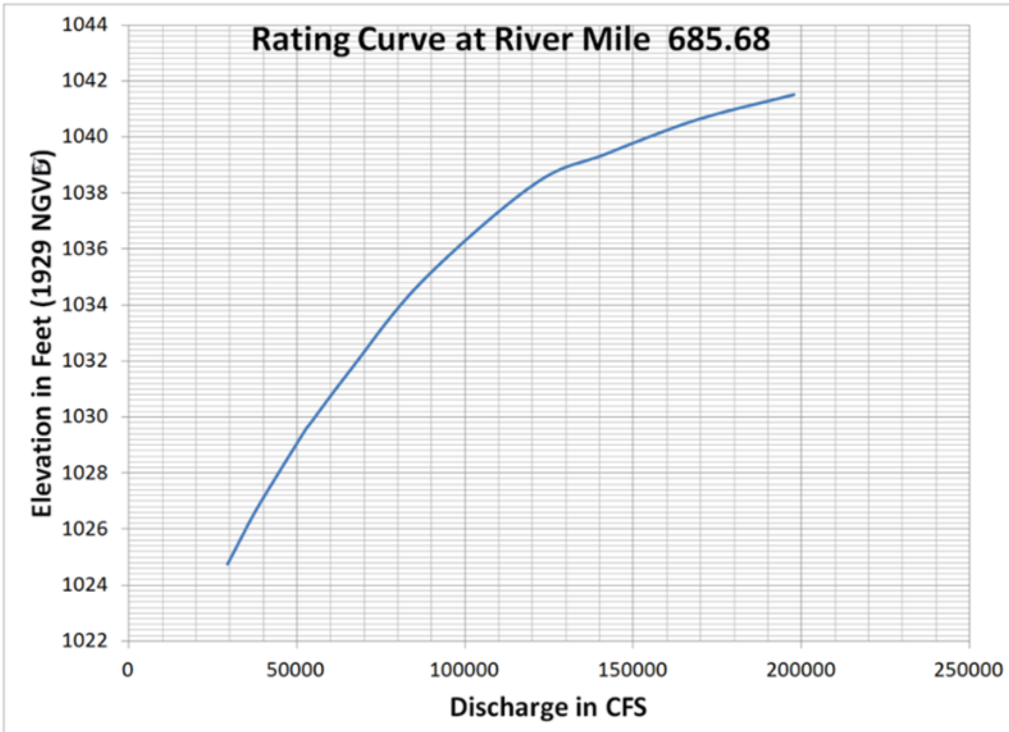
a. Boundary Conditions. A rating curve that was developed from an HEC-RAS model of the Missouri River was used to produce the downstream water surface elevation for the remaining events. Figure 8 shows the rating curve.

(1) The downstream water surface boundary was applied not only along the lower limit of the modeled main channel, but also extending around much of the southern model boundary where breakout flow is possible.

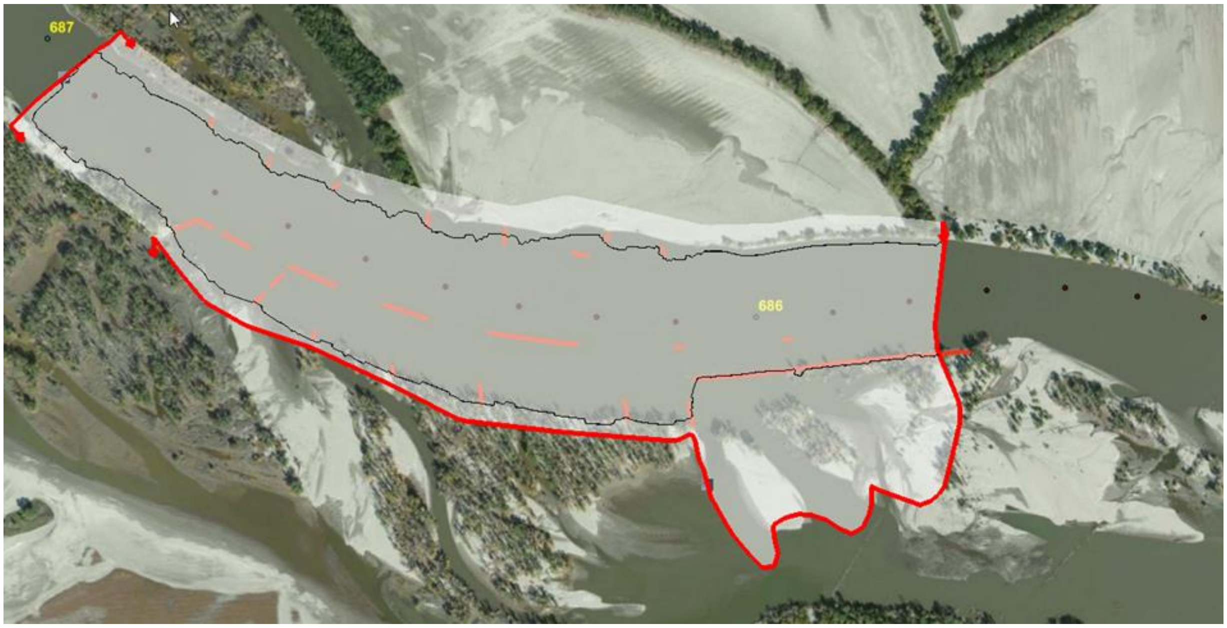
(2) For most conditions, the southern boundary is dry and does not provide an outlet to flow. For high flow conditions, the low parts of this southern boundary become active and allow flow to leave the model to the south. The southern portion of the boundary string is located along the high banks and breakout areas. The assumption is that the downstream boundary water

elevation is similar to the backwater surface elevation. This should be a reasonable assumption because during high flow conditions, most of the head loss would occur as water breaks across the lower breakout areas and over the river banks.

(3) Figure 9 shows both the upstream and downstream model boundaries as thick red lines.



Case Study 9A Figure 8. Downstream rating curve from HEC-RAS



Case Study 9A Figure 9. Location of upstream and downstream boundary strings

b. Simulation Events. Calibration and alternative design events simulated with the model are listed in Table 2.

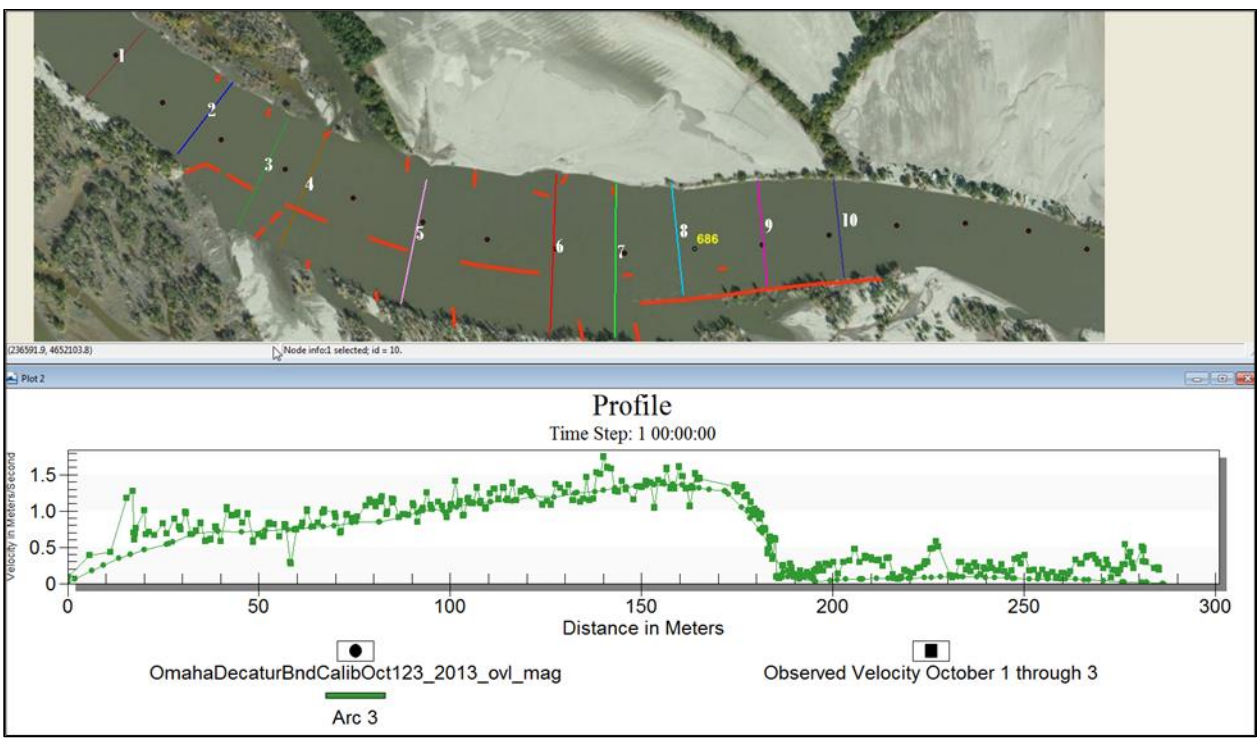
Case Study 9A Table 2
Simulation Events

Event	Discharge (cfs)	Discharge (cms)	Downstream Elev. (feet)	Downstream Elev. (meters)
Calibration Event	25,600	725	1,025.0	312.4
Navigation 90% Exceedance	23,200	657	1,023.2	311.9
Navigation 50% Exceedance	31,800	900	1,025.3	312.5
Navigation 10% Exceedance	58,200	1,648	1,030.5	314.1
10 Year	87,200	2,469	1,034.9	315.4

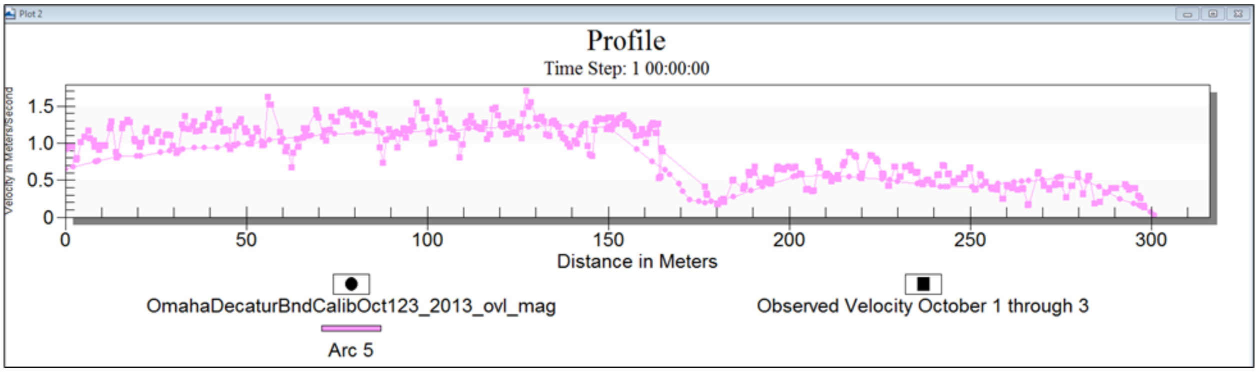
c. Calibration. Calibration was performed on the developed model. Chapter 9, paragraph 9-4 provides a thorough description of hydraulic and sediment model parameter calibration. Application of these guidelines was limited for this project due to the scarcity of data. Therefore, this model should be regarded as a computational experiment instead of a fully calibrated model as described in this document (Chapter 9) and Thomas and Chang (2008). This computational model is suitable for use with comparing alternatives.

(1) Hydraulic Calibration – Water Surface Elevation. Missouri River slope is typically in the range of 0.8 to 1.0 foot/mile. Measured data was available from recent surveys. Calibration was able to successfully match water surface elevations within 0.1 foot. While it is important to match the river slope, success does not necessarily imply that the model is suitable for further use due to the short model length and relatively change in water surface elevation.

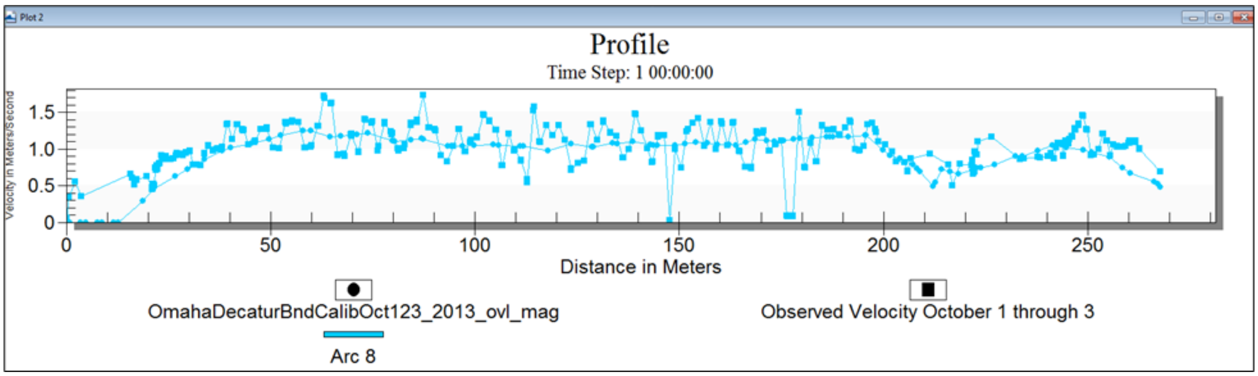
(2) Hydraulic Calibration – Measured Velocity Transects. Additional calibration was performed using measured velocity to verify flow distribution in the model. Velocity measurements were obtained on 26 September and 1–3 October 2013, with flow as stated in Table 2 for the calibration event. Figure 10, Figure 11, and Figure 12 show comparisons of the field-measured velocity with the computed velocity. The transect locations are color coded to match the arc located in the images. The model computed velocities are always more uniform than the measured values, which are affected more by local turbulence. Transects illustrate that the model can successfully reproduce the observed velocity variation. The velocities shown are in meters per second in preparation for sediment modeling.



Case Study 9A Figure 10. Calibration results transect 3



Case Study 9A Figure 11. Calibration results transect 5



Case Study 9A Figure 12. Calibration results transect 8

(3) Bed Displacement Results. Bed displacement data over a long time frame for calibration purposes was not available. Bed displacement was examined to verify that the model results for the existing condition was suitable for alternative analysis. A qualitative examination of bed change was performed after simulating the model for 50 days with a constant flow and determined that bed displacement was reasonable.

(4) Further Bed Displacement Examination. Additional qualitative examination of bed displacement was performed in the following focus areas:

(a) Simulation at normal flow resulted in reasonable levels of bed change. Navigation channel changes were less than two feet at most locations after a 50-day period.

(b) Navigation channel conveyance was preserved. The bed change determined by the model was relatively consistent.

(c) Bed gradation size used for the simulation was reasonable.

(d) The model reached bed continuity (none/minor change between time steps) during the 50-day simulation period.

(e) Excessive Scour Areas Associated with Dike Structures Filled to an Equilibrium Level. This is consistent with results that occurred in other sediment model applications on the

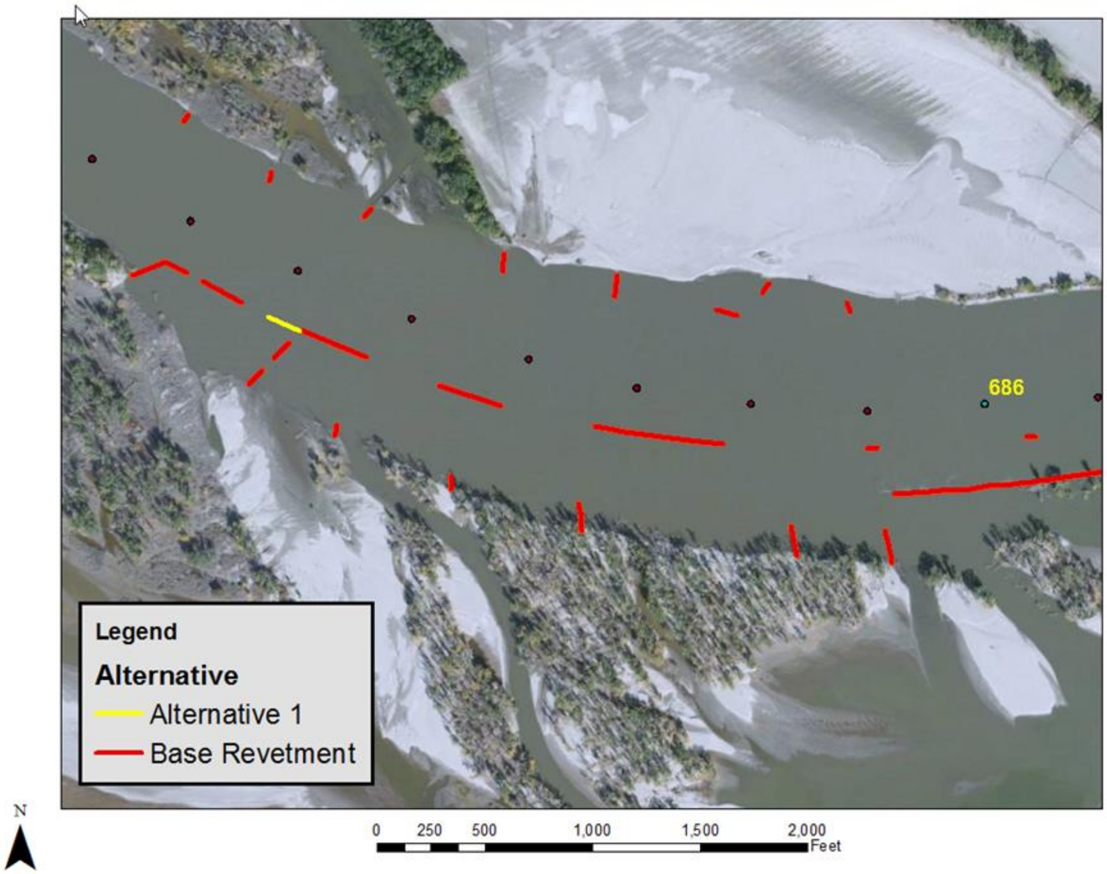
Missouri River. The dike structures cause scour downstream of the dike. Since the hydraulics that cause this to form are largely 3D with flow plunging over the structure, the depth average 2D model results generally retain only a portion of the scour depth to be maintained. Therefore, this limitation is critical to identify prior to simulating alternatives.

(5) Bed Displacement Lessons Learned. Results for this study supported the model use with reasonable bed displacement computations after completing the model development and calibration process. However, simulations that require use of sediment parameters beyond the normal range should be further reviewed. Examples include: (1) a model where bed material size is greatly increased to keep the model from scouring excessively; (2) model sediment input must be significantly reduced to prevent continual aggradation.

6. Alternative Modeling.

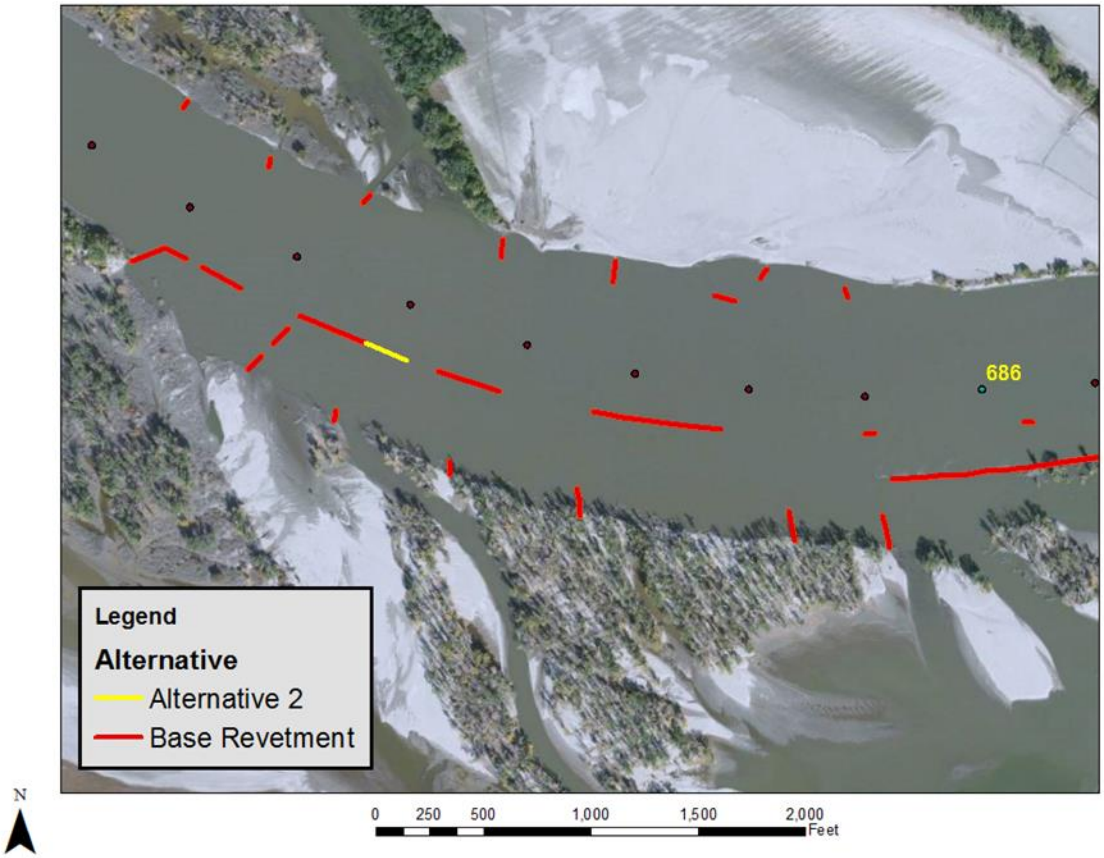
Three alternative actions were considered as a means to impact the velocities and scour/deposition characteristics of the river, particularly to the south of the primary east-west running revetment. This existing condition revetment provides a more isolated region that has lower velocity and improved water depths for aquatic wildlife.

a. Alternative 1 – Extend Revetment Segment 2 Upstream. Alternative 1 consists of extending revetment segment 2 upstream to reduce the opening size. The revetment would be extended upstream by about 150 feet. This alternative reduces the opening size of the most upstream major opening in the revetment to about 150 feet. The elevation of the extended rock segment is 1,031.1 feet (314.4 meters). Figure 13 shows this construction in yellow with the existing revetment segments and dike structures shown in red.



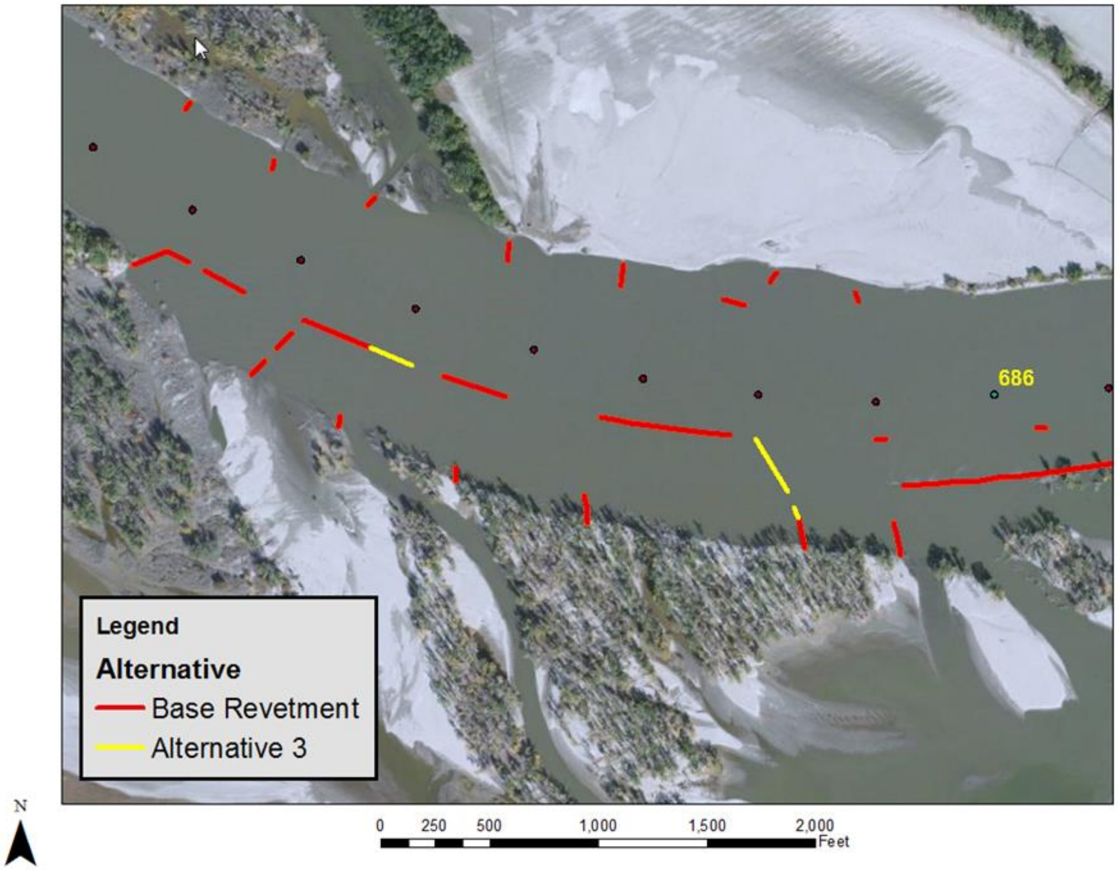
Case Study 9A Figure 13. Alternative 1 extend revetment segment 2

b. Alternative 2 – Extend Revetment Segment 2 Downstream. Alternative 2 extends revetment segment 2 downstream and reduces the size of the next downstream gap by 150 feet. This alternative extends the revetment segment 2 about 200 feet in the downstream direction. The elevation of the rock extension is 1,030.6 feet (314.2 meters). Figure 14 shows the location of this construction in yellow.



Case Study 9A Figure 14. Alternative 2 extend revetment segment 2 downstream

c. Alternative 3 – Extend Revetment Segment 2 Downstream Plus Reduce Opening. The third alternative includes the downstream revetment extension from Alternative 2 along with two additional features at the lower end of the revetment. First, one of the southern shore existing dikes is extended by 50 feet at an elevation of 1,025.3 feet (312.5 meters). Second, the crossing dike near revetment segment 4 is extended south toward the bank a distance of about 270 feet at the same elevation. This leaves about an 85 foot gap between the two near structures. Figure 15 shows the three features of Alternative 3 in yellow.



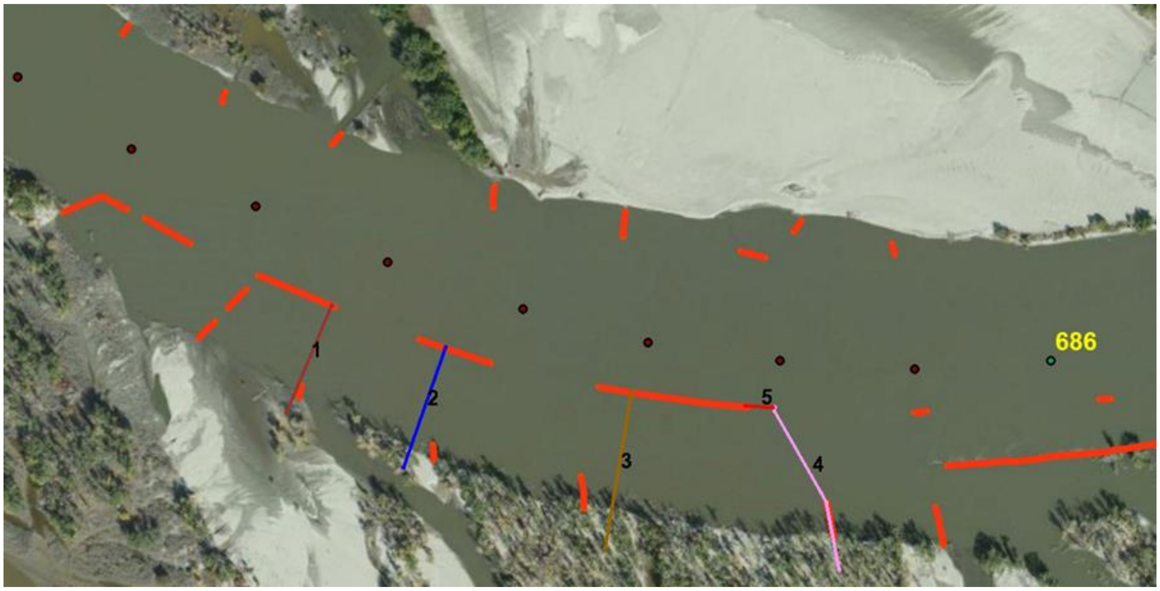
Case Study 9A Figure 15. Alternative 3 extend revetment segment 2 plus reduce opening

d. Impact of Alternatives on Discharge. AdH results determined that the three alternatives each have an effect on the discharges south of the primary revetment. Figure 16 is a map showing several arcs across the flow area south of the primary revetment. These arcs indicate discharge computation transects. Figure 17 shows a comparison of how discharge is changed by the alternative plans at these arcs. The 31,800 cfs discharge runs are used because that discharge was selected as critical for navigation channel structure performance. The effects are summarized as:

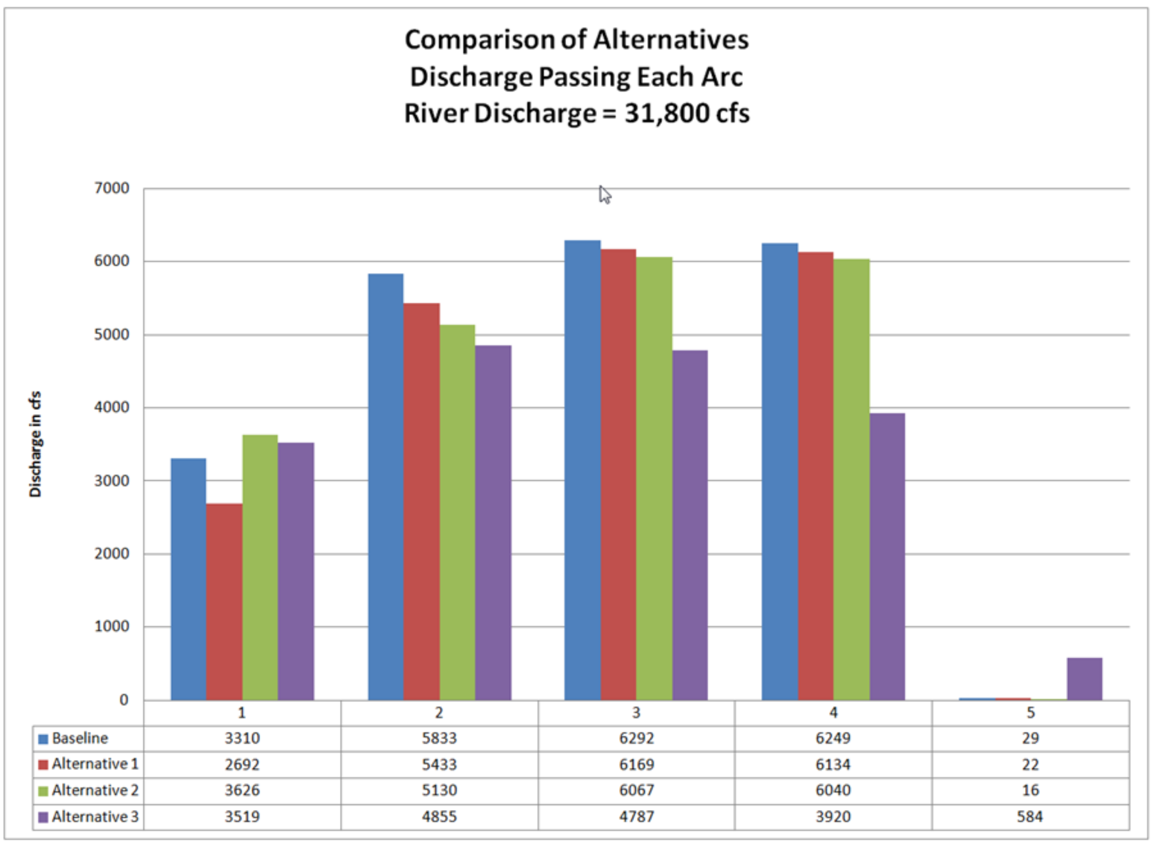
(1) Alternative 1 reduces flow most at the upstream reaches of protected area. Its impact is lost as lower inlets add water to make up for the reduced flow at the upstream restriction.

(2) Alternative 2 is similar in that the restriction of the second primary opening is partially offset by increased flow is the first inlet. Both Alternatives 1 and 2 have little effect below Arc 2.

(3) Alternative 3 has little effect at Arc 1 but is having an increasingly significant effect on reducing discharge at each of the remaining four downstream arc locations.



Case Study 9A Figure 16. Location of arcs for alternative discharge comparison

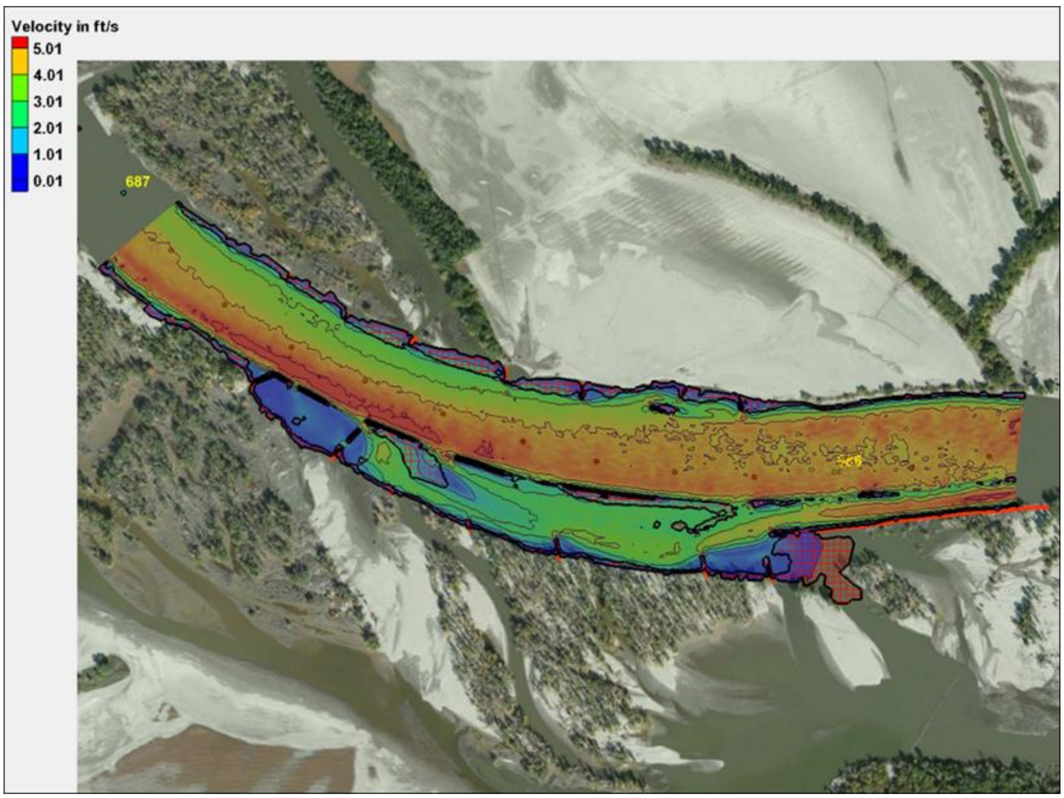


Case Study 9A Figure 17. Comparison of AdH alternatives – discharge passing arcs at total river discharge of 31,800 cfs

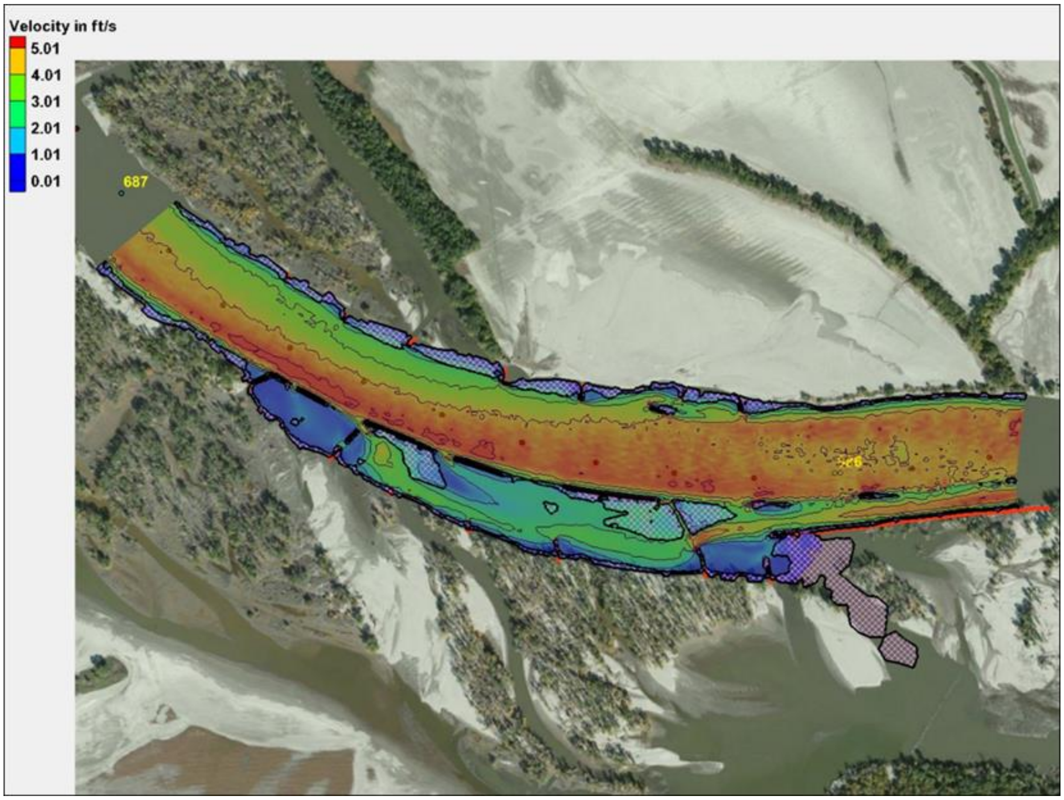
e. AdH Computed Velocity and Depth within Habitat Area. Velocity and depth computed by AdH in the off-channel area were examined to evaluate future sedimentation processes and potential impacts to the constructed habitat project. A velocity of less than 2 feet per second in areas with depths less than 5 feet is desired.

(1) Model results did not determine a significant difference in the zonal coverage between the baseline (Figure 18) and Alternatives 1 and 2. Alternative 3 (Figure 19) does show an increase in the zone particularly by the downstream crossing dike feature.

(2) Most of the habitat area located south of the primary revetment do not meet the depth criteria since water depths are greater than 5 feet due to the extreme scour that occurred during the 2011 event. Areas that fit into this compound range of parameters are shown in Figure 18 and Figure 19. These identified areas are shown with a colored cross-hatching pattern bounded by black borders in the figures.



Case Study 9A Figure 18. Baseline condition, areas with depth <5 and velocity <2



Case Study 9A Figure 19. Alternative 3 condition, areas with depth <5 and velocity <2

7. Sediment Modeling of Alternatives.

a. Sediment Modeling. Seven grain sizes, based on the Wentworth Classification system (documented in EM 1110-2-1100, Part 3), were used. These seven grain sizes are provided in Table 3.

Case Study 9A Table 3
Sediment Properties

Wentworth Classification	Size Range (mm)	Geometric Mean of Size (mm) (Size Modeled in AdH)
Coarse Silt	0.0312–0.0625	0.0441588
Very Fine Sand	0.0625–0.125	0.0883883
Fine Sand	0.125–0.250	0.1767767
Medium Sand	0.250–0.50	0.3535534
Coarse Sand	0.50–1.00	0.7071068
Very Coarse Sand	1.00–2.00	1.4142136
Granule	2.00–4.00	2.8284271

(1) Bed Layers. Six bed layers were modeled. Where erosion was allowed to occur, the bottom four layers were assigned thicknesses of 3.28 feet (1 meter), and the top two layers were assigned a thickness of 0.0003 feet (0.1 mm). ERDC (USACE 2017a) recommends two thin bed layers at the top of the sediment layer to aid with bed sorting. The four lower layers were assigned varying percentages of the grain sizes at different depths. In AdH each bed layer is assigned a layer number, with layer number 1 being the bottom layer.

(2) Bed Layer Properties. The properties of layer number 4 (the bed layer just below the two thin layers meant for sorting) were set according to available sieve data that was obtained from the upper portion of the bed. Detailed, deep-sediment samples were not available so the percentages for layer numbers 1 through 3 are assumed. For this analysis, the assumption made is that larger grain sizes will make up a larger percentage of the lower layers (the bed becomes coarser at greater depth). Results were not affected by this assumption. Studies where critical results are affected by bed material layers will likely need to acquire field data. The bed layer properties for areas allowed to erode are provided as shown in Table 4.

Case Study 9A Table 4
Modeled Bed Layer Properties

Layer # (top layer has highest number)	Layer thickness (ft)	% of Coarse Silt	% of Very Fine Sand	% of Fine Sand	% of Medium Sand	% of Coarse Sand	% of Very Coarse Sand	% of Granule
1	3.281	1	4	20	25	25	15	10
2	3.281	1	4	20	30	25	13	7
3	3.281	1	4	25	30	25	10	5
4	3.281	1	4	30	30	20	10	5
5	0.0003	25	25	25	25	0	0	0
6	0.0003	25	25	25	25	0	0	0

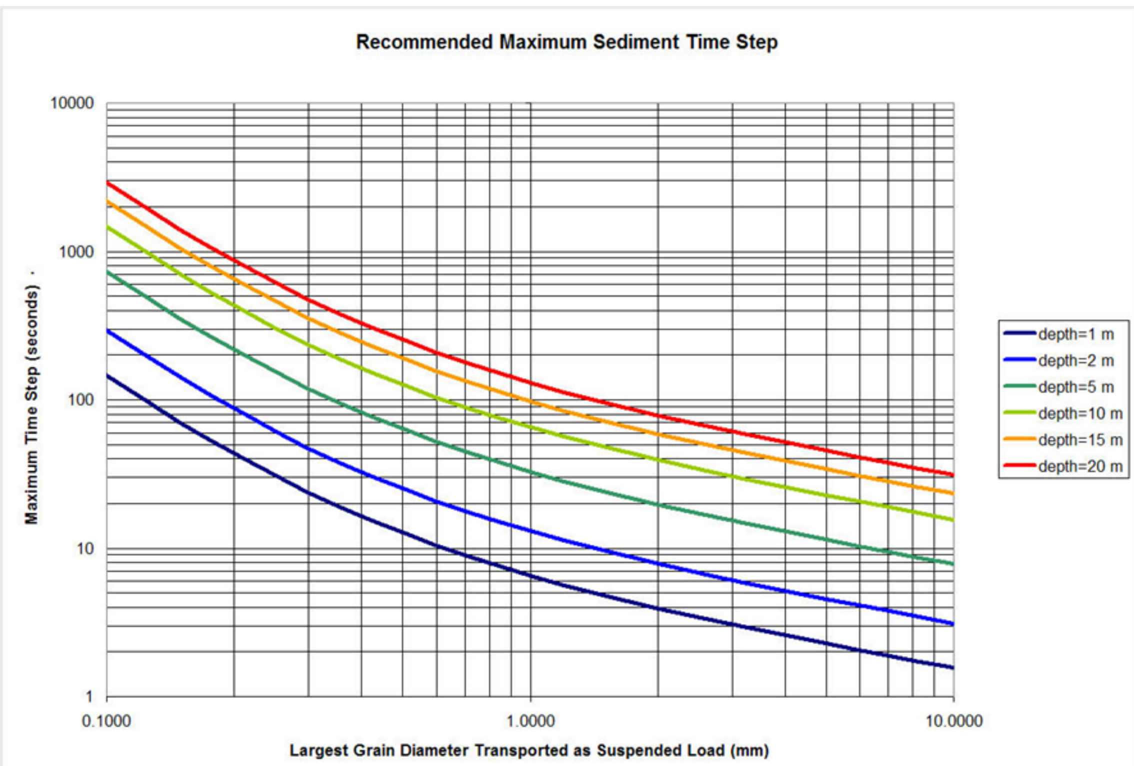
Note: Percentages were selected for AdH modeling purposes and do not represent actual field measurements. While the values are reasonable compared to actual sampling data, some adjustments were required for model stability and to produce expected bed armoring.

b. Structures and Non-erodible Layers. All structures were assumed to be constructed of non-erodible materials (rock and/or trees). In non-erodible areas, all six bed layers were assigned a thickness of 0.0003 feet (0.1 mm) so that essentially no erosion could occur. The tabulated bed layer information and the geometric mean data from was entered into the AdH boundary condition file.

c. Upstream Sediment Load. Available data included an upstream sediment load curve based on gage data. The gage is located about 50 river miles upstream of the study area without any significant tributaries between the gage and the site. However, initial results using the sediment load curve produced unrealistically large amount of channel deposition. Subsequently, the equilibrium transport boundary condition available in AdH was selected. The AdH manual (USACE 2017a) states that “when this condition is specified, the concentration that is required for a state of equilibrium at the location is applied in suspension. An equilibrium condition is one in which no sediment would erode or deposit.” Results with this condition to define the incoming sediment load were much more realistic.

d. Time Step Selection Guidance. In the AdH workshop modeling notes, ERDC provides recommendations for determining a maximum time step for AdH sediment runs as shown in Figure 20. This provides the recommended maximum time step based on the largest grain size and depth. For a 1 to 2 millimeter diameter for coarse sand and a water depth of 2 meters, the recommended time step is 8 to 10 seconds. ERDC workshop notes also recommend a minimum time step of 5 seconds for sediment runs. Earlier versions of AdH also allowed for a smaller sub-time step that is not allowed in current versions of the model.

e. Selected Time Step. For the purposes of this study, a 120-second time step was selected with constant discharges running for 50-day periods. The 120 seconds is more acceptable for the greater depth in some areas, and the smaller particle sizes used in the analysis (range of 0.1 to 0.2 mm). However, the selected time step does exceed ERDC and was largely driven by computational time and finding the largest time step where model results did not change significantly. The 50-day simulations combined with a 120-second time step still required a computer run time of several days, which was deemed to be barely feasible within study schedule limits.



Case Study 9A Figure 20. Sediment time step constraint (ERDC workshop notes)

f. Simulation Events. Table 5 shows the discharges used in the sediment modeling. Each discharge was held constant for the 50-day simulation period. Since the selected time step is higher than recommended values, the resulting sedimentation results should be considered qualitative instead of quantitative. The runs are best used to show comparative trends in deposition and erosion between different alternative options. A rough sensitivity analysis was done which indicated that smaller time steps increased the scale of deposition and erosion depths.

Case Study 9A Table 5
Discharges Used in Sediment Runs

Type of Event	Discharge in cfs
Navigation 10% Exceeded	58,200
Navigation and August 50% Exceeded	31,800
10 Year (10% Frequency)	87,200

g. Analysis Results – Alternative 1 and 2. Initial analysis results illustrated that Alternatives 1 and 2 were not effective for the project goals to maintain navigation channel depth and enhance deposition in the habitat area. Both alternatives resulted in less than desirable navigation channel depth.

h. Analysis Results Alternative 3. Figure 21, Figure 22, and Figure 23 illustrate displacement maps that show the effects of Alternative 3 on the displacement pattern after a 50-day model run. Difference displacement comparison (difference) was computed by subtracting the base condition displacement from the alternative. Therefore, positive values indicate more deposition for the alternative than the base condition. Specific modeling results are noted in the following sections:

(1) Although displacement differences are shown, the erosion and deposition patterns are often similar between baseline and the Alternative 3 conditions.

(2) Each of the three discharge conditions shows an effect on displacement due to Alternative 3. Many of the effects are similar across the discharges where the magnitude of the effects is reduced with higher discharges.

(3) The scour hole below the first inlet continues to scour for each of the flow conditions (label A in Figure 21).

(4) Below this scour area is a deposition zone that runs roughly to the transverse dike feature of Alternative 3 (see B and C in Figure 21). Results indicated that Alternative 3 could aid in producing reduced depth in this area. The alternative reduces the amount of displacement along the southern edge of the existing revetment for both the 31,800 and 87,200 cfs events.

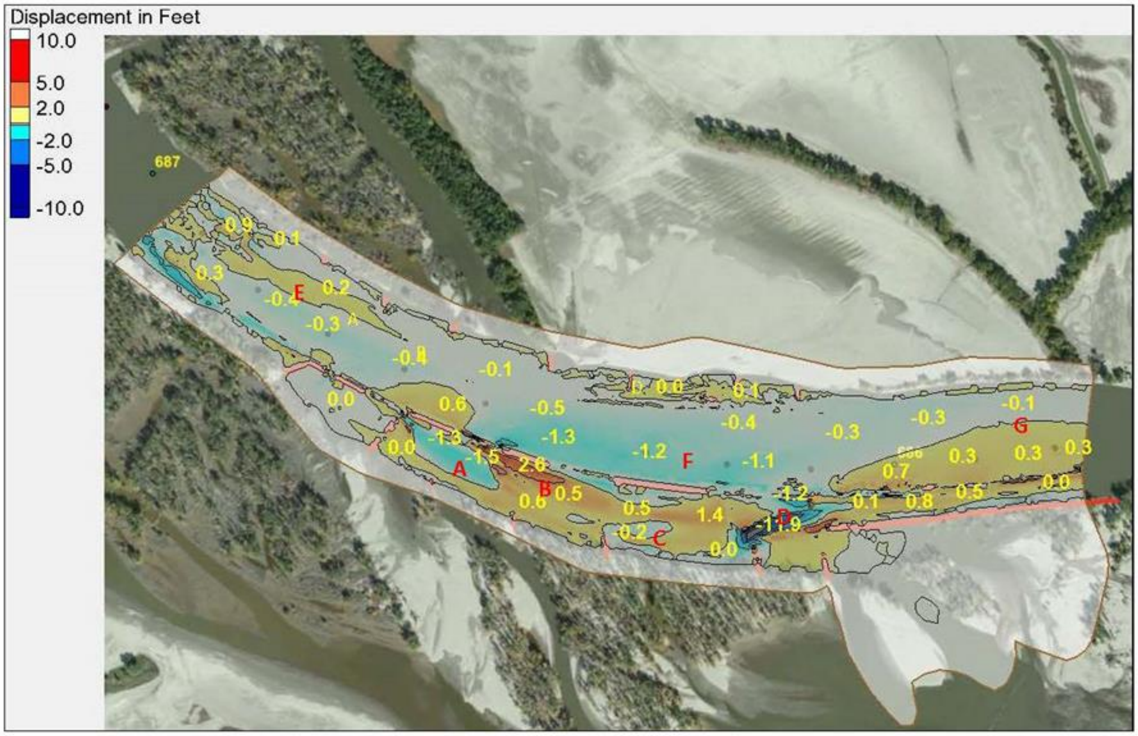
(5) In the existing condition, a deeper channel follows the southern shoreline in much of this reach. The modeling indicates that this channel should get shallower for most flows for both baseline and alternative condition. For Alternative 3, portions of this channel show about 0.5 feet of increased deposition at 31,800 cfs and 87,200 cfs, but almost no change at 58,200 cfs.

(6) The existing channel (D in Figure 21) near the transverse dike is a deposition zone under baseline conditions. This channel becomes an erosion zone under Alternative 3 at the 31,800 cfs event. For the 58,200 and 87,200 cfs events, this area is a deposition zone but receives less deposition with the alternative.

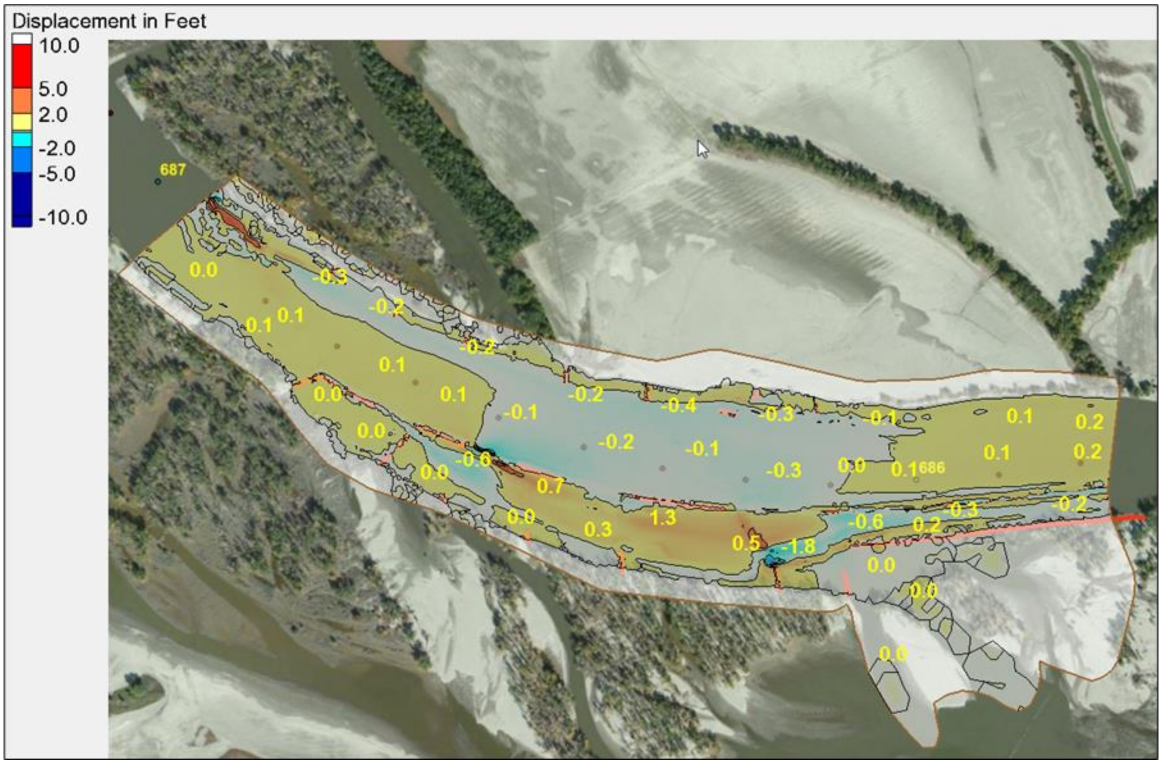
(7) Downstream of D between the revetment and the southern bank, the model results illustrate degradation for all flows for Alternative 3. This indicates that confining flow to the narrow gap with the new structures could result in downstream scour adjacent to the revetment. This could be detrimental to future revetment performance.

(8) Much of the main channel receives less displacement north of the existing revetment for Alternative 3. This is best seen to the north of the revetment segments 3 and 4 (F in Figure 21). This area was also the location of a problem sand shoal that has been occurring during the 2013 and 2014 navigation flow support seasons.

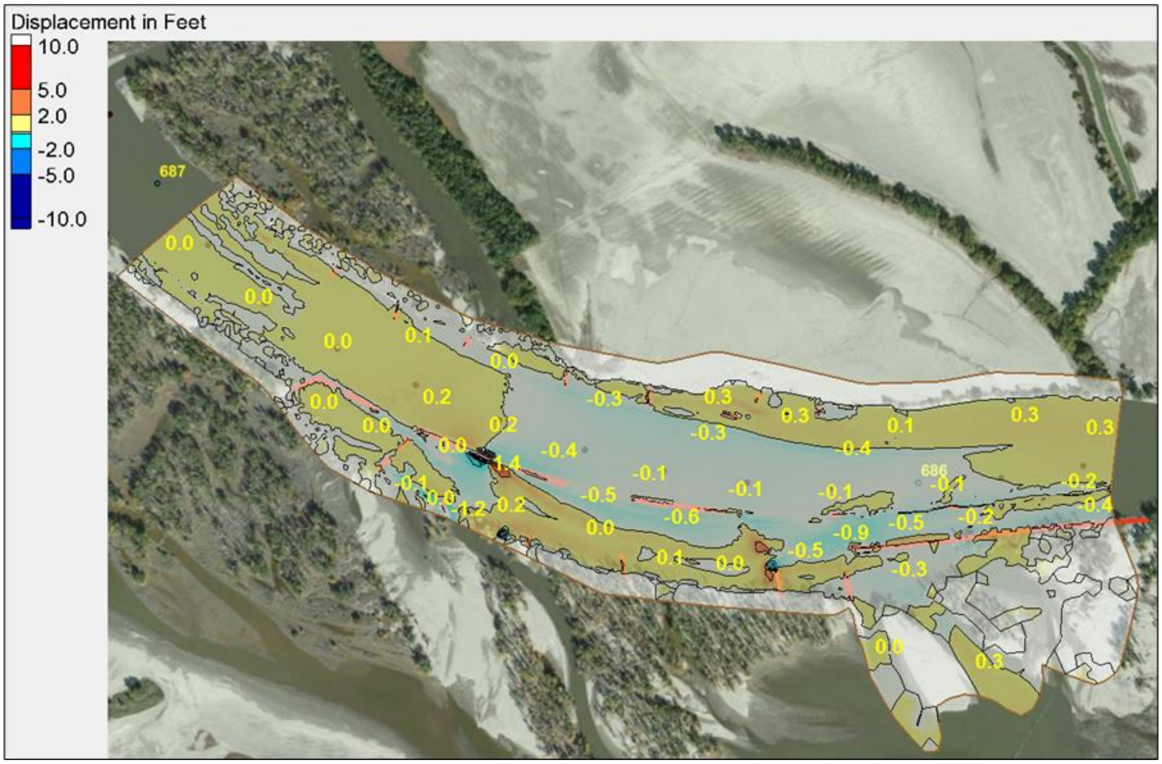
(9) Based on these results, implementation of Alternative 3 is recommended. However, continued monitoring during and following construction is required to track channel depth. Model results also illustrate that Alternative 3 is not likely to be wholly successful and the additional modifications will be required.



Case Study 9A Figure 21. Difference in displacement (computed as Alterative 3 − base, in feet) at 31,800 cfs (feet)



Case Study 9A Figure 22. Difference in displacement (computed as Alternative 3 – base, in feet) at 58,200 cfs (feet)



Case Study 9A Figure 23. Difference in displacement (computed as Alternative 3 – base, in feet) at 87,200 cfs (feet)

Case Study 10A
Extreme Events, Sedimentation Processes, and Uncertainty Considerations

1. Case Study.

This case study provides content condensed from multiple locations to illustrate extreme event consequences on sedimentation processes. Principles may be applied to USACE new project design and also the operation, maintenance, and repair of existing projects.

2. Introduction.

A difficult task for the design engineer is examining how extreme events may alter sedimentation processes and consequences to USACE flood damage reduction project performance. Many river systems will undergo a significant shift in sedimentation processes, cross a stability threshold, and experience a shift in the stage-damage relationship. These river alterations can have catastrophic impacts on the surrounding community.

a. Impact to Flood Risk Management Project. USACE flood risk management project designs are based on detailed hydrologic and hydraulic modeling. In most cases, these models are calibrated to available data that includes few, if any, extreme events. However, dramatically reducing damage from extreme events is the intended project function. Robust design methods, as explained in this manual, are employed to reduce the risk of project non-performance due to unanticipated hydrologic and sedimentation processes.

b. Case Study Purpose. The purpose of this case study is to present example project consequences from extreme events due to sedimentation processes. Extreme event impacts are usually project and event specific. Therefore, the intent of this case study is to present numerous extreme event consequences and highlight economic damages. Application to specific USACE projects will require thoughtful and detailed application through the USACE design process.

c. Example Events and Application to USACE Project Design. A series of example events are provided to illustrate the magnitude of problems that can be encountered. These examples are limited in both detail and discussion. The examples are meant to provide real-world problems that have occurred and implications for USACE project performance. Application to USACE project design presents a challenge for the Design Engineer to determine how a reasonable interpretation of extreme events may alter the river system. After this determination, the design may be improved to include appropriate features to mitigate risks to USACE project performance.

(1) River systems often undergo a significant change in sedimentation processes (sediment load, grain size, transport capacity) when comparing minor to extreme events.

(2) Extreme events may cross stability thresholds with rapid changes in river processes.

(3) Identification of some extreme events may require including risk mitigation features in USACE project design (such as buried rock revetment, levee toe scour protection, cutoff walls, flow diversion, debris basins, etc.).

(4) Extreme event sedimentation processes may be addressed through robust analysis and included in USACE risk and uncertainty analysis.

3. Sedimentation Processes and Threshold Effects.

USACE project construction can affect river sedimentation processes with potentially unforeseen responses during project life. In addition, relatively small changes in climate or land use may trigger large changes in river sedimentation processes. These phenomena are often expressed by the concept of geomorphic thresholds and channel evolution models. Further description of these processes in available in EM 1110-2-1418 (USACE 1994). Two examples are provided in the following sections.

a. Example – Corte Madera Creek. Corte Madera Creek, located in Marin County, California, discharges into San Francisco Bay about nine miles north of the Golden Gate bridge. A USACE flood damage reduction project was designed to contain the standard project flood with construction completed on the first three units in 1972.

(1) The downstream 2.2 miles of the project is an earthen channel, dredged to elevation -12.02 with a bottom width of 80 feet and side slopes of 1V: 6H. The next 0.7 mile of earthen channel has a bottom slope of 0.0007, a 30-foot bottom width, and 1V: 6H side slopes. The next mile of the project consists of a 33-foot-wide concrete channel with a stilling basin at the downstream end. The first 1,000 feet of this channel has a mild slope of 0.0007. The remainder of the concrete channel has a steep slope of 0.0038 and is designed for supercritical flow (Copeland 2000).

(2) Following project construction, additional studies were conducted to evaluate design of the upstream reach design (Unit 4), project performance of several floods in the 1980s, and the effect of sediment deposition in the project. Copeland (2000) summarized the results of this analysis.

(3) The sedimentation study noted deviation in hydraulic roughness in the concrete channel where the wall roughness was noted to vary depending on the presence of tube worms and barnacles. Tube worms and barnacles significantly increase the wall roughness as they protrude as much as 2 inches from the wall. Tube worm and barnacle growth exposed during a 1998 dredging operation is shown in Figure 1. In the numerical model, the lower wall (below elevation 0.1) was assigned a roughness value of 0.021. The upper wall was assigned a roughness value of 0.014.

Case Study 10A Figure 1. Corte Madera concrete-lined channel, tube worm and barnacle growth (Copeland 2000)

(4) The bottom or bed roughness of the concrete channel in reaches where there was no sediment deposition was set equal to 0.017. This value is higher than normally assigned to concrete. The higher roughness assigned to the bed of Corte Madera Creek is attributed to concrete abrasion (Figure 2) and to fish rests indented into the channel invert.

Case Study 10A Figure 2. Concrete channel invert showing effects of abrasion (Copeland 2000)

(5) The numerical study also noted that additional bed roughness occurs because of the movement of gravel bedload over the concrete surface. A relationship between gravel concentration and increase in bed roughness caused by gravel bedload transport was developed from flume studies at ERDC (Copeland, McVan, and Stonestreet 2000).

(6) Conclusions from the study noted several key concepts related to sedimentation processes (Copeland 2000).

(a) Sediment deposition occurred in the lower reaches of Corte Madera Creek due to the channel invert elevation below sea level. Sediment deposits and additional concrete channel roughness reduce the channel capacity.

(b) Study results determined that annual removal of sediment deposits and aquatic growth from the concrete channel would reduce hydraulic roughness such that most of the sediment deposited in the concrete channel from antecedent flow can be washed out of the channel by the time the flood peak occurs.

(c) The most important feature of the flood control project, in terms of allowing sediment deposits to be washed out, is upstream containment of breakout flows. Under existing conditions, when the discharge exceeds about 3,000 cfs, flow breaks out of Corte Madera Creek and flows away from the channel. Containment of breakout flows will require construction of floodwalls and/or channel improvement in the Unit 4 reach.

(d) Without upstream containment, channel flows are reduced, and the deposited sediment will not wash out of the concrete channel for the design flood even with annual cleanout.

(e) Another important feature of the flood control project, in terms of allowing sediment deposits to be washed out, is a sediment basin upstream from the concrete channel. To be effective, the sediment basin must have sufficient capacity to trap most of the coarsest sediment on the rising limb of the annual hydrograph.

(f) Coarse sediment deposits are more difficult to re-entrain and move out of the concrete channel than fine sediment deposits.

(g) A more detailed study to determine average annual maintenance quantities was recommended.

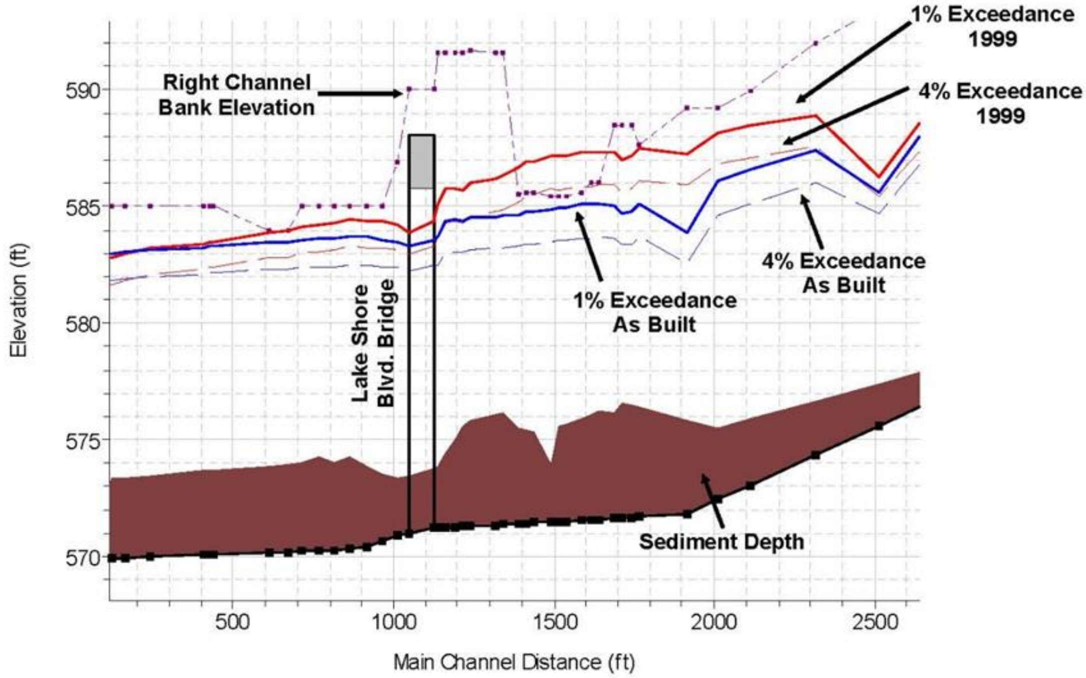
b. Example – Euclid Creek. Euclid Creek is located in the town of Euclid, Ohio, east of Cleveland. USACE (Buffalo District) designed and completed construction on a flood damage reduction project in 1985 to convey the 1% ACE (100-year). Following construction, shoaling concerns with sediment deposition were noted and a sedimentation analysis was conducted by the Buffalo District. Sediment deposition within the channel in 2004 is shown in Figure 3.

Case Study 10A Figure 3. Euclid Creek sediment deposition (2004)

(1) The sedimentation analysis reported several pertinent factors related to causes of deposition. The channel lining transitions between the concrete and the rip rap/earthen dam. In addition, the slope changes from steep to shallow, the channel widens, and the roughness of the channel increases. This area also corresponds with the location of the upstream end of the backwater effects of Lake Erie. All of these factors combine to result in a significant slowing of flow velocity in the slope and channel width transition area.

(2) The hydraulic model projects the occurrence of a hydraulic jump as the flow transitions from super critical to sub-critical flow between the upstream steep slope concrete lined channel and the deposition zone. Velocity reduction results in less energy and a reduction in sediment transport capacity with resulting deposition. Profile comparison of the as-built and a 1999 modeled condition are shown in Figure 4.

(3) Modeling results demonstrated the hydraulic conditions that are causing sediment deposition. Results also demonstrated a significant impact to FRM project performance.



Case Study 10A Figure 4. Euclid Creek, water surface profiles for the as-built (1985) and 1999 conditions

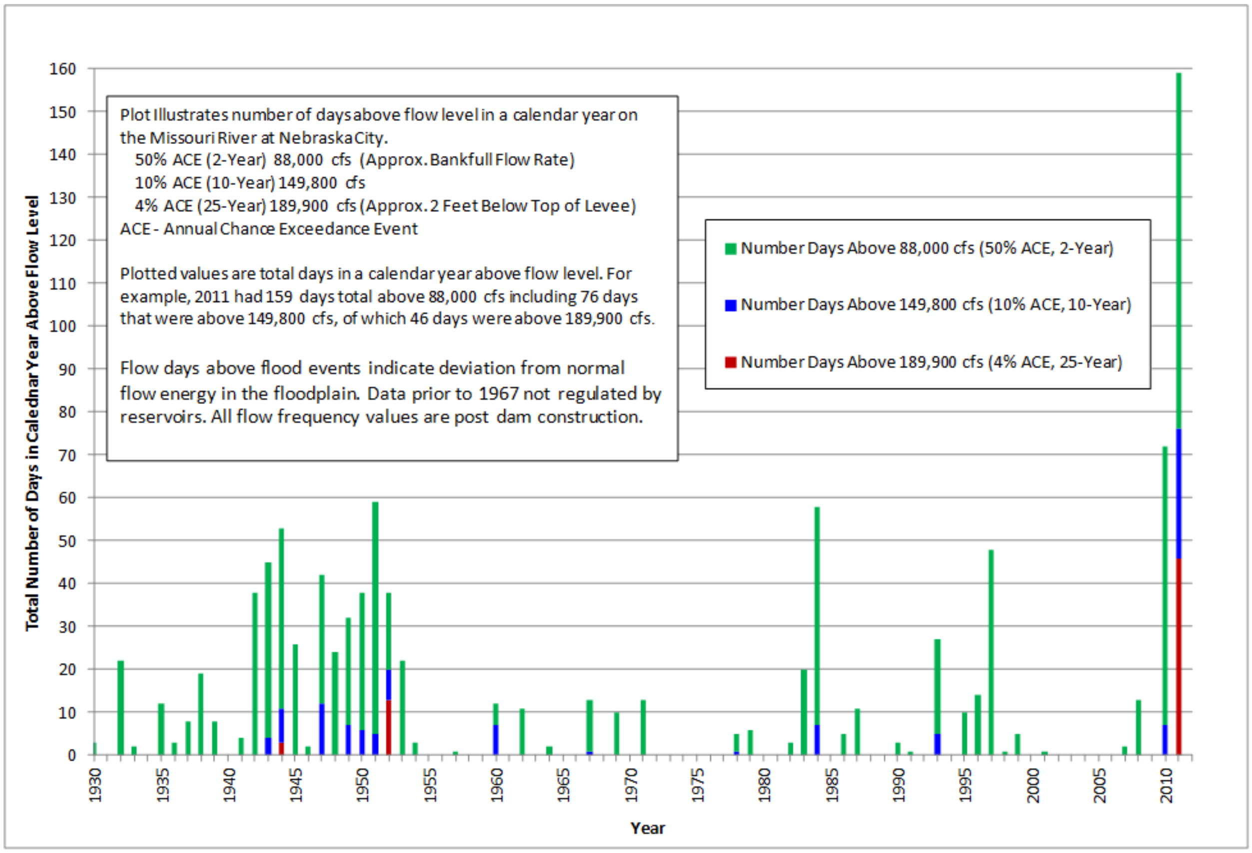
4. Sediment Processes and Duration – Missouri River Example.

Sedimentation models typically rely on historic data and hydrologic simulations to provide hydrologic information during project plan formulation. However, river sedimentation processes and USACE project design performance can be impacted by conditions that are greatly different than the historic record. The 2011 event on the Missouri River provides an example of the impact of sedimentation processes impacted by long-duration reservoir releases at a high level that were necessary to evacuate flood pool storage. Reservoir releases were sediment-free with excess sediment transport energy. Annual runoff volume frequency for the 2011 event has been characterized as a 0.2% annual chance exceedance (500-year) event (Grigg et al., 2012).

a. 2011 Event Flow Duration. On the Missouri River, the 2-year event can be assumed to roughly correspond with floodplain flow initiation in the current condition. In a simple method used to compare the 2011 event floodplain occurrence and flow duration to historic events, the number of days when the Missouri River flowed within the floodplain were tabulated.

(1) The 2011 event floodplain flow duration, as indicated by the number of days above channel capacity, was unprecedented in the 85 years of historic record. No flow events since upstream dam construction in the 1950s exceeded the 4% chance ACE (25-year) for a single day. The 2011 event had 44 consecutive days above that level.

(2) Data from the USGS gage at Nebraska City, Nebraska, which is located about 250 river miles downstream of Gavins Point Dam, is shown in Figure 5. The Missouri River channel and floodplain experienced high damages as sedimentation processes occurred over a long duration with large magnitudes of both scour and deposition.

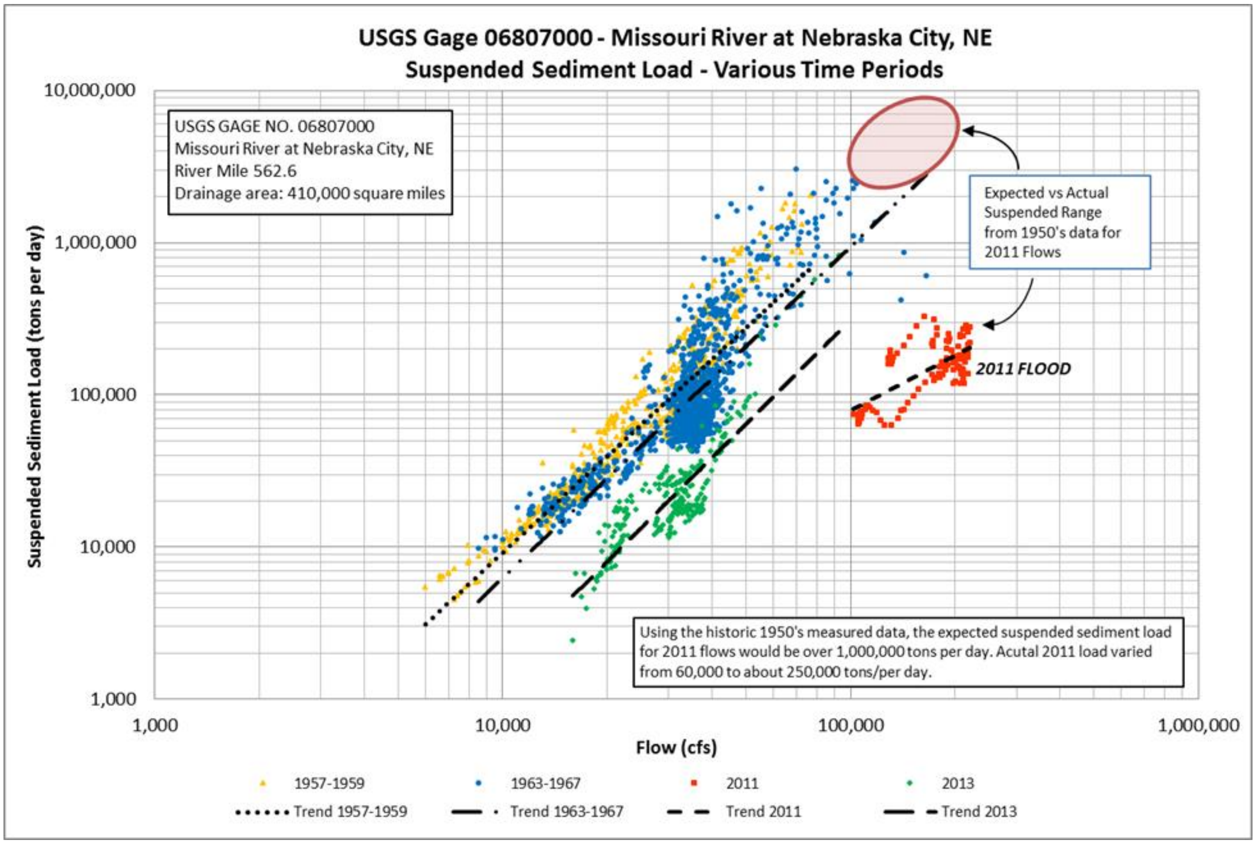


Case Study 10A Figure 5. Missouri River at Nebraska City, days above flow value by year

b. Suspended-Sediment Variation. Sediment measurements were conducted at a number of locations on the Missouri River during the 2011 event. The sediment measurements during the 2011 event were compared to historic measurements from both the pre- and post-dam era.

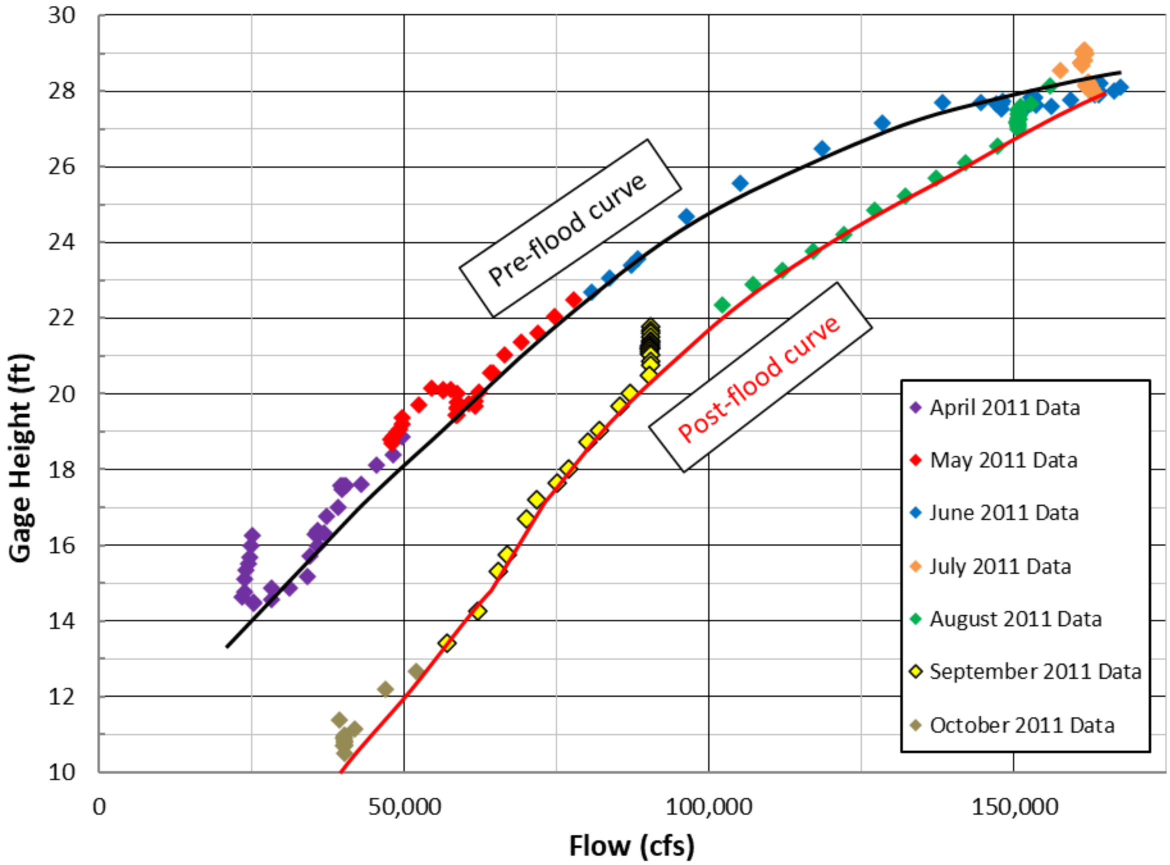
(1) Measurements indicate a dramatic difference in the suspended sediment vs. flow relationship for the 2011 event compared to historic data from the 1950s and 1960s. Suspended sediment is also much different than that measured in 2013. The 2011 suspended-sediment levels, which originated from upstream reservoir releases and were affected by sediment trapping within the reservoir system, were much lower than the historic data.

(2) Missouri River data illustrates that orders of magnitude difference can occur when comparing sediment processes for with and without project conditions. Sediment modeling should consider that historic sediment load may be significantly altered in the future. Suspended-sediment plots from 2011 and other periods for comparison are illustrated for Nebraska City, Nebraska in Figure 6.



Case Study 10A Figure 6. Missouri River at Nebraska City, suspended sediment variation with time

c. Event Scour and Deposition. Single-event scour or deposition can result in large magnitude impacts. Using the 2011 Missouri River event as a further example, the variation in the stage-flow relationship at Ponca, Nebraska, from pre- to post-event is shown in Figure 7. Ponca is located approximately 60 river miles below Gavins Point Dam. The vast difference between the pre- and post-flood rating curve shown at this site is the result of sedimentation processes. In this case, approximately 7 feet of scour occurred at normal flow level range of 30,000 to 50,000 cfs. Scour of this magnitude could severely impact adjacent USACE project features and infrastructure.



Case Study 10A Figure 7. Missouri River at Ponca, Nebraska, gage height vs. flow during 2011 event

5. Considerations for Economic Damages Affected by Sediment Transport in Extreme Events.

USACE project designs may need to consider economic damages related to sedimentation processes. For example, floodplain deposits can damage urban and agricultural areas that affect agricultural production and can incur substantial sediment cleanup costs. Two examples are included that illustrate sediment processes that results in high damage levels that may not be considered in typical USACE project plan formulation.

a. Example – Colorado 2013. In the steep gradient rivers of Colorado, significant damage resulted from not only high water levels but also the sediment processes that led to channel avulsion and large scale sediment transport through the urban floodplain areas.

(1) Background. A cold front moved across northeast and north central Colorado on 9 September, setting the stage for record rainfall. By 11 September, moderate rainfall began to intensify and become more widespread, eventually becoming very heavy in Jefferson, Boulder, and Larimer counties along the Colorado Front Range. The heavy rains resulted in flash flooding in many communities. The city of Boulder reported several precipitation records including a 24-

hour value of 9.1 inches. Seven-day rainfall totals of 13–17 inches occurred for much of central Boulder County.

(2) Event Impacts. Event impacts are illustrated in several areas with severe sedimentation consequences.

(a) In Jamestown, Colorado, deposited sediment completely filled the main channel and bridge opening after flows broke out upstream and reduced available energy for sediment transport (Figure 8).

Case Study 10A Figure 8. Jamestown, Colorado, October 2013, illustrating Main St. bridge blockage from sediment post flood and after cleanout

(b) Severe stream bank erosion (resulted in major road damage) and sediment deposition (caused both damage and additional cleanup costs) occurred in Jamestown, Colorado (Figure 9). Due to extreme road erosion, community access was heavily restricted with additional socio-economic impacts.

Case Study 10A Figure 9. Jamestown, Colorado, road damage and sediment deposition in community

(c) In Lyons, Colorado, the main bridge at McConnell Drive was bypassed when flood flows reconnected through several old mining pits (Figure 10). Severe damage resulted to McConnell Drive as a result of overtopping flood flows. Repairs consisted of sediment removal and a long linear dike to confine creek flows to the former channel. Post-flood repair projects that require sediment removal and channel realignment can encounter large expenses when infrastructure is damaged and large river training structures are required.

Case Study 10A Figure 10. McConnell Drive, Lyons, Colorado, post- and pre-flood

b. Example – Missouri River 2011. During the 1993 and 2011 floods, the Missouri River deposited so much sediment in the floodplain that, in some places, removing deposited material to restore agriculture productivity was cost-prohibitive.

(1) Deposit Impacts. Deposits of infertile sand on a highly productive silt loam that is high in organic matter and nutrients can significantly decrease the quality of the silt loam. Deep tillage may be required for soil mixing to restore productivity. Factors include thickness of sand deposits, size and cost of tillage equipment, soil type prior to the flood, and farm commodity prices. For instance, a sand deposit of 12 inches over fertile loam soil would require tillage depth of 22 inches (USDA 1993). For great depths, deposited material removal is likely required. Expenses for soil removal or remediation using techniques such as deep tillage can be significant.

(2) Material Location and Depth. Material deposition location and depth within the floodplain is typically highly variable, related to multiple factors including vegetation, bend geometry, levee alignment, infrastructure such as roads, and similar. Floodplain scour at depths in the range of 10 to 20 feet also occurred in the floodplain. Both deposition and scour are illustrated in Figure 11.

Case Study 10A Figure 11. (a) Missouri River 2011, sand deposition in floodplain; (b) Missouri River 2011, intermittent deep scour zones in floodplain

(3) Levee Breach Scour and Deposition. Levee breaches are often associated with large-scale scour and deposition in the floodplain. Levee breaches occurred in several locations during the 2011 event along the Missouri River. Breaches typically developed scour depths in the range of 30 to 50 feet. Post-flood repairs at these breach locations required large setbacks to avoid the deep scour depths at considerable expense. Damage to the area adjacent to the breach was extensive, with a large scour zone surrounded by large sediment depths on the floodplain. Upstream levee breaches typically result in extensive flooding, scour, and deposition throughout the levee unit, landward of the river, until reaching the downstream levee breach return area. Figure 12 illustrates both exit and return flow impacts on the Missouri River in Omaha District following the extreme 2011 flow event.

Case Study 10A Figure 12. (a) Return flow from levee breach area; (b) breach from river with deep scour hole, deposited sediment materials, and debris

c. Restoration Projects. USACE restoration projects are often designed with less stringent requirements for stability. Projects often have a goal to restore natural stream function which includes river dynamics. The 2011 Missouri river severely damaged several restoration projects with sediment deposition, erosion, and structure damage. At several projects, repair cost exceeded the initial project construction cost.

(1) At Tobacco Bend, the upstream chute entrance and rock control structure was damaged to the point that repair was necessary to maintain navigation channel reliability Figure 13.

Case Study 10A Figure 13. Missouri River chute restoration project at Tobacco Bend, pre- and post-2011 flood event

(2) Missouri River restoration included many projects that were constructed to create new floodplain chute and backwater areas. At several of the backwater projects, sediment deposition completely filled backwater to an elevation greater than the pre-project conditions. At several of the chute projects, portions of the chutes were filled with sediment that prevented flow-through connection to the main Missouri River. At other chute projects, the chute eroded and greatly exceeded design flow rates with possible impacts to the surrounding infrastructure and navigation channel depths. Restoration included sediment removal and new rock structures at significant cost.

6. Alluvial Fans.

Alluvial fans occur in many USACE project regions and can result in catastrophic damages. Alluvial fans are gently sloping landforms that are often fan- or cone-shaped, that have been created over a long-time scale by sediment deposition at the base of mountain ranges. Once in the valley, the stream is unconfined and can migrate back and forth, depositing alluvial sediments across a broad area.

a. Alluvial fan impacted areas are subjected to floods and debris flows induced intense and prolonged rainfall. Floods on alluvial fans commonly occur with little to no warning, and have high velocities and sediment-transporting capabilities (FEMA 2012, 2016). Communities on alluvial fans may be at risk of severe periodic sedimentation and flooding hazards.

b. During the spring of 1983, widespread flooding and mudflows caused an estimated $250 million in damages to Davis County communities located on numerous alluvial fans along the base of the Wasatch Mountains in Utah (Wieczorek et al., 1983). Refer to Chapter 7 of this manual for a further discussion of alluvial fans.

7. Debris Flows.

Debris flows are a high-density slurry mixture of water, sediments (boulders, cobbles, soil), woody debris, and mud that can have enormous destructive power, particularly when they are fast-moving. Refer to Chapter 7 of this manual for further details on evaluating debris flows. Two debris flow examples are provided to illustrate the catastrophic damage associated with these types of events.

a. Example – Mount St. Helens. Mount St. Helens erupted 18 May 1980, blasting more than three billion cubic yards of volcanic ash and debris, and triggering an immense landslide of mud and rock. The eruption dramatically altered the hydraulic and hydrologic regimes of the Cowlitz and Toutle River valleys. Ash fall and lateral blast from the eruption produced immediate and long-term effects on the hydrology of the Toutle watershed by changing its land cover and runoff characteristics.

(1) The excessive amount of sediment produced by the eruption and its aftermath was deposited downstream in the lower Toutle, Cowlitz, and Columbia rivers. The USACE Portland District has performed multiple studies to find long-term solutions to manage the continuing flow of sediment and reduce flood risk.

(2) Projects include the Spirit Lake Tunnel, completed in 1985, to help stabilize the lake's water levels. The sediment retention structure (SRS), completed in 1989, keeps hundreds of millions of cubic yards of sediment from the Toutle River, preventing significant flooding and navigation problems. Currently, sediment from the Mount St. Helens debris avalanche continues to cause flooding concerns to residents of several nearby communities. The Portland District raised the spillway of the existing sediment retention structure with construction completed in 2013 to increase trapping efficiency and sediment storage capacity (USACE 2012b). Figure 14 illustrates project locations.

**Case Study 10A Figure 14. (a) Mt St Helens debris flow;
(b) sediment retention structure, Toutle River, 1990 (USACE 2012b)**

b. Example – Caraballeda, Venezuela. The torrential flows that took place in the north coastal range of Venezuela (state of Vargas) in December 1999, resulted in the occurrence of extreme debris flows in about 20 streams along 50 km of a narrow coastal strip (Lopez et al., 2003). The disaster caused losses of more than $2 billion and killed an estimated 20,000 people. In terms of human losses, this was the worst natural disaster in Venezuelan history and one of the worst in South America (Wieczorek et al., 2001). An illustration of the debris flow and alluvial fan, at Caraballeda, Venezuela, is included in Figure 15 (USGS 2001).

**Case Study 10A Figure 15. (a) Caraballeda, Venezuela alluvial fan with debris flow, showing an estimated 1.8 million tons of deposited sediment (USGS 2001);
(b) apartment structure damage and debris flow material (USGS 2001)**

8. Summary.

USACE project design follows a detailed evaluation process as described throughout this manual. This case study provided examples of how USACE project performance was affected by large-scale system changes that occurred during extreme events. Project design features should be included to sediment processes as warranted. A robust risk analysis should consider how sediment processes can significantly alter the stage-flow and stage-damage relationships during extreme events. These factors should also be considered in the risk register.

a. General Considerations.

(1) It is not practical to attempt to identify all possible future scenarios, and caution should be used to prevent the adoption of overly conservative risk and uncertainty.

(2) A robust evaluation of risk and uncertainty should consider how project features have affected future sediment processes.

(3) Adequately considering sedimentation process transformation during extreme events, including both floods and droughts, is an important factor that should not be ignored when considering project performance.

(4) Sediment processes are generally episodic with large-scale system responses observed. Channel cutoffs, lateral migration, and large amounts of degradation or aggradation are common occurrences on river systems in extreme events.

b. Application to Project Design. The presented examples illustrate cases where sedimentation processes played a significant role in project performance and event damage. Highlights for consideration applicable to USACE project design are:

(1) USACE reservoirs with large flood storage volume significantly reduce downstream peak discharge. However, evacuation of the flood storage pool can result in sustained high flows with a much greater duration.

(2) Varying with trap efficiency, reservoir releases will have low suspended-sediment concentrations, and may be orders of magnitude less, leading to high rates of degradation and bank erosion in the downstream channel.

(3) High reservoir releases can result in large-scale sedimentation processes during a single event. Consequences due to large-magnitude scour and deposition should be considered.

(4) The magnitude of change due to sediment processes can overwhelm variability due to other processes. An example was presented of the 7 feet of scour that occurred on the Missouri River during 2011.

(5) Flow duration and sediment concentrations may be significantly altered in the future compared to the historic. The Missouri River example identified reservoir operations as one

factor. Watershed development, agricultural practices, climate change, and other water resource projects can also alter future conditions from the historic.

c. Alluvial Fans and Debris Flows. Hazards from infrequent events, such as the eruption of Mount St. Helens, are not normally included in a risk and uncertainty analysis. However, consequences from these types of extreme events can be catastrophic. Many extreme events can be anticipated and should be addressed in project design:

(1) Alluvial fans and debris flows, although rare, should be accounted for both in USACE project design and risk and uncertainty analysis.

(2) The presence of these features should be identified early in the USACE design process to allow for performing the additional project design necessary and including costs for necessary project features (such as debris basins).

Glossary

Accelerated Erosion

Erosion at a rate greater than normal, usually associated with human activities that reduce plant cover and increase runoff. See also geologic erosion.

Accuracy

The degree of conformity of a measure to a standard or true value.

Active Bed

The layer of material between the bed surface and a hypothetical depth at which no transport will occur for the given gradation of bed material and flow conditions. See also active layer.

Active Layer

The depth of material from bed surface to equilibrium depth continually mixed by the flow, but it can have a surface of slow-moving particles that shield the finer particles from being entrained by the flow.

Aggradation

The geologic process by which stream beds, flood plains, and the bottoms of other water bodies are raised in elevation by the deposition of material eroded and transported from other areas. It is the opposite of degradation.

Algorithm

A procedure for solving a mathematical problem in a finite number of steps that frequently involves repetition of an operation; a step-by-step procedure for solving a problem or accomplishing an end; or a set of numerical steps or routines to obtain a numerical output from a numerical input.

Aliquot

A fractional part representative of the whole.

Alluvial

Pertains to alluvium deposited by a stream or flowing water.

Alluvial Channel

See alluvial stream.

Alluvial Deposit

Clay, silt, sand, gravel, or other sediment deposited by running or receding water.

Alluvial Reach

A reach of river with a sediment bed composed of the same type of sediment material as that moving in the stream.

Alluvial Stream

A stream, the channel boundary of which is composed of appreciable quantities of the sediments transported by the flow, and which generally changes its bed forms as the rate of flow changes. Alluvium. A general term for all detrital deposits resulting directly or indirectly from the sediment transported of (modern) streams, thus including the sediments laid down in river beds, flood plains, lakes, fans, and estuaries.

Alluvium

A general term for all detrital deposits resulting directly or indirectly from the sediment transported by (modern) streams, thus including the sediments laid down in riverbeds, floodplains, lakes, fans, and estuaries.

Anti-Dune

A series of general sinusoidal-shaped bed forms that commonly move upstream accompanied by in-phase waves on the water surface. Anti-dunes develop in a sand-bed stream where the Froude number is close to or greater than one.

Armoring

The process of progressive coarsening of the bed layer by removal of fine particles until it becomes resistant to scour. The coarse layer that remains on the surface is termed the "armor layer." Armoring is a temporary condition; higher flows may destroy an armor layer and it may re-form as flows decrease. Or simply, the formation of a resistant layer of relatively large particles resulting from removal of finer particles by erosion.

Average End Concept

The averaging of the two end cross sections of a reach to smooth the numerical results.

Backwater Profile

Longitudinal profile of the water surface in a stream where the water surface is raised above its normal level by a natural or artificial obstruction.

Bank Sediment Reservoir

The portion of the alluvium on the sides of a channel.

Bed Forms

Irregularities found on the bottom (bed) of a stream that are related to flow characteristics. They are given names such as dunes, ripples, and anti-dunes. They are related to the transport of sediment and interact with the flow because they change the roughness of the stream bed. An analog to stream bed forms are desert sand dunes (although the physical mechanisms for their creation and movement may be different).

Bed Layer

An arbitrary term used in various procedures for computation of sediment transport. From observation of slow-motion movies of laboratory flume experiments, H. Einstein defined the "bed layer" as: "A flow layer, two grain diameters thick, immediately above the bed. The thickness of the bed layer varies with the particle size."

Bedload

Material moving on or near the stream bed by rolling, sliding, and sometimes making brief excursions into the flow a few diameters above the bed.

Bedload Discharge

The quantity of bedload passing a cross section in a unit of time.

Bedload Sampler

A device for sampling the bedload.

Bed Material

The sediment mixture of which the bed is composed. In alluvial streams, bed material particles are likely to be moved at any moment or during some future flow condition.

Bed Material Discharge

The total rate (tons/day) at which bed material (see bed material) is transported by a given flow at a given location on a stream.

Bed Material Load

The total rate (tons/day) at which bed material is transported by a given location stream. It consists of bed material moving both as bedload and suspended load.

Bed Rock

A general term for the rock, usually solid, that underlies soil or other unconsolidated, bed material.

Bed Sediment Control Volume

The source-sink component of sediment sources in a river system (the other component is the suspended sediment in the inflowing discharge). Its user-defined dimensions are the movable-bed width and depth, and the average reach length.

Bottomset Bed

Fine-grained material (usually silts and clays) that is slowly deposited on the bed of a quiescent body of water, and that may in time be buried by foreset beds and topset beds.

Boundary Conditions

Definition or statement of conditions or phenomena at the boundaries. Water surface elevations, flows, sediment concentrations, etc., that are specified at the boundaries of the area being modeled. The downstream water surface elevation and the incoming upstream water and sediment discharges are the standard HEC-6 boundary conditions.

Boundary Roughness

The roughness of the bed and banks of a stream or river. The greater the roughness, the greater the frictional resistance to flows and, hence, the greater the water surface elevation for any given discharge.

Boulder

See Glossary Table 1.

Braided Channel

A stream that is characterized by random interconnected channels divided by islands or bars. Bars that divide the stream into separate channels at low flows are often submerged at high flow.

Channel

A natural or artificial waterway that periodically or continuously contains moving water.

Channel Invert

The lowest point in the channel.

Channel Stabilization

A stable channel is neither progressively aggrading, degrading, or changing its cross-sectional area through time. It could aggrade or degrade slightly, but over the period of a year, the channel remains similar in shape and dimensions and position to previous times. Unstable channels are depositing or eroding in response to some exterior conditions. Stabilization techniques consist of bank protection and other measures that work to transform an unstable channel into a stable one.

Clay

See Glossary Table 1.

Coagulation

The agglomeration of colloidal or finely divided suspended matter, generally caused by the addition of a chemical coagulant.

Cobbles

See Glossary Table 1.

Cohesive Sediments

Sediments whose resistance to initial movement or erosion is affected mostly by cohesive bonds between particles.

Colloids

Finely divided solids that do not settle in a liquid, but that may be coagulated chemically or biochemically. See Glossary Table 1.

Composite Sample

A sample formed by combining two or more individual samples, or representative portions thereof.

Computational Hydrograph

Sequence of discrete steady flows, each having a specified duration in days, is used to represent the continuous discharge hydrograph. This is done to minimize the number of time steps needed to simulate a given time period, and thus minimize computer time.

Concentration of Sediment

The dry weight of sediment per unit volume of water-sediment mixture (such as mg/1). (*Note*: In earlier writings, concentration was calculated as the ratio of the dry weight of sediment in a water-sediment mixture to the total weight of the mixture divided by 1,000,000. It was expressed as parts per million (ppm). Either method gives the same result, within 1%, for concentrations up to 16,000 mg/1. A correction is needed for concentrations in excess of that value.)

Conceptual Model

A simplification of prototype behavior used to demonstrate concepts.

Consolidation

The compaction of deposited sediments caused by grain reorientation and by the squeezing out of water trapped in the pores.

Control Point

The downstream boundary of the main river segment and the junction point of each tributary.

Convergence

The state of tending to a unique solution. A given scheme is convergent if an increasingly finer computational grid leads to a more accurate solution.

Conveyance

A measure of the carrying capacity of the channel section. Flow is directly proportional to conveyance for steady flow. From Manning's equation, the proportionality factor is the square root of the energy slope.

Cover Layer

One of the two sublayers of the active layer. It lies above the subsurface layer (the second sublayer in the active layer.

Critical Bed Shear Stress

See critical tractive force.

Critical Depth

If discharge is held constant and the water depth allowed to decrease, as in the case of water approaching a free overfall, velocity head will increase, pressure head will decrease, and total energy will decrease toward a minimum value where the rate of decrease in the pressure head is just counter-balanced by the rate of increase in velocity head. This is the critical depth. More generally, the critical depth is the depth of flow that would produce the minimum total energy head.

Critical Flow

The state of flow where the water depth is at the critical depth and when the inertial and gravitational forces are equal.

Critical Tractive Force

The critical tractive force is the maximum unit tractive force that will not cause serious erosion of the material forming the channel bed on a level surface.

Cross Section

Depicts the shape of the channel in which a stream flows. Measured by surveying the stream bed elevation across the stream on a line perpendicular to the flow. Necessary data for the computation of hydraulic and sediment transport information.

Cross-Sectional Area

The area of a cross section between the stream bed and the water surface.

Degradation

The geologic process by which stream beds, flood plains, and the bottoms of other water bodies are lowered in elevation by the removal of material from the boundary. It is the opposite of aggradation.

Delta

A deposit of sediment formed where moving water (as from a stream at its mouth) is slowed by a body of standing water.

Density

The mass of a substance per unit volume. In the English system, the units are pounds-seconds square/feet to the fourth power. In the metric system, the units are kg/L. The Greek letter “rho” is the common symbol.

Density Current

A highly turbid mixture of water and very fine-grained sediment that flows into and along the bottom of a reservoir because its density is relatively larger than that of the standing water in the reservoir.

Deposition

The mechanical or chemical processes through which sediments accumulate in a resting place.

Depth-Integrated Sample

A sample of the water-sediment mixture collected at a vertical consistent with the technique of depth integration. The sample is used to determine the sediment discharge and the range of particle sizes in that discharge (such as the sediment load in that discharge).

Depth of Flow

The depth of flow is the vertical distance from the bed of a stream to the water surface.

Depth-Integrating, Suspended-Sediment Sampler

An instrument designed to be lowered to within a few inches of the stream bed while collecting a water-sediment mixture isokinetically into a bottle. Sampling starts automatically as the instrument enters the water and continues until the orifice breaks the water surface on the return trip from the bed. Hence, a sampler suitable for performing depth integration.

Depth Integration

A method of sampling the water-sediment mixture in a flowing stream whereby the sampling instrument is lowered down to the bottom and returned to the surface in a continuous motion and at the proper rate to collect a discharge-weighted sample of the mixture. Ordinarily, depth integration is performed by traversing the water column with a depth-integrating sampler. However, when the water is so deep or the current is so swift that a single bottle cannot contain the entire sample, depth integration is accomplished by partitioning the water column into layers, vertically, and lowering a point-integrating sampler through each layer separately. The valve on the point sampler allows the inflow orifice to be opened only for the layer being sampled. Depth integration has also been accomplished using a vertical-slot sampler.

Diameter, Standard Fall

See standard fall diameter.

Diameter, Standard Sediment

See standard sedimentation diameter.

Discharge

The discharge (Q) is the volume of a fluid or solid passing a cross section of a stream per unit time.

Discharge-Weighted Concentration

The dry mass (weight) of sediment in a unit volume of stream discharge, or the ratio of the mass discharge (dry) of sediment to the mass discharge of water-sediment mixture.

Disperse

To de-flocculate or disaggregate compound particles such as clays and fine silts into individual component particles (ultimate particles).

Dispersed System

A condition in particle size analyses whereby particles begin to settle from an initial uniform dispersion such that particles of equivalent fall diameters settle at the same rate.

Distributaries

Diverging streams that do not return to the main stream but discharge into another stream or the ocean.

Dissolved Load

The part of the stream load that is carried in solution, such as chemical ions yielded by weathering and erosion of the land mass.

Dissolved Solids

The mass of dissolved constituents in water determined by evaporating a sample to dryness, heating to 103-105 C for 2 hours, desiccating, and weighing.

Dominant Discharge

A particular magnitude of flow that is sometimes referred to as the “channel forming” discharge. Empirical relations have been developed between “equilibrium” stream width, depth, and slope and dominant discharge. It has been variously defined as the bankfull flow, mean annual discharge, etc.

Draft Depth

The depth measured perpendicularly from the water surface to the bottom of a boat, ship, etc. (such as a “clearance” depth).

Drainage Basin

The area tributary to or draining into a lake, stream, or measuring site. See also watershed.

Drop

A structure in an open conduit or canal installed for the purpose of dropping the water to a lower level and dissipating its energy. It may be vertical or inclined; in the latter case it is usually called a chute.

Dunes

Bed forms with triangular profile that advance downstream due to net deposition of particles on the steep downstream slope. Dunes move downstream at velocities that are small relative to the stream flow velocity.

Effective Grain Size

The diameter of the particles in an assumed rock or soil that would transmit water at the same rate as the rock or soil under consideration, and that is composed of spherical particles of equal size and arranged in a specific manner. The effective grain size is that single particle diameter that best depicts the bed material properties. The DSO grain size is often used as the effective grain size.

Entrainment

The carrying away of bed material produced by erosive action of moving water.

Equilibrium Depth

The minimum water depth for the condition of no sediment transport.

Equilibrium Load

The amount of sediment that a system can carry for a given discharge without an overall accumulation (deposit) or scour (degradation).

Equal-Discharge-Increment (Edi) Method

A procedure for obtaining the discharge-weighted SSC at a cross section by (1) collecting a depth-integrated sample at the center of equal-flow subsections across the cross section while (2) using vertical transit rates that provide the same volume of sample at each sampling vertical.

Equal Transite Rate

Obsolete, replaced by the term “equal width increment.”

Equal Width Increment (EWI) Method

A procedure for obtaining the discharge-weighted, suspended-sediment concentration of flow at a cross section by (1) performing depth integration at a series of verticals equally spaced across the cross section, and by (2) using the same vertical transit rate at all sampling verticals.

Erosion

The wearing away of the land surface by detachment and movement of soil and rock fragments through the action of moving water and other geological agents.

Fall Velocity

The falling or settling rate of a particle in a given medium.

Filtrate

The fluid that has passed through a filter.

Fixed-Bed Model

Model in which the bed and side materials are nonerodable. Deposition does not occur, as well.

Filtration

The process of passing a liquid through a filter to remove suspended matter that usually cannot be removed by settling. The filter may consist of granular material such as sand, magnetite, or diatomaceous earth, finely woven cloth, unglazed porcelain, or specially prepared paper.

Fine Material

Particles of a size finer than the particles present in appreciable quantities in the bed material; normally silt and clay particles (particles finer than 0.062 mm).

Fine Material Load

That part of the total sediment load that is composed of particles smaller than the particles present in appreciable quantities in the stream bed. Normally, that is of sediment particles smaller than 0.062 mm.

Flocculent

An agent that produces floes or aggregates from small suspended particles.

Flocculation Agent

A coagulating substance which, when added to water, forms a flocculent precipitate that will entrain suspended matter and expedite settling; for example, alum, ferrous sulfate, or lime.

Flow Duration Curve

A measure of the range and variability of a stream’s flow. The flow duration curve represents the percent of time during which specified flow rates are exceeded at a given location. This is usually presented as a graph of flow rate (discharge) vs. percent of time that flows are greater than, or equal to, that flow.

Fluvial

(1) Pertaining to streams, (2) growing or living in streams or ponds, or (3) produced by river action, as a fluvial plain.

Fluvial Geomorphology

The study of the origin and evolution of landforms shaped by river processes.

Fluvial Sediment

Particles derived from rocks or biological materials that are transported by, suspended in, or deposited by streams.

Foreset Bed

Included layers of sandy material deposited on or along an advancing and relatively steep frontal slope. A foreset bed progressively covers a bottomset bed, and in turn is covered by a topset bed.

Frequency

The number of repetitions of a periodic process in a certain time period.

Froude Number

A dimensionless number expressing the ratio between influence of inertia and gravity in a fluid.

Gaging Station

A selected cross section of a stream channel where one or more variables are measured continuously or periodically to index discharge and other parameters.

Geologic Erosion

The erosion process on a given landform that is not associated with the activities of man.

Geologic Control

A local rock formation or clay layer that limits (within the engineering time frame) the vertical and/or lateral movement of a stream at a particular point. Note that manufactured controls of each particle size, or the frequency distribution of various sizes, constituting a particulate material such as a soil sediment or sedimentary chosen arbitrarily. Four different gradations are significant: the gradation of the suspended load, the gradation of the bedload, the gradation of the material comprising the bed surface, and the gradation of material beneath the bed surface.

Geomorphology

The study of the origin and evolution of landforms.

Gradation Curve

Sediment samples usually contain a range of grain sizes, and it is customary to break this range into classes of percentages of the total sample weight contained in each class. After the individual percentages are accumulated, a graph, the "gradation curve," shows the grain size vs. the accumulated percent of material that is finer than that grain size. Movable boundary models use these curves to depict the bed sediment material properties

Grading

Degree of mixing of size classes in sedimentary material: well-graded implies a more or less uniform distribution from coarse to fine; poorly graded implies uniformity in size of lack of continuous distribution.

Grain Size

See particle size.

Grain Size Distribution (Gradation)

A measure of the variation in grain (particle) sizes within a mixture. Usually presented as a graph of grain diameter vs. percent of the mixture that is finer than that diameter.

Gravel

See Glossary Table 1.

Gross Erosion

The total of all sheet, gully, and channel erosion in a drainable basin, usually expressed in units of weight.

Historic Flows

The collection of recorded flow data for a stream during the period of time in which steam gages were in operation.

Hydraulic Model

A physical scale model of a river used for engineering studies.

Hydraulics

The study and computation of the characteristics such as depth (water surface elevation), velocity, and slope of water flowing in a stream or river.

Hydrograph

A graph showing, for a given point on a stream or conduit, the discharge, water surface elevation, stage, velocity, available power, or other property of water with respect to time.

Hydrology

The study of the properties, distribution, and circulation of water on the surface of the land, in the soil, and in the atmosphere.

Inactive Layer

The depth of material beneath the active layer.

Incipient Motion

The flow condition at which a given size bed particle just begins to move. Usually related to a "threshold" shear stress.

Ineffective Flow

When high ground or other obstruction such as a levee prevents water from flowing into a subsection, the area up to that point is ineffective for conveying flow and is not used for hydraulic computations until the water surface exceeds the top elevation of the obstruction. The barrier can be a natural levee, manufactured levee, or some other structure.

Inflowing Load Curve

See sediment rating curve.

Initial Conditions

The value of water levels, velocities, concentrations, etc., that are specified everywhere in the mesh at the beginning of a model run. For an iterative solution, the initial conditions represent the first estimate of the variables the model is trying to solve.

In Situ

In (its original) place.

Instantaneous Sampler

A suspended-sediment sampler that instantaneously traps a sample of the water-sediment mixture in a stream at a desired depth.

Isokinetic Sampling

To collect a water-sediment mixture at the velocity of the approaching flow; that is, the velocity of the mixture experiences no acceleration or deceleration as it leaves the ambient flow and enters the sampler intake.

Left Overbank

See overbank.

Load

See sediment load.

Local Inflow/Outflow Point

Points along any river segment at which water and sediment enter or exit that segment as a local flow. Each local inflow/outflow point is designated (n,m), where n is the segment number and m is the sequence number (going upstream) of the local inflow/outflow points located along segment n.

Local Scour

Erosion caused by an abrupt change in flow direction or velocity. Examples include erosion around bridge piers, downstream of stilling basins, at the ends of dikes, and near snags.

Ml and M2 Curves

Ml and M2 curves represent mild sloping water surface profiles.

Main Stem

The primary river segment with its outflow at the downstream end of the model.

Manning's Equation

The empirical Manning's equation commonly applied in water surface profile calculations defines the relationship between surface roughness, discharge, flow geometry, and rate of friction loss for a given stream location.

Manning's n Value

n is the coefficient of roughness that accounts for energy loss due to the friction between the bed and the water. In fluvial hydraulics (movable boundary hydraulics), the Manning's n value includes the effects of all losses, such as grain roughness of the movable bed, form roughness of the movable bed, bank irregularities, vegetation, bend losses, and junction losses. Contraction and expansion losses are not included in Manning's n but are typically accounted for separately.

Mathematical Model

A model that uses mathematical expressions (such as a set of equations, usually based on fundamental physical principles) to represent a physical process.

Meander Geometry

The five parameters commonly used in the description of meander patterns, including wavelength, radius of curvature, arc length, amplitude, and belt width.

Meander Length

The product of the meander wavelength and the valley slope divided by the channel slope.

Meandering Stream

An alluvial stream characterized in planform by a series of pronounced alternating bends. The shape and existence of the bends in a meandering stream are a result of alluvial processes and not determined by the nature of the terrain (geology) through which the stream flows.

Measured Sediment Discharge

The quantity of sediment passing a stream cross section in a unit of time as computed with data measured by sampling. Sampling with suspended-sediment samplers provides the measured sediment discharge of suspended-sediment. There will be an unmeasured sediment discharge that must be added to that value to obtain the total sediment discharge for the cross section.

Median Diameter

The sediment particle diameter for which one-half of the weight of the material is composed of particles larger than the median diameter, and the other half is composed of particles smaller than the median diameter.

Model

A representation of a physical process or thing that can be used to predict the process' or thing's behavior or state. Examples: A conceptual model: If one throws a rock harder, it will go faster; a mathematical model: F = ma; a hydraulic model: Columbia River physical model.

Movable Bed

That portion of a river channel cross section that is considered to be subject to erosion or deposition.

Movable-Bed Limits

The lateral limits of the movable bed that define where scour or deposition occur.

Movable-Bed Model

Model in which the bed and/or side material is erodible and transported in a manner similar to the prototype.

Native Water

Water from a water body that has been unaffected by sampling, handling, and preservation.

Network Model

A network model is a network of main stem, tributary, and local inflow and/or outflow points that can be simulated simultaneously and in which tributary sediment transport can be calculated.

Nephelometer

An instrument that measures the amount of light scattered in a suspension.

Noncohesive Sediments

Sediments consisting of discrete particles. For given erosive forces, the movement of such particles depends only on the properties of shape, size, and density, and on the position of the particle with respect to surrounding particles.

Normal Depth

The depth that would exist if the flow were uniform is called normal depth.

Numerical Experiments

Varying the input data, or internal parameters, of a numerical model to ascertain the impact on the output.

Numerical Model

A numerical model is the representation of a mathematical model as a sequence of instructions (program) for a computer. Given approximate data, the execution of this sequence of instructions yields an approximate solution to the set of equations that comprise the mathematical model.

One-Dimensional Energy Equation

This equation has the same form as the Bernoulli Equation and the same terms are present. In addition, a term has been added to correct for velocity distribution.

Operating Policy

See operating rule.

Operating Rule

The rule that specifies how water is managed throughout a water resource system. They are often defined to include target system states, such as storage, above which one course of action is implemented and below which another course is taken.

Optical Opacity

An expression for the amount of light absorbed and scattered by a suspension reported as (1) extinction coefficient, (2) percent of incident light scattered at 90 degrees, and/or (3) percent of incident light transmitted at 180 degrees over a standard distance.

Overbank

In a river reach, the surface area between the bank on the main channel and the limits of the floodplain.

Overdredging

The additional depth dredged beyond the minimum dredging depth used to provide sufficient navigational depth, to minimize re-dredging, and to help compensate for the sloughing off and resettling of sediment after dredging occurs.

Parameter

Any set of physical properties whose values determine the characteristics or behavior of something.

Particle Shape Factor

The particle shape factor of a perfect sphere is 1.0 and can be as low as 0.1 for very irregular shapes.

Particle Size

A linear dimension, usually designated as “diameter,” used to characterize the size of a particle. The dimension may be determined by any of several different techniques, including sedimentation sieving, micrometric measurement, or direct measurement. See Glossary Table 1.

Particle Size Classification

See sediment grade scale.

Particle Size Distribution

The frequency distribution of the relative amounts of particles in a sample that are within specified size ranges, or a cumulative frequency distribution of the relative amounts of particles coarser or finer than specified sizes. Relative amounts are usually expressed as percentages by weight.

Particle Size, Intermediate Axis

The size of a rock or sediment particle determined by direct measurement of the axis normal to a plane representing the longest and shortest axes.

Permeability

The property of a soil that permits the passage of water under a gradient of force.

Plane Bed

A sedimentary bed without elevations or depressions larger than the maximum size of the bed material.

Planform

Horizontal alignment of a channel; the shape and size of channel and overbank features as viewed from directly above.

Point Bar

A depositional area formed on the inside bank of a meander that sometimes remains bare of vegetation due to the frequent recurrence of the bankfull discharge.

Point-Integrating Sediment Sampler

An instrument capable of collecting a water-sediment mixture isokinetically for a specified period of time by opening and closing while under water. An instrument suitable for performing point integration.

Point-Integrated Sample (Point Sample)

A sample of water-sediment mixture collected at a relatively fixed point consistent with the technique of point integration. A point-integrated sample is discharge weighted. However, because the sample is obtained from a single point, the concentration of any component of the mixture that is transported exactly at stream velocity can be considered as either a spatial or a discharge-weighted concentration. Samples collected with instruments that instantaneously capture a quantity of water-sediment mixture are not true point-integrated samples.

Point Integration

A method of sampling at a relatively fixed point whereby the water-sediment mixture is withdrawn isokinetically for a specified period of time.

Pollution

The condition caused by the presence of substances of such character and in such quantities that the quality of the environment is impaired. See also water pollution.

Primary Tributary

A tributary that is directly connected to or that joins with the main river segment.

Prototype

The full-sized structure, system process, or phenomenon being modeled.

Pumping Sampler

A sampler with which the water-sediment mixture is withdrawn through a pipe or hose, the intake of which is placed at the desired sampling point.

Qualitative

Relating to or involving quality or kind.

Reservoir

An impounded body of water or controlled lake where water is collected and stored.

Residue

Material that remains after gases, liquids, or solids have been removed.

Rating Curve

See stage-discharge curve.

Reach

(1) The length of a channel, uniform with respect to discharge, depth, area, and slope (such as "study reach," "typical channel reach," or "degrading reach," etc.) or (2) the length of a stream between two specified gaging stations.

Right Overbank

See overbank.

Rill Erosion

Land erosion forming small, well-defined incisions in the land surface less than 30 centimeters in depth.

Ripple

Small triangular-shaped bed forms that are similar to dunes but have much smaller heights and lengths of 0.3 meter or less. They develop when the Froude number is less than approximately 0.3.

River Segment

See stream segment.

Runoff

Flow that is discharged from the area by stream channels; sometimes subdivided into surface runoff, ground water runoff, and seepage.

Sl and S2 Curves

Sl and S2 curves represent steep sloping water surface profiles.

Sand

See Glossary Table 1.

Sampled Zone

The part of a vertical transect presumed to be wholly represented by sediment samples.

Sampling Vertical

An approximately vertical path from the water surface to the bottom along which one or more samples are collected to define various properties of the flow, such as sediment concentration.

Sand

See Glossary Table 1.

Saturation

The degree to which voids in soil are filled with water.

Scale of Particle Sizes

The scale recommended is essentially that prepared by Lane (1947), for the Subcommittee on Sediment Terminology, AGU. See Glossary Table 1.

Scour

The enlargement of a flow section by the removal of boundary material through the action of the fluid in motion.

Secondary Currents (or Flow)

The movement of water particles on a cross section normal to the longitudinal direction of the channel.

Sediment

(1) Particles derived from rocks or biological materials that have been transported by a fluid, or (2) solid material (sludge) suspended in or settled from water.

Sediment Budget Analysis

A quantitative sediment assessment of watershed sediment processes. May be performed as a sediment impact assessment to evaluate stability and sediment-transporting flows done by comparing the mean annual sediment load for the project channel with that of the supply reach.

Sediment Continuity

The concept that difference between the sediment entering and leaving a control volume must be stored (deposited) or removed (eroded) from storage; continuity within a reach during a given period of time is computed as the difference between the volumes of sediment entering and leaving the reach.

Sediment Discharge

The mass or volume of sediment (usually mass) passing a stream cross section in a unit of time. The term may be qualified, for example, as suspended-sediment discharge, bedload discharge, or total sediment discharge.

Sediment Discharge Relationship

Tables that relate inflowing sediment loads to water discharge for the upstream ends of the main stem, tributaries, and local inflows.

Sediment Grade Scale

The grouping of sediment particles into size classes based on particle diameters uses the American Geophysical Union size classification scale of 1947. See Glossary Table 1.

Sediment Load

A general term that refers to material in suspension and/or in transport. It is not synonymous with either discharge or concentration. See also total sediment load.

Sedimentology

The scientific study of sediment, sedimentary rocks, and the processes by which they are formed; more specifically, it is a study of detachment, transport, and deposition of sediment particles in streams and other water bodies.

Sediment Particles

Fragments of mineral or organic material in either a singular or aggregate state.

Sediment Production

An unacceptable term. Use erosion. See also sediment yield.

Sediment Sample

A quantity of water-sediment mixture or deposited sediment that is collected to characterize some property or properties of the sampled medium.

Sediment Transport (Rate)

See sediment discharge.

Sediment Transport Function

A formula or algorithm for calculating the sediment transport rate given the hydraulics and bed material at a cross section. Most sediment transport functions compute the bed material load capacity. The actual transport may be less than the computed capacity due to armoring, geologic controls, etc.

Sediment Transport Potential

Sediment transport potential is the transportable mass of a particular grain class in response to cross channel hydraulic parameters; analysis via HEC-RAS or similar programs computes transport potential for each grain class with the selected sediment transport equations.

Sediment Transport Routing

The computation of sediment movement for a selected length of stream (reach) for a period of time with varying flows. Application of sediment continuity relations allow the computation of aggradation and deposition as functions of time.

Sediment Trap Efficiency

See trap efficiency.

Sediment Yield

The total sediment outflow from a drainage basin in a specific period of time. It includes bedload as well as suspended load, and usually is expressed in terms of mass, or volume per unit of time.

Sedimentation

A broad term that embodies the dynamic processes of erosion, entrainment, transportation, deposition, and the compaction of sediment. Sediment investigations are performed to evaluate sedimentation within river and reservoir applications.

Sedimentation Diameter

The diameter of a sphere of the same specific weight and the same terminal settling velocity as the given particle in the same fluid.

Sedimentary Delivery Ratio

The ratio of sediment yield to gross erosion.

Settling

The downward movement of suspended-sediment particles.

Shape Factor

See particle shape factor.

Shear Intensity

A dimensionless number that is taken from Einstein's bedload function. It is the inverse of Shield's parameter.

Shear Stress

Frictional force per unit of bed area exerted on the bed by the flowing water. An important factor in the movement of bed material.

Sheet Erosion

The more or less uniform removal of soil from an area by raindrop splash and overland flow without the development of water channels. Included with sheet erosion, however, are the numerous conspicuous small rills that are caused by minor concentrations of runoff.

Shield's Deterministic Curve

A curve of the dimensionless tractive force that s is plotted against the grain Reynolds number (such as Uo*D/v where, Uo = turbulent shear velocity, D_s = characteristic or effective size of the grains or roughness elements, v = kinematic viscosity) and that is used to help determine the critical tractive force.

Shield's Parameter

A dimensionless number referred to as a dimensionless shear stress.

The beginning of motion of bed material is a function of this dimensionless number.

$$\frac{\boldsymbol{\tau_c}}{(\boldsymbol{\gamma_s} - \boldsymbol{\gamma})\, \boldsymbol{D_s}}$$

where:

τ_c = critical tractive force
γ_s = specific weight of the particle specific
γ = fluid specific weight
D_s = characteristic or effective size of the grains or roughness elements

Sieve Diameter

The smallest standard sieve opening size through which a given particle of sediment will pass.

Simulate

To express a physical system in mathematical terms.

Sinuosity

A measure of meander "intensity." Computed as the ratio of the length of a stream measured along its thalweg (or centerline) to the length of the valley through which the stream flows.

Silt

A granular material of a size between sand and clay. See Glossary Table 1.

Siltation

An unacceptable term. Use sediment deposition, sediment discharge, or sediment yield as appropriate.

Soil

Unconsolidated mineral and organic surface material that has been sufficiently modified and acted on by physical, chemical, and biological agents so that it will support plant growth.

Sorting

The dynamic process by which sedimentary particles having some particular characteristic (such as similarity of size, shape, or specific gravity) are naturally selected and separated from associated, but dissimilar particles by the agents of transportation. See also gradation.

Spatial Concentration

The dry mass of sediment in a unit volume of water-sediment mixture in place.

Specific Gravity

Ratio of the mass of any volume of a substance to the mass of an equal volume of water at 4 degrees C.

Specific Weight of Sediment Deposits

The dry weight of sediment particles within a unit volume of the deposit expressed as pounds per cubic foot.

Specific Weight of Sediment Particles

The dry weight of sedimentary material per cubic foot of volume assuming no voids.

Split Flow

Flow that leaves the main river flow and takes a completely different path from the main river. Split flow can also occur in the case of flow bifurcation around an island.

Split Sample

A single sample separated into two or more individual parts in a manner that each part is representative of the original sample.

Stable Channel

A stream channel that does not change in planform or bed profile during a particular period of time. For purposes of this glossary, the time period is years to tens of years.

Stage-Discharge Rating Curve

Defines a relationship between discharge and water surface elevation at a given location.

Standard Fall Diameter

Sometimes simply fall diameter. The diameter of a sphere that has a specific gravity of 2.65 and has the same standard fall velocity as the given particle.

Standard Step Method

Method where the total distance is divided into reaches by cross sections at fixed locations along the channel and, starting from one control, profile calculations proceed in steps from cross section to cross section to the next control.

Steady-State Model

Model in which the variables being investigated do not change with time.

Stream Bank Erosion

The removal of bank material by the force of flowing water and the caving of stream banks.

Stream Discharge

The quantity of flow passing a stream cross section in a unit of time. The discharge contains water, dissolved solids, organic sediment, and inorganic sediment.

Stream Gage

A device that measures and records flow characteristics such as water discharge and water surface elevation at a specific location on a stream. Sediment transport measurements are usually made at stream gage sites.

Stream Power

The product of bed shear stress and mean cross-sectional velocity at a cross section for a given flow.

Stream Profile

A plot of the elevation of a stream bed vs. distance along the stream.

Stream Segment

A stream segment is a specified portion of a river with an upstream inflow point and with a downstream termination at a control point. Primary inflow points are designated by In, where n is the segment number. Primary inflow points are always at the upstream most end of a tributary or main stem segment.

Subcritical Flow

The state of flow where the water depth is above the critical depth. Here, the influence of gravity forces dominates the influences of inertial forces, and flow, having a low velocity, is often described as tranquil.

Subsurface Layer

The subsurface layer is composed of well-mixed sediments brought up from the inactive layer plus sediment that has deposited from the water column. It will replenish the cover layer and thereby supply bed sediment as required to meet sediment transport capacity. When the weight in the subsurface layer becomes less than the weight required to cover 100% of the bed surface to a depth of two times the size of the largest particle in transport, a new subsurface layer is brought up from the inactive layer.

Supercritical Flow

The state of flow where the water depth is below the critical depth, inertial forces dominate the gravitational forces, and the flow is described as rapid or shooting.

Supernate (or Supernatant)

The liquid (such as water) above the surface of settled sediment.

Suspended Bed Material Load

That portion of the suspended load that is composed of particle sizes found in the bed material.

Suspended Load

That part of the sediment load that is suspended sediment. See also sediment load.

Suspended Sediment

Sediment that is carried in suspension by the turbulent components of the fluid or by Brownian movement.

Suspended Sediment Concentration

See concentration of sediment.

Suspended Sediment Discharge

The quantity of suspended sediment passing a cross section in a unit of time.

Suspended Sediment Sample

See sediment sample.

Suspended Sediment Sampler

Device to sample flow and its suspended-sediment load.

Tail Water

The water surface elevation downstream from a structure, such as below a dam, weir, or drop structure.

Thalweg

The line connecting the lowest or deepest points along a stream bed, valley, or reservoir, whether under water or not.

Topset Bed

A layer of sediments deposited on the top surface of an advancing delta that is continuous with the landward alluvial plain.

Total Sediment Discharge

The total quantity of sediment passing a section in a unit of time.

Total Sediment Load (Total Load)

All of the sediment in transport; that part moving as suspended load plus that moving as bedload.

Tractive Force

When water flows in a channel, a force is developed that acts in the direction of flow on the channel bed. This force, which is simply the pull of water on the wetted area, is known as the tractive force. In a uniform flow, the equation for the unit tractive force (such as the average value to the tractive force per unit wetted area).

Transect

A sample line or sub-area chosen as the basis for studying one or more characteristics of the water discharge mixture. (*Note*: Some documents use transect interchangeably with cross section, but that is not consistent with other areas of hydraulics and, therefore, is discouraged.)

Transmissometer

An instrument that measures the energy of a light ray that has passed through a suspension.

Transportation (Sediment)

The complex processes of moving sediment particles from place to place. The principal transporting agents are flowing water and wind.

Transmissive Boundary

A boundary (cross section) that will allow sediment that reaches it to pass without changing that cross section.

Transport Capacity

The ability of the stream to transport a given volume or weight of sediment material of a specific size per time for a given flow condition. The units of transport capacity are usually given in tons per day of sediment transported passed a given cross section for a given flow. Transport capacity for each sediment grain size is the transport potential for that size material multiplied by the actual fraction of each size class present in the bed and bank material.

Transport Potential

Transport potential is the rate at which a stream could transport sediment of a given grain size for given hydraulic conditions if the bed and banks were composed entirely of material of that size.

Trap Efficiency

Proportion of sediment inflow to a stream reach (or reservoir) that is retained within that reach (or reservoir). Computed as inflowing sediment volume minus outflowing sediment volume divided by inflowing sediment volume. Positive values indicate aggradation; negative values, degradation.

Tributary

A river segment other than the main stem in which sediment transport is calculated. More generally, a stream or other body of water, surface or underground, that contributes its water to another and larger stream or body of water.

Turbidity

Only a general definition is possible because of the wide variety of methods in use. This term has been used as an expression of the optical properties of a sample that causes light rays to be scattered and absorbed rather than transmitted through the sample. See also optical opacity.

Turbidity Current

See density current.

Turbulence

In general terms, the irregular motion of a flowing fluid.

Unmeasured Sediment Discharge

The difference between total sediment discharge and measured suspended-sediment discharge. See also total load.

Unsampled Depth

The unsampled part of the sampling vertical; usually within 8 to 15 centimeters of the stream bed depending on the kind of sampler used.

Unsampled Zone

The bottom part of a vertical transect that cannot be reached by sediment samplers. See also sampled zone.

Volume Weight

Use density.

Wash Load

The part of the suspended load that is finer than the bed material. Wash load is limited by supply rather than hydraulics. What grain sizes constitute wash load varies with flow and location in a stream. Sampling procedures that measure suspended load will include both wash load and suspended bed material load. Normally, that is of sediment particles smaller than 0.062 mm.

Water Discharge

See stream discharge.

Water Column

An imaginary vertical column of water used as a control volume for computational purposes. Usually the size of a unit area and as deep as the depth of water at that location in the river.

Water Discharge

See stream discharge.

Watershed

A topographically defined area drained by a river/stream or system of connecting rivers/streams such that all outflow is discharged through a single outlet. Also called a drainage area.

Water Pollution

The addition of harmful or insufficient quantities to adversely affect point on a stream. See drainage objectionable material to water its usefulness.

Weir

A structure designed to raise the water level or to divert its flow through a desired channel.

Wetted Perimeter

The wetted perimeter is the length of the wetted contact between a stream of flowing water and its containing channel, measured in a direction normal to the flow.

Glossary Table 1
American Geophysical Union (AGU) Sediment Size Classification System (revised from Lane 1947)

Sediment	Size Range (mm)	Geometric Mean
Very large boulders	4,096–2,048	2,896 mm
Large boulders	1,024–2,048	1,488 mm
Medium boulders	1,024–512	724 mm
Small boulders	512–256	362 mm
Large cobbles	256–128	181 mm
Small cobbles	128–64	90.5 mm
Very coarse gravel	64–32	45.3 mm
Coarse gravel	32–16	22.6 mm
Medium gravel	16–8	11.3 mm
Fine gravel	8–4	5.66 mm
Very fine gravel	4–2	2.83 mm
Very coarse sand	2.0–1.0	1.41 mm
Coarse sand	1.0–0.5	707 µm
Medium sand	0.5–0.25	354 µm
Fine sand	0.25–0.125	177 µm
Very fine sand	0.125–0.0625	88.4 µm
Coarse silt	0.0625–0.031	44.1 µm
Medium silt	0.031–0.016	22.6 µm
Fine silt	0.016–0.008	11.0 µm
Very fine silt	0.008–0.004	5.52 µm
Coarse clay	0.004–0.002	2.76 µm
Medium clay	0.002–0.001	1.38 µm
Fine clay	0.0010–0.0005	0.691 µm
Very fine clay	0.0005–0.00024	0.345 µm