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	Engineering and Design CONFINED DISPOSAL OF DREDGED MATERIAL	
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US Army Corps
of Engineers

Confined Disposal of Dredged Material

ENGINEER MANUAL

DEPARTMENT OF THE ARMY
U. S. Army Corps of Engineers
Washington, D. C. 20314-1000

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Engineer Manual
No. 1110-2-5027

30 September 1987

Engineering and Design
CONFINED DISPOSAL OF DREDGED MATERIAL

1. Purpose. This manual provides guidance for planning, designing, constructing, operating, and managing dredged disposal areas.

2. Applicability. This manual applies to all HQUSACE/OCE elements and field operating activities (FOA) having civil works responsibilities.

3. General. Subjects covered in this manual are field investigations, laboratory testing, containment area design, long-term capacity and management methods.

FOR THE COMMANDER:

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CHAPTER 1

INTRODUCTION

1-1. Purpose. This manual provides guidance for planning, designing, constructing, operating, and managing confined dredged material disposal areas* to retain suspended solids during disposal operations and to provide adequate storage volume for both short-term and long-term disposal needs.

1-2. Applicability. This manual applies to all field operating activities concerned with administering Corps dredging programs.

1-3. References. The references listed below provide guidance to personnel concerned with design, construction, operation, and management of dredged material containment areas.

a. ER 200-2-2

b. ER 1105-2-10

c. ER 1105-2-20

d. ER 1105-2-50

e. ER 1110-2-1300

f. EM 1110-1-1802

g. EM 1110-2-1902

h. EM 1110-2-1903

i. EM 1110-2-1906

j. EM 1110-2-1907

k. EM 1110-2-1908

l. EM 1110-2-1911

m. EM 1110-2-2300

n. EM 1110-2-5025

o. Hydraulics Design Criteria Sheets 224-1/2 to 224-1/4. Available from Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180.

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* The terms "confined disposal area," "confined disposal site," "diked disposal area," "containment area," and "confined disposal facility" all refer to an engineered structure for containment of dredged material.

EM 1110-2-5025 is an overview of Corps dredging and dredged material disposal practice. This manual supplements EM 1110-2-5025 by providing detailed guidance for confined dredged material disposal.

1-4. Bibliography. Bibliographic references are indicated as needed in the text and are listed in Appendix A. The US Army Engineer Waterways Experiment Station (WES) reports listed in the Bibliography may be obtained from the Technical Information Center, US Army Engineer Waterways Experiment Station, PO Box 631, Vicksburg, MS 39180-0631. They are available for loan by request from the WES Technical Information Center Library. In addition, copies of the reports are available through the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22161.

1-5. Background.

a. General. In fulfilling its mission to maintain, improve, and extend waterways and harbors, the US Army Corps of Engineers (CE) is responsible for the dredging and disposal of large volumes of dredged material each year. Dredging is a process by which sediments are removed from the bottom of streams, rivers, lakes, and coastal waters; transported via ship, barge, or pipeline; and discharged to land or water. Annual quantities of dredged material average about 300 million cubic yards* in maintenance dredging operations and about 100 million cubic yards in new work dredging operations with the total annual cost now exceeding $500,000,000. Much of this volume is placed in aquatic disposal sites, in wetlands creation or nourishment, or in unconfined disposal areas. Although no breakdown of these figures is routinely maintained, about 30 percent of the total maintenance volume, or 90 million cubic yards, is placed in diked disposal areas annually. This figure includes the majority of the maintenance for major ports along the Atlantic and gulf coasts and numerous harbors on the Great Lakes. The magnitude of confined dredged material disposal requires careful planning, design, construction, and management of containment areas that are compatible with future land-use goals and ensure environmental protection.

b. Scope.

(1) Confined disposal sites are engineered structures designed to provide required storage volume and to meet required effluent solids standards. This manual provides guidelines for designing, operating, and managing dredged material containment areas. These guidelines are applicable to the design of new containment areas as well as the evaluation of existing sites, and they include data collection and sampling requirements, description of testing procedures, and design, operational, and management procedures.

(2) The testing procedures described in this manual include column tests necessary for sedimentation design, chemical clarification for improvement of effluent quality, and consolidation tests for evaluating long-term storage capacity. Design procedures include the consideration of dredged material

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* The US customary units of measurement are used in lieu of metric (SI) units for those cases common in dredging practice. Metric (SI) units are used in this report when consistent with standard usage.

sedimentation and consolidation/dewatering behavior and potential consolidation of foundation soils. Guidelines for containment area design for sedimentation were developed primarily for fine-grained material generated in maintenance dredging operations. Factors that improve containment area efficiency are presented and include weir design and location, effects of area size and shape, and use of interior spur dikes. Guidelines for containment areas during the dredging operation include weir operation and maintenance of adequate ponding depth. Guidelines for containment area management before, during, and after dredging operations to maximize sedimentation efficiency and long-term storage capacity are also presented. In addition, the manual contains guidance on the design of chemical clarification systems for removal of additional suspended solids that are not effectively eliminated by gravity settling. Guidance is also provided on the design of containment area dikes, Many of the design procedures in this manual have been incorporated into the Automated Dredging and Disposal Alternatives Management System (ADDAMS), a centralized computer-program and data management system (item 19).

(3) Although not specifically covered in this manual, guidelines have been developed for odor control, for mosquito and other insect control, and for minimizing the adverse visual impact of disposal areas and aspects of confined disposal for contaminated sediments. These factors should be considered in the earliest planning and design stages and carried through during construction and management phases. Information on these specialized topic areas is found in the Bibliography (items 9, 12, 17, 18, and 23).

c. Authority. The authority for implementing the planning, design, and operation and management approaches described in this manual is recognized in Section 148 of PL 94-587: Sec. 148:

> The Secretary of the Army, acting through the Chief of Engineers, shall utilize and encourage the utilization of such management practices as he determines appropriate to extend the capacity and useful life of dredged material disposal areas such that the need for new dredged material disposal areas is kept to a minimum. Management practices authorized by this section shall include, but not be limited to, the construction of dikes, consolidation and dewatering of dredged material, and construction of drainage and outflow facilities.

Authority to implement management practices under Sec. 148 may be limited in some instances. If the disposal area is Federally owned, management practices can be pursued under Sec. 148. If the site is owned by others or if dikes are provided by others (such as the project sponsor), authority under Sec. 148 may be limited. Also, In some cases, the ownership of dredged material (once removed from the navigation channel) is in question. Current Corps policy on implementation of Sec. 148 should be determined on a case-by-case basis.

1-6. <u>Considerations Associated with Confined Dredged Material Disposal</u>.

a. Diked containment areas are used to retain dredged material solids while allowing the carrier water to be released from the containment area. The two objectives inherent in the design and operation of a containment area are to provide adequate storage capacity to meet dredging requirements and to attain the highest possible efficiency in retaining solids during the dredging operation in order to meet effluent suspended solids requirements. These considerations are basically interrelated and depend upon effective design, operation, and management of the containment area.

b. The major components of a dredged material containment area are shown schematically in Figure 1-1. Constructed dikes form a confined surface area, and the dredged channel sediments are normally pumped into this area hydraulically. Both the influent dredged material slurry and effluent water can be characterized by suspended solids concentration, suspended particle size gradation, type of carrier water (fresh or saline), and rate of flow.

c. In some dredging operations, especially in the case of new work dredging, sand, clay balls, and/or gravel may be present. This coarse material (>No. 200 sieve) rapidly falls out of suspension near the dredge inlet pipe, forming a mound. The fine-grained material (<No. 200 sieve) continues to flow through the containment area with most of the solids settling out of suspension, thereby occupying a given storage volume. The fine-grained dredged material is usually rather homogeneous and is easily characterized.

d. The clarified water is usually discharged from the containment area over a weir. Effluent flow rate is approximately equal to influent flow rate for continuously operating disposal areas. Flow over the weir is controlled by the static head and the weir length provided. To promote effective

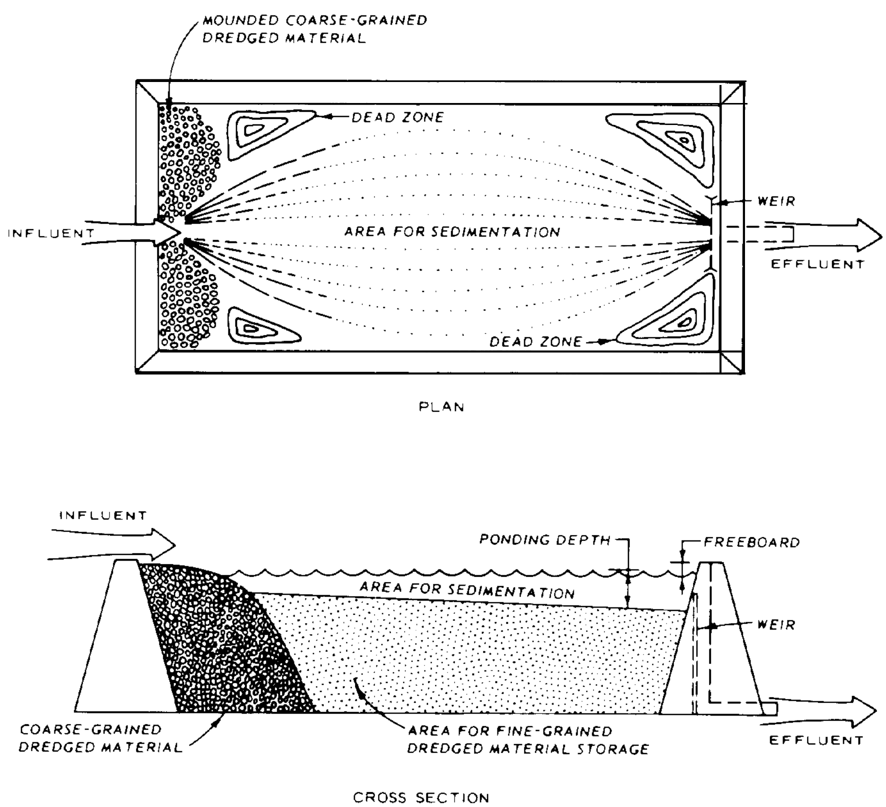


Figure 1-1. Conceptual diagram of a dredged material containment area

sedimentation, ponded water is maintained in the area with the depth of water controlled by the elevation of the weir crest. The thickness of the dredged layer increases with time until the dredging operation is completed. Minimum freeboard requirements and mounding of coarse-grained material result in a ponded surface area smaller than the total surface area enclosed by the dikes. Dead spots in corners and other hydraulically inactive zones reduce the surface area effectively involved with the flow to considerably less than the total ponded surface area.

e. Effluent standards may be imposed as a requirement for water quality certification. Standards in terms of suspended solids or turbidity may be used. Procedures in this manual allow containment areas to be designed to meet such effluent standards.

f. In most cases, confined disposal areas must be used over a period of many years, storing material dredged periodically over the design life. Long-term storage capacity of these areas is therefore a major factor in design and management. Consolidation of the layers continues for long periods following disposal, causing a decrease in the volume occupied by the layers and a corresponding increase in storage capacity for future disposal. Once water is decanted from the area following active disposal, natural drying forces begin to dewater the dredged material, adding additional storage capacity. The gains in storage capacity are therefore influenced by consolidation and drying processes and the techniques used to manage the site both during and following active disposal operations.

CHAPTER 2

FIELD INVESTIGATIONS

2-1. General. Field investigations are necessary to provide data for containment area design. The channel must be surveyed to determine the volume of material to be dredged, and channel sediments must be sampled to obtain material for laboratory tests. Site investigations must be conducted to provide information for dike design and 'evaluation of potential foundation settlement, an important parameter in long-term storage capacity estimates. This chapter of the manual describes field investigations required to obtain the necessary samples for laboratory testing. The methods in common use for determining volumes of channel sediment to be dredged are well known and are not described in this manual. Basic considerations regarding sampling and volume determination are described in EM 1110-2-5025. The potential for the presence of contaminants should be evaluated when planning field investigations, and appropriate safety measures should be considered.

2-2. Channel Sediment Investigations.

a. Sample Type and Location.

(1) Samples of the channel sediments to be dredged are required for adequate characterization of the material and for use in laboratory testing. The level of effort required for channel sediment sampling is highly project-dependent. In the case of routine maintenance work, data from prior samplings and experience with similar material may be available, and the scope of field investigations may be reduced. For unusual maintenance projects or new work projects, more extensive field investigations will be required.

(2) For maintenance work, channel investigations may be based on grab samples of sediment. Since bottom sediments are in an essentially unconsolidated state, grab samples are satisfactory for sediment characterization purposes and are easy and inexpensive to obtain. Grab sampling may indicate relatively homogeneous sediment composition, segregated pockets of coarse- and fine-grained sediment, and/or mixtures. If segregated pockets are present, samples should be taken at a sufficient number of locations in the channel to adequately define spatial variations in the sediment character. In any case, results of grab sampling must allow estimation of the relative proportions of coarse- and fine-grained sediments present. Caution should be exercised in interpreting conditions indicated by grab samples since sediment surface samples do not indicate variation in sediment character with depth. For more detailed information, additional samples may be taken using conventional boring techniques.

(3) Water samples should be taken at several locations near the sediment-water interface in the area to be dredged. Subsequent salinity tests on these samples indicate whether the dredging will be done in a freshwater or saltwater environment. Potential changes in salinity because of tides or seasonal flooding should also be considered.

(4) Samples of sediment taken by conventional boring techniques are normally required only in the case of new work dredging. Based on information gained from initial grab sampling, locations for borings should be selected. Samples should be taken from within the major zones of spatial variation in sediment type or along the proposed channel center line at constant spacing to define stratification within the material to be dredged and to obtain representative samples. Borings should be advanced to the full depth of anticipated dredging if possible. This is normally done on a routine basis for new work projects to indicate the type of material to be dredged and the degree of dredging difficulty, since this information is required for the dredging contractor to use as a basis for bidding on the project. Test pits using a clamshell dredge can also prove useful for accessing dredgability and can be used to obtain larger sample quantities.

(5) Vibracore samplers have also proved successful in obtaining core samples of sediments. The Vibracore sampler is not a standardized piece of equipment, but it usually consists of a core-barrel and a vibratory driving mechanism mounted on a four-legged tower guide and platform. The entire assembly is lowered to the sediment surface below the water by a crane/cable hoist system. After the device has been accurately positioned on the bottom, compressed air is supplied to the vibratory unit through flexible hoses extending from the floating plant down to the Vibracore. Upon application of the compressed air, the oscillating hammer (vibrator) propels the core-barrel into the subbottom materials. The Vibracore can be equipped with a penetration recording device that will provide a record of the penetration depth and time. After the core-barrel has been extended to its full length, the sampler is retracted from the sediment and returned to the floating plant deck. The removable plastic core-barrel containing the sample is then removed from the sampling device, and the ends are capped for sample preservation. Typically 3-inch-diameter cores of up to 20 feet in length are obtained; some devices may be modified to take samples of 30- or 40-foot lengths. This device is generally used to sample sands. It has also been used to sample some fine-grained materials.

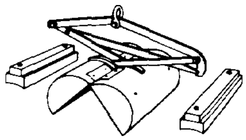
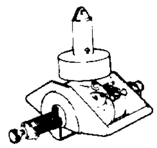
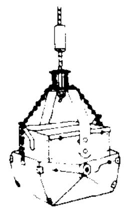
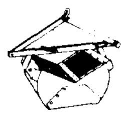
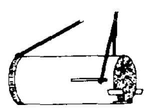
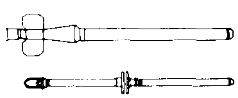
(6) Pertinent information regarding sediment samplers is summarized in Table 2-1. Grab samplers as described in Table 2-1 will allow retrieval of sufficient sediment to perform characterization tests and sedimentation and consolidation tests, if required.

b. Sample Quantity.

(1) The quantity of sediment samples to be collected should be determined by the designer, based on the requirements for the laboratory tests to be performed. A quantity of sediment sufficient to perform the necessary characterization tests and to provide some material for the composite sample for the column settling tests described in Chapter 3 should be collected from each established sampling point. If at all possible, the sampling efforts should be coordinated with other requirements for determining the presence of contaminants or for contaminant-related testing. In this case, appropriate procedures for sample collection, handling, and preservation should be followed. For grab samples, it is recommended that at least 5 gallons of sediment be collected at each sampling station. Five-gallon containers are generally recommended for collecting all grab samples; since most sampling

Table 2-1

Summary of Sediment Sampling Equipment

Sampler		Weight	Remarks
Peterson		39-93 lb	Samples 144-in.^{2} area to a depth of up to 12 in., depending on sediment texture
Shipek		150 lb	Samples 64-in.^{2} area to a depth of approximately 4 in.
Ekman		9 lb	Suitable only for very soft sediments
Ponar		45-60 lb	Samples 81-in.^{2} area to a depth of less than 12 in. Ineffective in hard clay
Drag bucket		Varies	Skims an irregular slice sediment surface. Available in assorted sizes and shapes
Phlegar tube		Variable 17-77 lb; fixed in excess of 90 lb	Shallow core samples may be obtained by self-weight penetration and/or pushing from boat. Depth of penetration dependent on weight and sediment texture
Conventional soil samplers			Conventional soil samplers may be employed using barge- or boat-mounted drilling equipment. Core samples attainable to full depth of dredging

will be performed from small motorboats, containers of this size are about the largest that can be handled efficiently. For core samples, a cornpositing scheme must be developed, depending on the number of cores taken, size of cores, and nature of the material.

(2) A smaller sample of sediment should be collected from each fine-grained grab sample and placed in a small (about g-ounce) watertight jar for water content and specific gravity tests. Care should be taken to collect small sediment samples that are representative of the sediment sample as a whole. Similar procedures for core samples are standard soil sampling practice.

(3) After the characterization tests have been performed on samples from each sampling point, samples can be combined to meet requirements for the settling tests described in Chapter 3. Approximately 15 gallons of channel sediment is required to perform the column settling tests.

c. Sample Preservation.

(1) The laboratory tests described in this manual do not require sophisticated sample preservation measures. There are two requirements:

(a) Collect the samples in airtight and watertight containers.

(b) Place the samples in a cold room (6° to 8° C) within 24 hours after sampling until the organic content can be determined. If the organic content is above 10 percent, the samples should remain in the cold room until testing is complete; otherwise, the samples need not be stored in the cold room. The in situ water content of the small samples must be maintained. These samples should not be allowed to drain, nor should additional water be added when they are placed into the containers.

(2) All sample containers should be clearly identified with labels, and the sample crew should keep a field log of the sampling activity. Laboratory testing should be accomplished on the samples as soon as practicable after sampling.

2-3. Containment Area Investigations.

a. Field investigations must be performed at the containment area to define foundation conditions and to obtain samples for laboratory testing if estimates of long-term storage capacity are required. The extent of required field investigations is dependent upon project size and upon foundation conditions at the site. It is particularly important to define foundation conditions (including depth, thickness, extent, and composition of foundation strata) ground-water conditions, and other factors that may influence construction and operation of the site. For new containment areas, the field investigations required for estimating long-term storage capacity should be planned and accomplished along with those required for the engineering design of the retaining dikes as described in Chapter 6.

b. For existing containment areas, the foundation conditions may have been defined by previous subsurface investigations made in connection with dike construction. However, previous investigations may not have included sampling of compressible soils for consolidation tests; in most cases, suitable samples of any previously placed dredged material would not be available. Field investigations must therefore be tailored to provide those items of information not already available.

c. Undisturbed samples of the compressible foundation soils can be obtained using conventional soil sampling techniques and equipment. If dredged material has previously been placed within the containment area, undisturbed samples must be obtained from borings taken within the containment area but not through existing dikes. The major problem in sampling existing containment areas is that the surface crust will not normally support conventional drilling equipment, and personnel sampling in these areas must use caution. Below the surface crust, fine-grained dredged material is usually soft, and equipment will sink rapidly if it breaks through the firmer surface. Lightweight drilling equipment, supported by mats, will normally be required if crust thickness is not well developed. In some cases, sampling may be accomplished manually if sufficient dried surface crust has formed to support crew and equipment. More detailed information regarding equipment use in containment areas may be found in Appendix I.

d. Water table conditions within the containment area must be determined in order to estimate loadings caused by placement of dredged material. This information must be obtained by means of piezometers, which may also be used for measurement of ground-water conditions during the service life of the area. Other desired instrumentation such as settlement plates may also be installed within the containment area for monitoring various parameters.

e. Additional information regarding conventional sampling techniques and equipment and development of field exploration programs is given in EM 1110-2-1907 and in Chapter 6 of this manual. Procedures for installation of piezometers and other related instrumentation are given in EM 1110-2-1908.

2-4. <u>Site Selection for Avoidance of Ground-Water Impacts.</u>

a. As water percolates through in-place dredged material, leachate may be produced. This leachate water may be the result of precipitation or entrained water resulting from the dredging operation. Available data for the characterization of leachate produced from dredged material are very limited. Potential adverse water quality impacts will most likely be caused by the increases of chloride, potassium, sodium, calcium, total organic carbon, alkalinity, iron, and manganese. These factors should be considered even for dredged material that is considered uncontaminated. This is especially true if a saltwater dredged material may be placed over a freshwater aquifer.

b. Site location is an important, if not the most important, consideration in minimizing any adverse impact to underlying ground water. Selection of a technically sound site may reduce or eliminate the need for any restrictions or controls. Site characteristics affecting ground-water impacts are presented in Table 2-2. Site characteristics that are particularly important in the evaluation of ground-water impacts at potential upland disposal sites are discussed in the following paragraphs:

Table 2-2

Site Characteristics Affecting Ground-water Impacts

Site volume	Depth to bedrock
Site area	Depth to aquicludes
Site configuration	Direction and rate of ground-water flow
Dredging method	Existing land use
Climate (precipitation, temperature wind, evaporation)	Depth of ground water
Soil texture and permeability	Ecological areas
Soil moisture	Drinking water wells
Topography	Receiving streams (lakes, rivers, etc.)
Drainage	Level of existing contamination
Vegetation	Nearest receptors

(1) Location. While the significant characteristics of a given site are usually unique, useful hypotheses about pathways of migration and estimates of parameters needed to calculate migration rate can often be developed from available regional data and keyed to location, topography, surface drainage patterns, flood potential, subsurface stratigraphy, ground-water flow patterns, and climate.

(2) Topography. Topographic variables are important in evaluating surface drainage and run-on and runoff potential of the site. This information would be helpful in determining the amount of water that may be available to percolate through the in-place dredged material.

(3) Stratigraphy. The nature of subsurface soils, determined by examination of soil core borings to bedrock, is an important input to evaluation of pathways of migration in both the unsaturated and saturated zones.

(4) Ground-water levels (equipotential surfaces). Seasonal maps of water table contours and piezometric surfaces, developed by analysis of ground-water monitoring well data, are important in predicting ground-water flow directions and hydraulic gradients, as these can vary greatly at upland or nearshore sites.

(5) Ground-water flow. Information on permeability and porosity of subsurface strata, combined with data on hydraulic gradients, is important in predicting ground-water flow velocities and direction.

(6) Meteorology and climate. Precipitation, including annual, seasonal, or monthly rain and snowfall, is an important parameter in determining a water balance for the site and in evaluating leachate potential. Evapotranspiration is also important in developing a water balance for the site. It is often estimated from temperature and the nature of vegetative growth at the site.

(7) Soil properties. An important variable in evaluating mobility of many metal contaminants is pH. Cation exchange capacity (CEC) is an important determinant of the mobility of metallic species in soils; if the CEC is sufficiently high to adequately immobilize the heavy metals present in the soil, no adverse ground-water impacts may result. Redox potential (Eh) is important in determining the stability of various metallic and organic species in the subsurface environment of the site. Organic carbon content is a major variable affecting adsorption, and hence mobility, of organic species in the subsurface environment. Soil type (e.g., clay, till, sand, fractured bedrock) is a major variable affecting rates and routes of ground-water migration.

(8) Potential ground-water receptors and sensitive ecological environments. Ground-water and surface water usage, especially downgradient of the site, is important in evaluating adverse impacts. Size of population and nature of ecological resources downgradient of the site are also important variables in determining adverse impacts.

c. Examples where site location alone can be used to reduce or eliminate adverse impacts to ground water include:

(1) Selection of sites that have natural clay underlying formations that can minimize potential ground-water contamination concerns.

(2) Selection of sites to avoid aquifer recharge areas that can minimize potential ground-water contamination concerns. Another consideration associated with site location is that some fine-textured dredged material tends to form its own liner as particles settle with percolation drainage water; however, it may require considerable time for self-sealing to develop. For this reason, if an artificial liner is considered useful, a temporary liner subject to gradual deterioration with time may be adequate in many cases.

CHAPTER 3

LARORATORY TESTING

3-1. General.

a. Laboratory tests as described are required primarily to provide data for sediment characterization, containment area design, and long-term storage capacity estimates. The laboratory tests and procedures described in this chapter include standard tests that generally follow procedures found in Standard Methods (item 2) and EM 1110-2-1906. A flowchart illustrating the complete laboratory testing program for sediment samples is shown in Figure 3-1. Sediment characteristics and requirements for settling data and for long-term storage capacity estimates will dictate which laboratory tests are required.

b. The required magnitude of the laboratory testing program is highly project dependent. Fewer tests are usually required when dealing with a relatively homogeneous material and/or when data are available from previous tests and experience. This is frequently the case in maintenance work. For unusual maintenance projects where considerable variation in sediment properties is apparent from samples or for new work projects, more extensive laboratory testing programs are required. Laboratory tests should always be performed on representative samples selected using sound engineering judgment. The potential presence of contaminants should be evaluated when planning a laboratory testing program, and appropriate safety measures should be considered.

c. In some cases, recurring maintenance dredging is performed on given channel reaches. Laboratory test data from previous sampling efforts may be available. Under such conditions, sediment characterization tests may be the only laboratory testing required. Additional settling tests or consolidation tests are not required if it has been satisfactorily determined by prior testing that the settling and consolidation properties of the sediment to be dredged have not changed.

3-2. Sediment Characterization Tests.

a. General. A number of sediment characterization tests are required before the tests essential to design can be performed. Visual classification will establish whether the sediment sample is predominantly fine-grained (more than half <No. 200 sieve) or coarse-grained (more than half >No. 200 sieve). Tests required on fine-grained sediments include natural water content, Atterberg limits (LL), organic content, and specific gravity. The coarse-grained sediments require only grain size analyses. Results of these tests can be used to classify the sediments according to the Unified Soil Classification System (USCS) (item 33).

b. Salinity. Near-bottom water samples from the area where water will be mixed with sediment during the dredging or pump-out operation (usually dredging site water) should be tested for salinity. In estuarine environments, the salinity may vary with depth, flow, wind, tidal cycle, and season. Therefore, it is important to know the expected range of salinity during the dredging project. If the dredging site water is saline (>1 part per

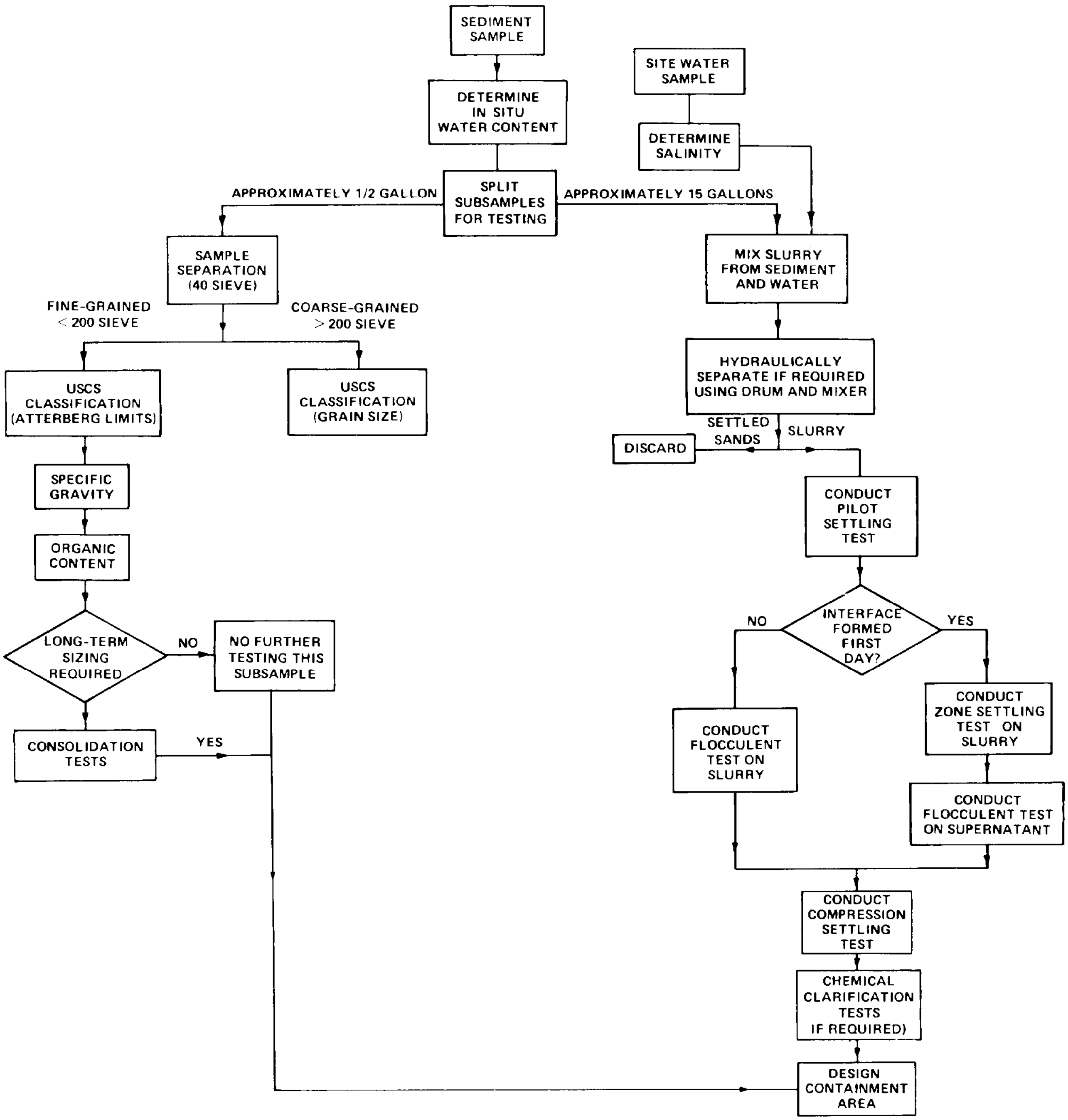


Figure 3-1. Flowchart depicting laboratory testing program for sediment samples

thousand), water gathered during the field investigation or reconstituted salt water should be used when additional water is required in all subsequent characterization tests and in the settling tests. Salinity may be measured in two ways:

(1) Conductivity. The salinity may be measured directly by a salinity-conductivity meter that electronically converts temperature-adjusted electrical conductivity into salinity.

(2) Dissolved solids or nonfiltrable residue. A detailed procedure is presented in Standard Methods for the Examination of Water and Wastewater (item 2). Briefly,

(a) Filter water through a filter that has a pore size of 1 micron or less.

(b) Pipette a known volume (about 25 millilitres) into a weighed dish and evaporate the sample 4 to 6 hours in a drying oven at 103° to 105° C.

(c) Cool the dish in a desiccator and then weigh immediately.

(d) Salinity (in parts per thousand) is equal to the residue (in milligrams) divided by the sample (in millilitres).

c. Water Content. Water content* is an important factor used in sizing dredged material containment areas. Water content determinations should be made on representative samples from borings or grab samples of fine-grained sediment obtained in the field investigation phase. The water content of the sample should be determined prior to sample homogenization and separation as described below. The detailed test procedure for determining the water content is found in Appendix I of EM 1110-2-1906. The water content is expressed on a dry weight basis as follows:

$$w = \frac{W_w}{W_s} \times 100 \text{ percent} \qquad (3\text{-}1)$$

where**

w = water content, percent*

W_w = weight of water in sample, grams

W_s = weight of solids in sample, grams

d. Solids Concentration.

(1) General. The suspended solids concentration is the most frequently measured parameter in the laboratory procedures. This measurement is made during preparation of slurries and suspensions and during evaluation of settling characteristics, treatment effectiveness, etc. Three methods may be

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* It should be noted that the term "water content" as used in this manual refers to the engineering water content commonly used in geotechnical engineering and may exceed 100 percent.

** For convenience, symbols and unusual abbreviations are listed and defined as appropriate throughout the text.

used to measure suspended solids: evaporation, filtration, and centrifugation. Each is applicable under different circumstances. Evaporation (direct drying) measures total solids, i.e., the sum of both suspended and dissolved solids. The dissolved solids concentration, if significant, must be measured separately and subtracted from the total solids concentration. Filtration directly measures suspended solids. Centrifugation is a blend of the other two methods. It attempts to measure suspended solids by measuring the total solids after washing the dissolved solids out of a known volume of sample. The procedures outlined below are adapted from the methods given in Palermo, Montgomery, and Poindexter (item 26). In practice, there has been confusion concerning the method of reporting suspended solids. The terms "concentration in grams per litre," "percent solids by weight," "percent solids by volume," and "percent solids by apparent volume" have been used. These methods of reporting suspended solids concentration are discussed and compared in Table 3-1. The relationship of percent suspended solids by weight and volume, concentration in grams per litre, and water content is illustrated in Figure 3-2. Figure 3-2 does not account for salinity in the sample. Suspended solids concentration in grams per litre or milligrams per litre is used throughout this manual. If suspended solids determinations are to be made on samples with a solids concentration of 1 gram per litre or less, the centrifugation or the filtration method should be used. The total solids method or

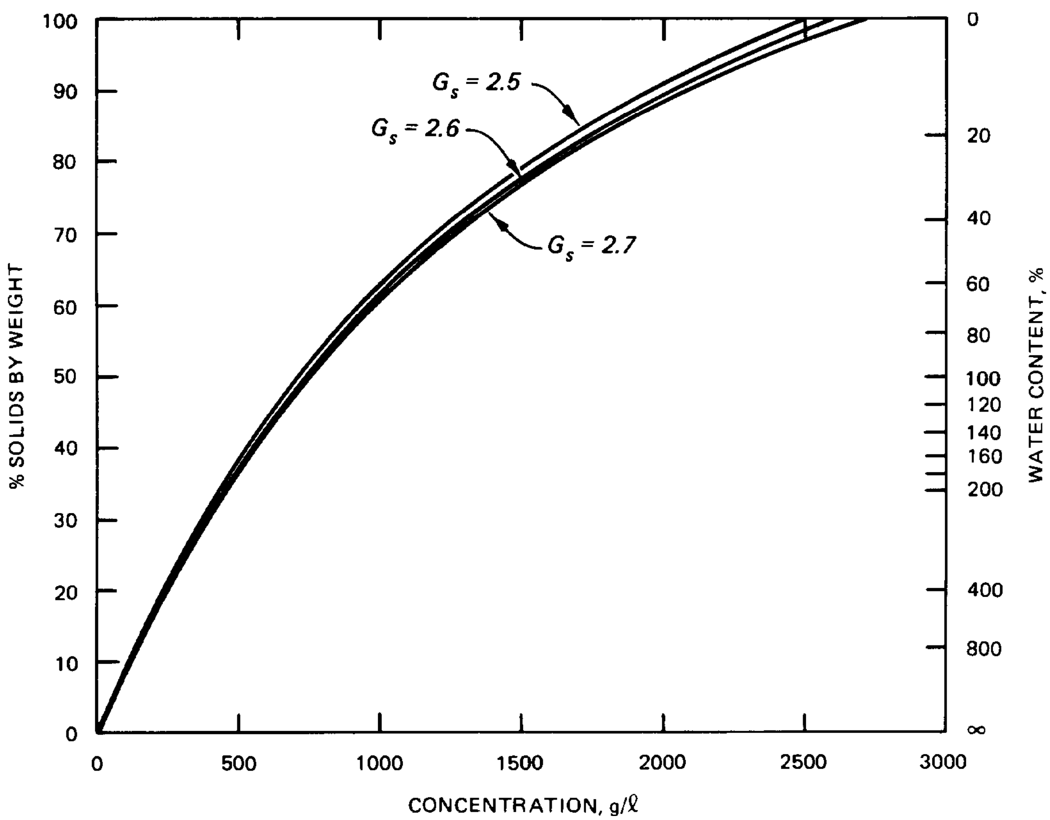
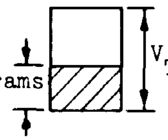
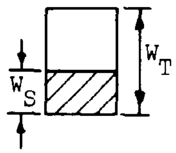
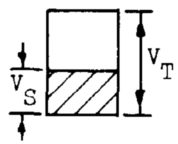
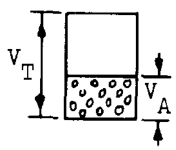


Figure 3-2. Relationship of concentration in percent solids by weight, percent solids by volume, concentration in grams per litre, and water content

Table 3-1

Methods of Reporting Suspended Solids

Method of Reporting Suspended Solids	Weight-Volume Relationship	Method of Computation	Remarks
		Preferred Method	
grams per litre or milligrams per litre	 W_S, grams; V_T = 1 litre	$S = \frac{W_S}{V_T}$	Common method for reporting dissolved chemical concentrations. Best method for engineering purposes
		Other Methods	
percent by weight	 W_S; W_T	$S = \frac{W_S}{W_T} 100$	Easy to determine by laboratory test. Does not require value for specific gravity
percent by volume	 V_S; V_T	$S = \frac{V_S}{V_T} 100$	Easy to determine by laboratory test. Requires determination of percent by weight and value for specific gravity
percent by apparent volume	 V_T; $V_A = V_S + V_I$	$S = \frac{V_A}{V_T} 100$	Apparent volume determined by settled solids for a bottle or flask. No standardized procedure available. Void ratio of settled solids varies with type of sediment. Can lead to errors because of nonstandard test. Not recommended. Value is meaningless in engineering calculations

Note: W_S = ovendry weight of solid particles
V_T = total volume
W_T = total weight
V_S = volume of solid particles
V_A = apparent volume of settled solids
V_I = volume of interstitial water

the centrifugation method should be used for slurries with solids concentrations of 1 gram per litre or more.

(2) Definitions and conversions.

(a) The percent of total solids by weight is the weight of solids both nonfiltrable and filtrable (both dissolved and suspended) in a sample divided by the weight of the sample.

$$\%S = \frac{W_s}{W_t} (100 \text{ percent}) \tag{3-2}$$

where

%S = percent total solids by weight, percent

W_t = total weight of sample, grams

(b) The percent of suspended solids by weight is the weight of solids less the weight of dissolved solids in a sample divided by the weight of the sample.

$$\%SS = \frac{W_s - \frac{(W_w \text{ Sal})}{1,000}}{W_t} (100 \text{ percent}) \tag{3-3}$$

where

%SS = percent suspended solids by weight, percent

Sal = salinity, parts per thousand

(c) Solids concentration is the weight of solids (dissolved and suspended) in a sample divided by the volume of sample.

$$C_s = \frac{W_s}{V_t} \tag{3-4}$$

where

C_S = solids concentration, grams per litre

v_t = sample volume, litre

(d) Suspended solids concentration is the weight of suspended solids in a sample divided by the volume of sample.

$$C = \frac{W_{ss}}{V_t} \tag{3-5}$$

where

C = suspended solids concentration, grams per litre

W_{SS} = weight of suspended solids in sample, grams

$= W_s - [W_w$ (Sal/1,000 parts per thousand)]

(e) The percent of suspended solids by weight may be converted to concentrations in units of grams per litre by the following formula:

$$C = \frac{(1{,}000\ \text{g}/\ell)\ G_s \left[1 + \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right]}{G_s\left[\left(\frac{100\%}{\%SS}\right) - 1\right] + \left[1 + \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right]} \quad \text{(3-6a)}$$

where

G_s = specific gravity of suspended solids particles

(f) Suspended solids concentrations presented in units of grams per litre may be converted to percent of suspended solids by the following formula:

$$\%SS = \frac{100\%\ G_s \left(\frac{C}{1{,}000\ \text{g}/\ell}\right)}{G_s\left(\frac{C}{1{,}000\ \text{g}/\ell}\right) + \left[G_s - \left(\frac{C}{1{,}000\ \text{g}/\ell}\right)\right]\left[1 + \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right]} \quad \text{(3-6b)}$$

(g) Suspended solids concentrations can be calculated from total solids concentrations by the following equations if the salinity is known and the total solids concentration is presented in percent of solids by weight.

$$\%SS = \%S - \left[(100\% - \%S)\left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right] \quad \text{(3-6c)}$$

$$\%S = \frac{\%SS + 100\% \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)}{1 + \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)} \quad \text{(3-6d)}$$

$$C_{ss} = \frac{(1{,}000\ \text{g}/\ell)\ G_s \left[1 + \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right]}{\left[1 + \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right] + G_S\left\{\frac{-1 + 1}{\left[\left(\frac{\%S}{100\%}\right)\left(1 + \frac{\text{Sal}}{1{,}000\ \text{ppt}}\right) - \left(\frac{\text{Sal}}{1{,}000\ \text{ppt}}\right)\right]}\right\}} \quad \text{(3-6e)}$$

(3) Total solids method. This test is used when the suspended solids concentration is large, compared to dissolved solids. It may be used in other cases where the dissolved solids or salinity is known or measured separately. To ensure accuracy, the test should generally be used only for a suspension suspension with a suspended solids concentration greater than 1 gram per litre. These steps should be followed:

(a) Obtain tared weight of a sample dish.

(b) Thoroughly mix sample and pour into sample dish.

(c) Weigh dish and sample and place in drying oven at 105" C until sample has dried to a constant weight (about 4 to 6 hours).

(d) Cool in desiccator and then weigh immediately.

(e) Calculate suspended solids concentration C , in grams per litre, as follows:

$$C = \frac{W_{ss}\ (1{,}000\ g/\ell)}{\left(\frac{W_{ss}}{G_s}\right) + W_w} \tag{3-7}$$

from before

$W_{ss} = W_s - [W_w$ (Sal/1,000 parts per thousand)]

W_s = [(weight of dry sample and dish) - (weight of dish)]

Sal = salinity, parts per thousand, or dissolved solids, grams per litre; if unknown in freshwater environments, use zero

G_s = specific gravity of solids; use 2.67 if unknown

W_w = [(weight of wet sample and dish, grams) - (weight of dry sample and dish, grams)]

(4) Filtration method. This method should be used for suspensions having suspended solids concentrations of less than 1.0 gram per litre. Any quantitative filtering apparatus using a filter paper that has a pore size of 1 micron or less can be used for the test. The two most common setups use either a Gooch crucible with a glass fibre filter paper or a membrane filter apparatus. These steps should be followed:

(a) Weigh the filter.

(b) Filter a measured volume of the sample. The volume should be sufficient to contain 5 milligrams of suspended solids.

(c) Filter 10 millilitres of distilled water twice to wash out dissolved solids.

(d) Place the filter in a drying oven at 105° C until the sample has dried to constant weight (usually 1 to 2 hours).

(e) Cool in a desiccator and weigh.

(f) Calculate suspended solids concentration C , in grams per litre, as follows:

$$C = \{[(\text{weight of filter and dry solids, grams}) - (\text{weight of filter, grams})]\ (1{,}000\ \text{millilitres per litre})/(\text{volume of sample, millilitres})) \qquad (3\text{-}8)$$

(5) Centrifugation method. This method is recommended for samples from saltwater environments that have a suspended solids concentration greater than 1 gram per litre. It is particularly useful when the dissolved solids concentration or salinity is unknown but is expected to be significant (greater than 10 percent of the suspended solids concentration). This method is preferable to the total solids method when the dissolved solids concentration is several times greater than the suspended solids concentration. These steps should be followed:

(a) Centrifuge a measured volume of sample until the liquid and solids have separated, yielding clear supernatant (several minutes should be sufficient).

(b) Pour off the supernatant, being careful not to lose any of the solids.

(c) Resuspend the settled solids in distilled water by diluting the sample to its initial volume.

(d) Repeat steps (a) through (c) twice to wash out all dissolved solids.

(e) Pour the sample into a preweighed dish and then wash all remaining solids from the centrifuge tube into the dish, using distilled water.

(f) Place the dish in a drying oven at 105" C until the sample has dried to constant weight (usually 4 to 6 hours).

(g) Cool in a desiccator and weigh.

(h) Calculate suspended solids concentration C , in grams per litre, as follows:

$$C = \left\{\frac{[(\text{weight of dish and dry solids, grams}) - (\text{weight of dish, grams})] \times 1{,}000\ \text{millilitres per litre}}{(\text{volume of sample, millilitre})}\right\} \qquad (3\text{-}9)$$

(6) Correlation of suspended solids with turbidity. In some cases, effluent quality standards are specified in terms of turbidity, an optical property. Relationships between suspended solids concentration and turbidity are sediment-specific and can be determined only by preparing a correlation curve. The correlation curve is developed by determining turbidity and suspended solids concentration of samples prepared over a sufficiently wide range of concentrations.

e. Sample Cornpositing and Separation.

(1) Following determination of in situ water content, the sediment sample(s) must be homogenized, split, and possibly separated into coarse- and

fine-grained fractions prior to further testing. Sediment characterization tests such as plasticity, grain size determination, specific gravity and organic content may be performed on grab samples from each of several sampling locations. Other tests, such as consolidation and settling tests, should be performed on an appropriately composited and homogenized sample. The need for and methods of compositing are highly project-dependent, but should be aimed toward producing a sample for testing that is representative of the material to be dredged. If composite samples are to be used for further testing, they must be thoroughly mixed. Samples for settling tests (approximately 15 gallons) may require addition of some water to aid in mixing.

(2) Sediment character as determined from in situ samples is not indicative of dredged material behavior after dredging since the fine-grained (<No. 200 sieve) fraction will undergo natural segregation within the containment area and will behave independently of the coarse-grained (>No. 200 sieve) fraction. Therefore, the relative percentage (dry weight basis) of coarse- and fine-grained material should be determined by separation of a small portion of the sample using a No. 200 sieve and following procedures generally described in EM 1110-2-1906.

(3) If the coarse-grained fraction is less than 10 percent by dry weight, the sediment sample is considered to be fine grained and is treated as though all the material passed the No. 200 sieve; separation for further characterization tests is not required. If the coarse-grained fraction is greater than 10 percent by dry weight, the entire sample should be separated into coarse- and fine-grained fractions prior to further testing. Separation can be accomplished for small sample volumes (e.g., those intended for classification or consolidation testing) by using the No. 200 sieve as described above. However, the larger sample volume required for sedimentation tests makes the use of a sieve impractical. For such volumes, slurry (sediment plus water) can be thoroughly mixed in a large barrel and then allowed to separate by differential settling. After initial mixing is stopped, coarse material will quickly accumulate on the bottom. The slurry remaining above the coarse material can be pumped into a second barrel, where it can be remixed and loaded into the testing column.

(4) In conducting the various tests and during sample separation and preparation activities, it will be necessary to make up slurries of various solids concentrations. In doing so, it is advisable to begin the testing sequence with slurry of higher concentration and add the required volume of water to obtain the desired lower concentration. The following simple relationship is useful in calculating the volume of additional water required:

$$C_1V_1 = C_2V_2 \tag{3-10}$$

where

C_1 and C_2 = solids concentrations
V_1 and V_2 = slurry volumes (water plus solids)

f. Grain Size Analyses. Grain size analyses should be performed on coarse-grained samples or on the coarse-grained fraction of samples that are mixtures of coarse- and fine-grained material. These analyses are used to classify the coarse-grained portion of the sediments. The fine-grained material (passing the No. 200 sieve) should be used in the other characterization and consolidation tests if required. Grain size analyses should follow the procedures contained in EM 1110-2-1906. Hydrometer analyses can be used to define the grain size distribution of the fine-grained fraction if desired.

g. Plasticity Analyses. In order to evaluate the plasticity of fine-grained samples of sediment, the Atterberg liquid limit (LL) and plastic limit (PL) must be determined. The LL is that water content above which the material is said to be in a semiliquid state and below which the material is in a plastic state. Similarly, the water content that defines the lower limit of the plastic state and the upper limit of the semisolid state is termed the PL. The plasticity index (PI), defined as the numerical difference between the LL and the PL, is used to express the plasticity of the sediment. Plasticity analyses should be performed on the fine-grained fraction (<No. 200 sieve) of sediment samples. A detailed explanation of the LL and PL test procedures and apparatus can be found in Appendix III of EM 1110-2-1906.

h. Organic Content. A knowledge of whether significant organic matter is present is required. The following dry-combustion test procedure is recommended to determine the organic content expressed as the percentage of weight lost on ignition:

(1) Dry a 40-gram sample at 105° C until there is no further weight loss (usually 4 to 6 hours).

(2) Place it in a desiccator to cool for 15 minutes.

(3) Weigh the sample and place it in the oven at 440° C for 4 hours.

(4) Place it in the desiccator to cool for 15 minutes.

(5) Weigh the sample and determine the organic content by dividing the weight lost by the sample while in the oven at 440° C by the total weight of the sample at the time it was placed in the oven.

i. Specific Gravity. Values for the specific gravity of solids for fine-grained sediments and dredged material are required for determining void ratios, conducting hydrometer analyses, and consolidation testing. Procedures for conducting the specific gravity test are given in Appendix IV of EM 1110-2-1906.

j. USCS Classification. When classifying sediment samples, the fine-grained portion that passes the No. 200 sieve should be classified separately from the coarse-grained portion retained on the No. 200 sieve, regardless of which fraction comprises the greatest percentage by weight. Additional information regarding the USCS classification may be found in WE'S Technical Memorandum No. 3-357 (item 33).

3-3. Settling Tests. Dredged material placed in disposal areas by hydraulic dredges or pumped into disposal areas by pump-out facilities enters the disposal area as a slurry (mixture of dredged solids and dredging site water). Settling refers to those processes in which the dredged material slurry is separated into supernatant water of low solids concentration and a more concentrated slurry. Laboratory sedimentation tests provide data for designing the containment area to meet effluent suspended solids criteria and to provide adequate storage capacity for the dredged solids.

a. Settling Processes.

(1) Settling types. The settling process can be categorized according to four basic classifications: discrete settling where the particle maintains its individuality and does not change in size, shape, or density during the settling process; flocculent settling where particles agglomerate during the settling period with a change in physical properties and settling rate; zone settling where the flocculent suspension forms a lattice structure and settles as a mass, the high solids concentration partially blocks the release of water and hinders settling of neighboring particles, and a distinct interface between the slurry and the supernatant water is exhibited during the settling process; and compression settling where settling occurs by compression of the lattice structure. All of the above sedimentation processes may occur simultaneously in a disposal area, and any one may control the design of the disposal area.

(2) Governing factors. Discrete settling describes the sedimentation of coarse particles. The important factors governing the sedimentation of fine-grained dredged material are the initial concentration of the slurry, salinity of the carrier water, and the flocculating properties of the solid particles. Because of the high influent solids concentration and the tendency of fine-grained particles to flocculate, either flocculent or zone settling behavior normally describes sedimentation in containment areas. Sedimentation of freshwater sediments at slurry concentrations of 100 grams per litre can generally be characterized by flocculent settling properties. As slurry concentrations or salinity is increased, the sedimentation process may be characterized by zone settling properties. Compression settling occurs in the lower layers of settled material for both the flocculent and zone settling cases. As more settled material accumulates, excess pore pressures develop in the lower layers and further consolidation occurs as water is expelled and the excess pore pressures dissipate.

(3) Zone versus flocculent settling as a function of salinity. The tendency of a fine-grained dredged material slurry to settle by zone or flocculent behavior in the initial stages of settling is strongly influenced by the presence of salt as a coagulant. If salinity is less than 1 part per thousand, indicative of freshwater conditions, flocculent processes normally describe the initial settling, and no clearly defined interface is seen. If salinity is greater than 1 part per thousand, indicative of brackish or saltwater conditions, zone settling processes normally describe the initial settling, and a clear interface between the clarified supernatant water and the more concentrated slurry is evident. For the zone settling case, some of the fine particles remain in the supernatant water as the interface falls. Flocculent processes then describe the settling of these fine particles from the supernatant.

b. Testing Equipment and Procedures.

(1) Test objective. The objective of running settling tests on sediments to be dredged is to define their settling behavior in a dredged material containment area. The tests provide numerical values for the design criteria that can be projected to the size and design of the containment area. Procedures for computer-assisted plotting and reduction of settling column data are available as discussed in Chapter 8.

(2) Settling column. The settling column shown in Figure 3-3 should be used for dredged material settling tests. The column is constructed of 8-inch-diameter Plexiglas tubing and can be sectioned for easier handling and cleaning. Ports are provided for extraction of samples at various depths during sampling. A bottom-mounted airstone is also provided for agitation and mixing of slurries in the column by using compressed air. Shop drawings of the column with bills of materials are shown in Appendix B.

(3) Samples. Samples used to perform settling tests should consist of fine-grained (<No. 200 sieve) material. Any coarse-grained (>No. 200 sieve) material present in the sample would normally be hydraulically separated when the sample is mixed prior to sedimentation testing. A composite of several sediment samples may be used to perform the tests if this is thought to be more representative of the dredged material. Approximately 15 gallons of sediment is usually required for the tests. Water used to mix the slurries can be taken from the proposed dredging site or can be prepared by mixing tap-water and salt to the known salinity of the dredging site water.

(4) Pilot test. A pilot test conducted in a graduated cylinder (4 litres is satisfactory) is a useful method for determining if flocculent or zone process will describe the initial settling. The pilot test should be run at a slurry concentration of approximately 150 grams per litre. If an interface forms within the first few hours of the test, the slurry mass is exhibiting zone settling, and the fall of the interface versus time should be recorded. The curve will appear as shown in Figure 3-4. The break in the curve will define the concentration at which compression settling begins. Only lower concentrations should be used for the zone settling test in the 8-inch column. If no break in the curve is evident, the material has begun settling in the compression zone, and the pilot test should be repeated at a lower slurry concentration. It should be emphasized that use of a small cylinder as in the pilot test is not acceptable for use in design. Wall effects for columns of small diameter affect zone settling velocities, and data obtained using small-diameter columns will not accurately reflect field behavior. If no interface is observed in the pilot test within the first few hours, the slurry mass is exhibiting flocculent settling. In this case, the pilot test should be continued until an interface is observed between the turbid water above and more concentrated settled solids below. The concentration of the settled solids (computed assuming zero concentration of solids above) is an indication of the concentration at which the material exhibits compression settling.

(5) Required number of column loadings for tests. Three types of settling tests in the B-inch column may be needed to fully define the settling properties of the dredged material. However, in most cases the B-inch

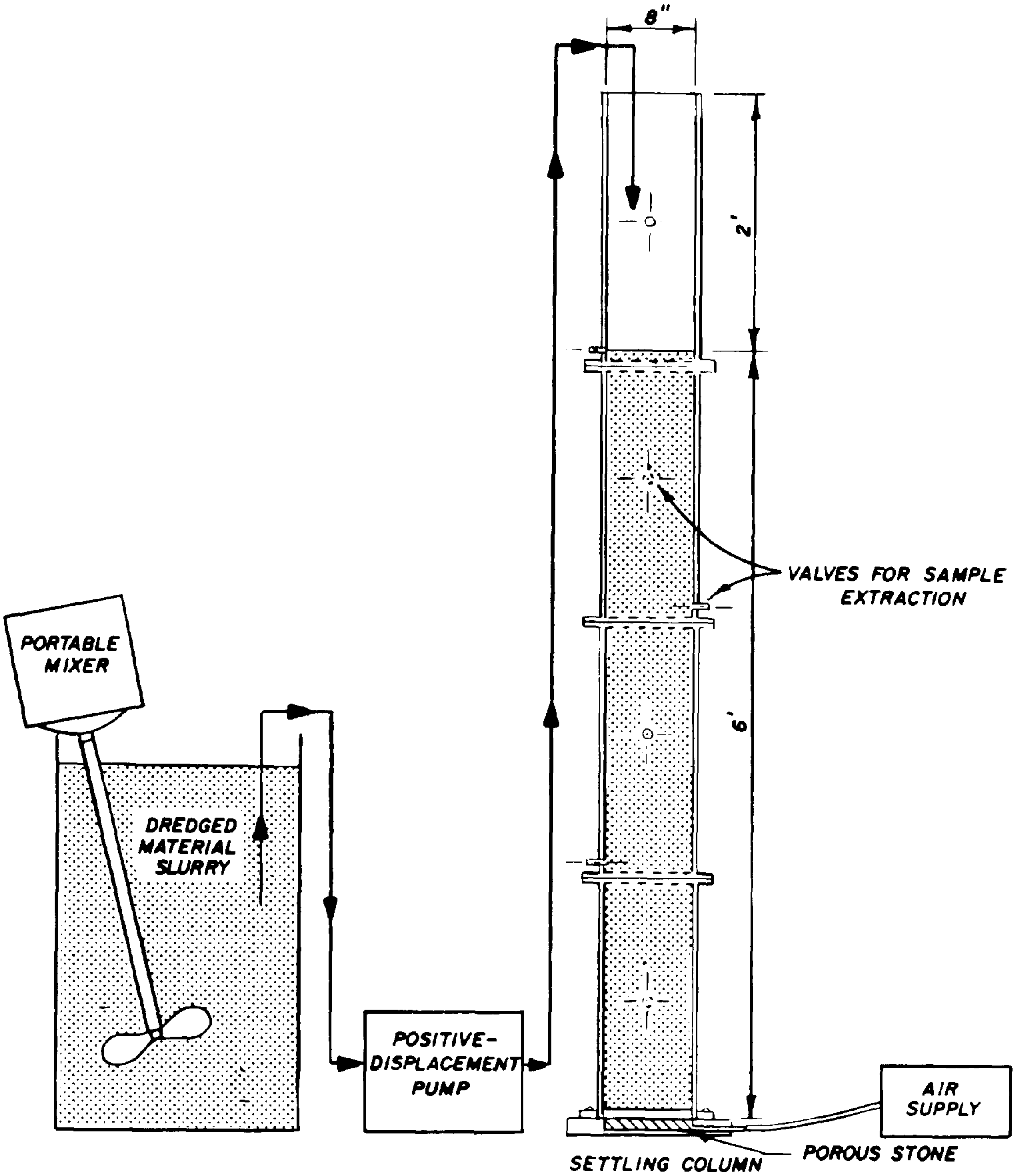


Figure 3-3. Schematic of apparatus for settling tests

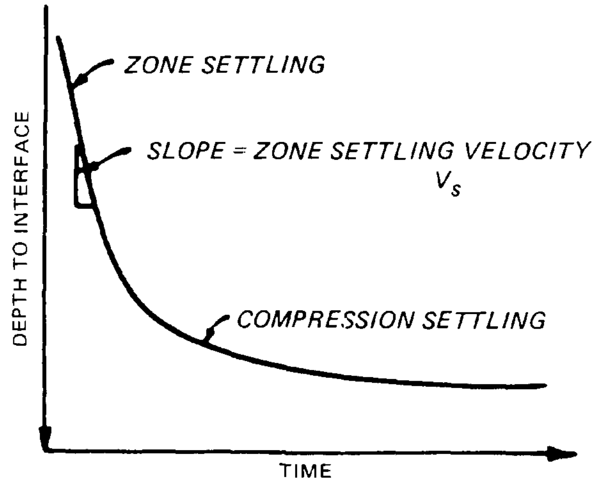


Figure 3-4. Conceptual plot of interface height versus time

settling column used for the settling tests needs to be loaded with slurry only once. A compression settling test is needed to define the volume that will be occupied in the disposal area by a newly deposited dredged material layer at the end of the disposal operation. A flocculent settling test for either the slurry mass or for the supernatant water above any interface is required to predict effluent suspended solids concentrations. A zone settling test is required to define the minimum surface area needed for effective zone settling. These tests should be conducted at a slurry concentration equal to the expected influent concentration; therefore, only one loading of the test column would be required to collect data for all purposes.

c. Flocculent Settling Test.

(1) The flocculent settling test consists of measuring the concentration of suspended solids at various depths and time intervals in a settling column. If an interface forms near the top of the settling column during the first day of the test, sedimentation of the material below the interface is described by zone settling. In that case, the flocculent test procedure should be continued only for that portion of the column above the interface.

(2) Information required to design a containment area for the flocculent settling process can be obtained using the following procedure:

(a) A settling column such as shown in Figure 3-3 is used. The slurry depth used in the test column should approximate the effective settling depth of the proposed containment area. A practical limit on depth of test is 6 feet. The column should be at least 8 inches in diameter with sample ports at 0.5 foot intervals (minimum). The column should have provisions for slurry agitation with compressed air from the bottom to keep the slurry mixed during the column filling period.

(b) Mix the sediment slurry to a suspended solids concentration C equal to the expected concentration of the dredged material influent C_i . The slurry should be mixed in a container with sufficient volume to fill the test column. Field studies indicate that for maintenance dredging of fine-grained material, the disposal concentration will average about 150 grams per litre. This concentration should be used in the test if better data are not available.

(c) Pump or pour the slurry into the test column using compressed air or mechanical agitation to maintain a uniform concentration during the filling period.

(d) When the slurry is completely mixed in the column, cut off the compressed air or mechanical agitation and immediately draw off samples at each sample port and determine their suspended solids concentration. Use the average of these values as the initial slurry concentration at the start of the test. The test is considered initiated when the first samples are drawn.

(e) If an interface has not formed on the first day, flocculent settling is occurring in the entire slurry mass. Allow the slurry to settle and withdraw samples from each sampling port at regular time intervals to determine the suspended solids concentrations. Substantial reductions of suspended solids will occur during the early part of the test, but reductions will lessen at longer retention times. Therefore, the intervals can be extended as the test progresses. Recommended sampling intervals are 1, 2, 4, 6, 12, 24, 48 hours, etc., until the end of the test. As a rule, a 50-millilitre sample should be taken from each port. Continue the test until an interface can be seen near the bottom of the column and the suspended solids concentration in the fluid above the interface is 1 gram per litre. Test data are tabulated and used to plot a concentration profile diagram as shown in Figure 3-5. Examples are shown in Appendix C.

(f) If an interface forms the first day, zone settling is occurring in the slurry below the interface, and flocculent settling is occurring in the supernatant water. For this case, samples should be extracted from all side ports above the falling interface. The first of these samples should be extracted immediately after the interface has fallen sufficiently below the uppermost port to allow extraction. This sample can usually be extracted within a few hours after the beginning of the test, depending on the initial slurry concentration and the spacing of ports. Record the time of extraction and port height for each port sample taken. As the interface continues to fall, extract samples from all ports above the interface at regular time intervals. As an alternative, samples can be taken above the interface at the desired depths using a pipette or syringe and tubing. As before, a suggested sequence of sampling intervals would be 1, 2, 4, 6, 12, 24, 48, 96 hours, etc. The samples should continue to be taken until the suspended solids concentration of the extracted samples shows no decrease. For this case, the suspended solids in the samples should be less then 1 gram per litre, and filtration will be required to determine the concentrations. The data should be expressed in milligrams per litre for these samples. Tabulate the data and plot a concentration profile diagram as shown in Figure 3-5. In reducing the data for this case, the concentration of the first port sample taken above the falling interface is considered the initial concentration C_o . Examples are shown in Appendix C.

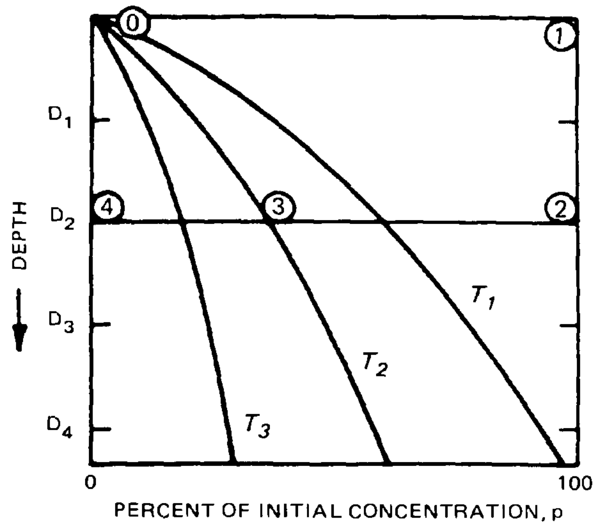


Figure 3-5. Conceptual concentration profile diagram

d. Zone Settling Test.

(1) The zone settling test consists of placing a slurry in a sedimentation column and reading and recording the fall of the liquid-solids interface with time. These data are plotted as depth to interface versus time. The slope of the constant velocity settling zone of the curve is the zone settling velocity, which is a function of the initial test slurry concentration. This test is required if the material exhibits an interface within the first day. The test should be run at the expected influent slurry concentration, or the highest expected to persist for several hours if a range is expected.

(2) Information required to design a containment area for the zone settling process can be obtained by using the following procedure:

(a) A settling column such as shown in Figure 3-3 is used. It is important that the column diameter be sufficient to reduce the "wall effect" and that the test be performed with a test slurry depth near that expected in the field. Therefore, a 1-litre graduated cylinder should never be used to perform a zone settling test for sediment slurries representing dredging disposal activities.

(b) Mix the slurry to the desired concentration and pump or pour it into the test column. Air may not be necessary to keep the slurry mixed if the filling time is less than 1 minute.

(c) Record the depth to the solid-liquid interface as a function of time. Readings must be taken at regular intervals to gain data for plotting the curve of depth to interface versus time as shown in Figure 3-4. It is important to take enough readings to clearly define this curve.

(d) Continue the readings until sufficient data are available to define the maximum point of curvature of the depth to interface versus time plot. The test may require from 8 to 48 hours to complete.

(e) Calculate the zone settling velocity v_s as the slope of the constant velocity settling zone, as shown in Figure 3-4 (straight-line portion of curve). The velocity should be in feet per hour.

(f) Compression Settling Test.

(1) A compression settling test must be run to obtain data for estimating the volume required for initial storage of the dredged material. For slurries exhibiting zone settling, the compression settling data can be obtained from the zone settling test with interface height versus time recorded. The only difference is that the test is continued for a period of 15 days, so that a relationship of log of concentration versus log of time in the compression settling range as shown in Figure 3-6 is obtained. For slurries exhibiting flocculent settling behavior, the test used to obtain flocculent settling data can be used for the compression settling test if an interface is formed after the first few days of the test. If not, an additional test is required with the slurry concentration for the test sufficiently high to initially induce compression settling. This concentration can be determined by the pilot test.

(2) Information required to design a containment area for the compression settling process can be obtained using the following procedures:

(a) Tabulate the interface height H_t for various times of observation during the 15-day test period.

(b) Calculate concentrations for various interface heights as follows:

$$C = \frac{C_o H_i}{H_t} \qquad (3\text{-}11)$$

where

C = slurry concentration at time t, grams per litre

C_o = initial slurry concentration, grams per litre

H_i = initial slurry height, feet

H_t = height of interface at time t, feet

Neglect solids in the water above the interface to simplify calculations.

(c) Plot concentration versus time on log-log paper as shown in Figure 3-6.

(d) Draw a straight line through the data points.

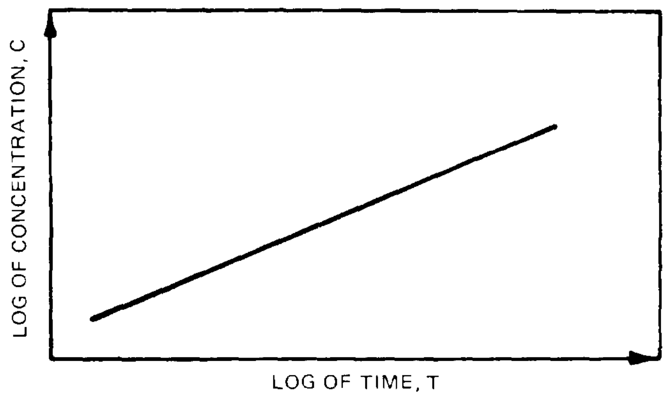


Figure 3-6. Conceptual time versus concentration plot

3-4. Chemical Clarification Tests.

a. General. If for a given disposal area, gravity sedimentation does not remove sufficient suspended solids to meet effluent standards, chemical clarification may be required. This may be especially true when a freshwater sediment with a significant clay fraction is dredged. The effluents from such sites may contain several grams per liter of suspended solids after gravity settling prior to chemical clarification.

b. Jar Tests.

(1) Jar tests have traditionally been used to evaluate the effectiveness of various flocculants under a variety of operating conditions for water treatment, and these procedures have been applied to the disposal of dredged material. Jar tests are used to provide information on the most effective flocculant, optimum dosage, optimum feed concentration, effects of dosage on removal efficiencies, effects of concentration of influent suspension on removal efficiencies, effects of mixing conditions, and effects of settling time.

(2) The general approach used in the jar test procedures is as follows:

(a) Using site-specific information on the sediment, dredging operation, containment areas, and effluent requirements, select mixing conditions, suspension concentration, settling time, and polymers for testing. Test a small number (four to six) of polymers that have performed well on similar dredged material.

(b) Prepare stock suspension of sediment.

(c) The tests should be run on 2-grams-per-litre suspensions of sediment, which is a typical concentration for effluents from a well-designed containment area for freshwater clay sediments. If good removals are obtained at low dosages (10 milligrams per litre of polymer or less), then select the most cost-effective polymer. If good removals are not obtained, examine the polymer under improved mixing and settling conditions and test the performance of other flocculants.

(d) After selecting a polymer and its optimum dosage, examine the effect of polymer-feed concentration over the range of sediment concentrations of 1 to 30 grams per litre, typical concentrations found in the field, at the optimum dosage.

(e) Determine the dosage requirements for the expected range of suspended solids concentrations to be treated at the primary weir.

(f) Examine the effects of the range of possible mixing conditions on the required dosage of flocculant for a typical suspension.

(g) Examine the effects of settling time on the removal of suspended solids from a suspension of average suspended solids concentration using the selected dosage and likely mixing conditions.

(3) The purpose of the approach described is to select an effective polymer for a suspension of a standard suspended solids concentration, 2 grams per litre, which is a typical effluent solids concentration. In this manner, the effectiveness and dosage requirements of various polymers are easy to compare. The other test variables are set to simulate anticipated field conditions. After a polymer is selected, other variables are examined: polymer-feed concentration, solids concentration of suspension to be treated, mixing, and settling time. The approach may be changed to fit the needs and conditions of the specific study. Detailed jar test procedures are found in Appendix E.

3-5. Consolidation Testing. Determination of containment area long-term storage capacity requires estimates of settlement due to self-weight consolidation of newly placed dredged material and due to consolidation of compressible foundation soils. Consolidation test results must be obtained, including time-consolidation data, to estimate the average void ratios at completion of 100 percent primary consolidation.

a. Consolidation tests for foundation soils should be performed as described in EM 1110-2-1906.

b. Controlled-rate-of-strain tests or fixed-ring consolidometers should be used for consolidation testing of sediment samples because of their fluid-like consistency. The only major modifications for the conventional fixed-ring testing procedure concern the sample preparation and the method of loading. Detailed procedures are found in Appendix D.

CHAPTER 4

CONTAINMENT AREA DESIGN FOR RETENTION OF SOLIDS AND INITIAL STORAGE

4-1. General.

a. This chapter presents guidelines for designing a new containment area for suspended solids retention and for evaluating the suspended solids retention potential of an existing containment area. Intermittent dredging, with higher costs, may be required if dredging flow rates exceed the solids retention capacity of a disposal area. This condition can be avoided by following the design guidelines in this chapter. The focus in this section is on fine-grained dredged-material. Guidelines presented here will provide the necessary guidance for designing a containment area for adequate space and volume for retaining the solids within the containment area through settling and providing storage capacity of dredged solids for a single dredged material disposal operation. The major objective is to provide solids removal by the process of gravity settling to a level that permits discharge of the transporting water from the area. Although ponding is not feasible over the entire surface area of many sites, an adequate ponding depth must be maintained over the design surface area as determined by these design procedures to assure adequate retention of solids. Guidance is also presented in this chapter for the design of weirs for the release of ponded water and for chemical clarification systems for additional removal of suspended solids.

b. The design procedures presented here are for gravity settling of dredged solids. However, the process of gravity sedimentation will not completely remove the suspended solids from the containment area effluent since wind and other factors resuspend solids and increase effluent solids concentration. The settling process, with proper design and operation, will normally provide removal of fine-grained dredged material down to a level of 1 to 2 grams per litre in the effluent for freshwater conditions. The settling process will usually provide removal of fine-grained dredged material down to a level of several hundred milligrams per litre or lower for saltwater conditions. If the required effluent standard is not met by gravity settling, the designer must provide for additional treatment of the effluent, e.g., flocculation or filtration.

c. The generalized flowchart shown in Figure 4-1 illustrates the design procedures presented in the following paragraphs. These steps were adapted from procedures used in water and wastewater treatment and are based on field and laboratory investigations on sediments and dredged material at active dredged material containment areas. The procedures in this chapter are presented in the manner required to calculate the minimum required disposal area geometry for a given inflow rate (dredge size) and dredged volume. The same procedures would be used in reverse fashion to calculate a maximum flow rate (dredge size) allowable for a given disposal area geometry. Numerical examples of both approaches are presented in Appendix C. Procedures for computer-assisted design for sedimentation and initial storage are available as discussed in Chapter 8.

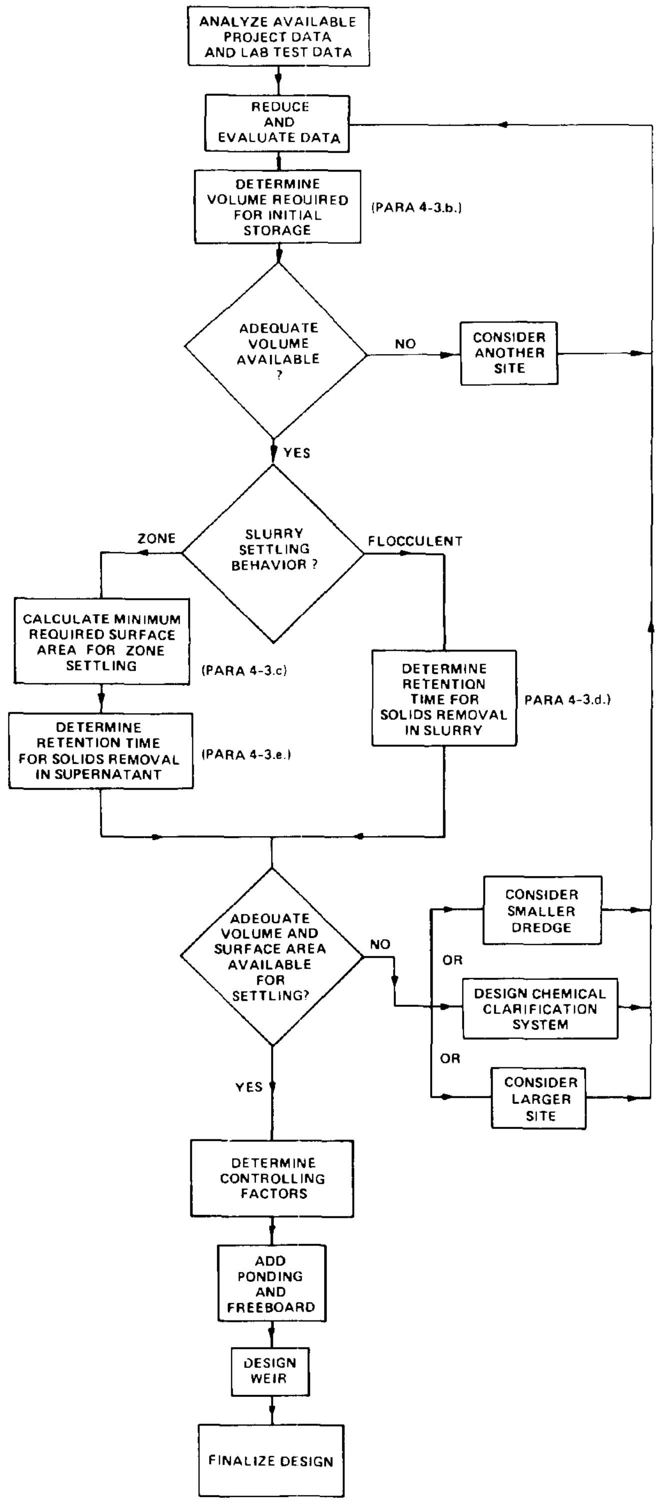


Figure 4-1. Flowchart of design procedure for settling and initial storage

4-2. Data Requirements.

a. General. The data required to use the design guidelines are obtained from field investigations (Chapter 2), laboratory testing (Chapter 3), project-specific operational constraints, and experience in dredging and disposal activities. The types of data required are described in the following paragraphs.

b. In Situ Sediment Volume. The initial step in any dredging activity is to estimate the total in situ channel volume of sediment to be dredged V_c . Sediment quantities are usually determined from routine channel surveys.

c. Physical Characteristics of Sediments. Field sampling and sediment characterization should be accomplished according to the laboratory tests described in Chapters 2 and 3 of this manual. Adequate sample coverage is required to provide representative samples of the sediment. Also required are in situ water contents of the fine-grained maintenance sediments. Care must be taken in sampling to ensure that the water contents are representative of the in situ conditions. Water contents of representative samples w are used to determine the in situ void ratios e_i as follows:

$$e_i = \frac{wG_s}{S_D} \qquad (4\text{-}1)$$

where

e_i = in situ void ratio of sediment

w = water content of the sample, percent

G = specific gravity of sediment solids

S_D = degree of saturation, percent (equal to 100 percent for sediments)

A representative value for in situ void ratios is used later to estimate volume for the containment area. Grain size analyses are used to estimate the quantities of coarse- and fine-grained material in the sediment to be dredged. The volume of sand V_{sd} can be estimated as a percentage of the total volume V_c to be dredged by using the percent coarser than No. 200 sieve. The in situ volume of fine-grained sediment V_i is equal to $V_c - V_{sd}$.

d. Proposed Dredging and Disposal Data. The designer must obtain and analyze data concerning the dredged material disposal rate. For hydraulic pipeline dredges, the type and size of dredge(s) to be used, average distance to containment area from dredging activity, depth of dredging, and average solids concentration of dredged material when discharged into the containment area must be considered. If the size of the dredge to be used is not known, the largest dredge size that might be expected to perform the dredging should be assumed. The time required for the dredging can be estimated, based on experience. If no data on past experience are available, Figure 4-2, which shows the relationship among solids output, dredge size, and pipeline length for various dredging depths, should be used. It was developed from data provided for Ellicott dredges for dredging in sand (item 32). Additional guidance on dredge production rates is found in ER 1110-2-1300. For hopper dredge

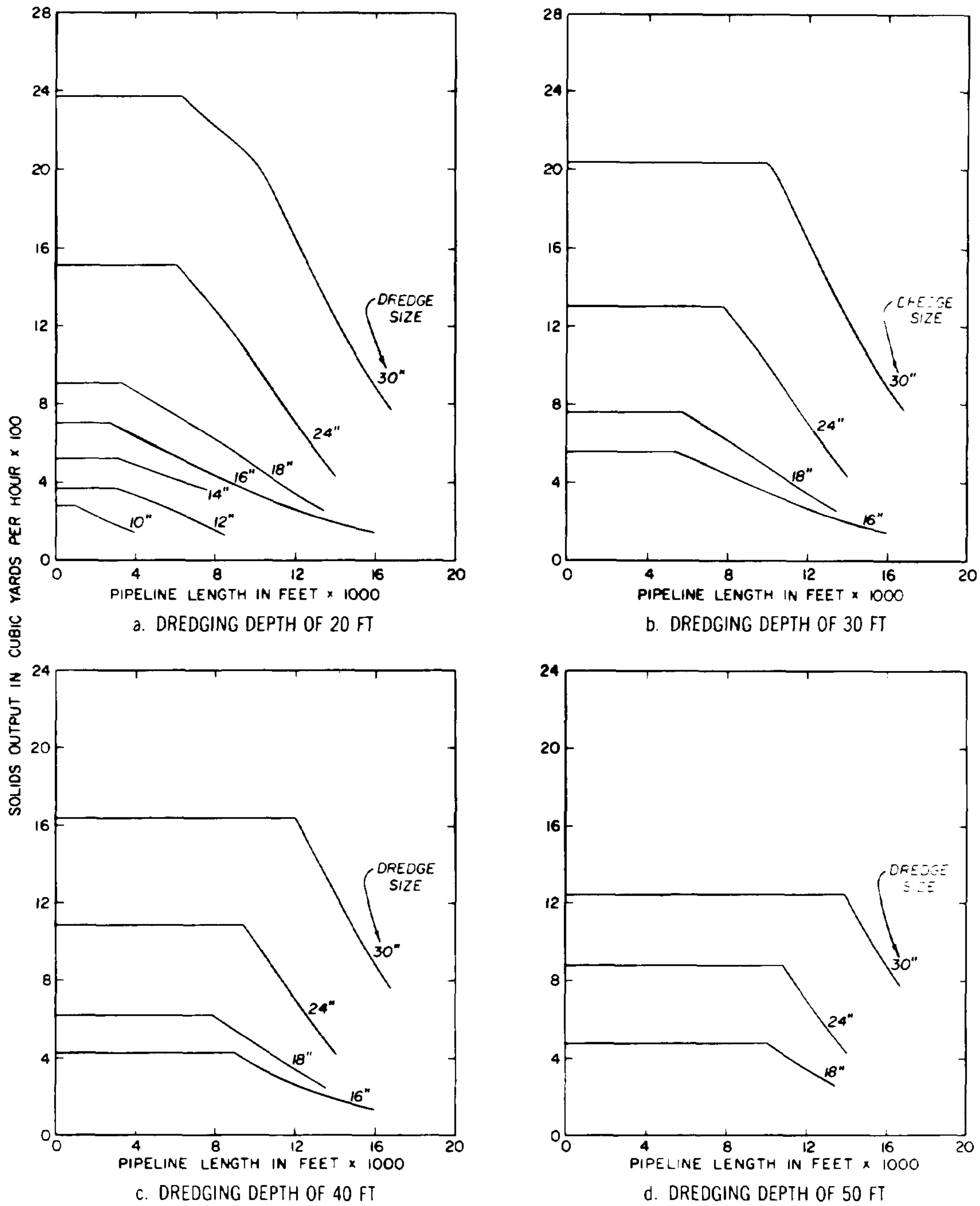


Figure 4-2. Relationships among solids output, dredge size, and pipeline length for various dredging depths

or barge pump-out operations, an equivalent disposal rate must be estimated based on hopper or barge pump-out rate and travel time involved. Based on these data, the designer must estimate or determine containment area influent rate, influent suspended solids concentration, effluent rate (for weir sizing), and time required to complete the disposal activity. For hydraulic pipeline dredges,-if no other data are available, an influent suspended solids concentration C_i of 150 grams per litre (14 percent by weight) should be used for design purposes. The influent flow rate Q_i can be estimated using the following tabulation or from other available data:

Discharge Pipeline Diameter, in.	Discharge Rate (for Flow Velocity of 15 ft/sec)* cfs	gal/min
8	5.3	2,350
10	8.1	3,640
12	11.8	5,260
14	16.0	7,160
16	20.6	9,230
18	26.5	11,860
20	32.7	14,660
24	47.1	21,090
27	59.5	26,630
28	64.1	28,700
30	73.6	32,950
36	106.0	47,500

* To obtain discharge rates for other velocities, multiply the discharge rate shown in this tabulation by the desired velocity and divide by 15.

e. Laboratory Settling Test Data. The guidelines for sedimentation tests are given in Section 3-3. Depending on the results of the sedimentation tests, the dredged material slurry will settle by either zone processes (common for saltwater sediments) or flocculent processes (common for freshwater sediments). Regardless of the salinity, flocculent processes govern the concentration of solids in the effluent.

4-3. Sedimentation Basin Design.

a. Selection of Minimum Average Ponding Depth. Before a disposal site can be designed for effective settling or before the required disposal area geometry can be finalized, a ponding depth H_{pd} during disposal must be assumed. The design procedures in the following paragraphs call for an average ponding depth in estimating the residence time necessary for effective settling. A minimum average ponding depth of 2 feet should be used for the design. If the design objective is to minimize the surface area required, selection of a deeper ponding depth may be desirable. If conditions will allow for the greater ponding depth throughout the operation, the greater value can be used. For most cases, constant ponding depth can be maintained by raising the pond surface as settled material accumulates in the containment area by raising the elevation of the weir crest.

b. Calculation of Volume for Initial Storage.

(1) General. Containment areas must be designed to meet volume requirements for a particular disposal activity. The total volume required in a containment area includes volume for storage of dredged material, volume for sedimentation (ponding depths), and freeboard volume (volume above water surface). Volume required for storage of the coarse-grained (>No. 200 sieve) material must be determined separately since this material behaves independently of the fine-grained (<No. 200 sieve) material.

(2) Calculation of design concentration. The design concentration C_d is defined as the average concentration of the dredged material in the containment area at the end of the disposal activity and is estimated from the compression (15-day) settling test described in Chapter 3. This design parameter is required both for estimating initial storage requirements and for determining minimum required surface areas for effective zone settling. The following steps can be used to estimate C_d from the compression settling test.

(a) Estimate the time of dredging by dividing the dredge production rate into the volume of sediment to be dredged. Use Figure 4-2 for estimating the dredge production rate if no specific data are available from past dredging activities. (Note that curves in Figure 4-2 were developed for sand.) The total time required for dredging should allow for anticipated downtime.

(b) Enter the concentration versus time plot as shown in Figure 4-3 and determine the concentration at a time t equal to one-half the time required for the disposal activity determined in step (a).

(c) The value computed in step (b) is the design solids concentration C_d. Examples are shown in Appendix C.

(3) Volume estimation. The volume computed in the following steps is the volume occupied by dredged material in the containment area immediately after the completion of a particular disposal activity. This value is critical in determining the dike height requirements for the containment area. The volume is not an estimate of the long-term needs for multiple-disposal activities. Estimates for long-term storage capacity can be made using the procedures outlined in Chapter 5. The design for initial storage may be a controlling factor regardless of the settling behavior exhibited by the material. If the material initially exhibits compression settling at the expected inflow concentration, the design for initial storage is the only consideration (this is expected to be an exceptional case).

(a) Compute the average void ratio of the fine-grained dredged material in the containment area at the completion of the dredging operation using the

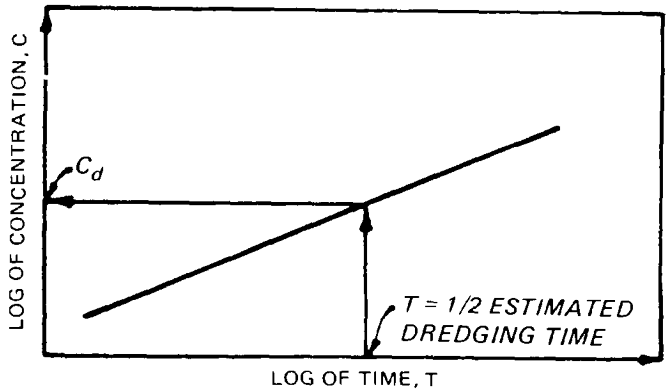


Figure 4-3. Conceptual time versus concentration plot

design concentration C_d determined in 4-3.b. Use the following equation to determine the void ratio:

$$e_o = \frac{G_s \gamma_w}{C_d} - 1 \qquad (4\text{-}2)$$

where

e_o = average void ratio of the dredged material in the containment area at the completion of the dredging operation

γ_w = density of water, grams per litre (normally 1,000 grams per litre)

(b) Compute the volume of the fine-grained channel sediments after disposal in the containment area:

$$V_f = V_i \left[\frac{e_o - e_i}{1 + e_i} \right] + 1 \qquad (4\text{-}3)$$

where

V_f = volume of the fine-grained dredged material after disposal in the containment area, cubic feet

V_i = volume of the fine-grained channel sediments, cubic feet

e_i = average void ratio of the in situ channel sediments

(c) Compute the volume required to store the dredged material in the containment area:

$$V = V_f + V_{sd} \qquad (4\text{-}4a)$$

where

V = total volume of the dredged material in the containment area at the end of the dredging operation, cubic feet

V_{sd} = volume of sand (use 1:1 ratio), cubic feet

(d) If there are limitations on the surface area available for disposal or if an existing disposal site is being evaluated, check whether the site conditions will allow for initial storage of the volume to be dredged. First determine the maximum height at which the material can be placed $H_{dm(max)}$ using the following equation:

$$H_{dm(max)} = H_{dk(max)} - H_{pd} - H_{fb} \quad (4\text{-}4b)$$

where

$H_{dk(max)}$ = maximum allowable dike height due to foundation conditions, feet

H_{pd} = ponding depth, feet

H_{fb} = freeboard (minimum of 2 feet can be assumed), feet

Compute the minimum surface area that could be used to store the material:

$$A_{ds} = \frac{V}{H_{dm(max)}\ (43{,}560)} \quad (4\text{-}4c)$$

where

A_{ds} = design surface area for storage, acres

If A_{ds} is less than the available surface area, then adequate volumetric storage is available at the site.

c. Calculation of Minimum Surface Area for Effective Zone Settling.

(1) General. If the sediment slurry exhibited zone settling behavior at the expected inflow concentration, the zone settling test results are used to calculate a minimum required ponded surface area in the containment for effective zone settling to occur. The method is generally applicable to dredged material from a saltwater environment, but the method can also be used for freshwater dredged material if the laboratory settling tests indicate that zone settling describes the initial settling process. Additional calculations using flocculent settling data for the solids remaining in the ponded supernatant water are required for designing the containment area to meet a specific effluent quality standard for suspended solids.

(2) Compute area required for zone settling. The minimum surface area determined according to the following steps should provide removal of fine-grained sediments so that suspended solids levels in the effluent do not exceed several hundred milligrams per litre. The area is required for the zone settling process to remove suspended solids from the surface layers at

the rate sufficient to form and maintain a clarified supernatant that can be discharged.

(a) Determine the zone settling velocity v_s at the influent suspended solids concentration C_i as described in paragraph 3-3.d.

(b) Compute area requirements as

$$A_z = \frac{Q_i\ (3600)}{V_s} \qquad (4\text{-}5)$$

where

A_z = containment surface area requirement for zone settling, square feet
Q_i = influent flowrate in ft^3/sec
3600 = conversion factor hours to seconds
V_s = zone settling velocity at influent solids concentration C_i, feet per hour

(c) Multiply the area by a hydraulic efficiency correction factor HECF to compensate for containment area inefficiencies:

$$A_{dz} = \frac{(HECF)A_z}{43{,}560} \qquad (4\text{-}6)$$

where

A_{dz} = design basin surface area for effective zone settling, acres
HECF = hydraulic efficiency correction factor (determined as described in 4-3.g.)
A_Z = area determined from Equation 4-5, square feet

d. Calculation of Required Retention for Flocculent Settling.

(1) Sediments dredged from a freshwater environment normally exhibit flocculent settling properties. However, in some cases, the concentration of dredged material slurry is sufficiently high that zone settling will occur. The method of settling can be determined from the laboratory tests.

(2) Sediments in a dredged material containment area are cornprosed of a broad range of particle floc sizes and surface characteristics. In the containment area, larger particle flocs settle at faster rates, thus overtaking finer flocs in their descent. This contact increases the floc sizes and enhances settling rates. The greater the ponding depth in the containment area, the greater is the opportunity for contact among sediments and flocs. Therefore, flocculent settling of dredged sediments is dependent on the ponding depth as well as the properties of the particles. For this reason, it is important that settling tests be performed with column heights corresponding to ponding depths expected under field conditions.

(3) The concentration of suspended solids in the effluent will depend on the total depth at which fluid is withdrawn at the weir, which is related to the hydraulic characteristics of the weir structure. The depth of withdrawal is equivalent to the depth of ponded water for weir configuration and flow rates that are normally encountered in containment areas. For this reason, the term "ponding depth" is used interchangeably with withdrawal zone in this manual in the context of effluent quality evaluations.

(4) Evaluation of the sedimentation characteristics of a sediment slurry exhibiting flocculent settling is accomplished as discussed in Chapter 3. The design steps to determine the required retention time for a desired effluent quality are as follows:

(a) Calculate the removal percentage at the selected minimum average ponding depth H_{pd} for various times using the concentration profile plot as shown in Figure 3-5. As an example, the removal percentage for H_{pd} = depth d_2 and time t_2 is computed as follows:

$$R = \frac{\text{Area right of profile}}{\text{Area total}}(100) = \frac{\text{Area } 0,\ 1,\ 2,\ 3,\ 0^*}{\text{Area } 0,\ 1,\ 2,\ 4,\ 0}(100) \qquad (4\text{-}5)$$

where R is the removal percentage. Determine these areas by either planimetering the plot or by direct graphical measurements and calculations. This approach is used to calculate removal percentages for the selected ponding depth as a function of time.

(b) Plot the solids removal percentages versus time as shown in Figure 4-4.

(c) Mean detention times can be selected from Figure 4-4 for various solids removal percentages. Select the residence time T_d that gives the desired removal percentage.

(d) The required mean residence time T_d should be multiplied by an appropriate hydraulic efficiency correction factor HECF to compensate for the fact that containment areas, because of inefficiencies, have field mean detention times less than theoretical (volumetric) detention times. The HECF is determined as described in 4-3.g. The basin volumetric or theoretical residence time is estimated as follows:

$$T = \text{HECF } T_d \qquad (4\text{-}8)$$

where T is the volumetric or theoretical residence time and T_d is selected from Figure 4-4.

(e) Note that for the case of flocculent settling of the entire slurry mass, the solids will be removed by gravity sedimentation to a level of 1 to

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* These numbers correspond to the numbers used in Figure 3-5 to indicate the area boundaries for the total area down to depth d_2 and the area to the right of the line for t_2 .

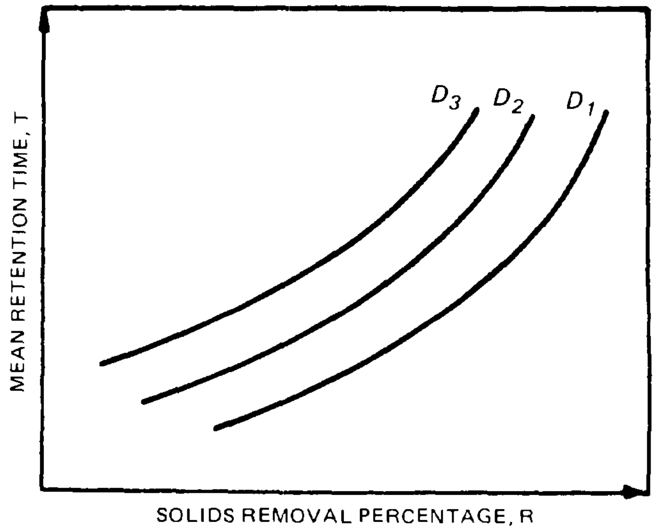


Figure 4-4. Conceptual plot of solids removal versus time for slurries exhibiting flocculent settling

2 grams per litre. For this case, the selection of a required residence time for a percentage removal is more convenient. For the case of flocculent settling in the supernatant water, where the slurry mass is undergoing zone settling, selection of a required residence time for an effluent suspended solids standard is more appropriate. Examples are shown in Appendix C.

e. Calculation of Required Retention Time for Flocculent Settling in Supernatant Water.

(1) Data analyses. For slurries exhibiting zone settling, flocculent settling behavior describes the process occurring in the supernatant water above the interface. Therefore, a flocculent data analysis procedure as outlined in the following paragraphs is required. The steps in the data analysis are as follows:

(a) Use the concentration profile diagram as shown in Figure 3-5 to graphically determine percentages removed, R , for the various time intervals and for the minimum ponding depth. This is done by graphically determining the areas to the right of each concentration profile and its ratio to the total area above the depth as described for the case of flocculent settling above.

(b) Compute the percentages remaining as follows:

$$P = 100 - R \tag{4-9}$$

(c) Compute values for the average suspended solids concentration in the supernatant at each time of extraction as follows:

$$C_t = P_t C_o \tag{4-10}$$

where

C_t = suspended solids concentration at time t , milligrams per litre

P_t = percentage remaining at time t

C_o = initial concentration in the supernatant, milligrams per litre

(d) Tabulate the data and plot a relationship for suspended solids concentration versus time using the value for each time of extraction as shown in Figure 4-5. An exponential curve fitted through the data points is recommended.

(2) Determination of retention time to meet an effluent suspended solids concentration. The relationship of supernatant suspended solids versus time developed from the column settling test is based on quiescent settling conditions found in the laboratory. The anticipated retention time in an existing disposal area under consideration can be used to determine a predicted suspended solids concentration from the relationship. This predicted value can be considered a minimum value able to be achieved in the field, assuming little or no resuspension of settled material. The relationship in Figure 4-5 can also be used to determine the required retention time to meet a standard for effluent suspended solids. For dredged material slurries exhibiting flocculent settling behavior, the concentration of particles in the ponded water is 1 gram per litre or higher. The resuspension resulting from normal wind conditions will not significantly increase this concentration; therefore, an adjustment for resuspension is not required for the flocculent settling case. However, an adjustment for anticipated resuspension is appropriate for dredged material exhibiting zone settling. The minimum expected value and the value adjusted for resuspension would provide a range of anticipated suspended solids concentrations in the effluent. The following procedure should be used:

(a) A standard for effluent suspended solids C_{eff} must be met considering anticipated resuspension under field conditions. A corresponding maximum concentration under quiescent laboratory conditions is calculated as:

$$C_{col} = \frac{C_{eff}}{RF} \tag{4-11}$$

where

C_{col} = Maximum suspended solids concentration of effluent as estimated from column settling tests, milligrams suspended solids per litre of water

C_{eff} = Suspended solids concentration of effluent considering anticipated resuspension, milligrams suspended solids per litre of water

RF = Resuspension factor selected from Table 4-1

Table 4-1 summarizes recommended resuspension factors based on comparisons of suspended solids concentrations as predicted from column settling tests and field data from a number of sites with varying site conditions.

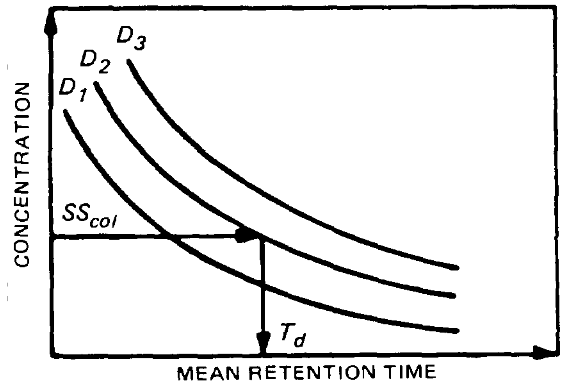


Figure 4-5. Conceptual plot of supernatant suspended solids concentration versus time from column settling test

Table 4-1

Recommended Resuspension Factors for the Zone Settling Case for Various Ponded Areas and Depths

	Anticipated Average Ponded Depth	
Anticipated Ponded Area	less than 2 feet	2 feet or greater
Less than 100 acres	2.0	1.5
Greater than 100 acres	2.5	2.0

(b) Using Figure 4-5, determine the required minimum mean residence time corresponding to C_{col} .

(c) As in the case for flocculent settling of the entire slurry mass, the mean residence time should be increased by an appropriate hydraulic efficiency factor HECF using Equation 4-8. The resulting minimum volumetric or theoretical residence time T can be used to determine the required disposal area geometry.

f. Computation of Design Surface Area for Flocculent Settling. The design surface area for flocculent settling can be calculated as follows:

$$A_{df} = \frac{T\ Q_i}{H_{pd}\ (12.1)} \tag{4-12}$$

where

A_{df} = design surface area for flocculent settling, acres

T = minimum mean residence time, hours

Q_i = average inflow rate, cubic feet per second

H_{pd} = average ponding depth, feet

12.1 = conversion factor acre-feet per cubic feet per second to hours

g. Estimation of Hydraulic Efficiency Correction Factor.

(1) Estimates of the field mean retention time for expected operational conditions are required for prediction of suspended solids concentrations in the effluent. Estimates of the retention time must consider the hydraulic efficiency of the disposal area, defined as the ratio of mean retention time to theoretical retention time, Field mean retention time T_d for given flow rate and ponding conditions and the theoretical residence time T are related by a hydraulic efficiency correction factor as follows:

$$T_d = \frac{T}{(HECF)} \tag{4-13}$$

where

T_d = mean residence time, hours

T = theoretical residence time, hours

HECF = hydraulic efficiency correction factor (HECF > 1.0) defined as the inverse of the hydraulic efficiency, T_d/T .

(2) The hydraulic efficiency correction factor HECF can be estimated by several methods. The most accurate estimate is that made from dye tracer studies to determine T_d at the actual site under operational conditions at a previous time, with the conditions similar to those for the operation under consideration (see Appendix J). This approach can be used only for existing sites.

(3) Alternatively, the ratio T_d/T = 1/HECF can be estimated from the equation:

$$\frac{T_d}{T} = 0.9 \left[1 - \exp\left(-0.3 \frac{L}{W}\right)\right] \tag{4-14}$$

where L/W is the length-to-width ratio of the proposed basin. The L/W ratio can be increased greatly by the use of internal spur dikes, resulting in a higher hydraulic efficiency and a lower required total area.

h. Determination of Disposal Area Geometry. Previous calculations have provided minimum required surface area for storage A_{ds} , a minimum required surface area for zone settling (if applicable) A_{dz} , and a minimum required surface area for flocculent settling A_{df} . A ponding depth H_{pd} was also assumed. These values are then used, as described in the following paragraphs, to determine the required disposal area geometry. Throughout the design process, the existing topography of the containment area site must be considered since it can have a significant effect on the resulting geometry of the containment area. Any limitations on dike height should also be determined based on an appropriate geotechnical evaluation of dike stability (See Chapter 6).

(1) Select the design surface area. Select the design surface area A_d as the largest of A_{ds}, A_{dz}, and/or A_{df}. If A_d exceeds the real estate available for disposal, consider a smaller flow rate (dredge size), deeper average ponding depth, chemical clarification, or an alternate site, and repeat the design. If the surface area for an existing site exceeds A_d, the existing surface area may be used for A_d.

(2) Compute height of the dredged material and dikes. The following procedure should be used:

(a) Estimate the thickness of the dredged material at the end of the disposal operation:

$$H_{dm} = \frac{V}{A_d} \tag{4-15}$$

where

H_{dm} = thickness of the dredged material layer at the end of the dredging operation, feet

V = volume of dredged material in the basin, cubic feet (from Equation 4-4)

A_d = design surface area, square feet (as determined above)

(b) Add the ponding depth and freeboard depth to H_{dm} to determine the required containment area depth (dike height):

$$H_{dk} = H_{dm} + H_{pd} + H_{fb} \tag{4-16}$$

where

H_{dk} = dike height, feet

H_{pd} = average ponding depth, feet (a minimum of 2 feet is recommended)

H_{fb} = freeboard above the basin water surface to prevent wave overtopping and subsequent damage to confining earth dikes, feet (a minimum of 2 feet is recommended)

4-4. Weir Design and Operation. The purpose of the weir structure is to regulate the release of ponded water from the containment area. Proper weir design and operation can control resuspension and withdrawal of settled solids.

a. Guidelines for Weir Design.

(1) Weir design and containment sizing. Weir design is based on providing the capability for selective withdrawal of the clarified upper layer of ponded water. The weir design guidelines as developed in the following paragraphs are based on the assumptions that the design of the containment area has provided sufficient area and volume for sedimentation and that short-circuiting is not excessive.

(2) Effective weir length and ponding depth.

(a) Ponding depth and effective weir length are the two most important parameters in weir design. The weir design guidelines presented in this section allow evaluation of the trade-off involved between these parameters.

(b) In order to maintain acceptable effluent quality, the upper layers containing low levels of suspended solids should be ponded at depths greater than or equal to the minimum depth of the withdrawal zone, which will prevent scouring settled material. The withdrawal zone is the area through which fluid is removed for discharge over the weir as shown in Figure 4-6. The size of the withdrawal zone affects the approach velocity of flow toward the weir and is generally equal to the depth of ponding.

(c) The weir shape or configuration affects the dimensions of the withdrawal zone and consequently the approach velocity. Since weirs do not extend across an entire side of the containment area, flow concentrations of varying degree occur near the weir, resulting in higher local velocities and possible resuspension of solids. Longer effective weir lengths result in less concentration of flow. The minimum width through which the flow must pass may be termed the effective weir length L_e .

(d) The relationship between effective weir length and ponding depth necessary to discharge a given flow without significantly entraining settled material is illustrated by the nomograph in Figure 4-7.

(3) Design procedure. To design a new weir to meet a given effluent suspended solids level, the following procedure should be used:

(a) Select the appropriate operating line in the lower portion of the nomograph based on the governing settling behavior of the dredged material slurry (zone or flocculent).

(b) Construct horizontal lines at the design inflow rate Q_i and the ponding depth expected at the weir as shown in the key in Figure 4-7. This ponding depth may be larger than the average ponding depth for large containment areas as the result of a slope taken by the settling material. The ponding depth at the weir may be estimated by using the following equation:

$$H_{pd(weir)} = H_{pd} + 1/2 \text{ Lps } (0.001) \tag{4-17}$$

where

$H_{pd(weir)}$ = estimated ponding depth at the weir, feet

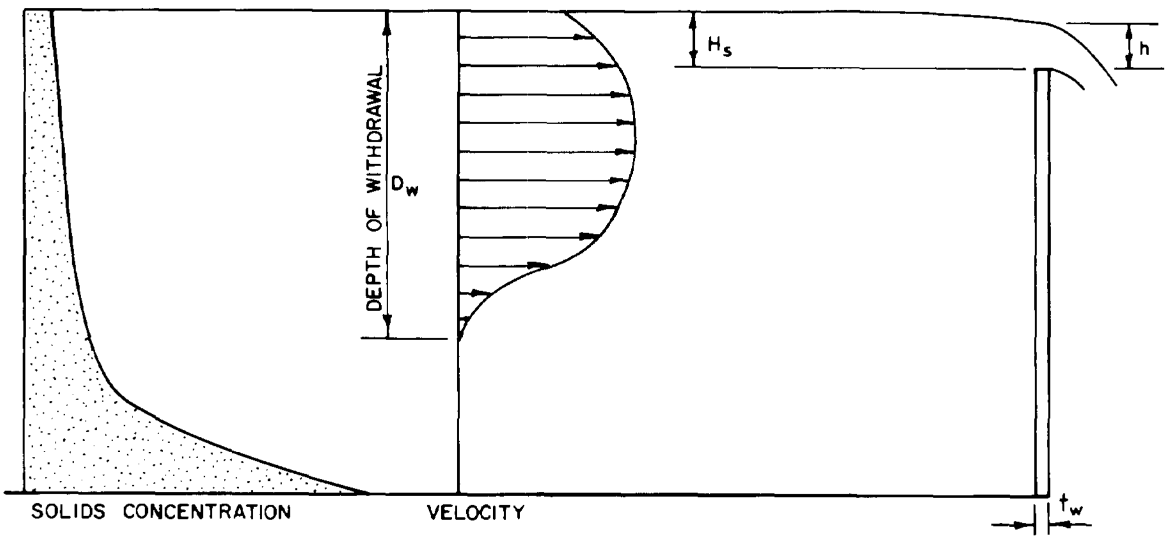


Figure 4-6. Conceptual illustration of withdrawal depth and velocity profile

H_{pd} = average ponding depth, feet

L_{ps} = length of ponded surface between inflow point and weir, feet

(c) Construct a vertical line from the point of intersection of the horizontal ponding depth line and the selected operating line of the nomogram. The required effective weir length is found at the intersection of the vertical line and the horizontal design flow line. An example is shown in the key in Figure 4-7.

(d) Determine the number of weir structures, the physical dimensions of each, and the locations, based on the weir type to be used and the configuration of the containment area. If a satisfactory balance between effective weir length and ponding depth cannot be achieved, intermittent operation or use of a smaller dredge may be required to prevent resuspension at the weir as the containment area is filled. An illustrative problem is given in Appendix C.

(4) Effect of weir type.

(a) Rectangular weirs. Rectangular weirs are the commonly used weir type and may consist of a rectangular wood- or metal-framed inlet(s) or half-cylindrical corrugated metal pipe riser(s). The effective weir length is equal to the actual weir crest length for rectangular weirs as illustrated in Figure 4-8a.

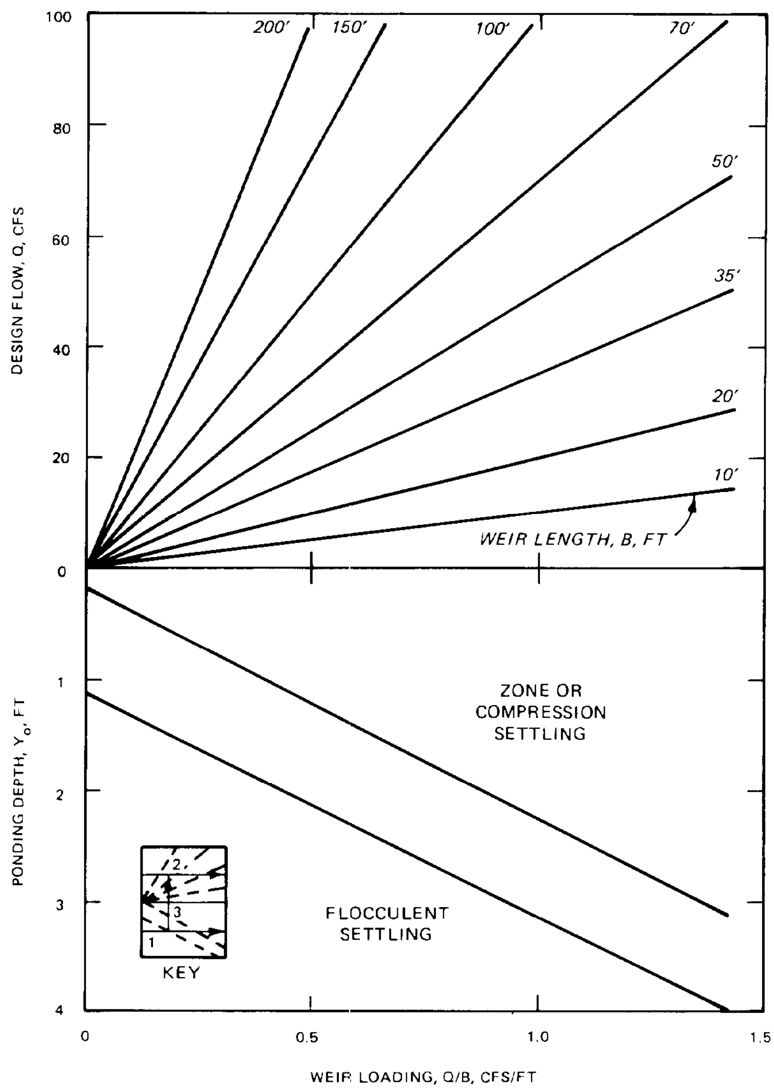


Figure 4-7. Weir design nomograph

(b) Jutting weirs. A modified form of the rectangular weir is the jutting weir (see Figure 4-8b). It is possible to achieve a greater effective weir length using a jutting weir since the effective length L_e equals L + 2J as shown in Figure 4-8b.

(c) Polygonal (labyrinth) weirs. Polygonal (labyrinth) weirs have been used to reduce the depth of flow over the weir. However, use of such weirs has little impact on effluent suspended solids concentrations since the controlling factor for the depth of withdrawal is usually not the flow over the weir but the approach velocity. Therefore, the approach velocity and the withdrawal depth for the rectangular weir in Figure 4-8a would be the same as that for the polygonal weir in Figure 4-8c since both weirs have the same effective length L_e , even though the total weir crest length for the polygonal weir is considerably greater. Use of polygonal weirs is not recommended

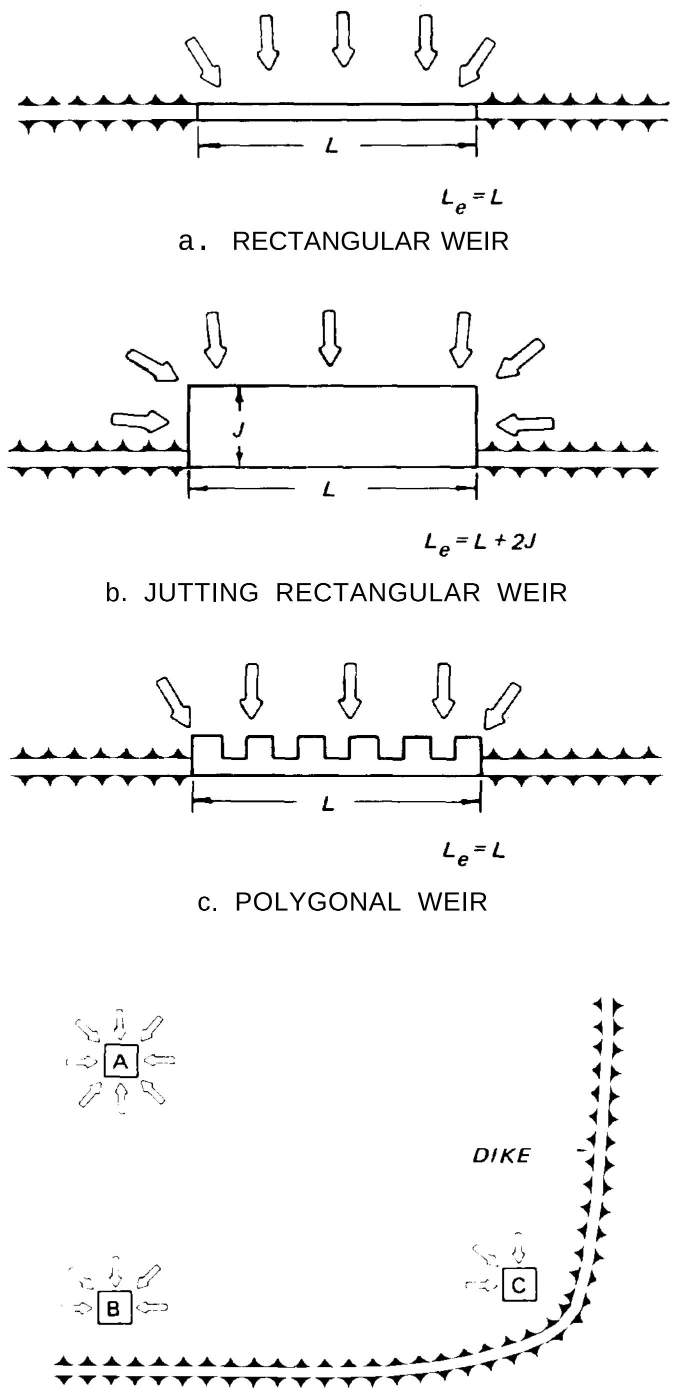


Figure 4-8. Effective lengths of various weir types

because of the greater cost and the marginal improvement of effluent quality realized when using such a weir.

(d) Shaft-type weirs. In some cases, the outflow structure is a four-sided drop inlet or shaft located within the containment area as shown in Figure 4-8d. In evaluating the effective weir length for shaft-type weirs, the approach velocity is a key consideration. To minimize the approach velocity and hence the withdrawal depth, the shaft weir should not be placed too near the dike. In Figure 4-8d, location A is the most desirable since flow can approach from all sides (four effective sides). Location B is less desirable since flow can approach from only three directions (three effective sides). Location C is the least desirable since it has only two effective sides. Since effluent pipes must run from the shaft weir under the dike to the receiving stream, a location such as A in Figure 4-8 may not be optimal since it is far from the dike and will require a longer pipe than Location B.

(e) Converting weir length. To convert the weir length determined from the design nomographs to length L_s of a side of the square shaft weir, use the following formula:

$$L_s = \frac{L_e}{n} \tag{4-18}$$

where n is the number of effective sides of a shaft-type weir. A side is considered effective if it is at least 1.5 L_s feet away from the nearest dike, mounded area, or other dead zone. This distance is generally accepted as being sufficient to prevent the flow restriction caused by the flow contraction and bending due to the walls.

(5) Structural design. Weirs should be structurally designed to withstand anticipated loadings at maximum ponding elevations. Considerations should be given to uplift forces, potential settlement, access, corrosion protection, and potential piping beneath or around the weir. Additional information regarding structural design of weirs is found in WES TR D-77-9 (item 16). Outlet pipes for the weir structure must be designed to carry flows in excess of the flow rate for the largest dredge size expected. Larger flow capacity of the outlet pipes may be needed if an emergency release of ponded water is required.

b. Weir Operation.

(1) Weir boarding.

(a) Adequate ponding depth during the dredging operation is maintained by controlling the weir crest elevation. Weir crest elevations are usually controlled by placing boards within the weir structure. The board heights should range in size from 2 to 10 inches, and thickness should be sufficient to avoid excessive bending as the result of the pressure of the ponded water.

(b) Weir boarding should be determined based on the desired ponding elevation as the dredging operation progresses. Small boards (e.g., 2 inches) should be placed at the top of the weir in order to provide more flexibility in controlling ponding depth. Use of larger boards in this most critical area may result in increased effluent suspended solids concentrations as weir boards are manipulated during the operation. Figure 4-9 shows the recommended weir boarding used for a minimum ponding depth of 2 feet.

(2) Operational guidelines for weirs. Some basic guidelines for weir operation are given below:

(a) If the weir and the disposal site are properly designed, intermittent dredging operation should not become necessary unless the required ponding depth cannot be maintained.

(b) While the weir is in operation, floating debris should be periodically removed from the front of the weir to prevent larger withdrawal flows at greater depths.

(c) If multiple weirs or a weir with several sections is used in a basin, the crests of all weirs or weir sections should be maintained at equal elevations, in order to prevent local high velocities and resuspension in front of the weir with lower elevation.

(d) If the effluent suspended solids concentration increases above acceptable limits, the ponding depth should be increased by raising the elevation of the weir crest. However, if the weir crest is at the maximum ponding elevation and the effluent quality is still unacceptable, the flow into the basin should be decreased by operating intermittently.

(e) The weir may be controlled in the field by using the head over the weir as an operational parameter since the actual volumetric flow over the weir cannot easily be measured.

(3) Operating head. The static head with the related depth of flow over the weir is the best criterion now available for controlling weir operation in the field. Weirs utilized in containment areas can usually be considered sharp crested where the weir crest thickness t_w is less than two-thirds the depth of flow over the weir h as seen in Figure 4-6. The ratio of depth of flow over the weir to the static head h/H_s equals 0.85 for rectangular sharp-crested weirs. Other values for the ratio of depth of flow to static head for various weir configurations may be found in the Handbook of Applied Hydrology (item 7). The weir crest length L , static head H_s , and depth of flow over the weir h are related by the following equations for rectangular sharp-crested weirs:

$$H_s = \left[0.3\ \frac{Q}{L}\right]^{2/3} \tag{4-19}$$

and

$$h = 0.85H_s \tag{4-20}$$

where

H_S = static head above the weir crest, feet

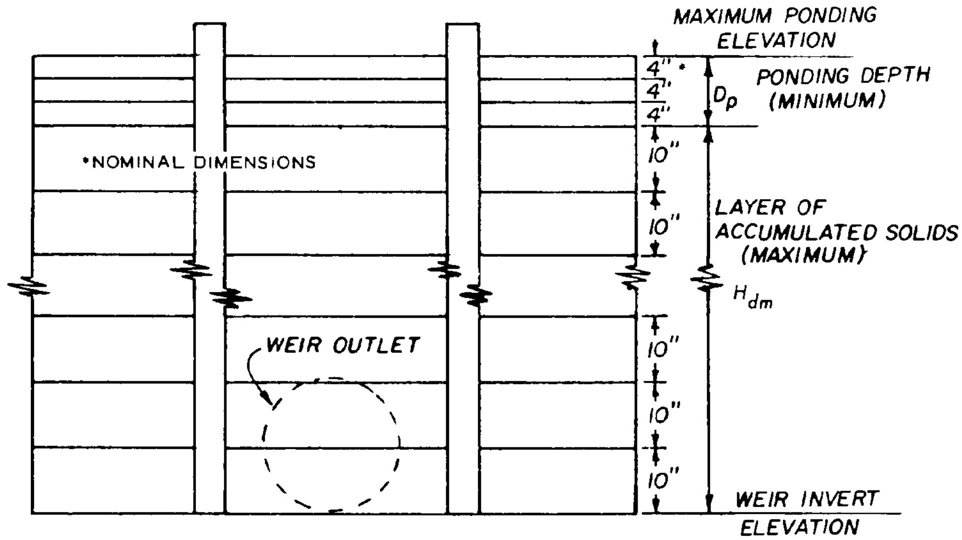


Figure 4-9. Recommended boarding configuration

Q = flow rate, cubic feet per second ($Q = Q_i = Q_e$ for continuous operation)

Q_e = clarified effluent rate, cubic feet per second

L = weir crest length, feet

h = depth of flow over the weir crest, feet

These relationships are shown graphically in Figure 4-10. If a given flow rate is to be maintained, Figure 4-10 can be used to determine the corresponding head and depth of flow. If the head in the basin exceeds this value, additional weir boards can be added, or the dredge can be operated intermittently until sufficient water is discharged to lower the head to an acceptable level. Since the depth of flow over the weir is directly proportional to the static head, it may be used as an operating parameter. The operator need not be concerned with head over the weir if effluent suspended solids concentrations are acceptable.

(4) Weir operation for undersized basins. If the basin is undersized and/or inefficient settling is occurring in the basin, added residence time and reduced approach velocities are needed to achieve efficient settling and to avoid resuspension, respectively. Added residence time can be obtained by raising the weir crest to its highest elevation to maximize the ponding depth or by operating the dredge intermittently. The residence time with intermittent dredging can be controlled by maintaining a maximum allowable static head or depth of flow over the weir based on the effluent quality achieved at various weir crest elevations.

(5) Weir operation for decanting, Once the dredging operation is completed, the ponded water must be removed to promote drying and consolidation of dredged material. Weir boards should be removed one row at a time to slowly decant the ponded water. Preferably, 2- by 4-inch boards should be located as described in previous paragraphs in order to minimize the withdrawal of settled solids. A row of boards should not be removed until the

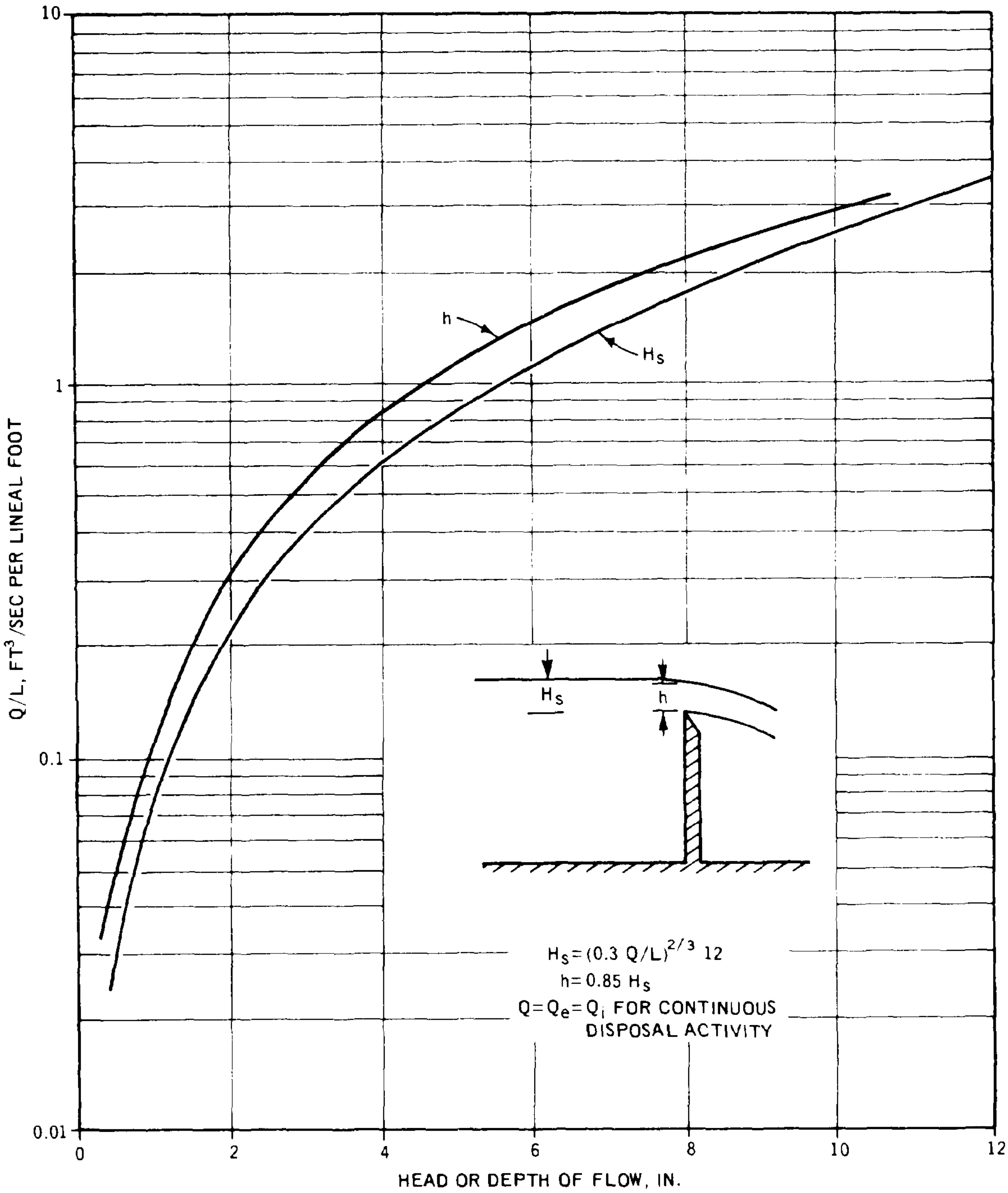


Figure 4-10. Relationship of flow rate, weir length, and head

water level is drawn down close to the weir crest and the outflow is low. This process should be continued until the decanting is completed. It is desirable to eventually remove the boards below the dredged material surface so that rainwater can drain from the area. These boards can be removed only after the material has consolidated sufficiently so that it will not flow from the basin. If it begins to do so, the boards should be replaced. In the final stages of decanting ponded water, notched boards may be placed in the weir, allowing low flow for slow removal of surface water.

4-5. Design of Chemical Clarification Systems. Pipeline injections of chemicals for clarification into the dredge inflow pipeline have shown only limited effectiveness and require much higher dosages of chemicals, This section therefore presents only the design procedures for chemical clarification of primary containment area effluents. The design is composed of three subsystems: the polymer feed system including storage, dilution, and injection; the weir and discharge culvert for mixing; and the secondary basin for settling and storage. The treatment system should be designed to minimize equipment needs and to simplify operation. Detailed procedures and examples are presented in Appendix G.

CHAPTER 5

LONG-TERM STORAGE CAPACITY OF CONTAINMENT AREAS

5-1. Factors Affecting Long-Term Storage Capacity.

a. General.

(1) In order that the maximum benefits can be derived from areas constructed for the confined disposal of dredged material, the design and operation plan must accurately account for the long-term increase in storage capacity in containment areas resulting from decreases in the height of dredged fill deposited. The height of the dredged fill decreases by three natural processes: sedimentation, consolidation, and desiccation. Sedimentation is a relatively short-term process, whereas consolidation and desiccation are long-term processes. Design of containment areas for effective sedimentation was discussed in Chapter 4. This chapter presents guidelines for estimating long-term containment area storage capacity, considering both dredged material consolidation and dewatering (evaporative drying). The storage capacity is defined as the total volume available to hold additional dredged material and is equal to the total unoccupied volume minus the volume associated with ponding requirements and freeboard requirements. The estimation of long-term storage capacity is an important consideration for long-term planning and design of new containment areas or evaluation of the remaining service life of existing sites.

(2) After dredged material is placed within a containment area, it undergoes sedimentation and self-weight consolidation, resulting in gains in storage capacity. The placement of dredged material also imposes a loading on the containment area foundation; therefore, additional settlement may result from consolidation of compressible foundation soils. Settlement due to consolidation is therefore a major factor in the estimation of long-term storage capacity. Since the consolidation process is slow, especially in the case of fine-grained materials, it is likely that total settlement will not have taken place before the containment area is required for additional placement of dredged material. Settlement of the containing dikes may also significantly affect the available storage capacity and should be considered. Once a given active dredging operation ends, the ponded surface water required for settling is decanted, exposing the dredged material surface to desiccation (evaporative drying). This process can further add to long-term storage capacity and is a time-dependent and climate-dependent process. Active dewatering operations such as surface trenching can speed the natural dewatering process. A conceptual diagram illustrating these processes is shown in Figure 5-1.

(3) Guidelines for estimation of gains in long-term capacity due to settlement within the containment area are based on the fundamental principles of consolidation theory modified to consider the self-weight consolidation behavior of newly placed dredged material. The guidelines are presented in the following paragraphs; illustrative examples are found in Appendix F.

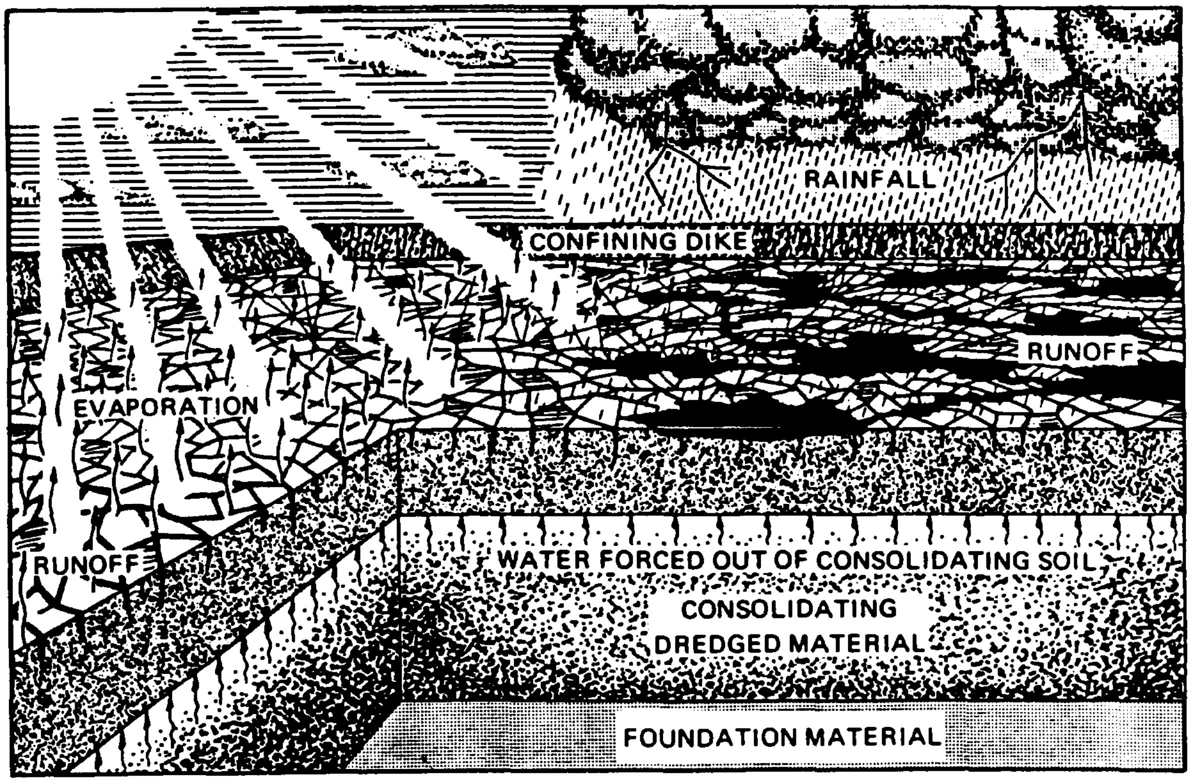


Figure 5-1. Conceptual diagram of dredged material consolidation and dewatering processes.

b. Dredged Material Consolidation. Three types of consolidation may occur in dredged material containment areas. These are primary consolidation, secondary consolidation, and consolidation resulting from desiccation.

(1) Primary consolidation. The Terzaghi standard theory of one-dimensional consolidation has received widespread use among geotechnical engineers and continues to be the first choice for estimation of settlements. The Terzaghi or "small strain theory" has received widespread application for consolidation problems in which the magnitude of settlement is small in comparison to the thickness of the consolidating layer. In contrast to the small strain theory, a "finite strain theory" for one-dimensional consolidation is better suited for describing the large settlements common to the primary consolidation of soft fine-grained dredged material. Calculation techniques are discussed in 5-2.

(2) Secondary consolidation. The process of secondary consolidation or "creep" refers to the rearrangement of soil grains under load following completion of primary consolidation. This process is not normally considered in settlement analyses and is not considered in this manual.

(3) Desiccation consolidation. There are basically two phenomena that control the amount of consolidation caused by desiccation of fine-grained dredged material. The first is the evaporation of water from the upper

sections of the dredged material. The resulting reduction in its moisture content causes a reduction in void ratio or volume occupied due to the negative pore water pressure induced by the drying. This can be referred to as the dewatering process and is discussed in 5-1.c.

(4) Consolidation in underlying material. An additional process influencing settlement involves the primary consolidation in underlying material when the free water surface is lowered. As the water surface moves downward, the unit weight acting on lower material changes from buoyant unit weight to effective unit weight. The material below the new water level is therefore subjected to an additional surcharge.

c. Dredged Material Dewatering Processes.

(1) General process description.

(a) Desiccation of dredged material is basically removal of water by evaporation and transpiration. In this report, plant transpiration is considered insignificant due to the recurrent deposition of dredged fill and is therefore disregarded. Evaporation is mainly controlled by such variables as radiation heating from the sun, convective heating from the earth, air temperature, ground temperature, relative humidity, and wind speed.

(b) However, other factors must also be taken into account. For instance, the evaporation efficiency is normally not a constant but some function of depth to which the layer has been desiccated and also is dependent on the amount of water available for evaporation.

(c) It is practical to make desiccation calculations on a monthly basis because of the availability of long-term monthly average rainfall and pan evaporation data. Rainfall and pan evaporation data have been tabulated and published in climatic summaries by the US Weather Bureau for many areas of this country. Tables of average monthly rainfall for select stations are available from the National Oceanic and Atmospheric Administration (NOAA) (item 25). Maps of monthly pan evaporation are presented in Appendix H. In the absence of more site-specific data, these sources can be used for specification of climatic data.

(2) Evaporative stages.

(a) Evaporative drying of dredged material leading to the formation of a desiccated crust is a two-stage process. The removal of water occurs at differing rates during the two stages as shown in Figure 5-2. The first stage begins when all free water has been decanted or drained from the dredged material surface. The void ratio at this point e_{oo} corresponds to zero-effective stress as determined by laboratory sedimentation and consolidation testing. This initial void ratio has been empirically determined to be at a water content of approximately 2.5 times the Atterberg liquid limits (LL) of the material.

(b) First-stage drying ends and second stage begins at a void ratio that may be called the "decant point or saturation limit" e_{SL} . The e_{SL} of

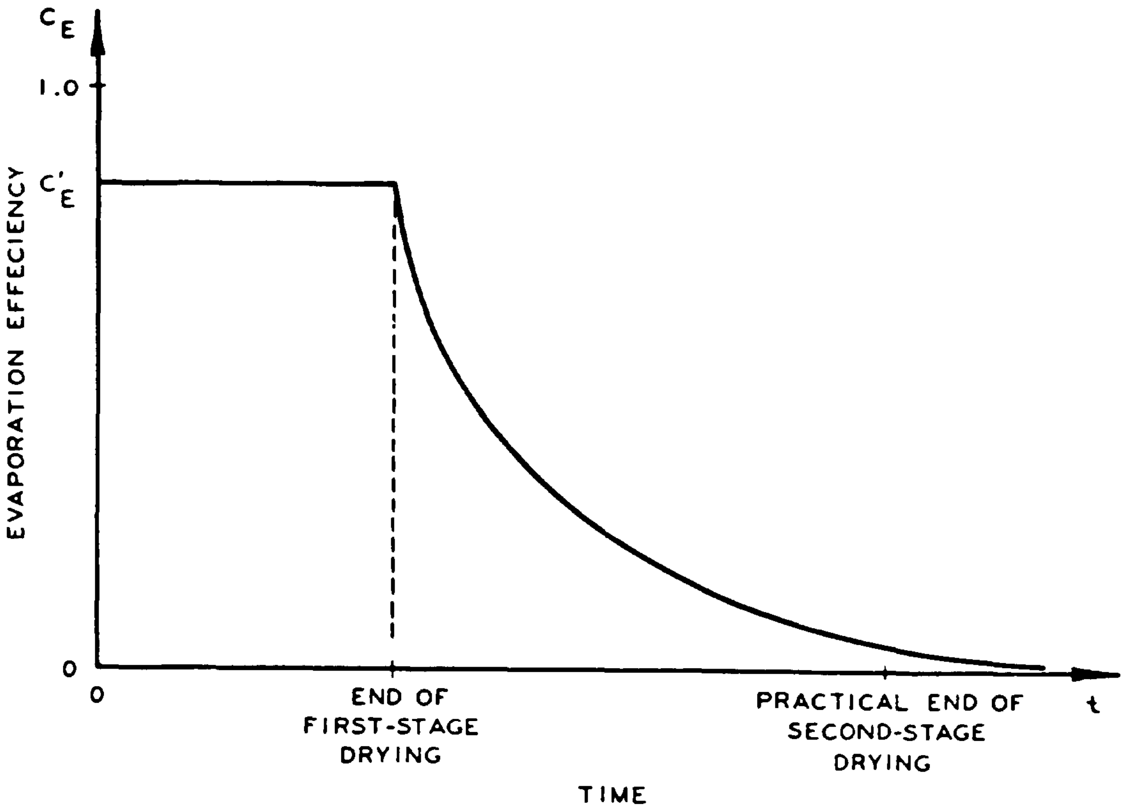


Figure 5-2. Dredged material evaporative efficiency as a function of time

typical dredged material has been empirically determined to be at a water content of approximately 1.8 LL.

(c) Second-stage drying will be an effective process until the material reaches a void ratio that may be called the "desiccation limit" or e_{DL} . When the e_{DL} reaches a limiting depth, evaporation of additional water from the dredged material will effectively cease. Any additional evaporation will be limited to excess moisture from undrained rainfall and that water forced out of the material as a result of consolidation of material below the crust. The e_{DL} of typical dredged material may roughly correspond to a water content of 1.2 plastic limit (PL). Also associated with the e_{DL} of a material is a particular percentage of saturation that probably varies from 100 percent to something slightly less, depending on the material.

5-2. Estimation of Long-Term Storage Capacity.

a. Data Requirements. The data required to estimate long-term storage capacity include the consolidation and desiccation properties of the fine-grained dredged material, the consolidation properties of compressible foundation soils, and project data. Any system of units is permissible as long as the dimensions are consistent. For example, if layer thickness is in feet and time is in days, then permeability must be in feet per day. The data required are as follows:

(1) Compressible foundation characteristics.

(a) Specific gravity of the soil solids.

(b) Initial thickness of the compressible foundation.

(c) Relationship between the void ratio and the effective stress.

(d) Relationship between the void ratio and the permeability.

(2) Fine-grained dredged material characteristics.

(a) Specific gravity of the soil solids.

(b) Thickness of the initial material deposit.

(c) Initial void ratio of the deposit.

(d) Unit weight of water.

(e) Relationship between the void ratio and the effective stress.

(f) Relationship between the void ratio and the permeability.

(g) Filling sequence, including average thickness of deposit, time of disposal, and estimated time until the material is exposed to evaporative drying.

(h) Elevation of a permanent water table.

(i) Void ratio corresponding to the end of maximum evaporation.

(j) Void ratio at the end of effective material drying.

(k) Efficiency of surface runoff drainage in the area.

(l) Monthly average pan evaporation values.

(m) Monthly average rainfall values.

(n) Average pan-to-field evaporation coefficient.

(o) Percent saturation of the material when dried to the desiccation limit, including the desiccation crack volume.

(p) Maximum thickness of the dried crust.

(3) Incompressible foundation characteristics.

(a) Void ratio at the upper surface.

(b) Permeability at the upper surface.

(c) Drainage path length.

(d) Elevation of the upper surface.

b. Storage Capacity-Time Relationship.

(1) The estimated time-settlements due to dredged material consolidation and dewatering and foundation consolidation may be combined to yield a time-total settlement relationship for a single lift as shown in Figure 5-3.

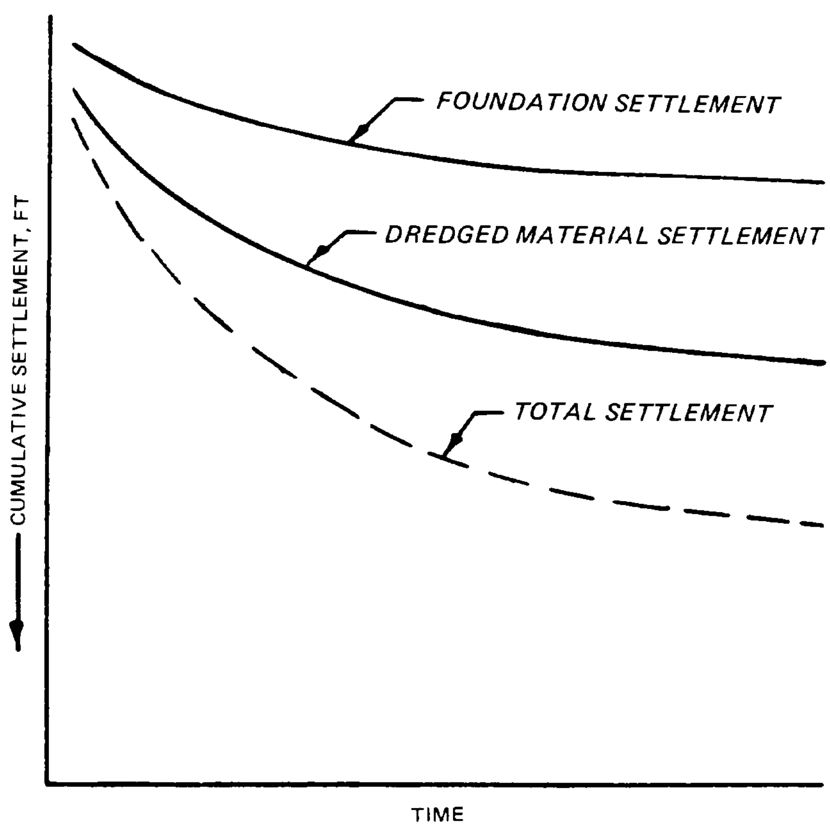


Figure 5-3. Illustrative time-consolidation relationships

These data are sufficient for estimation of the remaining capacity in the short term. However, if the containment area is to be used for long-term placement of subsequent lifts, a projected plot of dredged material surface height versus time should be developed. This plot can be developed using time-settlement relationships for sequential lifts combined as shown in

Figure 5-4. Such data may be used for preliminary estimates of the long-term service life of the containment area.

(2) The maximum dike height as determined by foundation conditions or other constraints and the containment surface area will dictate the maximum available storage volume. The increases in dredged material surface height during the dredging phases and the decreases during settlement phases correspond to respective decreases and increases in remaining containment storage capacity, shown in Figure 5-5. Projecting the relationships for surface height or for remaining capacity to the point of maximum allowable height or exhaustion of remaining capacity, respectively, will yield an estimate of the containment area service life. Gains in capacity due to anticipated dewatering or material removal should also be considered in making the projections.

(3) The complex nature of the consolidation and desiccation relationships for multiple lifts of compressible dredged material and the changing nature of the resulting loads imposed on compressible foundation soils may result in errors in projections of remaining storage capacity over long time periods. Accuracy can be greatly improved by updating the estimates every few years using data from newly collected samples and laboratory tests. Observed field behavior should also be routinely recorded and used to refine the projections.

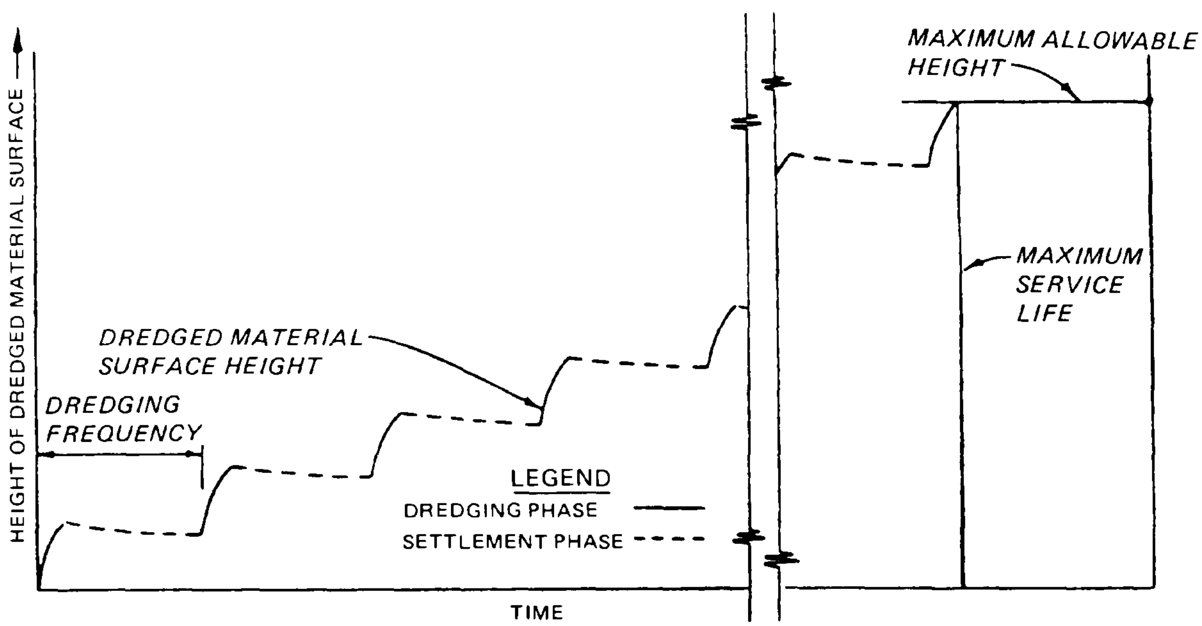


Figure 5-4. Projected surface height for determination of containment area service life

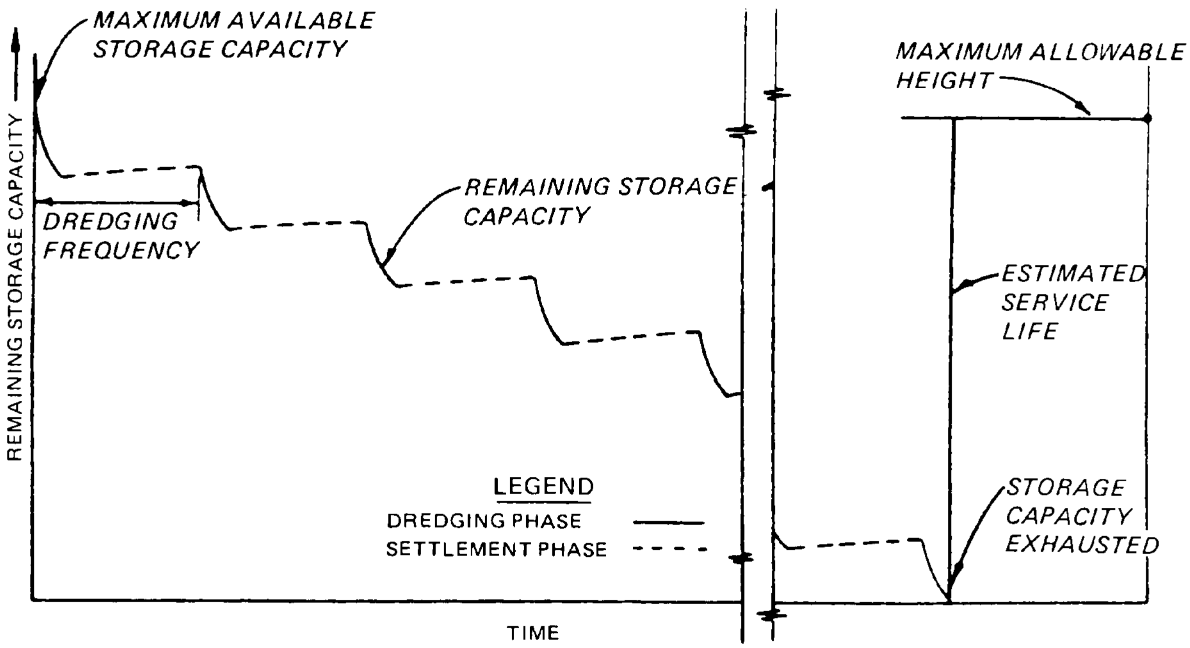


Figure 5-5. Projected storage capacity for determination of containment area service life

c. Overview of Estimation Techniques.

(1) Small strain versus finite strain consolidation.

(a) The most applicable procedure for estimating consolidation in soft dredged material is the finite strain consolidation theory. The magnitude of consolidation as determined by small strain techniques is equivalent to that determined by the finite strain technique. However, the time rate of consolidation is overly conservative for small strain in that the rate of consolidation as predicted is slow when compared to field behavior. Details on the theoretical background for the finite strain theory are given in WES TR D-83-1 and TR D-85-4 (items 5 and 6).

(b) The advantages of using the finite strain technique for the estimation of dredged material consolidation settlement are summarized in Table 5-1. The technique accounts for the nonlinearity of the void ratio, permeability, and coefficient of consolidation relationships that must be considered when large settlements of a layer are involved. Hand calculations using the finite strain approach have been developed and are presented in this manual. However, the technique is more easily applied using a computer program.

(2) Empirical methods for estimating desiccation behavior. Empirical equations for estimating the settlement of a dredged material layer due to desiccation and the thickness of dried crust were developed for the purpose of determining feasibility and benefits of active dewatering operations (item 14). The empirical relationships have been refined (Item 6) to consider the two stage process of desiccation and the overall water balance

Table 5-1

Comparison of Small Strain and Finite Strain Consolidation Techniques

Consideration	Finite strain	Small strain
Range of void ratios	Very large	Very small
Self-weight	Included	Not included
Void ratio/effective stress relationship	Nonlinear	Linear
Void ratio/permeability relationship	Variable	Constant

relationships that exist within a dredged material disposal area. The interaction of the desiccation process with dredged material consolidation due to self-weight has been incorporated in computer programs for estimating long-term storage capacity. The refined empirical relationships can be easily applied in determining the benefits of dewatering programs and provide increased accuracy in storage capacity evaluations.

(3) Hand calculation versus computer solution.

(a) The use of computer models can greatly facilitate the estimation of storage capacity for containment areas. Although the computations for simple cases can be easily and quickly done by hand, the analyses often require computations for a multiyear service life with variable disposal operations and possibly material removal or dewatering operations occurring intermittently throughout the service life. These complex computations can be done more efficiently using a computer model.

(b) The use of computer models holds added advantage when considering the additional settlements that occur as the result of dredged material desiccation (dewatering). The estimation of desiccation behavior can also be done by means of hand calculations; however, the interaction between desiccation and consolidation cannot be handled by direct hand computation, but instead would require cumbersome iterative calculations. A computer program is well suited to handle the calculations of both consolidation and desiccation and the interaction between the two processes.

(c) Methods of hand calculation for finite strain consolidation and desiccation are presented in Appendix F. These calculations are manageable for estimation of settlements in one dredged material layer. However, if storage capacity estimates must be made for multiple disposal operations, the use of computer programs is recommended.

d. Computer Solutions for Consolidation and Desiccation.

(1) The recommended computer model for use in predicting the long term capacity of disposal areas is documented in the Automated Dredging and Disposal Alternatives Management System (ADDAMS) described in Chapter 8 of this manual. The program is entitled "Primary Consolidation and Desiccation of Dredged Fill (PCDDF)" and incorporates the concepts described in this chapter. Theoretical documentation, description of solution techniques, and a user guide are available (item 6). An expanded user guide with plotting routines for results is found in the ADDAMS instruction report (item 19).

(2) Examples of the results obtained using the PCDDF model are shown in Figures 5-6 and 5-7. These are plots of dredged material surface elevation versus time for several cases including multiple layers deposited at varying times. Field data collected at the respective sites are also shown for comparison.

5-3. Dredged Material Dewatering Operations.

a. General.

(1) Surface trenching for improved drainage and use of underdrains are the only technically feasible and economically justifiable dewatering techniques for dredged material containment areas. The use of underdrains has been successfully applied on a small scale; however, their use in large disposal areas has not been proven economical as compared with surface drainage techniques. Accordingly, this section describes only techniques recommended for improvement of surface drainage through trenching. Guidance for application of underdrains is found in WES Technical Report DS-78-11 (item 14).

(2) Four major reasons exist for dewatering fine-grained dredged material placed in confined disposal areas:

(a) Promotion of shrinkage and consolidation, leading to creation of more volume in the existing disposal site for additional dredged material.

(b) Reclamation of the dredged material into more stable soil form for removal and use in dike raising, other engineered construction, or other productive uses, again creating more available volume in the existing disposal site.

(c) Creation of stable fast land at a known final elevation and with predictable geotechnical properties.

(d) Benefits for control of mosquito breeding.

b. Conceptual Basis for Dewatering by Progressive Trenching. The following mechanisms were found to control evaporative dewatering of fine-grained dredged material placed in confined disposal areas:

(1) Establishment of good surface drainage allows evaporative forces to dry the dredged material from the surface downward, even at disposal area locations where precipitation exceeds evaporation (negative net evaporation).

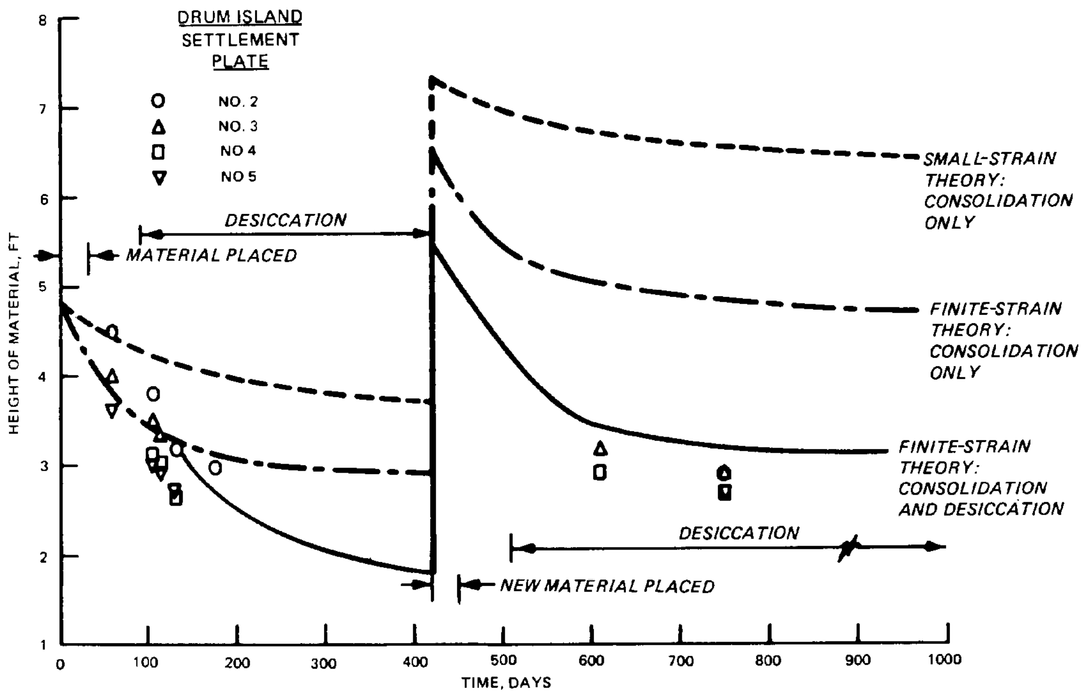


Figure 5-6. Measured and predicted material heights at Drum Island

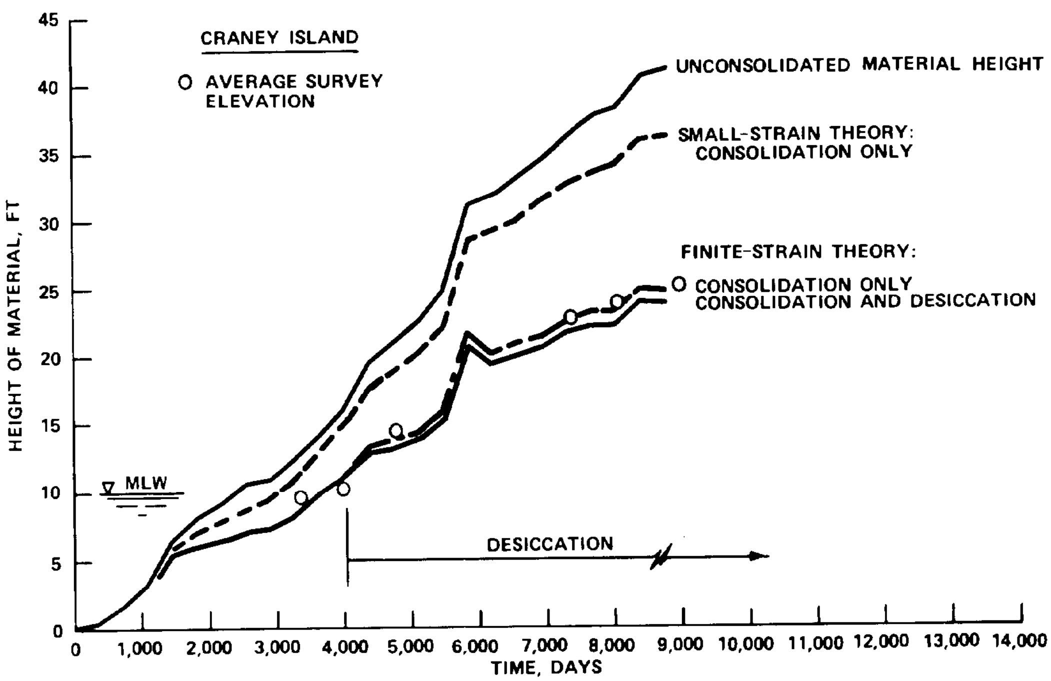


Figure 5-7. Measured and predicted material heights at Craney Island

(2) The most practical mechanism for precipitation removal is by runoff through crust desiccation cracks to surface drainage trenches and off the site through outlet weirs.

(3) To maintain effective drainage, the flow-line elevation of any surface drainage trench must always be lower than the base of crust desiccation cracks; otherwise, ponding will occur in the cracks. As drying occurs, the cracks will become progressively deeper.

(4) Below the desiccation crust, the fine-grained subcrust material may be expected to exist at water contents at or above the liquid limit (LL). Thus, it will be difficult to physically construct trenches much deeper than the bottom of the adjacent desiccation crust.

(5) To promote continuing surface drainage as drying occurs, it is necessary to progressively deepen site drainage trenches as the water table falls and the surface crust becomes thicker; thus, the name "progressive trenching" was developed for the concept.

(6) During conduct of a progressive trenching program, the elevation difference between the internal water table and the flow line of any drainage trench will be relatively small. When the relatively low permeability of fine-grained dredged material is combined with the small hydraulic gradient likely under these circumstances, it appears doubtful that appreciable water can be drained from the dredged material by gravity seepage. Thus, criteria for trench location and spacing should be based on site topography so that precipitation is rapidly removed and ponding is prevented, rather than to achieve marked drawdown from seepage.

c. Effects of Dewatering. The net observable effects of implementing any program of dewatering by improved surface drainage will be as follows:

(1) Disappearance of ponded surface water.

(2) Runoff of the majority of precipitation from the site within a few hours.

(3) Gradual drying of the dredged material to more stable soil form.

(4) Vertical settlement of the surface of the disposal area.

(5) Ability to work within the disposal area with conventional equipment.

d. Initial Dewatering (Passive Phase).

(1) Once the disposal operation is completed, dredged material usually undergoes hindered sedimentation and self-weight consolidation (called the "decant phase"), and water will be brought to the surface of the consolidating material at a faster rate than can normally be evaporated, During this phase, it is extremely important that continued drainage of decant water and/or precipitation through outlet weirs be facilitated. Weir flow-line elevations may have to be lowered periodically as the surface of the newly placed dredged

material subsides. Guidelines for appropriate disposal site operation during this passive dewatering phase, to maximize decant and precipitation water release while maintaining appropriate water quality standards, are described in Chapter 7.

(2) Once the fine-grained dredged material approaches the decant point water content, or saturation limit as described previously, the rate at which water is brought to the surface will gradually drop below the climatic evaporative demand. If precipitation runoff through site outflow weirs is facilitated, a thin drying crust or skin will form on the newly deposited dredged material. The thin skin may be only several hundredths of a foot thick, but its presence may be observed by noting small desiccation cracks that begin to form at 3- to 6-foot intervals, as shown in Figure 5-8. Once the dredged material has reached this consistency, active dewatering operations may be initiated.

e. Dewatering by Progressive Trenching.

(1) Three procedures have been found viable to initiate active dredged material dewatering by improved surface drainage, once the material has achieved consistency conditions shown in Figure 5-8: periodic perimeter

Figure 5-8. Surface of fine-grained dredged material at the earliest time when surface trenching should be attempted; initial cracks are spaced at 3- to 6-foot intervals, and the surface water content approaches 1.8 x LL

trenching by dragline, with draglines working initially from perimeter dikes and subsequently from berms established inside the perimeter dikes; periodic interior site trenching; or a combination of these two methods. This section presents information necessary to properly conduct dewatering operations by these procedures. Only the last two procedures will result in total site dewatering at the maximum rates. The first procedure would have, in many instances, an effective interior dewatering rate considerably less than the predicted maximum rate, though the exact lower rate would be highly site-specific.

(2) Perimeter dragline trenching operations.

(a) Construction of trenches around the inside perimeter of confined disposal sites is a procedure that has been used for many years to dewater and/or reclaim fine-grained dredged material. In many instances, the purpose of dewatering has been to obtain convenient borrow for use in perimeter dike raising activities. Draglines and backhoes have been found to be adaptable to certain activities because of their relatively long boom length and/or method of operation and control. The perimeter trenching scheme should be planned carefully so as not to interfere with operations necessary for later dewatering or other management activities.

(b) When initiating dragline trenching operations, the largest size, longest boom length dragline that can be transported efficiently to the disposal site and can operate efficiently on top of disposal site dikes should be obtained. Operations should begin at an outflow weir location, where the dragline, operating from the perimeter dike, should dig a sump around the weir extending into the disposal area to maximum boom and bucket reach. The very wet excavated material is cast against the interior side of the adjacent perimeter dike. It may be necessary to board up the weir to prevent the very wet dredged material from falling into the weir box during the sump-digging operation. A localized low spot some 1 or 2 inches in elevation below the surrounding dredged material can be formed. Once the sump has been completed, weir boards should be removed to the level of the dredged material, and, if necessary, handwork should be conducted to ensure that any water flowing into the sump depression will exit through the outflow weir.

(c) Once the sump has been completed, the dragline should operate along the perimeter dike, casting its bucket the maximum practicable distance into the disposal area, dragging material back in a wide shallow arc to be cast on the inside of the perimeter dike. A wide shallow depression 1 to 2 inches lower than the surrounding dredged material will be formed. The cast material will stand on only an extremely shallow (1 vertical on 10 horizontal or less) slope. A small dragline should be able to accomplish between 200 and 400 linear feet of trenching per working day.

(d) Dredged material near the ditch edge will tend to dry slightly faster than material located farther out in the disposal site, with resulting dredged material shrinkage giving a slight elevation gradient from the site interior toward the perimeter trenches, also facilitating drainage (Figure 5-9). In addition, desiccation crack formation will be more pronounced near the drainage trenches, facilitating precipitation runoff through the cracks to the perimeter trenches.

Figure 5-9. Shallow initial perimeter trench constructed by dragline operating from perimeter dike

(e) Once appreciable desiccation drying has occurred in the dredged material adjacent to the perimeter trench and the material cast on the interior slope of the perimeter dike has dried, the perimeter trenches and weir sumps should be deepened. The exact time between initial and secondary trench deepening will vary according to the engineering properties of the dredged material and existing climatological conditions, ranging from 2 or 3 weeks during hot, dry summer months up to 8 or 10 weeks in colder, wetter portions of the year. Inspection of the existing trenches is the most reliable guideline for initiating new trench work, since desiccation cracks 1 or 2 inches deep should be observed in the bottom of existing trenches before additional trenching is begun. Depending on the size of the disposal area, relative costs of mobilization and demobilization of dragline equipment, and the relative priority and/or need for dewatering, it may prove convenient to employ one or more draglines continuously over an interval of several months to periodically work the site. A second trenching cycle should be started upon completion of an initial cycle, a third cycle upon completion of the second cycle, etc., as needed.

(f) During the second trenching, wide shallow trenches with a maximum depth of 2 to 6 inches below the surface of adjacent dredged material can be constructed, and sumps can be dug to approximately 8 to 12 inches below surrounding dredged material. These deeper trenches will again facilitate more rapid dewatering of dredged material adjacent to their edges, with resulting shrinkage and deeper desiccation cracks providing a still steeper drainage flow gradient from the site Interior to the perimeter trenches.

(g) After two or perhaps three complete periodic perimeter dragline trenching cycles, the next phase of the trenching operation may be initiated. In this phase, the dragline takes the now dry material placed on the interior of the perimeter dike and spreads it to form a low berm adjacent to the dike inside the disposal area. The dragline then moves onto this berm, using single or double mats if required and using the increased digging reach now available, and widens and extends the ditch into the disposal site interior, as shown in Figure 5-10. The interior side of the ditch is composed of material previously dried, and a ditch 12 to 18 inches deep may be constructed, as shown in Figure 5-11. Material excavated from this trench is again cast on the interior slope of the perimeter dike to dry and be used either for raising the perimeter dike or for subsequent berming farther into the disposal area.

(h) After two or more additional periodic trench deepenings, working from the berm inside the disposal area, trenches up to 3 to 5 feet deep may be completed. Trenches of this depth will cause accelerated drying of the dredged material adjacent to the trench and produce desiccation cracks extending almost the entire thickness of the adjacent dredged material, as shown in Figure 5-12. A well-developed perimeter trench network leading to outflow weirs is now possible, as shown in Figure 5-13, and precipitation runoff is facilitated through gradual development of a network of desiccation cracks which extend from the perimeter trenches to the interior of the site.

(i) Once a perimeter trench system such as that shown in Figure 5-13 is established, progressive deepening operations should be conducted at less frequent intervals, and major activity should be changed from deepening perimeter trenches and weir sumps to that of continued inspection to make sure that the ditches and sumps remain open and facilitate free drainage. As a desiccation crack network develops with the cracks becoming wider and deeper, precipitation runoff rate will be increased and precipitation ponding in the site interior will be reduced. As such ponding is reduced, more and more evaporative drying will occur, and the desiccation crack network will propagate toward the

Figure 5-10. Small dragline on mats working on berm deepens shallow perimeter drainage trench

Figure 5-11. Construction of ditch 12 to 18 inches deep with excavated material cast on interior slope of perimeter dike

Figure 5-12. Desiccation crust adjacent to perimeter 3- to 5-foot-deep drainage trench

Figure 5-13. A well-developed perimeter trenching system, Morris Island Disposal Site, Charleston District

disposal area interior. Figure 5-13 is a view of the 500-acre Morris Island Disposal Site of the Charleston District, where a 3-foot lift of dredged material was dewatered down to approximately a 1.7-foot thickness at the perimeter over a 12-month period by an aggressive program, undertaken by the District, of site drainage improvement with dragline perimeter trenching. Figure 5-14 shows the 12-inch desiccation crust achieved at a location approximately 200 yards from the disposal area perimeter. The dredged material was a CH clay with an LL over 100. However, despite the marked success with perimeter trenching, a close inspection of Figure 5-13 shows that ponded water still exists in the site interior.

(3) Interior trenching.

(a) Riverine utility craft. The high water content of dredged material during the initial dewatering stages requires the use of some type of amphibious or low-ground-pressure equipment for construction of trenches in the site interior. The Riverine Utility Craft (RUC), an amphibious vehicle using twin screws for propulsion and flotation, can successfully construct shallow trenches in fine-grained dredged material shortly after formation of a thin surface crust. It can also be effective in working with other equipment in constructing sump areas around outflow weirs for collection of surface water. The RUC was initially developed in the 1960's as a reconnaissance vehicle for military applications and was used on an experimental basis for trenching operations. RUC vehicles have since been successfully applied in dewatering operations in the Mobile, Charleston, and Norfolk Districts for both trenching and surveying/sampling applications. Even though this vehicle is perhaps the only tool that can be used to construct shallow trenches in dredged material

Figure 5-14. Desiccation crust achieved in highly plastic clay dredged material 200 yards into disposal area by perimeter trenching over 12-month period

with little or no developed surface crust, its potential use in dewatering operations is limited. The RUC is susceptible to maintenance problems because of the nature of the drive train and frame, which were not designed for heavy use in trenching operations on a production basis. The nonavailability of RUC vehicles limits their potential widespread use for routine dewatering operations. Only two vehicles are available Corps-wide. Also, field experience has shown that the early stages of evaporative dewatering and crust development occur at acceptable rates considering only the natural drying processes, perhaps aided by perimeter trenching as described previously. Once a surface crust of 4 to 6 inches has developed, more productive trenching equipment as described in the following paragraphs can be used.

(b) Rotary trenchers. The use of trenching equipment with continuously operating rotary excavation devices and low-ground-pressure chassis is recommended for routine dewatering operations. This type of equipment has been used successfully in dewatering operations in the Savannah District and in the other numerous locations along the Atlantic Coast for mosquito control. The Charleston, Norfolk, and Philadelphia Districts have also used this equipment for dewatering operations. The major features of the equipment include a mechanical excavation implement with cutting wheel or wheels used to cut a trench up to 3 feet deep. The low-ground-pressure chassis may be tracked or rubber tired. The major advantage of rotary trenchers is their ability to continuously excavate while slowly moving within the containment area. This allows them to construct trenches in areas where the use of dragline or backhoe equipment would cause mobility problems. Photographs of tracked and rubber-tired trenchers are shown in Figures 5-15 and 5-16. The excavating wheels can be arranged in configurations that create hemispherical or trapazodial trench cross sections and can throw material to one or both sides of the trench. The material is spread in a thin layer by the throwing action, which allows it to dry quickly and prevents the creation of a windrow which might block drainage to the trench. Photographs of the excavating devices, ongoing trenching operations, and configuration of constructed trenches are shown in Figures 5-17 through 5-22. Based on past experience, an initial crust thickness of 4 to 6 inches is required for effective mobility of the equipment. This crust thickness can be easily formed within the first year of dewatering effort if surface water is effectively drained from the area, assisted by perimeter trenches constructed by draglines operating from the dikes. A suggested scheme for perimeter and interior trenching using a combination of draglines and a rotary trencher or other suitable equipment is shown in Figure 5-23.

(c) Trench spacing. The minimum number of trenches necessary to prevent precipitation ponding on the disposal area surface should be constructed. These trenches should extend directly to low spots containing ponded water. However, the greater the number of trenches per unit of disposal site area, the shorter the distance that precipitation runoff will have to drain through desiccation cracks before encountering a drainage trench. Thus, closely spaced trenches should produce more rapid precipitation runoff and may slightly increase the rate of evaporative dewatering. Conversely, the greater the number of trenches constructed per unit of disposal site area, the greater the cost of dewatering operations and the greater their impact on subsequent dike raising or other borrowing operations. However, the rotary trenchers have a relatively high operational speed, and it is therefore recommended that the maximum number of drainage trenches be placed consistent with the specific trenching plan selected. Trench spacings of 100 to 200 feet have normally been used. If topographic data are available for the disposal site interior, they may be used as the basis for preliminary planning of the trenching plan.

(d) Parallel trenching. The most common trench pattern would employ parallel trenching. A complete circuit of the disposal area with a perimeter trench is joined with parallel trenches cut back and forth across the disposal area, ending in the perimeter trench. Spacing between parallel trenches can be varied as described above. A parallel pattern is illustrated in Figure 5-22. A schematic of a parallel trenching pattern with radial combinations is shown in Figure 5-23.

(e) Radial trenching pattern. Small disposal areas or irregularly-shaped disposal areas may be well suited for a radial trenching pattern for effective drainage of water to the weir structures. The radial patterns should run parallel to the direction of the surface slopes existing within the area. Radial trenching patterns can also be used to provide drainage from localized low spots to the main drainage trench pattern. When the disposal area is extremely large in area1 extent or when interior cross dikes or other obstructions exist within the disposal area, sequential sets of radial trenches may be constructed, with the sets farthest into the disposal area

Figure 5-15. Rubber-tired rotary trencher

Figure 5-16. Track-mounted rotary trencher used in mosquito control activities

Figure 5-17. View of hemispherical rotary trenching implement

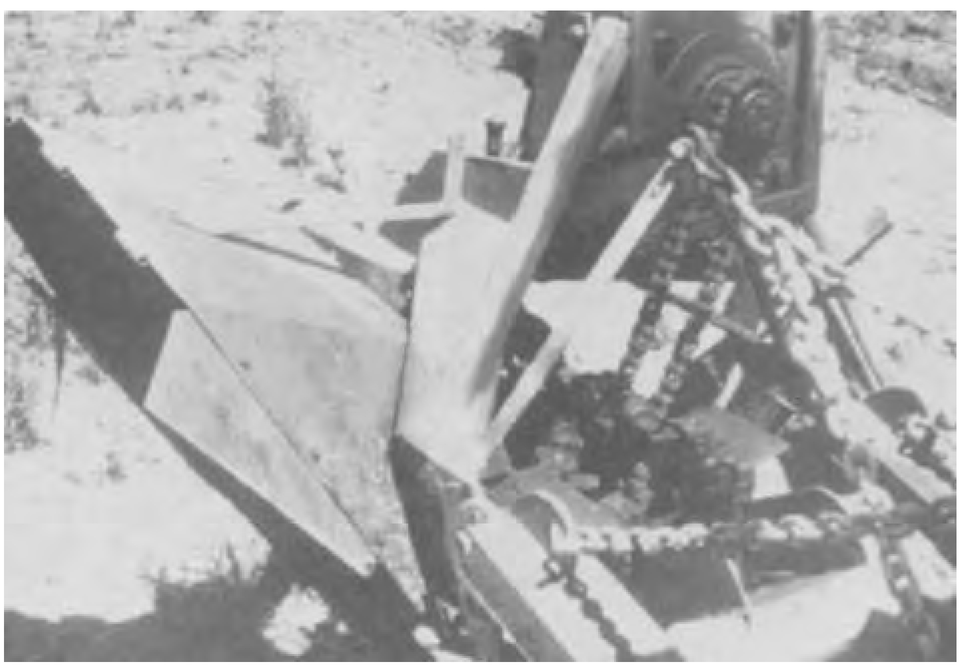


Figure 5-18. View of the trapazodial rotary
trenching implement

Figure 5-19. View of rotary trenching device in operation

Figure 5-20. General view of trenches formed by rotary trencher

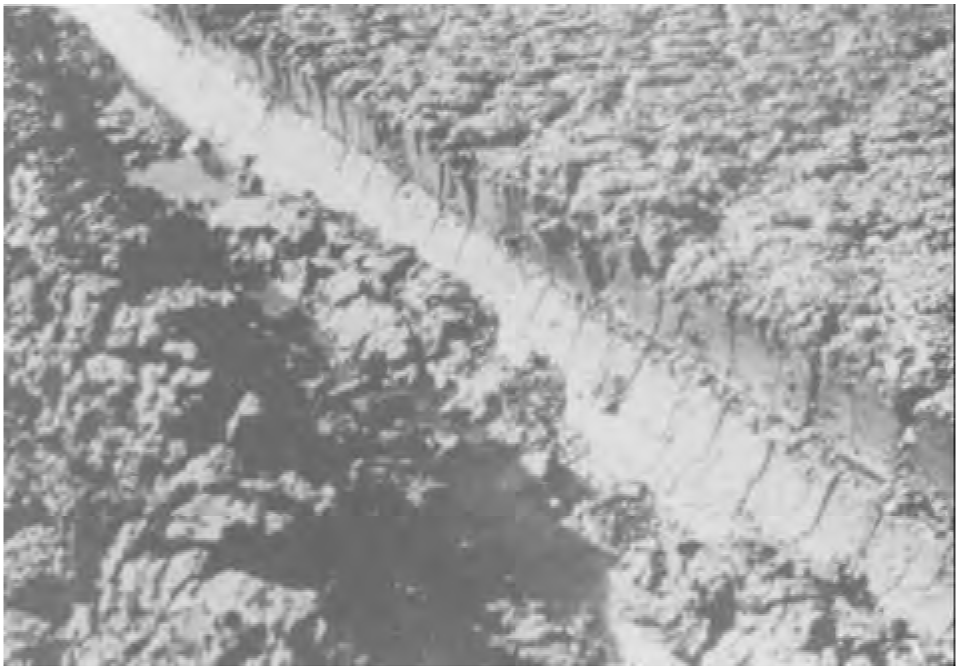


Figure 5-21. Closeup view of trenches formed by rotary trencher

Figure 5-22. General view of confined disposal area showing parallel trenches in place

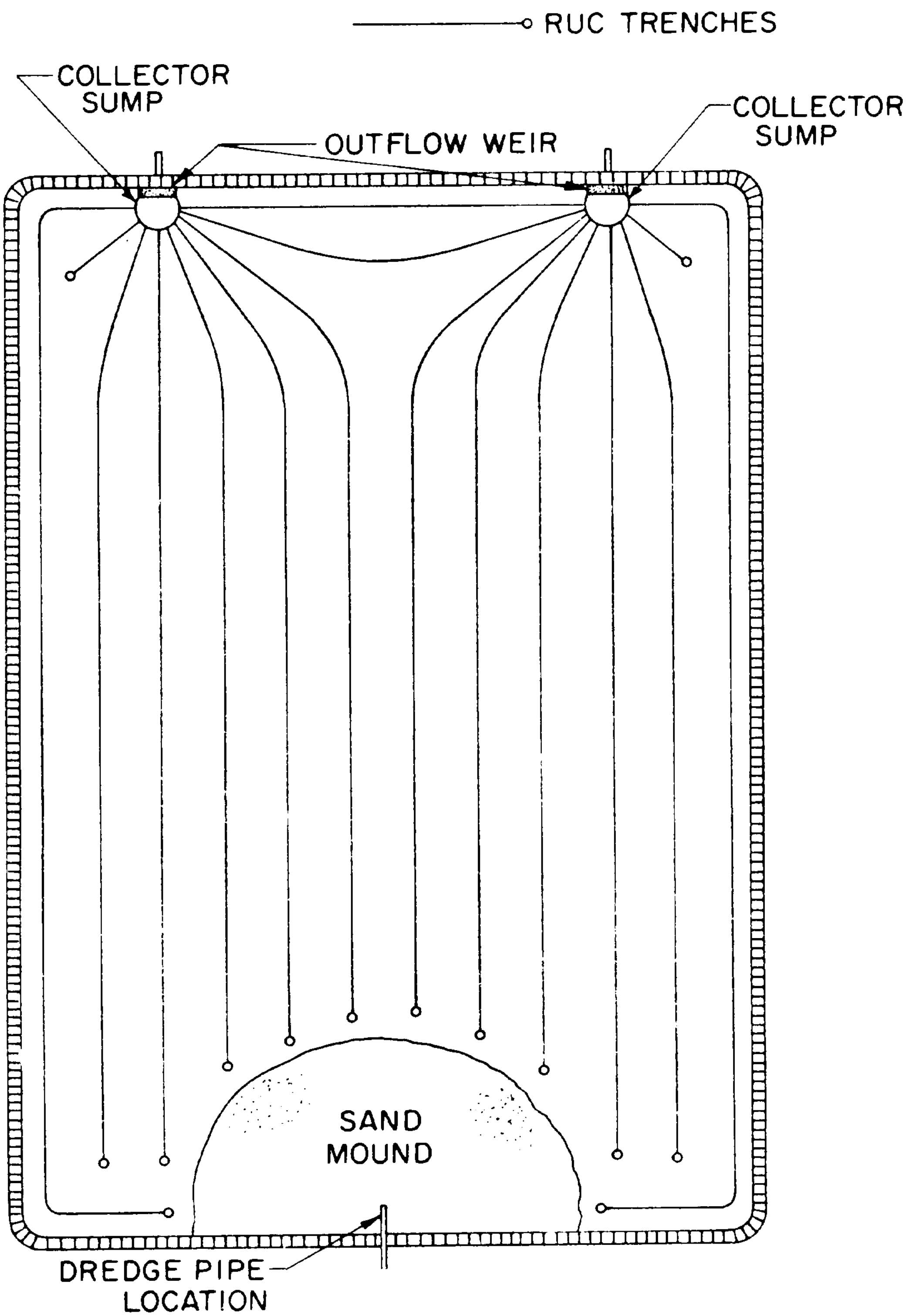


Figure 5-23. Combination radial-parallel trenching scheme

interior acting as collectors funneling into one of the radial trenches extending from the outflow weir. This sequential radial trenching procedure is shown in Figure 5-24, as constructed in the South Blakely Island Disposal Site of the Mobile District.

Figure 5-24. Aerial view of sequential radial trenching procedure used when interior cross dikes are encountered, South Blakely Island Disposal Site of the Mobile District

CHAPTER 6

DESIGN AND CONSTRUCTION OF DIKES FOR CONTAINMENT OF DREDGED MATERIAL

6-1. Purpose. Containment dikes are retaining structures used to form confined disposal facilities. They consist primarily of earth embankments and can be constructed in upland or nearshore areas or on nearshore islands. The principal objective of a dike is to retain solid particles and pond water within the disposal area while at the same time allowing the release of clean effluent to natural waters. The location of a containment dike will usually be established by factors other than foundation conditions and available borrow material (i.e., proximity to dredge, only land available, etc.) from which there will be little deviation. The heights and geometric configurations of containment dikes are generally dictated by containment capacity requirements, availability of construction materials, and prevailing foundation conditions.

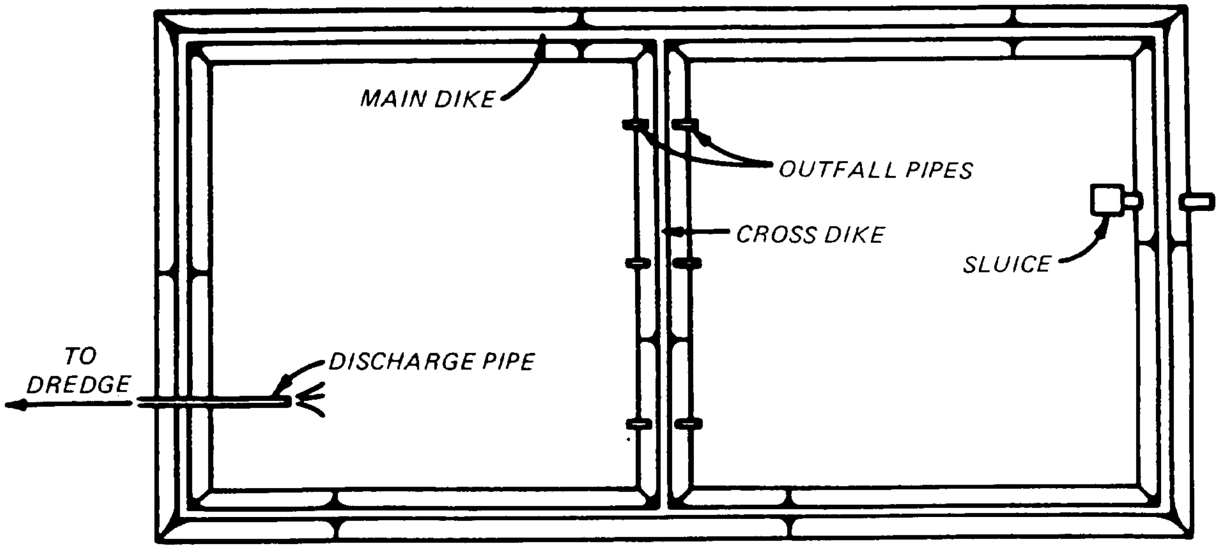
a. Types of Containment Dikes.

(1) Main dike. The predominant retaining structure in a containment facility extends around the outer perimeter of the containment area and is referred to as the main dike. Except as otherwise noted, all discussion in this chapter applies to the main dike. The main dike and two other type dikes, cross and spur dikes, which serve primarily as operational support structures for the main dike, are shown in Figure 6-1.

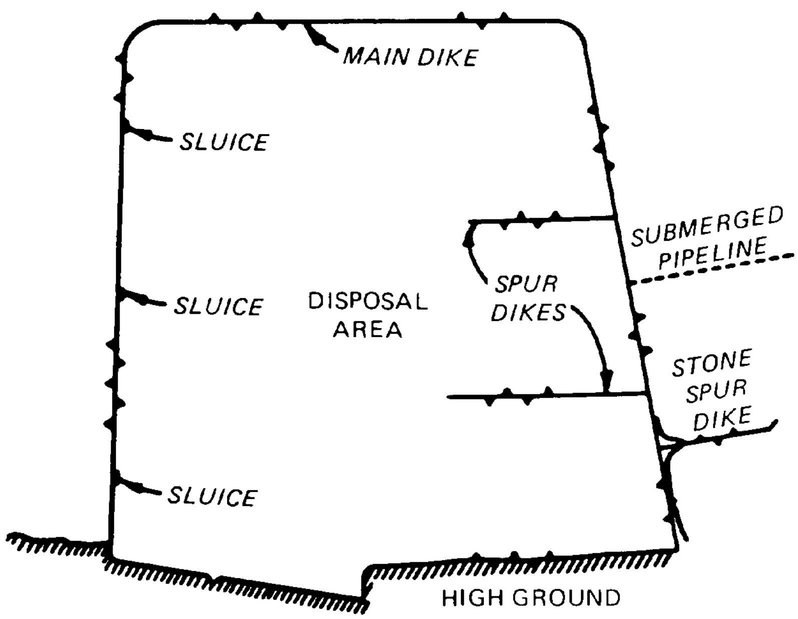
(2) Cross dike. A cross or lateral dike (Figure 6-1) is placed across the interior or the containment area connecting two sides of the main dike. This permits the use of one area as an active disposal area while another area may be used solely for dewatering. Another use of cross dikes is to separate the facility so that the slurry in one area is subjected to initial settling prior to passing over or through the cross dike to the other area. In order to accomplish this, the cross dike is placed between the dredged discharge point and the sluice discharge. A cross dike can also be used with a Y-discharge line to divide an area into two or more areas, each receiving a portion of the incoming dredged material.

(3) Spur dike. Spur or finger dikes protrude into, but not completely across, the disposal area from the main dike as shown in Figure 6-1. They are used mainly to prevent channelization by breaking up a preferred flow path and dispersing the slurry into the disposal area. Spur dikes are also used to allow simultaneous discharge from two or more dredges by preventing coalescence of the two dredged material inputs and thereby discouraging an otherwise large quantity of slurry from reaching flow velocities necessary for channelization.

b. Factors affecting design. The engineering design of a dike includes selection of location, height, cross section, material, and construction method. The selection of a design and construction method are dependent on project constraints, foundation conditions, material availability, and availability of construction equipment. The final choice will be a selection among feasible alternatives.



a. DIKED DISPOSAL AREA WITH CROSS DIKE



b. DIKED DISPOSAL AREA WITH SPUR DIKES

Figure 6-1. Examples of cross and spur dikes

(1) Project constraints. Several constraints on design are placed by the overall project needs. Available construction time and funding are always factors. The location, height, and available space for the containment dike are usually dictated by project requirements that are discussed elsewhere in this manual. The design factor of safety against structural failure is usually specified. Environmental safety and aesthetics must be considered.

(2) Foundation conditions. The lateral and vertical distribution of shear strength, compressibility, permeability, and stratification of potential foundation materials are major factors in dike design.

(3) Availability of materials. All potential sources of construction materials for the embankment should be characterized according to location, type, index properties, and ease of recovery. Available disposal sites are often composed in the near-surface of soft clays and silts of varying organic content. Since economical dike construction normally requires the use of material from inside the disposal area and/or immediately adjacent borrow areas, initial dike heights may be limited, or it may be necessary to use rather wide embankment sections, expensive foundation treatment, or expensive construction methods.

(4) Availability of equipment. Although common earthwork equipment is generally available, the specialized equipment for the soft soils desirable for use at containment sites may not be available to meet the project schedule, or the mobilization cost may be excessive. Less expensive alternatives should then be considered.

c. Construction Methods. Each type of construction method has characteristics that can strongly affect dike design. Transportation of the soil material to be placed in the dike section is either by hauling, casting, or dredging. The soil is then compacted, semicompacted, or left uncompacted. The selection of a construction method, even though based on economics, must also be compatible with available materials, available equipment, geometry of the final dike section, and environmental considerations.

6-2. Foundation Investigation. The extent to which the site investigation(s) and design studies are carried out is dependent, in part, on the desired margin of safety against failure. This decision will usually be made by the local design agency and is affected by a number of site-specific factors. Table 6-1 lists some general factors, based on engineering experience, that can be used as general guidelines in the planning stage of a project.

a. Foundation Exploration. The purpose of the foundation exploration is similar to that for the containment area as defined in Section 2-3, i.e., to define dike foundation conditions including depth, thickness, extent, composition, and engineering properties of the foundation strata. The exploration is made in stages, each assembling all available information from a given source prior to the planning and start of the next, more expensive stage. The usual sequence of the foundation exploration is shown in Table 6-2.

Table 6-1
Factors Affecting the Extent of Field Investigations and Design Studies

Factor	Field Investigations and Design Studies Should Be More Extensive Where:
Construction experience	There is little or no construction experience in the area, particularly with respect to dikes.
Consequence of failure	Consequences of failure involving life, property, or damage to the environment are great.
Dike height	Dike heights are substantial.
Foundation conditions	Foundation deposits are weak and compressible. Foundation deposits are highly variable along the alignment. Underseepage and/or settlement problems are severe.
Borrow materials	Available borrow is of poor quality, water contents are high, or borrow materials are variable along the alignment.
Structures in dikes	Sluices or other structures are incorporated into the dike embankment and/or foundation.
Utility crossings	Diked area is traversed by utility lines.

Additional guidance on the number, depth, and spacing of exploratory and/or final phase borings is given in WES TR D-77-9 (item 16), EM 1110-2-2300, ASCE Manual No. 56 (item 3), and various geotechnical engineering textbooks. Geophysical exploration methods are described in EM 1110-1-1802.

b. Field and Laboratory Tests. Field soils tests are often made during exploratory boring operations. Commonly used field tests are given in Table 6-3. Disturbed samples from exploratory and final phase borings are used for index properties tests. Samples from undisturbed sample borings are used in laboratory tests for engineering properties. Commonly used laboratory tests are given for fine-grained soils in Table 6-4 and for coarse-grained soils in Table 6-5. Additional guidance on field soil sampling methods is given in EM 1110-2-1907 and on laboratory soils testing in EM 1110-2-1906.

6-3. Construction Materials.

a. Acceptable Materials.

(1) Almost any type of soil material is acceptable (even though not the most desirable) for construction of a retaining dike, with the exception of very wet fine-grained soils and those containing a high percentage of organic

Table 6-2
Stages of Field Investigation

Stage	Features
Preliminary geological investigation	Office study Collection and study of: Topographic, soil, and geological maps Aerial photographs Boring logs and well data Information on existing engineering projects Field survey Observations and geology of area, documented by written notes and photographs, including such features as: Riverbank and coastal slopes, rock outcrops, earth and rock cuts or fills Surface materials Poorly drained areas Evidences of instability of foundations and slopes Emerging seepage and/or soft spots Natural and man-made physiographic features
Subsurface exploration and field testing and more detailed geologic study	Exploratory phase Widely but not uniformly spaced disturbed sample borings (may include split-spoon penetration tests) Test pits excavated by backhoes, farm tractors, or dozers Geophysical surveys to interpolate between widely spaced borings Borehole geophysical tests Water table observations Final phase Additional disturbed sample borings including split-spoon penetration tests Undisturbed sample borings Field vane shear tests for soft materials

Table 6-3
Preliminary Appraisal of Foundation Strengths

Method	Remarks
Penetration resistance from standard penetration test	In clays, provides data helpful in a relative sense, i.e., in comparing different deposits. Generally not helpful where number of blows per foot N* is low In sand, N-values less than about 15 indicate low relative densities
Natural water content of disturbed or general type samples	Useful when considered with soil classification and previous experience is available
Hand examination of disturbed samples	Useful where experienced personnel are available who are skilled in estimating soil shear strengths
Position of natural water contents relative to LL and PL	If natural water content is close to PL, foundation shear strength should be high
Field pumping tests used to determine field permeability	Natural water contents near LL indicate sensitive soils with low shear strengths
Torvane or pocket penetrometer tests on intact portions of general samples	Easily performed and inexpensive, but results may be excessively low; useful for preliminary strength estimates
Vane shear tests	Useful where previous experience is available Used to estimate shear strengths

Table 6-4

Laboratory Testing of Fine-Grained Cohesive Soils

Type Test	Purpose	Scope of Testing
Visual classification	To visually classify the soil in accordance with the USCS	All samples
Water content	To determine the water content of the soil in order to better define soil profiles, variation with depth, and behavioral characteristics	All samples
Atterberg limits	Foundation soils: for classification, comparison with natural water contents, or correlation with shear or consolidation parameters Borrow soils: for classification, comparison with natural water contents, or correlations with optimum water content and maximum dry densities	Representative samples of foundation and borrow soils. Sufficient samples should be tested to develop a good profile with depth
Compaction	To establish maximum dry density and optimum water content	Representative samples of all borrow soils for compacted or semicompacted dikes: Compacted - perform standard 25-blow test Semicompacted - perform 15-blow test
Consolidation	To determine parameters necessary to estimate settlement of dike and/or foundation and time-rate of settlement. Also, to determine whether soils are normally consolidated or overconsolidated and to aid in estimating strength gain with time	Representative samples of compacted borrow where consolidation of dike embankment itself is expected to be significant. Representative samples of foundation soils where such soils are anticipated to be compressible On samples of fine-grained adjacent and/or underlying materials at structure locations
Permeability	To estimate the perviousness of borrow and/or foundation soils in order to calculate seepage losses and time-rate of settlement	Generally not required for fine-grained cohesive soils as such soils can be assumed to be essentially impervious in seepage analyses. Can be computed from consolidation tests
Shear strength	To provide parameters necessary for input into stability analysis Pocket penetrometer. miniature vane, unconfined compression, and Q-tests to determine unconsolidated-undrained strengths R-tests to determine consolidated-undrained strengths S-teats to determine consolidated-undrained strengths	Pocket penetrometer and miniature vane (Torvane) for rough estimates Unconfined compression tests on saturated foundation clays without joints, fissures, or slickensides Appropriate Q- and R-trisxial and S-direct shear tests on representative samples of both foundation and compacted borrow soils

Table 6-5

Laboratory Testing of Coarse-Grained Noncohesive Soils

Test	Purpose	Scope of Testing
Visual classification	To visually classify the soil in accordance with the USCS	All samples
Gradation	To determine grain-size distribution for classification and correlation with permeability and/or shear strength parameters	Representative samples of foundation and borrow materials
Relative density or compaction	To determine minimum-maximum density values or maximum density and optimum water content values; should use the test which gives greatest values of maximum density	Representative samples of all borrow materials
Consolidation	To provide parameters necessary for settlement analysis	Not generally required as pervious soils consolidate rapidly under load and post-construction magnitude is usually insignificant
Permeability	To provide parameters necessary for seepage analysis	Not usually performed as correlations with grain size are normally of sufficient accuracy. Where underseepage problems are very serious, best to use results from field pumping test
Shear strength	To provide parameters necessary for stability analysis	Representative samples of compacted borrow and foundation soils. Consolidated-drained strengths from S-direct shear or triaxial tests are appropriate for free-draining pervious soils

matter. High plasticity clays may present a problem because of detrimental swell-shrink behavior when subjected to cycles of wetting and drying.

(2) Either fine-grained soil materials of high water content must be dried to a water content suitable for the desired type of construction, or the embankment design must take into account the fact that the soil has a high water content and is, therefore, soft and compressible. Because the drying of soils is very expensive, time consuming, and highly weather dependent, the design should incorporate the properties of the soil at its natural water content or should require only a minimum of drying. When the dike fill is to be compacted, the borrow material must have a sufficiently low water content so that placement and machine compaction can be done effectively. Semicompacted fill can tolerate fine-grained soils with higher water contents, while uncompacted (cast) fine-grained fill can be placed at even higher water contents. Since dike construction is normally done in low, wet areas, problems with materials being too dry are rarely encountered.

b. Material Sources. A careful analysis of all available material sources, including location, material type, and available volume should be made. Possible sources include any required excavation area, the material adjacent to the dike toe, a central borrow area, and material from maintenance dredging operations.

(1) Required excavation. Soil material from required excavations should be given first consideration since it must be excavated and disposed of anyway. Included in this category is material from adjacent ditches, canals, and appurtenant structures, as well as material from inside the containment area. This usage also eliminates the problem of dealing with borrow areas left exposed permanently after project completion.

(2) Material adjacent to dike toe. This is the most common source of dike material because it involves a short-haul distance. Hauling can be eliminated by the use of a dragline-equipped crane. Dike stability can be seriously affected if the excavation is made too close to the toe. A berm is usually left in place between the toe of the dike and the excavation to ensure dike stability and to facilitate construction. The required width of the berm should be based on a stability analysis.

(3) Central borrow area. When sufficient material cannot be economically obtained from required excavations or the dike toe, a central borrow area is often used. This may be within the containment area or may be offsite. A central borrow pit within the containment area serves to increase available containment volume. Central borrow areas can be used for either hauled or hydraulic fill dikes. Dredging from a water-based central borrow pit is usually economical for hydraulic fill dikes. Usually a deeper pit with smaller surface area is preferred since this requires less movement of the dredge.

(4) Maintenance dredging. Maintenance dredging can be a very economical source of borrow material. The coarse-grained materials from maintenance dredging are desirable for dike construction. Zones around the dredge discharge usually will provide the highest quality of material. However, fine-grained soils may not be suitable because of their very high water content and

may require considerable drying. The use of previously placed dredged material from maintenance operations has been commonly used to raise existing dikes. It is readily available and serves to increase the capacity of the containment area.

c. Materials Exploration and Testing. All discussion of field investigation procedures, including exploratory investigation of strength, and of laboratory index properties tests given in Tables 6-2, 6-3, 6-4, and 6-5 is applicable to the characterization of potential embankment materials. The objective is to develop sufficient information regarding the various sources of fill material for a comparison among feasible alternatives.

6-4. Embankment Design Considerations. The development of an investigation for the dike foundation and for proposed borrow areas, the selection of a foundation preparation method, and the design of the embankment cross section require specialized knowledge in soil mechanics. Therefore, all designs and specifications should be prepared under the direct supervision and guidance of a geotechnical engineer and should bear his approval.

a. Factors in Design. In addition to the project constraints described in 6-1.c.(1), the site-specific factors that should be considered in the design of containment dikes are foundation conditions; dike stability with respect to shear strength, settlement, seepage, and erosion; available dike materials; and available construction equipment.

b. Dike Geometry. The height and crown width of a dike are primarily dependent on project constraints generally unrelated to stability. Side slopes and materials allocation within the cross section are functions of foundation conditions, materials availability, and time available for construction.

c. Embankment and Foundation Stability. Proposed cross-section designs should be analyzed for stability as it is affected by foundation and/or embankment shear strength, settlement caused by compression of the foundation and/or the embankment, and external erosion. The analytic methods described and referenced herein contain procedures that have proven satisfactory from past use, and most are currently employed by the CE. Specific details concerning methods for analyzing dike stability are reported in TR D-77-9 (item 16) and in EM 1110-2-1902. Several computer programs are available to CE districts to assist in stability analyses, either on mainframe or on microcomputers.

d. Causes of Dike Instability.

(1) Inadequate shear strength. Overstressing of low shear strength soils in the dike and/or the foundation (often coupled with seepage effects) is the cause of most dike failures. Failures of this type can be the most catastrophic and damaging of all since they usually occur quickly and can result in the loss of an entire section of the dike along with the contained dredged material. These failures may involve the dike alone, or they may involve both the dike and the foundation. Thus, two forms of instability may occur:

(a) Where the foundation is much stronger than the embankment, the dike slope can fail in a rotational slide tangent to the firm base as shown in Figure 6-2. However, if a much weaker horizontal plane or layer exists at or near the contact between the dike fill and the foundation, the failure may be a translation type, taking the form of a sliding wedge as shown in Figure 6-3.

(b) When the strength of the foundation is equal to or less than that of the fill, a rotational sliding failure that involves both the fill and the foundation may occur, as shown in Figure 6-4. If the foundation contains one or more weaker horizontal planes or layers, then a translation type failure, in the form of a sliding wedge, may occur as shown in Figure 6-5.

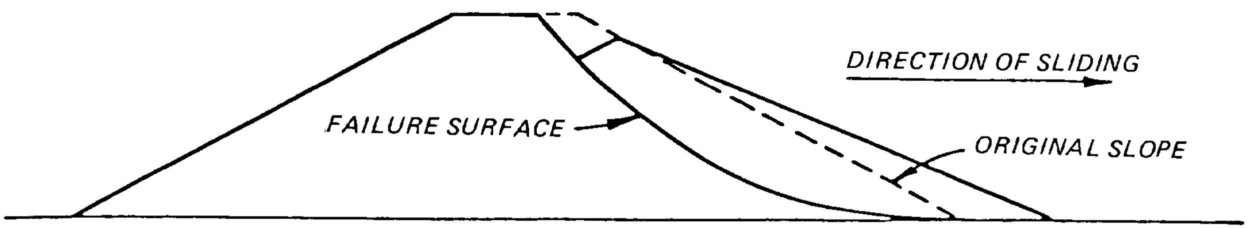
(c) Recommended minimum factors of safety and applicable shear strength tests for slope stability analyses of containment dikes are given in Table 6-6. These values are to be used where reliable subsurface data from a field exploration and laboratory testing program are available for input to a stability analysis. The factors of safety given in Table 6-6 are applicable to dikes less than 30 feet in height where the consequences of failure are not severe. For dikes greater than 30 feet in height and where the consequences of failure are severe, the criteria given in Table 1 of EM 1110-2-1902 should be used.

(d) When the foundation soils are very soft, as is often the case, various design sections are used to provide stability, as shown in Figure 6-6. A floating section may be used, with very flat slopes and often a berm. The settlement of this section may become detrimental. The soft foundation may be displaced by the firmer dike material, or the soft foundation may be removed and replaced with compacted fill.

(2) Seepage. Potentially detrimental seepage can occur through earth dikes and foundations consisting of pervious or semipervious materials unless prevented by positive means such as impervious linings, blankets, or cutoffs. Seepage effects can create instability through internal erosion (piping) of the dike or foundation materials, or they may lead to a shear failure by causing a reduction in the shear strength of the dike and/or foundation materials through increased pore water pressure or by the introduction of seepage forces. The following conditions may create or contribute to seepage problems in containment dikes:

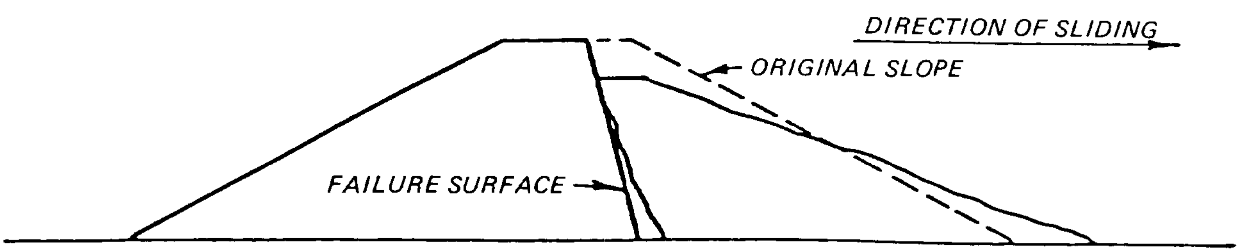
(a) Dikes with steep slopes composed of coarse-grained pervious materials or fine-grained silt. The seepage surface through the embankment may exit on the outer slope above the dike toe, as shown in Figure 6-7, resulting in raveling of the slope. If the dike contains alternating layers of pervious and impervious materials, the seepage surface may even approach a horizontal line near the ponding surface elevation, as shown in Figure 6-8, creating a potentially severe seepage problem.

(b) Dikes built on pervious foundation materials or where pervious materials are near the surface or exposed as a result of nearby excavation. As shown in Figure 6-9, this is a common condition where material adjacent to the dike toe is used for the embankment. This condition may lead to the development of large uplift pressures beneath and at the outer toe of the dike,



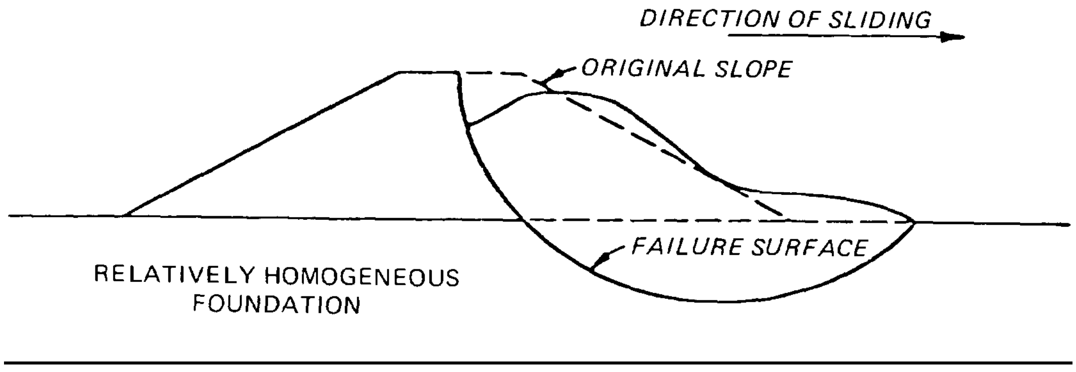
CROSS SECTION OF FAILURE

Figure 6-2. Rotational failure in dike



CROSS SECTION OF FAILURE

Figure 6-3. Translatory failure in dike



CROSS SECTION OF FAILURE

Figure 6-4. Rotational failure in both dike and foundation

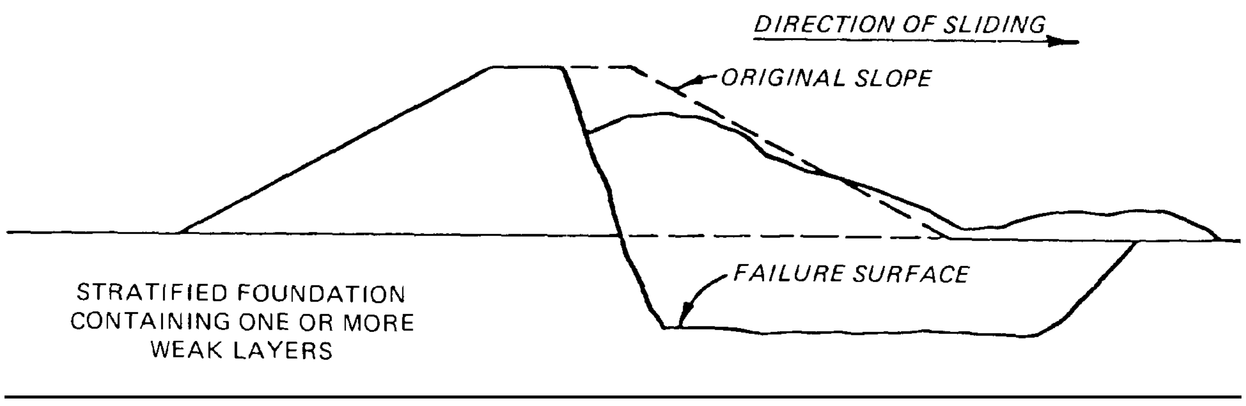


Figure 6-5. Translatory failure in both dike and foundation

Table 6-6
Applicable Shear Strengths and Recommended
Minimum Factors of Safety*

Condition	Shear Strength: Impervious Soils**	Shear Strength: Free-Draining Soils	Slope Analyzed	Minimum Factor of Safety†: Main Dikes	Minimum Factor of Safety†: Appurtenant Dikes
End of construction	Q	S	Exterior and interior	1.3‡	1.3
Steady seepage	Q, R††	S	Exterior	1.3	1.2
Sudden drawdown	Q, R††	S	Exterior	1.0	NA

* Criteria not applicable to dikes greater than 30 feet in height or where the consequences of failure are very severe. For such dikes use criteria given in Table 1 of EM 1110-2-1902.
** For low plasticity silt where consolidation is expected to occur rather quickly, the R strength may be used in lieu of the Q strength.
† To be applied where reliable subsurface data from exploration and testing are available; where assumed values are used, recommended minimum factors of safety should be increased by a minimum of 0.1.
†† Use Q strength where it is anticipated loading condition will occur prior to any significant consolidation taking place; otherwise use R strength.
‡ Use 1.5 where considerable lateral deformation of foundation is expected to occur (usually where foundations consist of soft, high-plasticity clay).

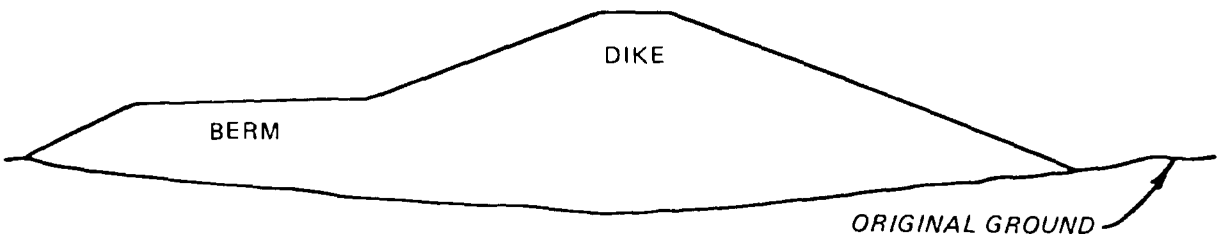
causing overall instability from inadequate shear strength or may result in piping near the embankment base. Methods for analyzing this condition are reported in WES TM 3-424 (item 34).

(c) Dikes constructed by casting methods with little or no compaction. When used with fine-grained soils, this method of construction may leave voids within the dike through which water can flow freely, resulting in piping of dike material.

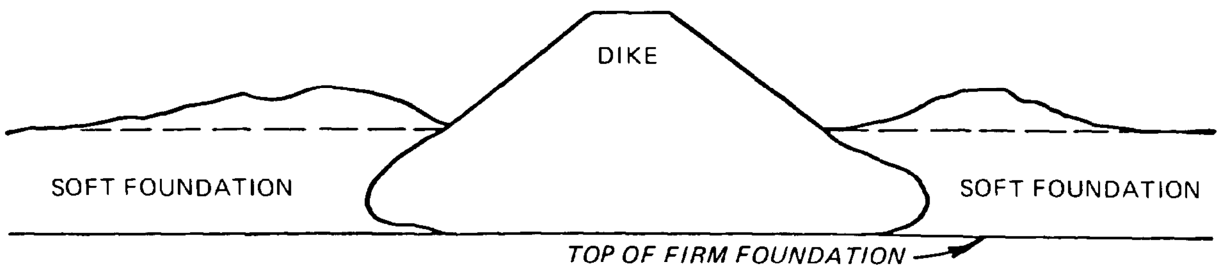
(d) The existence of seepage paths along the contact between structures touching the dike. This condition can be caused by inadequate compaction of the dike materials, shrinkage of material adjacent to structures, or differential settlement, As in the previous case, piping of the dike material often results in and normally leads to breaching of the dike.

e. Dike Settlement.

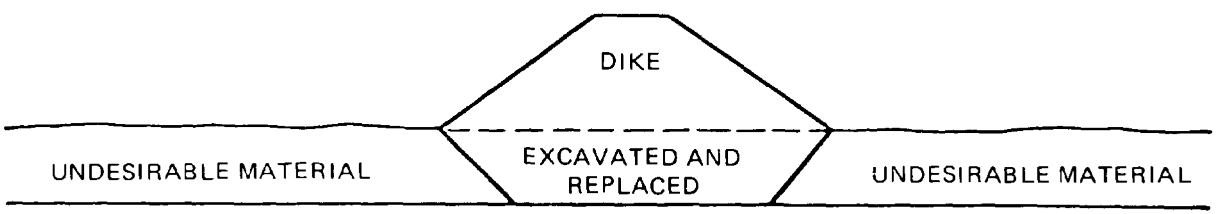
(1) Settlement of dikes can result from consolidation of foundation and/or embankment materials, shrinkage of embankment materials, or lateral spreading of the foundation. Like uncontrolled seepage, settlement of a dike can result in failure of the dike, but more likely will serve to precipitate



a. FLOATING SECTION



b. DISPLACED SECTION



c. SECTION FORMED BY EXCAVATION AND REPLACEMENT

Figure 6-6. Basic methods of forming dike sections for stability

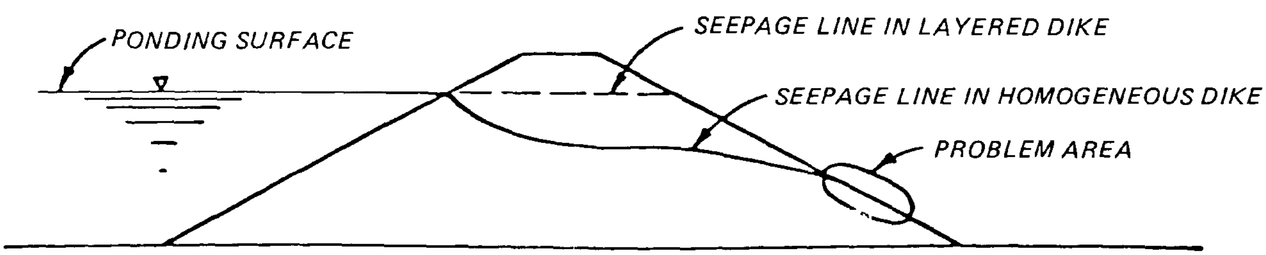


Figure 6-7. Seepage lines through dike

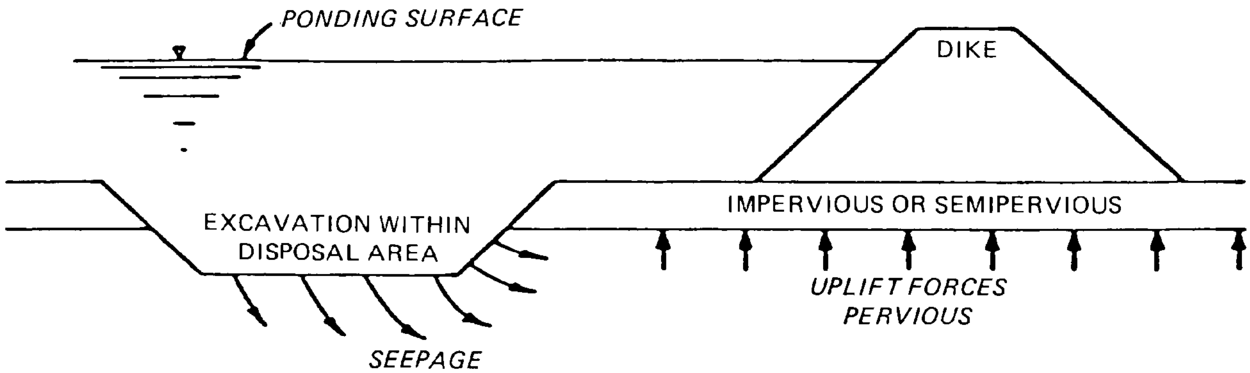


Figure 6-8. Seepage entrance through area excavated within disposal area

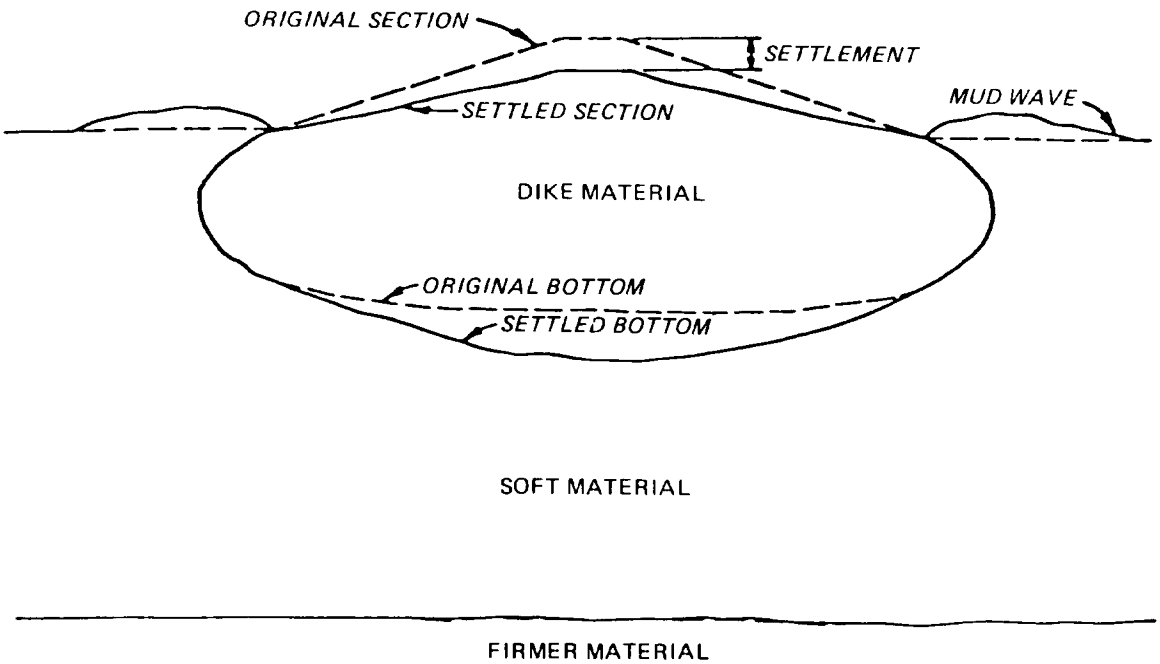


Figure 6-9. Example of excessive uniform settlement

failure by another mode such as seepage or shear failure. Consolidation, shrinkage, and some lateral deformation occur over a period of time, directly related to the soil permeability and the load intensity. Some lateral deformation can occur quickly, however, particularly during construction using the displacement method. Settlement problems are almost always related to fine-grained soils (silts or clays). Settlement and/or shrinkage of coarse-grained soils (sand and gravel) is generally much less than for fine-grained soils and occurs quickly, usually during construction.

(2) Specific forms of settlement that cause problems with dikes include: excessive uniform settlement, differential settlement, shrinkage of uncompacted embankment materials, and settlement resulting from lateral deformation, or creep, of soft foundation soils. Excessive uniform settlement can cause a loss in containment area capacity as a result of the loss of dike

height, as shown in Figure 6-9. Differential settlement can result in cracking of the dike, which can then lead to a shear or piping failure. This is an especially acute problem at the contact between a dike and an adjacent structure. Examples of differential settlement resulting from materials of different compressibility are shown in Figure 6-10. Embankment shrinkage in dikes built with fine-grained soils and placed by means of casting or hydraulic filling can result in volume reductions of as much as 35 percent as a result of evaporation drying.

f. Erosion. Retaining dike failures can be initiated by the effects of wind, rain, waves, and currents that can cause deterioration of exterior and interior dike slopes. The exterior slopes, which are exposed to constant or intermittent wave and/or current action of tidal or flood waters, are usually subject to severe erosion. Interior slopes may also suffer this form of erosion, particularly in large containment areas. The slopes of dikes adjacent to navigable rivers and harbors may be eroded by wave action from passing vessels.

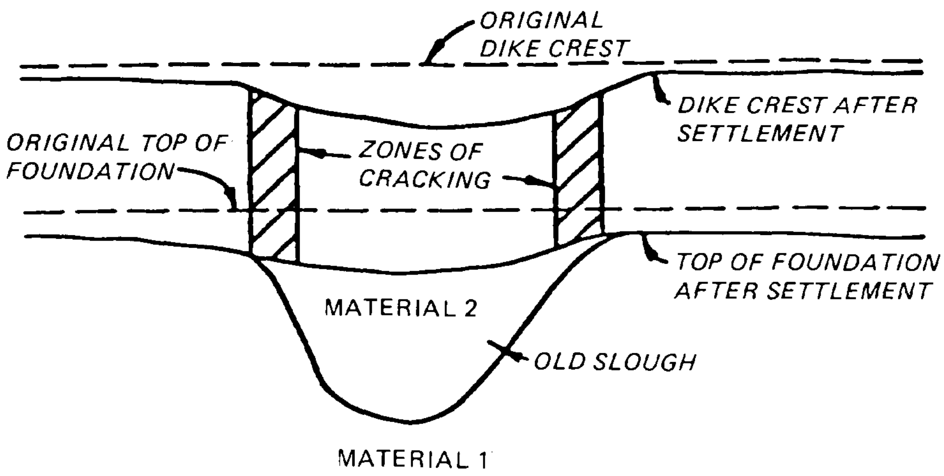
(1) Weathering. Erosion of dike slopes due to the effects of wind, rain, and/or ice is a continuing process. Although these forces are not as immediately severe as wave and current action, they can gradually cause extensive damage to the dike, particularly those dikes formed of fairly clean coarse-grained soils.

(2) Disposal operations. Normal disposal operations can cause erosion of interior dike slopes near the pipeline discharge and/or exterior slopes at the outlet structures. The pipeline discharge of dredged material is a powerful eroding agent, particularly if the flow is not dispersed.

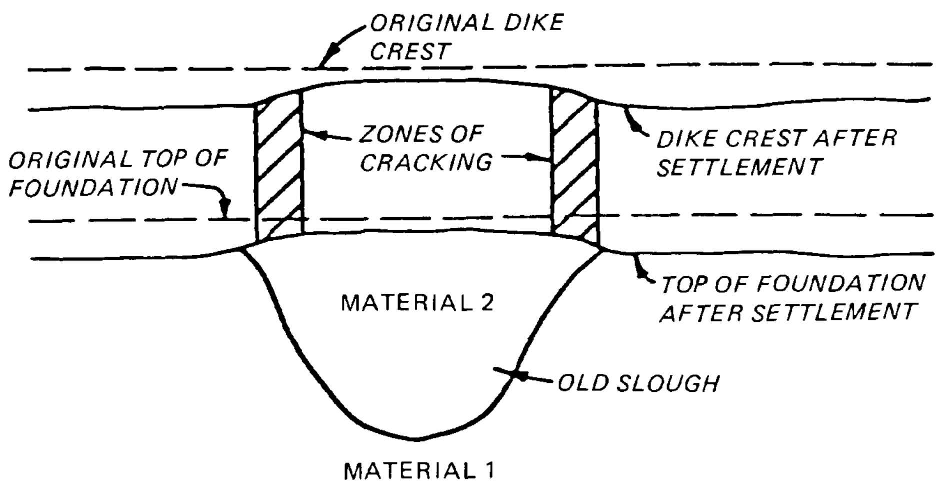
g. Use of geotextiles.

(1) Selection. Geotextiles (permeable textile materials) are being increasingly used in dike construction to provide tensile reinforcement where it will increase the overall strength of the structure. The selection of geotextiles for use in a containment dike is usually based on a substantial cost savings over feasible, practical, alternate solutions, or on the improvement in performance of a design (e.g., more effective installation, reduced maintenance, or increased life).

(2) Stability analyses with geotextile reinforcement. Although the use of a geotextile as reinforcement introduces a complex factor into stability analyses, no specific analytic technique has yet been developed. Therefore, the conventional limited equilibrium-type analyses for bearing capacity and slope stability are used for the design of geotextile reinforced dikes. The bearing capacity analysis, as given in EM 1110-2-1903, assumes the dike to be an infinitely long strip footing. Slope stability analyses, as described in EM 1110-2-1902, involve calculations for stability of a series of assumed sliding surfaces in which the reinforcement acts as a horizontal force to increase the resisting moment. Potential failure modes for fabric-reinforced dike sections are shown in Figure 6-11. Examples of stability analyses for geotextile reinforced embankments are given in item 10.

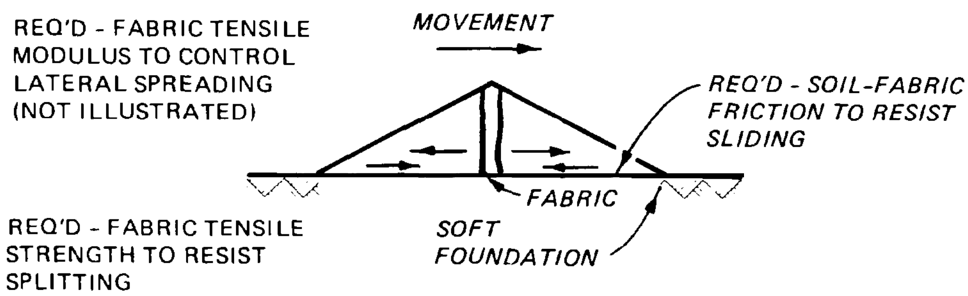


a. COMPRESSIBILITY OF MATERIAL 2 >> MATERIAL 1

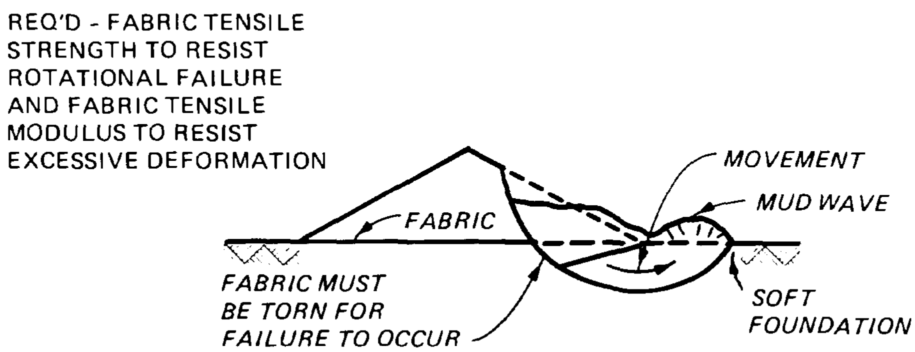


b. COMPRESSIBILITY OF MATERIAL 2 << MATERIAL 1

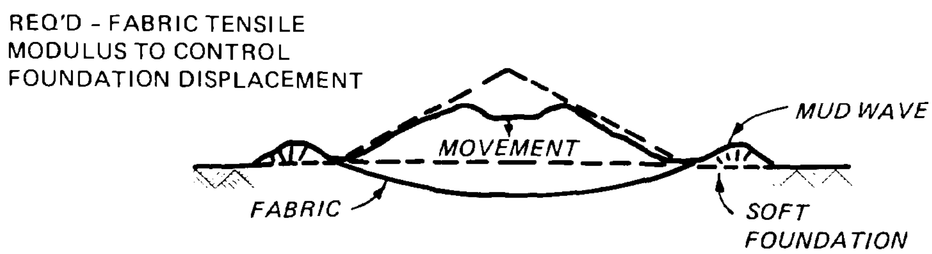
Figure 6-10. Differential settlement from foundation containing materials of different compressibility



A. POTENTIAL EMBANKMENT FAILURE FROM
LATERAL EARTH PRESSURE



B. POTENTIAL EMBANKMENT ROTATIONAL
SLOPE/FOUNDATION FAILURE



C. POTENTIAL EMBANKMENT FAILURE FROM
EXCESSIVE DISPLACEMENT

Figure 6-11. Potential fabric-reinforced embankment failure modes

h. Raising of existing dikes. The height to which a dike can be placed in one stage is sometimes limited by the weakness of the foundation. This limits the capacity of the containment area. The loading of the foundation due to the dike and/or dredged material causes consolidation, and consequent strength gain, of the foundation materials over a period of time. Thus, it is often possible to raise the elevation of an existing dike after some time. Construction of dikes in increments is usually accomplished by incorporating the initial dike into the subsequent dike, as shown in Figure 6-12a, or by constructing them on the dredged fill, at some distance from the inside toe of the existing dike, as shown in Figure 6-12b.

6-5. Construction Equipment.

a. Equipment Types. Types of equipment commonly used in dike construction are listed in Table 6-7 according to the operation they perform. Some types of equipment are capable of performing more than one task, with varying degrees of success. Most of the equipment listed is commonly used in earthwork construction. However, because many dikes are founded on soft to very soft ground, low-ground-pressure versions of the equipment must usually be used in those areas. Specific information on general construction equipment may be found in EM 1110-2-1911. Guidance on equipment available for use on soft soils is given in item 16 and item 13 and on dredging equipment in item 21.

b. Selection Criteria. In the selection of equipment for any particular task, consideration should be given to the following:

(1) Quantity of soil to be excavated, moved, or compacted.

(2) Type of soil to be excavated, moved, or compacted.

(3) Consistency of soils to be excavated, moved, or compacted.

(4) Distance soil must be moved.

(5) Trafficability of soils in borrow, transport, and dike placement areas.

(6) Availability of equipment to fit project time schedule.

(7) Purchase and operating costs.

(8) Auxiliary tasks or uses for equipment.

(9) Maintenance needs; availability of parts.

(10) Standby or backup equipment needs.

(11) Time available for construction of dike.

(12) Money available for construction of dike.

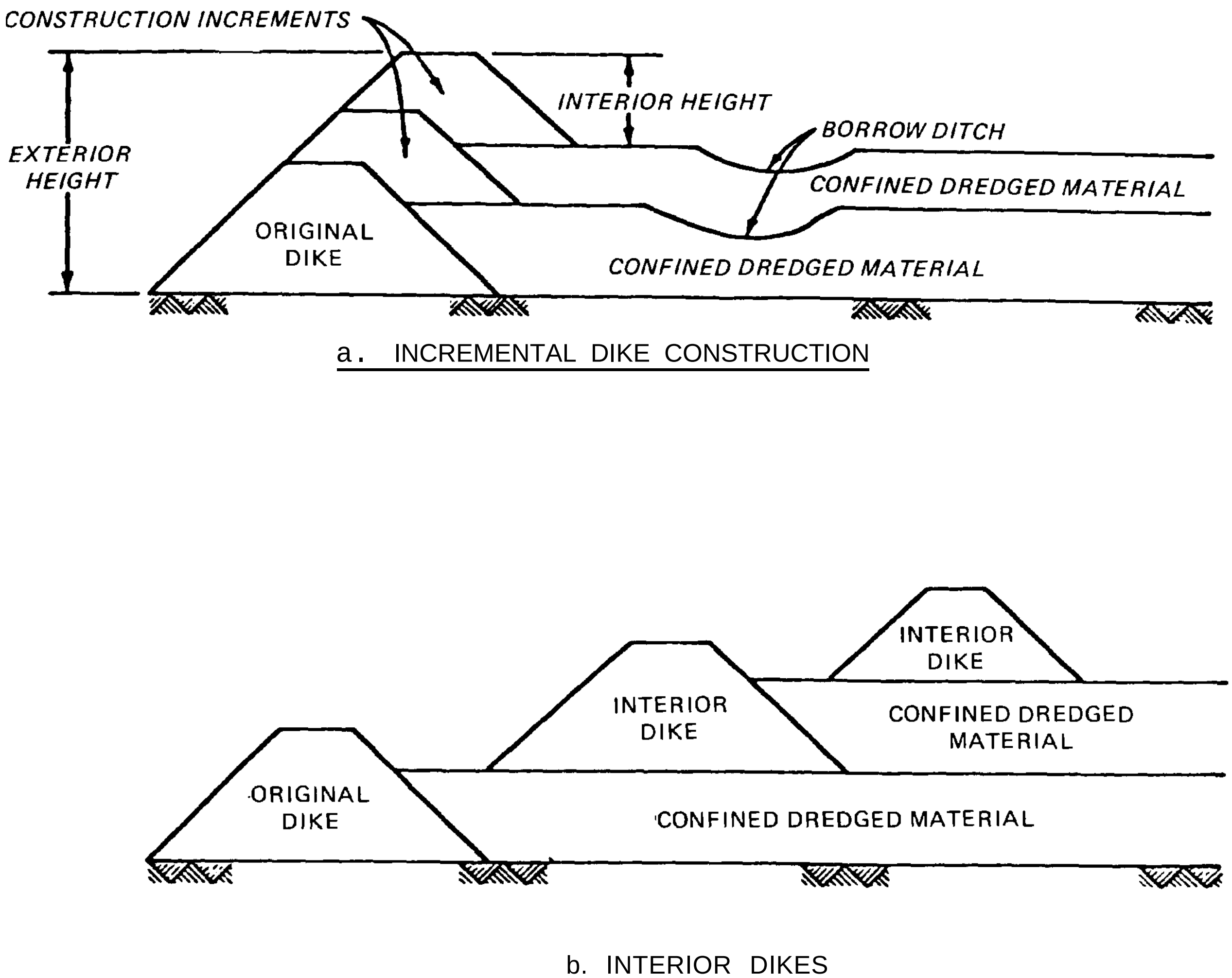


Figure 6-12. Dike raising methods

6-6. Dike Construction. The general construction sequence for a containment dike is normally foundation preparation, borrow area operations, transportation and placement of the dike materials in the embankment, and manipulation and possibly compaction of the materials to the final form and shape.

a. Factors in Method of Construction. The choice of construction method for a containment dike will be governed by available embankment materials, foundation conditions, trafficability of haul roads and the foundation, availability of construction equipment, and project economics.

b. Foundation Preparation. The preparation of a dike foundation usually involves clearing, grubbing, and stripping. Some degree of foundation preparation is desirable to help ensure the integrity of the structure. Clearing and grubbing should be a minimum treatment for all projects. However, in marshy areas where a surface mat of marsh grass and roots exists over a typical soft clay layer, experience has shown that it is often more beneficial from a stability and construction standpoint to leave the mat in place rather

Table 6-7
Equipment Commonly Used in Dike Construction

Operation	Equipment	Application
Excavation	Scraper	Firm to stiff soils; firm roadway
	Dragline	Soft soils that cannot support scrapers
	Dredge	Granular or soft soils below water
Transportation	Scraper	Hauling firm, moist soils
	Truck	Hauling firm, moist soils
	Dragline	Casting soft, wet soils
	Dredge	Pumping soils from below water
Scarification	Disc	Scarifying surface of compacted soil
Spreading	Scraper	Haul and spread from same machine
	Grader	Spread truck-hauled soils
	Crawler dozer	Used on soft terrain
Compaction	Sheepsfoot roller	Clays, silts, clayey or silty sands
	Pneumatic roller	Clays, silts, clayey or silty sands
	Vibratory roller	Clean sand; less than 10% fines
	Crawler tractor	All soils for semicompaction
	Hauling equipment	All soils for semicompaction
Shaping	Grader	Firm to stiff soils
	Crawler dozer	All soils; useful on soft soils
	Dragline	Rough shaping in very soft soils

than remove it, even though this will leave a highly pervious layer under the dike.

(1) Clearing. Clearing consists of the complete removal of all above-ground matter that may interfere with the construction and/or integrity of the dike. This includes trees, fallen timber, brush, vegetation, abandoned structures, and similar debris. Clearing should be accomplished well in advance of subsequent construction operations.

(2) Grubbing. Grubbing consists of the removal of below ground matter that may interfere with the construction and/or integrity of the dike. This includes stumps, roots, buried logs, and other objectionable matter. All holes and/or depressions caused by grubbing operations should have their sides flattened and should be backfilled to foundation grade in the same manner proposed for the embankment filling.

(3) Stripping. After clearing and grubbing, the dike area is usually stripped to remove low-growing vegetation and the organic topsoil layer. This will permit bonding of the fill soil with the foundation, eliminate a soft, weak layer that may serve as a translation failure plane, and eliminate a potential seepage plane. Stripping is normally limited to the dike location

proper and is not usually necessary under stability berms. All stripped material suitable for use as topsoil should be stockpiled for later use on dike and/or borrow area slopes. Stripping is not normally required for dikes on soft, wet foundations or for dikes built by other then full compaction.

(4) Disposal of debris. Debris from clearing, grubbing, and stripping operations can be disposed of by burning in areas where permitted. Where burning is not feasible, disposal is usually accomplished by burial in suitable areas such as old sloughs, ditches, and depressions outside the embankment limits (but never within the embankment proper). Debris should never be placed in locations where it may be carried away by streamflow or where it may block drainage of an area. Material buried within the containment area must be placed so that no debris may escape and damage or block the outlet structure. All buried debris should be covered by a minimum of 3 feet of earth.

(5) Foundation scarification. For compacted dikes on firm foundations only, the prepared foundation should be thoroughly scarified to provide a good bond with the embankment fill.

c. Borrow Area Operations. Factors that should be considered in the planning and operation of a borrow area are site preparation, excavation, drainage, and environmental considerations.

(1) Site preparation. The preparation of the surface of a borrow area includes clearing, grubbing, and stripping. The purpose of this effort is to obtain fill material free from such objectionable matter as trees, brush, vegetation, stumps, roots, and organic soil. In marshy areas, a considerable depth of stripping may be required due to frequently occurring 3- to 4-foot root mats, peat, and underlying highly organic soil. Often, marshy areas will not support the construction equipment. All stripped organic material should be wasted in low areas or, where useable as topsoil, stockpiled for later placement on outer dike slopes, berms, exposed borrow slopes, or other areas where vegetative growth is desired.

(2) Excavation. Planning for excavation operations in borrow areas should give consideration to the proximity of the areas to the dike, topography, location of ground-water table, possible excavation methods and equipment, and surface drainage.

(3) Drainage. Drainage of borrow areas (including control of surface and ground water) is needed to achieve a satisfactory degree of use. Often, natural drainage is poor, and the only choice is to start at the lowest point and work toward the higher areas, thereby creating a sump. Ditches are often effective in shallow borrow areas. Ditching should be done in advance of the excavation, particularly in fine-grained soils, to allow maximum drying of the soils prior to excavation.

(4) Environmental considerations. Permanently exposed borrow areas are usually surface treated to satisfy aesthetic and environmental protection considerations. Generally, projects near heavily populated or industrial areas will require more elaborate treatment than those in sparsely populated areas. Minimum treatment should include topographic shaping to achieve adequate drainage, smoothing and blending of the surface, treatment of the surface to

promote vegetation growth, and placement of vegetation to conform to the surrounding landscape. Item 23 should be consulted for more detailed information concerning landscaping methods.

d. Transportation and Placement of Materials. Three basic methods for transporting and placing dike materials in the embankment are hauling by means of trucks or scrapers, casting by means of a dragline, and pumping, or hydraulic filling, using a dredge. The relative advantages and disadvantages of these methods are summarized in Table 6-8.

e. Manipulation, Compaction, and Shaping. After placement, the dike materials may be compacted, semicompacted, or uncompacted. Many variations and combinations of these methods can and have been used. Classification by these methods does not necessarily refer to the end quality of the embankment; rather it refers to the amount of control of water content and compactive effort used during construction. The relative advantages and disadvantages of the methods of compaction are summarized in Table 6-9.

f. Construction Quality Control. The control of quality of construction operations is an extremely important facet of dike operations. Some of the more pertinent items to be inspected during construction of the dike are given in Table 6-10. For further guidance on control of earthwork operations, see EM 1110-2-1911.

6-7. Miscellaneous Features.

a. Discharge Facilities. Both excessive uniform and differential settlement of the dike can cause distortion and/or rupture of weir discharge pipes located under or through dikes (Figure 6-13) and can cause distortion of the weir box itself (Figure 6-14). The settlement effect can be somewhat mitigated by cambering (Figure 6-15) or raising one end (Figure 6-16) of the pipe during construction.

b. Seepage Control. Antiseepage devices, either metal fins or concrete collars, have been used in the past to inhibit seepage and piping along the outside wall of the outlet pipe. These have not proven effective. To aid in the prevention of piping failures along the pipe-soil interface, an 18-inch-minimum annular thickness of drain material (clean, pervious sand, or sand/gravel) should be provided around the outlet one-third of the pipe, as shown in Figure 6-17. This may be omitted where the outlet one-third of the pipe is located in sand.

c. Additional Uses of Geotextiles. The use of geotextiles to provide soil reinforcement was presented in section 6-4.g. In addition, geotextiles have been extensively used as filter fabrics to replace the filter materials (section 6-7.b.), drain materials, a separation medium, and an armor medium to inhibit erosion item 10. A brief summary of geotextile functions in dike construction is given in Table 6-11.

Table 6-8
Commonly Used Methods of Transporting Soils
in Dike Construction

Method	Advantages	Disadvantages
Hauling	May use central borrow area Permits use of high-speed, high-capacity equipment Allows better selection of soil type	All traveled surfaces must be firm to support equipment Cannot be used in soft, wet areas or underwater May require specialized low-pressure equipment
Casting	Dragline bucket can move very soft, wet soils Can operate on soft foundation	Low speed; low capacity Requires frequent movement of dragline equipment Short casting distance
Dredging	Move large quantities of soils from below water Permits use of dredged materials in dike May be used on soft foundation and roadway	Requires dredge and pipeline Soils cannot be compacted without drying; requires large sections with very flat slopes

Table 6-9
Commonly Used Methods of Compacting Soils in Dike Construction

Method	Advantages	Disadvantages
Compacted	Placed in thin layers and well compacted, strong dike, low compressibility Steep slopes, minimum space occupied Highest quality control	Requires that soils be dried to water content near plastic limit Requires competent foundation Highest cost
Semicompacted	Uses soils at natural water content, no drying needed May be used on weaker foundations Uses thick lifts May be hauled or cast	Requires flatter slopes May be limited in height Poorer quality control May require specialized low-pressure equipment
Uncompacted	Permits use of cast or dredged materials May be placed on very soft, wet foundation Fill placed at natural water content Lowest cost for dike	Requires very flat slopes May be severely limited in height or require stage construction Poorest quality control

Table 6-10
Operations or Items to Be Inspected During
Construction of Dikes

Type Construction	Items or Operation to Be Checked
Compacted	Proper fill material Loose lift thickness Disking Water content Type of compaction equipment and number of passes Density
Semicompacted	Proper fill material Loose lift thickness Water content (if required) Number of passes (if required) Routing of hauling and spreading equipment
Uncompacted (displacement technique)	Proper fill material Dumping and shoving techniques Ensuring fill is advanced in V-shape and with slopes as steep as possible Elevation of fill surface Prevention of rutting of fill surface by hauling equipment

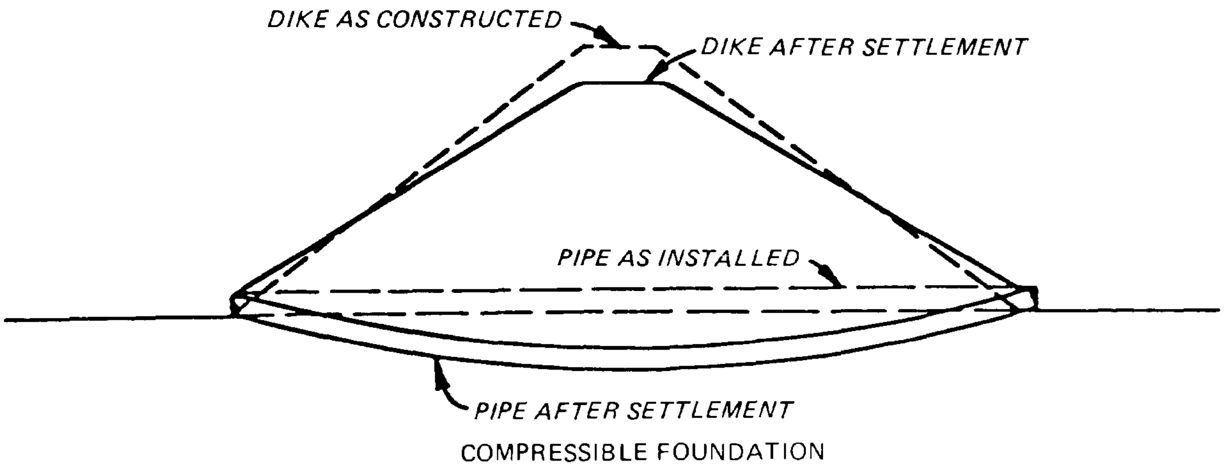


Figure 6-13. Swagging of pipe due to settlement of dike and foundation

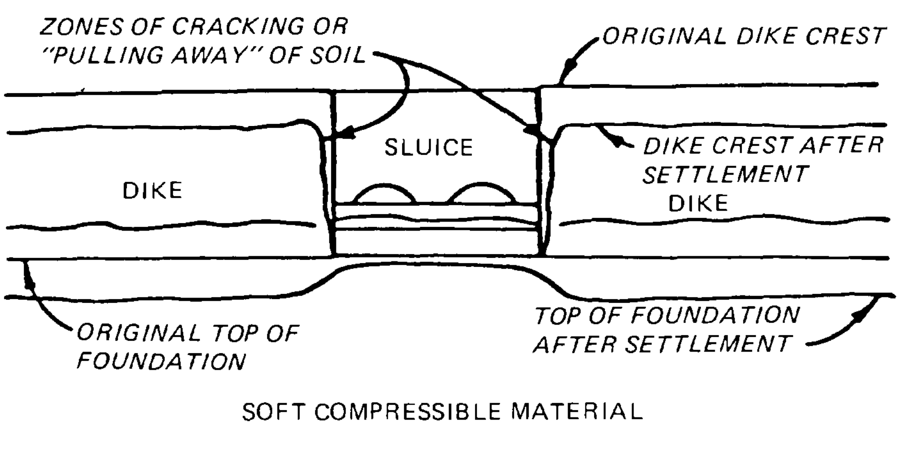


Figure 6-14. Cracking at dike-structure junction caused by differential settlement because dike load is much greater than weir load

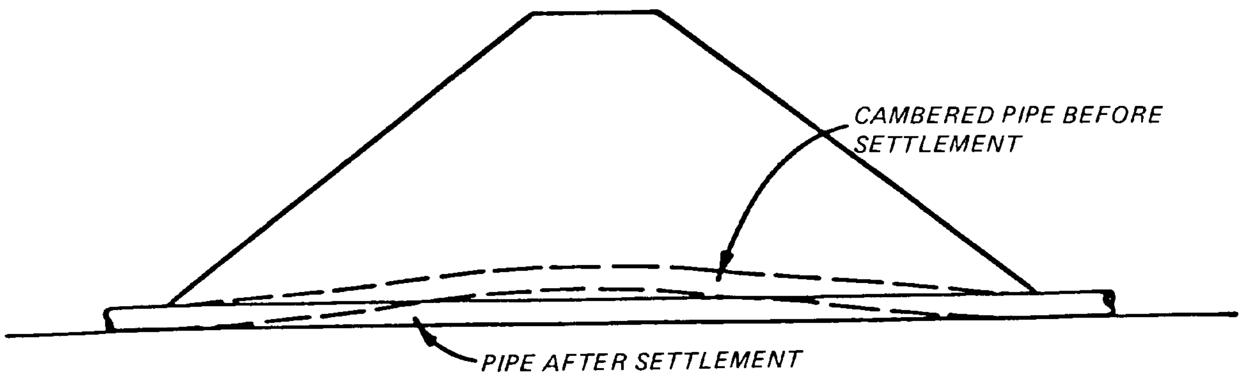


Figure 6-15. Cambered pipe beneath dike

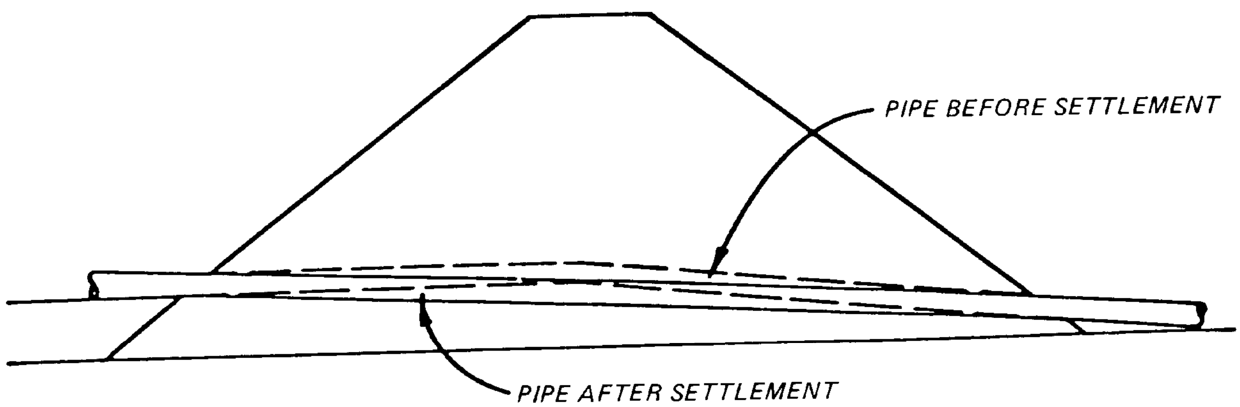
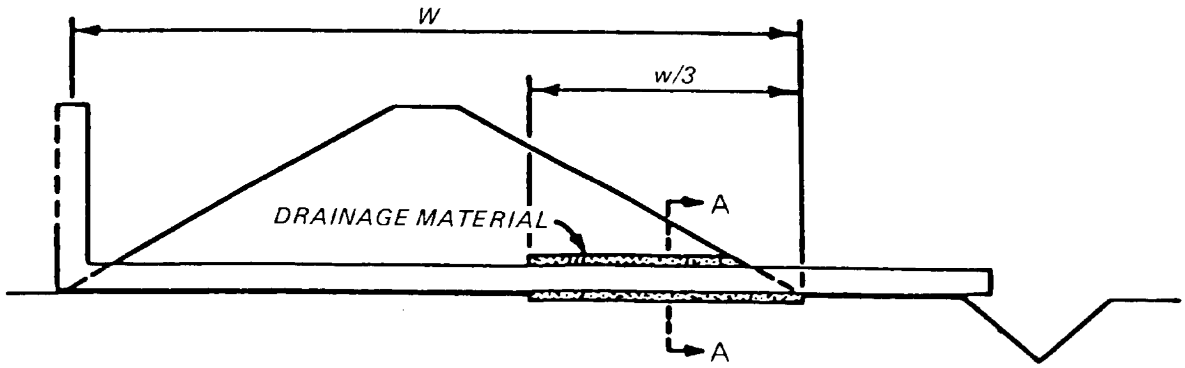
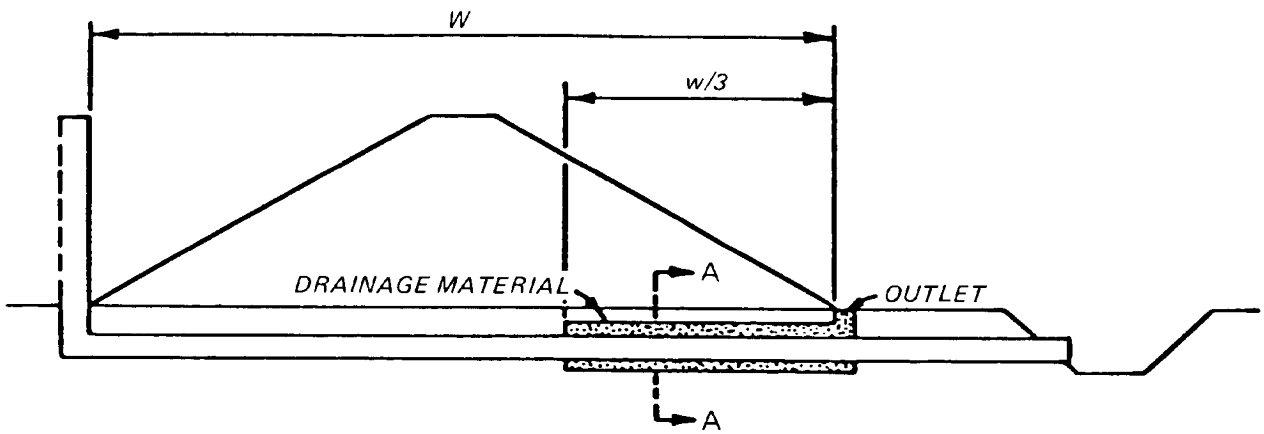


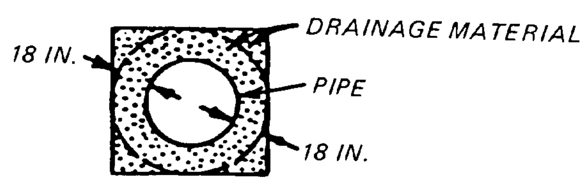
Figure 6-16. Cambered and raised pipe beneath dike



a. PIPE THROUGH EMBANKMENT



b. PIPE THROUGH FOUNDATION



c. SECTION A-A

NOTE: W = SECTION WIDTH

Figure 6-17. Annular drainage material around outlet one third of pipe

Table 6-11
Description of Geotextile Functions

Function	Description
Filter	The process of allowing water to escape easily from a soil unit while retaining the soil in place. The water is carried away by some other drain (e.g., rock or rock with pipe).
Drain	The situation where the fabric itself is to carry the water away from the soil to be drained.
Separation	The process of preventing two dissimilar materials from mixing. This is distinct from the filtration function, in that it is not necessary for water to pass through the fabric.
Reinforcement	The process of adding mechanical strength to the soil-fabric system.
Armor	The process of protecting the soil from surface erosion by some tractive force. Usually in these situations, the fabric serves only for a limited time.

CHAPTER 7

OPERATION AND MANAGEMENT OF CONTAINMENT AREAS

7-1. General Considerations. This chapter presents procedures for the effective management and operation of containment areas. Management activities are required before, during, and following the dredging operation to maximize the retention of suspended solids and the storage capacity of the areas. These activities include site preparation, removal and use of existing dredged material for construction purposes, surface water management, suspended solids monitoring, inlet and weir management, thin-lift placement, separation of coarse material, dredged material dewatering, and disposal area reuse management. Management activities described in this part are not applicable in all cases, but should be considered as possibilities for improving the efficiency and prolonging the service life of containment areas.

7-2. Predredging Management Activities.

a. Site Preparation. Immediately before a disposal operation, the desirability of vegetation within the containment area should be evaluated. Although vegetation may be beneficial because it helps dewater dredged material by transpiration and may improve the effluent quality by filtering, very dense vegetation may severely reduce the available storage capacity of the containment area and may restrict the flow of dredged slurry throughout the area, causing short-circuiting. Irregular topography within the containment area will directly affect resulting topography of the dredged material surface following the dredging operation. It may be beneficial to grade existing topography from planned inlet locations toward the weir locations to facilitate drainage of the area.

b. Use of Existing Dredged Material. If dikes must be strengthened or raised to provide adequate storage capacity for the next lift of dredged material, the use of the dried dredged material or suitable construction material from within the containment for this purpose will be beneficial. In addition to eliminating the costs associated with the acquisition of borrow, additional storage capacity is generated by removing material from within the area. Consideration should also be given to the use of any coarse-grained material present from previous dredging operations for underdrainage blankets or for other planned applications requiring more select material.

c. Placement of Weirs and Inflow Points.

(1) General placement for site operation and management control. Outflow weirs are usually placed on the site perimeter adjacent to the water or at the point of lowest elevation. The dredge pipe inlet is usually located as far away as practicable from these outflow weirs or at a location closest to the dredging areas. However, these objectives may sometimes be conflicting. If the disposal area is large or if it has irregular foundation topography, considerable difficulty may be encountered in properly distributing the material throughout the area and obtaining the surface elevation gradients necessary for implementation of a surface trenching program. One alternative is to use interior or cross dikes to subdivide the area and thus change the large

area into several smaller areas. Effective operation may require that the dredge pipe location be moved periodically from one part of the site to another, to ensure a proper filling sequence and obtain proper surface elevation gradients. Also, shifting inflow from one point of the site to another and changing outflow weir location may facilitate obtaining a proper suspended solids concentration in disposal site effluent.

(2) Installation and operation of multiple outflow weirs. In conjunction with provisions for moving the inflow point over the disposal site, it may also be worthwhile to contemplate installation of more outflow weirs than would be strictly required by design methods. Availability of more outflow points allows greater flexibility in site operation and subsequent drainage for dewatering, as well as greater freedom in movement of dredge inflow points while still maintaining the flow distances required to obtain satisfactory suspended solids concentrations in disposal site effluent. Also, a higher degree of flexibility in both disposal site inflow and outflow control will allow operation of the area in such a manner that desired surface topography can be produced, facilitating future surface trenching operations.

d. Interior Dike Construction.

(1) Need for interior dike construction. The basic rationale behind the construction of interior disposal area dikes is to subdivide the area into more manageable segments and/or to control the flow of dredged material through the disposal area. Control of material placement is normally to facilitate future disposal site operations, such as dewatering, or to provide proper control of disposal area effluent. Interior dikes may also be used as a haul road and access for movement of material for dike construction or other beneficial uses.

(2) Economics of interior dike construction. As a general rule, the use of interior cross dikes in any disposal area will increase the initial cost of construction and may result in increased operating costs. However, facilitation of disposal site operations, particularly future dewatering, may result in a general reduction in unit disposal cost over the life of the site. The benefit derived from dikes should be evaluated against the amount of disposal volume required for their construction. If the dikes can be constructed from dredged material or material available in the disposal site foundation and subsequently raised with dewatered dredged material, the net decrease in storage capacity will be approximately zero.

(3) Disposal site operation using subareas in series.

(a) Cross dikes may be used to control and direct the inflow and are normally built to allow site subcontainment area (subarea) operation either in series or in parallel. In series, the flow is routed first into one subarea, with sedimentation producing segregation of larger particles, and the overflow from the first subarea is routed to a second subarea where finer particles fall from suspension and then perhaps into another subarea, etc., with the outflow point being located at the end of the last subarea. In some instances, cross dikes are built across the entire site width, and a long overflow weir is provided to allow outflow into the next subarea in the series. In other instances, spur dikes are built into the containment area to cause a twisting path for the flow.

(b) In general, the use of series-oriented subdisposal areas should be considered carefully, since the actual result of such use may be the opposite of that desired by the designer. During disposal, coarse-grained sand and gravel will settle very quickly around the disposal pipe location. Other material will remain in suspension, depending on its effective particle size, water salinity, and flow velocity. A subarea can be effective in separating coarse material in an area where later recovery for other use will be easier. As a practical matter, a subarea or containment basin to trap or separate specific silt and clay sizes is rather impractical. A rational design for a series of subareas might require an initial subarea to trap sand and gravel, with the remainder of the material, i.e., the fine-grained fraction, going to a larger subarea. Then, if desired, a final subarea could be used for retention of fine material in conjunction with use of chemical flocculants, to maintain proper water quality in the disposal area effluent. When designing the series of subdisposal areas, care must be taken to obtain adequate size. If the first subarea in the series is filled, it will no longer function and provide the required residence time, and its function must be assumed by the next unit in the series.

(4) Disposal site operation using subareas in parallel. To facilitate site dewatering, operation of interior compartments on a parallel basis may be used. In this concept, flow is initially routed into one compartment; then, when it is filled to the proper depth or when suspended solids concentration standards in the effluent are exceeded, the flow is routed to another portion of the site. This procedure allows more carefully controlled placement of material to the desired thickness throughout the site. Parallel compartments also allow more efficient drying to occur in compartments not in active use since the water ponded for sedimentation is confined to the active compartment (see Figure 7-1).

(5) Sequential dewatering operations. If the disposal site is large enough to contain material from several periodic dredgings, each compartment may be used sequentially for a separate operation. In this manner, a sequence such as the following may be developed. The first compartment is filled, and, after decant, dewatering operations are initiated. As dewatering operations proceed, the next disposal is placed in the second compartment and subsequent disposal in the third, etc. While fresh material is being deposited in part of the site, the dewatered material from the initial placement may be borrowed and used to raise perimeter dikes, facilitating reuse of the initial subarea. This sequence of operations is shown in Figure 7-2.

e. Improvement of Site Access.

(1) Adequate provisions for site access are essential when the long-term operation and management plan for a disposal site includes provision for future dewatering activities and/or removal of dewatered material for dike raising or other productive use. General considerations for site access may include:

(a) Access roads on or adjacent to perimeter and interior dikes.

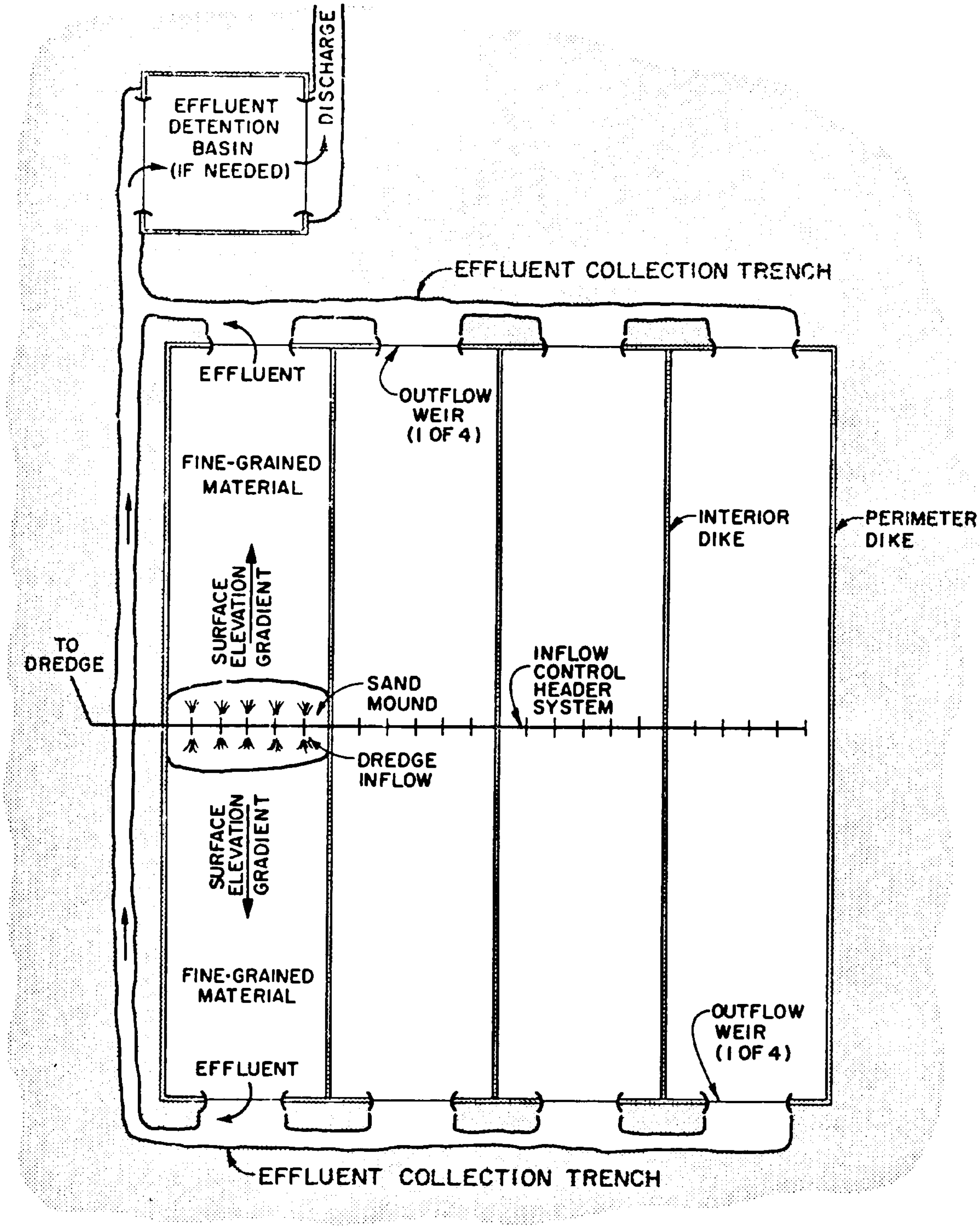


Figure 7-1. Conceptual illustration of disposal site layout to permit parallel compartment use and produce surface topography facilitating future dredged material dewatering

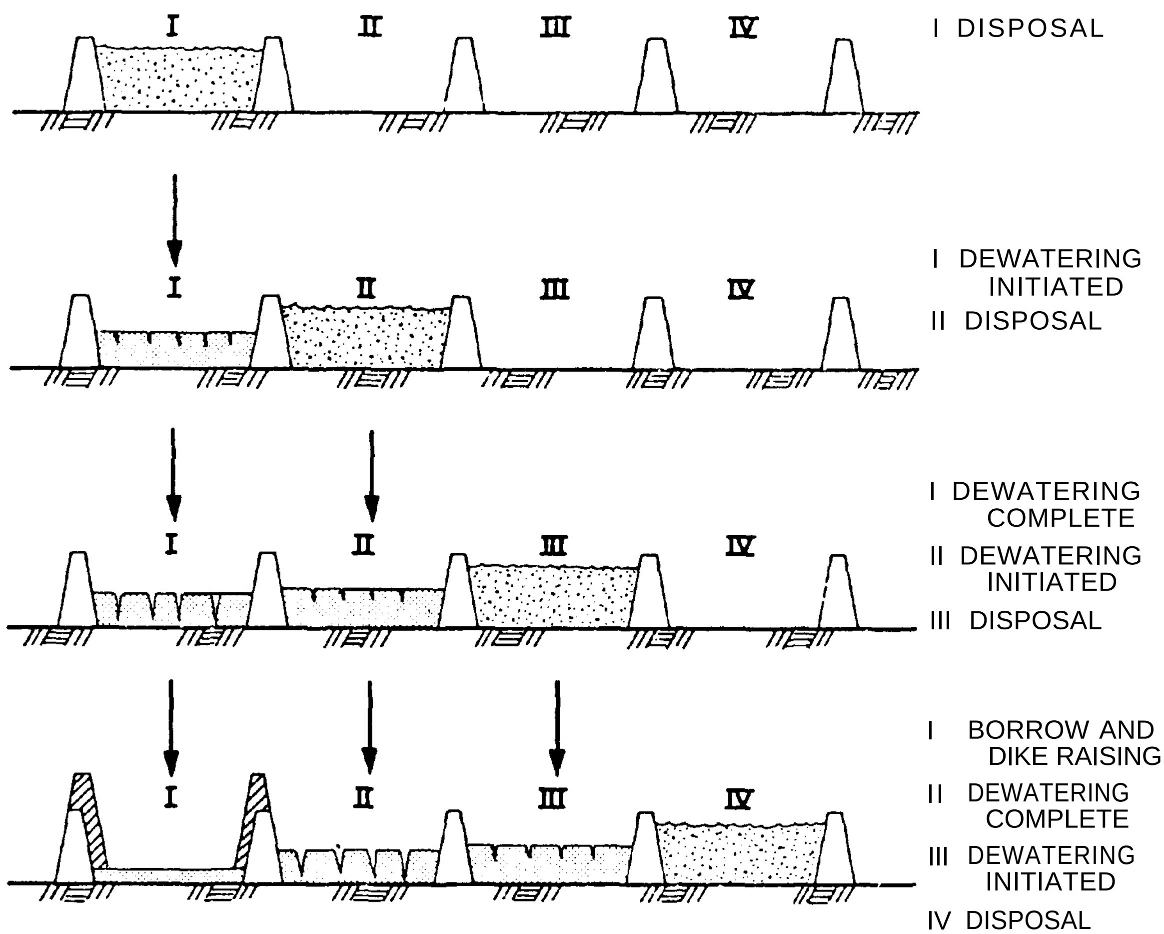


Figure 7-2. Conceptual illustration of sequential dewatering operations

(b) Crossing points on interior ditches used for drainage or dewatering.

(c) Access for equipment and personnel to reach weir structures for repair or maintenance.

(d) Ramps for access onto dikes from both inside and outside dike faces.

(e) Ramps for pipelines leading to inflow points.

(f) Equipment turnarounds.

(g) Stockpiles of materials for sandbagging and emergency dike repairs.

(h) Offloading ramps for equipment transported by water.

(2) If future borrow of interior dewatered dredged material is contemplated, it may be most cost-effective to construct small access roads into the area, as a substructure for future haul roads or dragline access. Such stable platforms may be covered with some fine-grained dredged material, but

their emplacement in the disposal area will allow subsequent equipment operation without immobilization.

f. Scheduling of Dredging Operations to Take Maximum Advantage of Climatic Conditions. Many nonengineering considerations affect the actual time during which disposal operations are conducted. They include:

(1) Expenditure of funds with respect to fiscal year.

(2) Relative priority of the operation with respect to other work.

(3) Lag time necessary to obtain proper specifications preparation and contract advertisement.

(4) Variation in time when the contractor must move on the job.

(5) Size of dredge.

(6) Existing weather conditions.

(7) Environmental considerations (i.e., dredging windows).

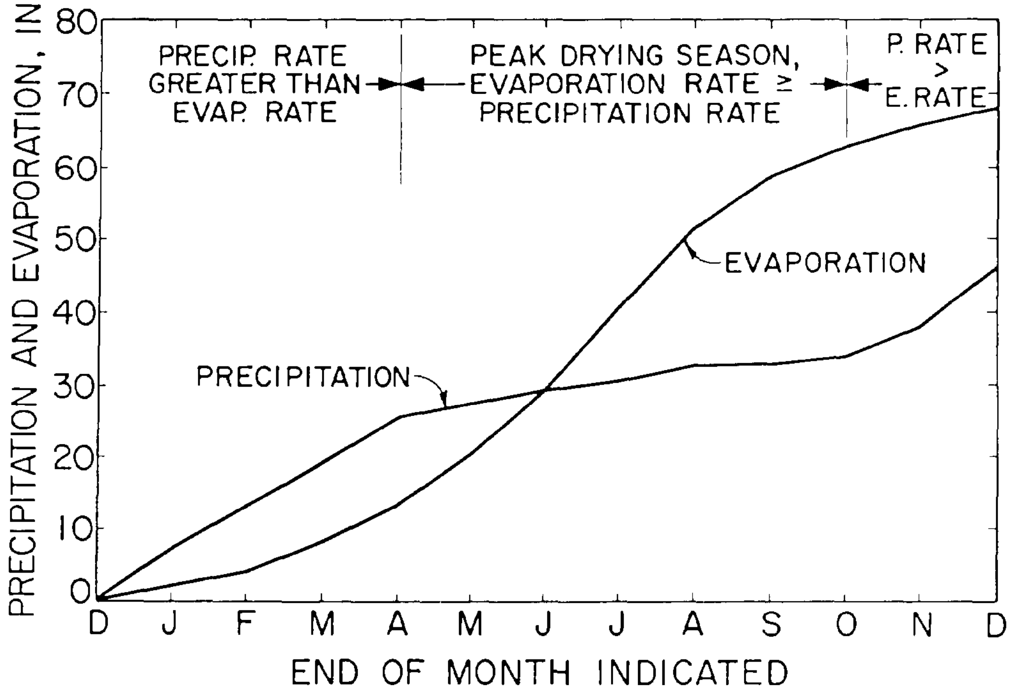
(8) Lag time required for preparation of the disposal site.

Nevertheless, considerable advantage may be gained, in an engineering sense, from scheduling disposal operations to occur at appropriate periods of the calendar year, depending upon prevailing climatic conditions. By conducting the disposal phase during a period of relatively low evaporative demands, the initial postdisposal activity (i.e., decanting and gradual reduction of ponded water depth) will occur when minimum evaporative forces are available for dewatering. If the disposal operation can be scheduled so that the material reaches the approximate decant-point water content when seasonal evaporation rates begin to be maximized, evaporative dewatering will be facilitated. Dramatic results can occur over short time periods when conditions are prime for drying. Estimation of the calendar period for optimum evaporation, based on projected climatic conditions, is illustrated in Figure 7-3. Examples are from the San Francisco, California, and Mobile, Alabama, areas. If possible, disposal operations should be terminated, ponded water removed, and the material sedimented/consolidated to the decant point by the time (calendar month) when the evaporation rate begins to increase.

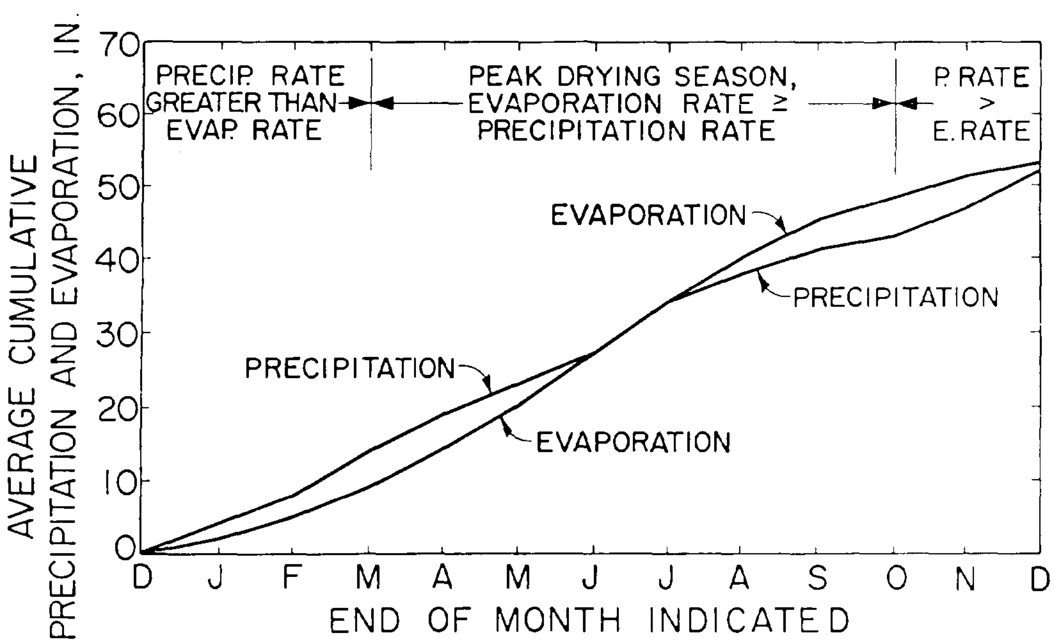
7-3. Management During Disposal.

a. Surface Water Management.

(1) The management of surface water during the disposal operation can be accomplished by controlling the elevation of the outlet weir(s) throughout the disposal operation to regulate the depth of water ponded within the containment area. Proper management of surface water is required to ensure containment area efficiency and can provide a means for access by boat or barge to the containment area interior.



a. San Francisco, California, area



b. Mobile, Alabama, area

Figure 7-3. Illustrations of method for estimating calendar periods when evaporation rates are maximized

(2) At the beginning of the disposal operation, the outlet weir is set at a predetermined elevation to ensure that the ponded water will be deep enough for settling as the containment area is being filled. As the disposal operation begins, slurry is pumped into the area; no effluent is released until the water level reaches the weir crest elevation. Effluent is then released from the area at about the same rate as slurry is pumped into the area. Thereafter, the ponding depth decreases as the thickness of the dredged material deposit increases. After completion of the disposal operation and the activities requiring ponded water, the water is removed as quickly as effluent water quality standards will allow. Figure 7-4 illustrates the concept.

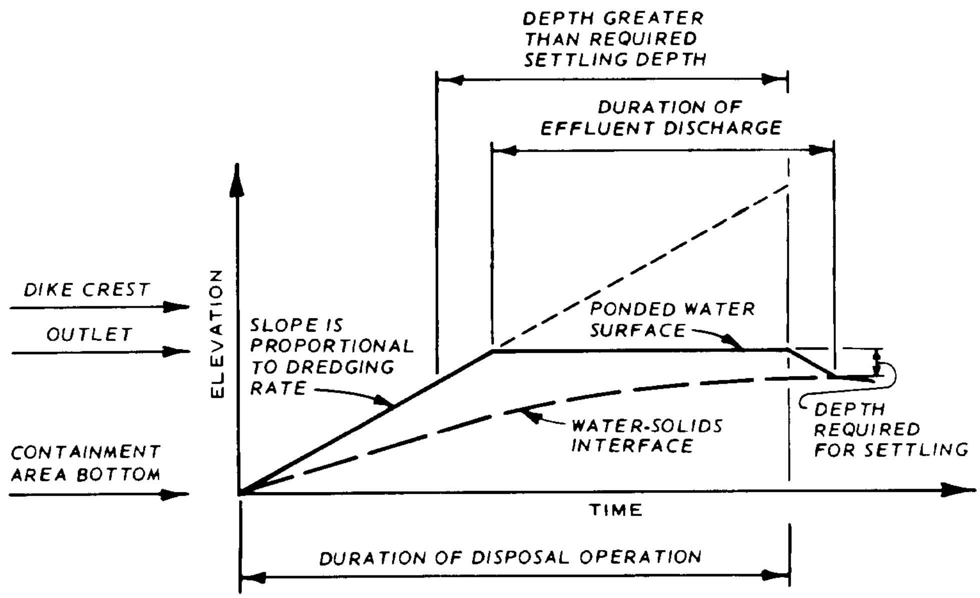


Figure 7-4. Surface water management

b. Suspended Solids Monitoring. A well-planned monitoring program during the entire dredging and decanting operation is desirable to ensure that effluent suspended solids remain within acceptable limits or to verify conditions for future design or site evaluations. Since suspended solids concentrations are determined on a grams per litre basis requiring laboratory tests, it is desirable to complete a series of laboratory tests during the initial stages of operation. Indirect indicators of suspended solids concentration, such as visual comparison of effluent samples with samples of known concentration or utilization of a properly calibrated instrument, may then be used during the remainder of the operation, supplemented with laboratory determination of effluent solids concentrations as needed for record purposes.

(1) Samples of both inflow and outflow can be taken for laboratory tests. The solids determination should be made on the samples using the procedure described in Chapter 3.

(2) When the dredging operation commences, samples should be taken from the inlet pipe at approximately 12-hour intervals to verify design assumptions. Effluent quality samples should be taken periodically at approximately

6-hour intervals during the dredging operation for laboratory solids determinations to supplement visual estimates of effluent suspended solids concentrations. The sampling interval may be changed based on the observed efficiency of the containment area and the variability of the effluent suspended solids concentrations. More frequent sampling will be necessary as the containment area is filled and effluent concentrations increase.

c. Inlet and Weir Management.

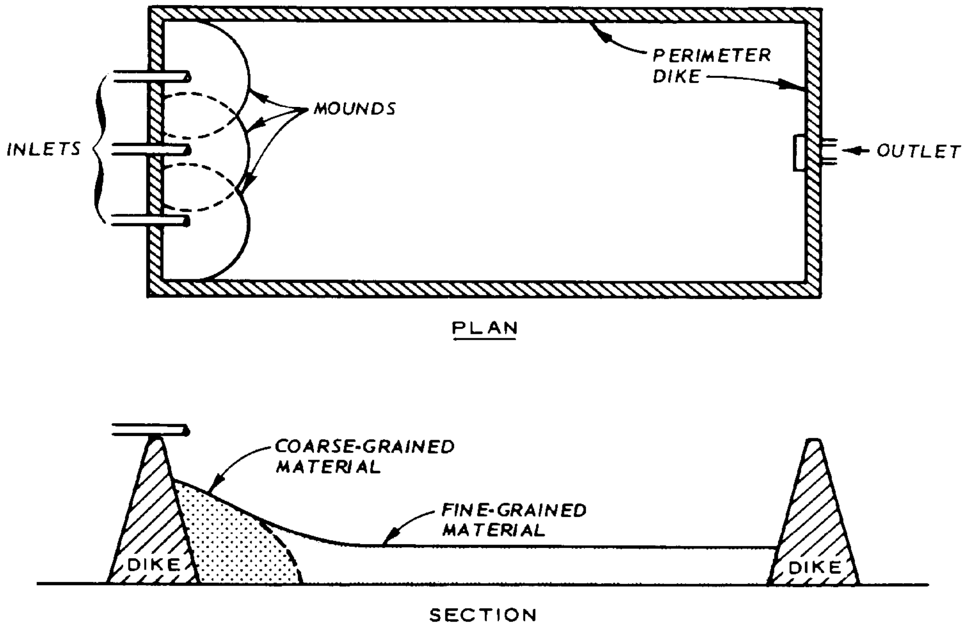
(1) If multiple weirs are used, discharging the weirs alternately is sometimes useful for preventing short-circuiting. As the area between the inlet and one outlet fills or as the inlet location is moved, the flow may channelize in a more or less direct route from inlet to weir. If this occurs, the flow should be diverted to another weir. Simultaneous discharge of slurry from several inlets located on the perimeter can also be advantageous, because the lower velocity of the slurry flow results in more pronounced mounding around the edge of the containment area. This mounding in turn increases the slope from inlet to outlet, and drainage will be improved.

(2) The removal of water following the dredging operation can be somewhat expedited by managing inlets and weirs during the disposal operation to place a dredged material deposit that slopes continually and as deeply as practical toward the outlets. Figure 7-5 shows a containment area with a weir in one end and an inlet zone in the opposite end. Inlets are located at various points in the inlet zone, discharging either simultaneously (multiple inlets) or alternately (single movable inlet or multiple inlets discharging singly). A common practice is to use a single inlet, changing its location between disposal operations. The result of this practice is the buildup of several mounds, one near each inlet location. By careful management of the inlet locations, a continuous line of mounds can be constructed, as shown in Figure 7-5. When the line of mounds is complete, the dredged material will slope downward toward the weir. If the mound area is graded between disposal operations, the process can then be repeated by extending the pipe over the previous mound area and constructing a new line of mounds, as shown in Figure 7-5.

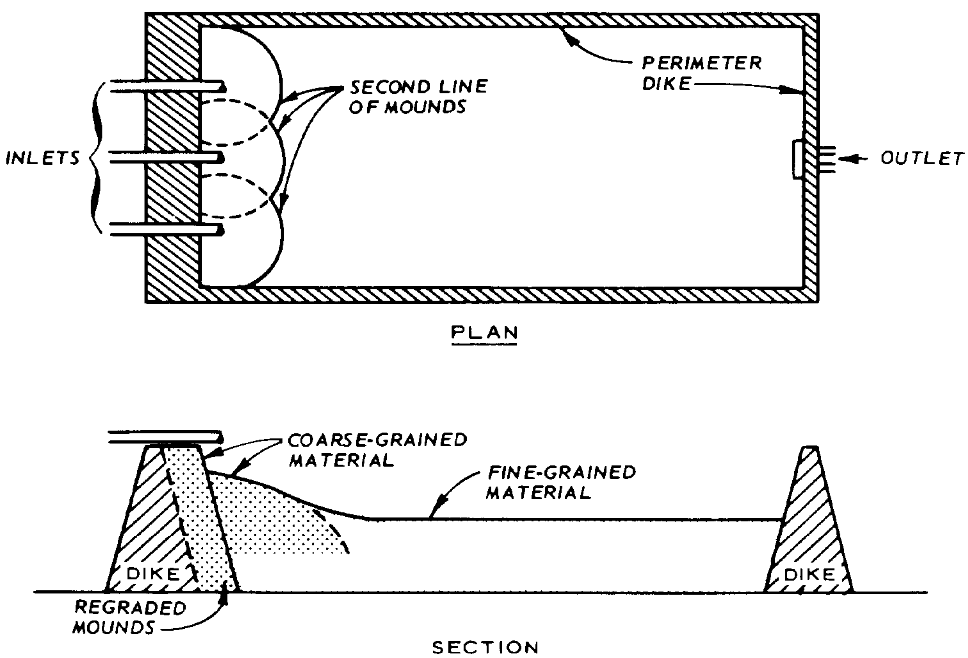
d. Thin-Lift Placement of Dredged Material. Gains in long-term storage capacity of containment areas through natural drying processes can be increased by placing the dredged material in thin lifts. Thin-lift placement also greatly enhances potential gains in capacity through active dewatering and disposal area reuse management programs.

(1) One approach to placing dredged material in thin lifts is to obtain sufficient land area to ensure adequate storage capacity without the need for thick lifts. Implementation of this approach requires careful long-range planning to ensure that the large land area is used effectively for dredged material dewatering, rather than simply being a containment area whose service life is longer than that of a smaller area.

(2) Large containment areas, especially those used nearly continuously, are difficult to manage for effective natural drying of dredged material. The practice of continuous disposal does not allow sufficient time for natural drying. However, dividing a large containment area into several compartments



a. FIRST LINE OF MOUNDS



b. SECOND LINE OF MOUNDS

Figure 7-5. Inlet-weir management to provide smooth slope for inlet to weir can facilitate operation because each compartment can be managed separately so that some compartments are being filled while the dredged material in others is being dewatered.

(3) One possible management scheme for large compartmentalized containments is shown conceptually in Figure 7-2. For this operation, thin lifts of dredged material are sequentially placed into each compartment. The functional sequence for each compartment consists of filling and settling, and surface drainage and dewatering, and dike raising (using dewatered dredged material). The operation must be designed to include enough compartments to ensure that each thin lift is dried before the next lift is placed.

7-4. Postdredging Management Activities.

a. Periodic site inspections and continuous site management following the dredging operation are desirable. Once the dredging operation has been completed and the ponded water has been decanted, site management efforts should be concentrated on maximizing the containment storage capacity gained from continued drying and consolidation of dredged material and foundation soils. To ensure that precipitation does not pond water, the weir crest elevation must be kept at levels allowing efficient release of runoff water. This will require periodic lowering of the weir crest elevation as the dredged material surface settles.

b. Removal of ponded water will expose the dredged material surface to evaporation and promote the formation of a dried surface crust. Some erosion of the newly exposed dredged material may be inevitable during storm events; however, erosion will be minimized once the dried crust begins to form within the containment area.

c. Natural processes often need man-made assistance to effectively dewater dredged material since dewatering is greatly influenced by climate and is relatively slow. When natural dewatering is not acceptable for one reason or another, then additional dewatering techniques should be considered.

d. Removal of coarse-grained material and dewatered fine-grained material for productive uses through Disposal Area Reuse Management (DARM) techniques will further add to capacity and may be implemented in conjunction with dike maintenance or raising. In the case of fine-grained dredged material, DARM is a logical follow-up to successful dewatering management activities. This concept has been successfully used by CE Districts and demonstrated in field studies. Guidelines for determining potential benefits through DARM are found in WES Technical Report DS-78-12 (item 24). Additional information on productive uses of dredged material is found in EM 1110-2-5025.

7-5. Long-Term Management Plans for Containment Areas.

a. Adequate dredged material disposal areas are becoming increasingly difficult to secure in many areas of the country. For this reason, it is necessary that the remaining resources of confined disposal sites be properly utilized and managed. A management plan is a vehicle that can be used to assure the most effective use of containment in future years.

b. The following objectives would normally be set in the plan development:

(1) Maximize volumetric disposal capacity.

(2) Dewater and densify fine,-grained material to the greatest extent feasible.

(3) Reclaim and remove useable material for productive use.

(4) Maintain acceptable water quality of effluent.

(5) Abide by all legal and policy and easement constraints.

c. Development of a management plan should include an extensive evaluation of management alternatives based on data accumulated through field investigations and laboratory testing. Integration of the disposal plan with overall navigation system needs is essential. The plan should be developed using the latest available technical approaches for evaluation of the benefits of management practices. A management plan developed for the Craney Island disposal area in the Norfolk District (item 27) is a well-documented example that illustrates how the procedures described in this manual can be used in developing management approaches.

d. A working group or management plan committee is an effective means to ensure that the plan benefits from the input of all District elements. The committee would logically be composed of representatives from Planning, Engineering, and Operation elements, Once a management approach is selected, a monitoring program should be initiated for use in evaluating the effectiveness of management techniques, especially dewatering activities. A monitoring program serves to verify benefits attained and to form a basis for updating or modifying the management approaches.

CHAPTER 8

AUTOMATED DESIGN AND MANAGEMENT PROCEDURES

8-1. Need for Automated Procedures.

a. In many of the analyses described in this manual, tedious repetitive calculations for alternative designs and analysis of the design sensitivity to various parameters are required to answer the many "what if" questions which arise. These repetitive calculations are naturally conducive to computerization to allow the evaluation of more alternatives and more detailed sensitivity analyses.

b. The blending of the engineering techniques for dredging design and management with the computerized approach resulted in a computer program called the Automated Dredging and Disposal Activities Management System (ADDAMS). This is a centralized program containing different computerized modules and an associated data management system. In creating ADDAMS, the developers agreed that the program must be easy-to-use, easy-to-modify, internally consistent, and well documented.

c. ADDAMS is set up so that users do not need to be computer experts to run the program. Logging into the computer is the most sophisticated step in using the program. Once inside ADDAMS, the user is led through the program with the aid of keywords and menus. ADDAMS has a data-base management system that can save and update a user's data from one run to the next, but in ADDAMS the system is essentially transparent to the user. All the user needs to do is assign a file name to the data file.

8-2. Current Status.

a. The ADDAMS program now performs a large number of different functions. The program, however, is modular in that the user need learn only that portion of the program needed to accomplish a given task. Current modules now available in ADDAMS include those related to short-term sizing (Chapters 3 and 4), long-term sizing (Chapter 5), a disposal area sequencing model, and other modules related to disposal area design and cost-estimating. Figure 8-1 is a schematic showing how the modules are related through an executive program that controls the overall program and manages the data. Another benefit of the modular nature of the program is that it is fairly easy to add new features or upgrade old modules. It is even possible to maintain old and new versions of a given module data-base update. Since ADDAMS will continually be improved and upgraded, as any often-used computer program, it is highly desirable to upgrade the program in one aspect without affecting other program features.

b. The ADDAMS program is currently running and available for CE users on the CDC Cybernet system. The user's guide and documentation are available in draft form (item 19). When these documents have been published, the program will be made available to the public through the Engineer Computer Program Library at WES. The program is being updated regularly. As it is applied to various studies, those using the program are identifying areas that can be upgraded, and the developers are incorporating these suggestions into the program. This should make the program more flexible as well as more relevant to real world problems.

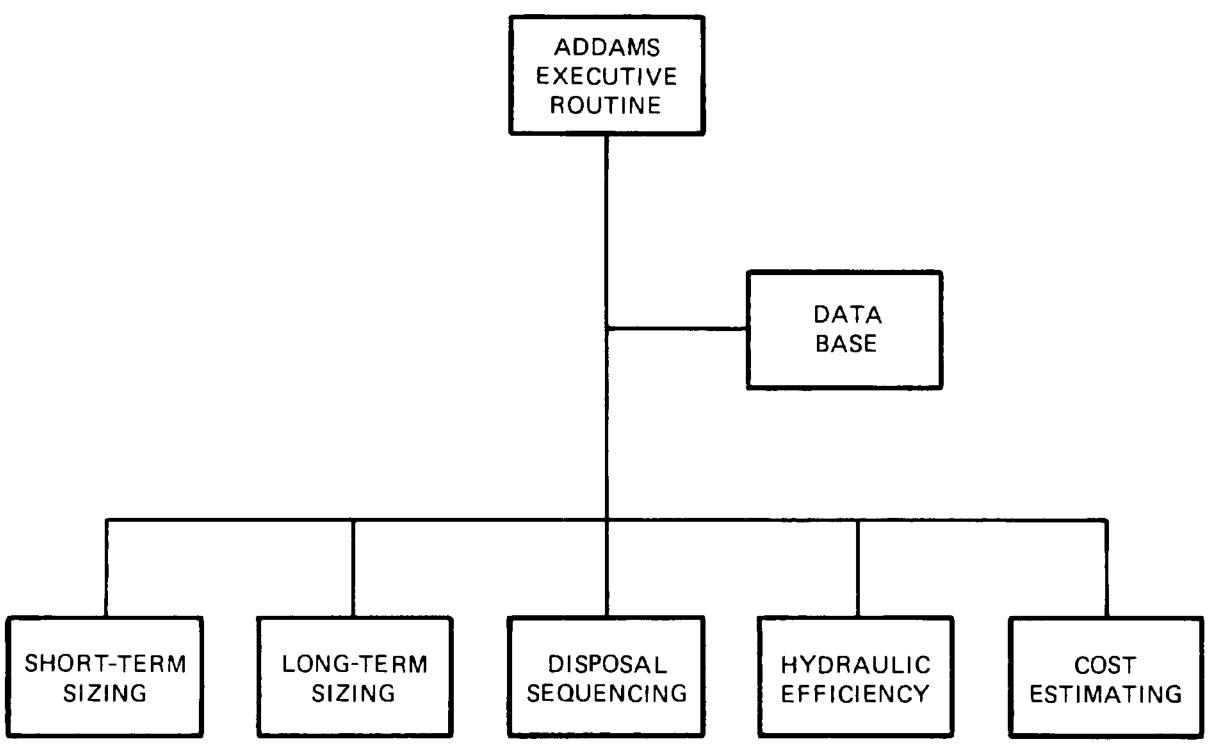


Figure 8-1. Schematic of current ADDAMS program

APPENDIX A

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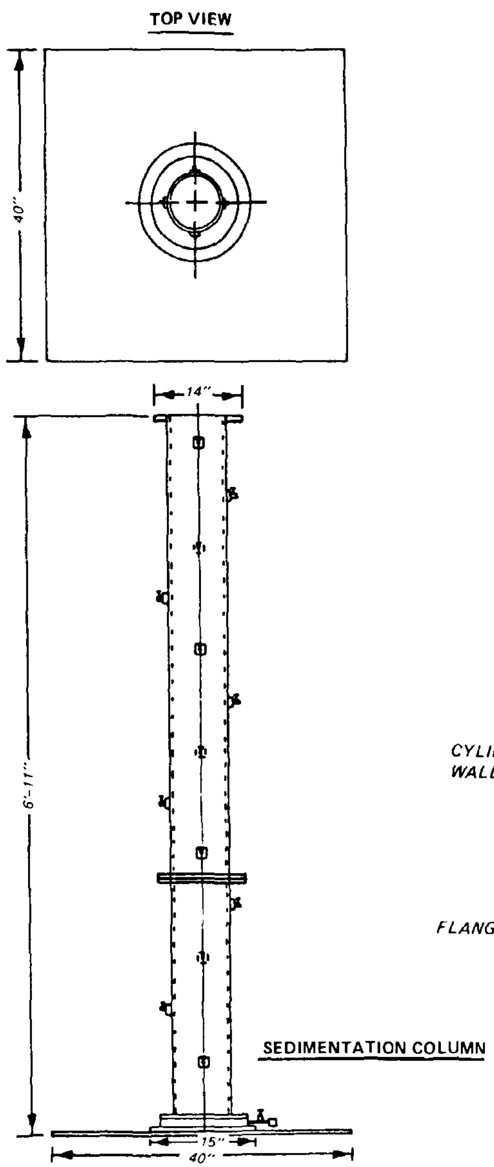
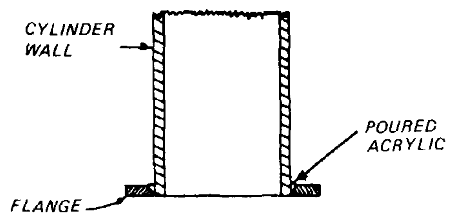
APPENDIX B

PLANS AND SPECIFICATIONS FOR SETTLING COLUMN





BILL OF MATERIAL		
QTY	MATERIAL	SIZE
1	AIR COUPLER	1/4" I.D.
1	BALL VALVE (STAINLESS STEEL)	1/4" I.D.
1	BALL VALVE (STAINLESS STEEL)	3/4" I.D.
13	SAMPLING VALVE ASSEMBLIES (STAINLESS STEEL)	3/8" I.D.
2	PIPE (STAINLESS STEEL)	1/4" I.D., 2 " LONG
1	SINTERED STEEL FILTER DISK 50-80 MICRON PORE DIAMETER	1/4" THICK 8.23 DIAMETER
1	MATERIAL FOR AT LEAST 2-10" DIAMETER O-RINGS	1/4" DIA. 12' LONG
1	WATERPROOF PLYWOOD	4' x 4' x 1/2"
1	WATERPROOF PLYWOOD PLATE FOR COLUMN BASE	16" DIAMETER
8	HEX HEAD BOLT W/WASHER AND NUT	1/4"-20 UNC x 1-1/2"
8	CARRIAGE HEAD BOLT W/WASHER AND NUT	1/4" x 3"
14	PLEXIGLASS REINFORCEMENT FOR COLUMN HOLES	3" x 3" x 1/4"
2	PLEXIGLASS FLANGE FOR BOTTOM COLUMN	1/2" THICK, 14" O.D. 8-1/2" I.D.
2	PLEXIGLASS FLANGE FOR TOP COLUMN	1/2" THICK, 14" O.D. 8-1/2" I.D.
1	PLEXIGLASS PLATE FOR COLUMN BOTTOM	1" THICK, 14" DIA
1	PLEXIGLASS CYLINDER	27" LONG, 1/4" THICK 8" I.D., 8-1/2" O.D.
1	PLEXIGLASS CYLINDER	54" LONG, 1/4" THICK 8" I.D., 8-1/2" O.D.

SUPPLIES	
MATERIAL	COMPANY
FILTER DISK	MOTT METALLURIGAICAL CORPORATION FARMINGTON INDUSTRIAL PARK FARMINGTON, CT 06032
O-RING	MISSISSIPPI RUBBER AND SPECIALTY COMPANY 819 SOUTH CONGRESS JACKSON, MS 39201
PLEXIGLASS	CADILLAC PLASTIC COMPANY/FAULKNER PLASTICS, INC. 4504 E. HILLSBORO AVE. P.O. BOX 1126 TAMPA, FL 33601

Figure B-1. Specifications for settling column and plan for sedimentation column

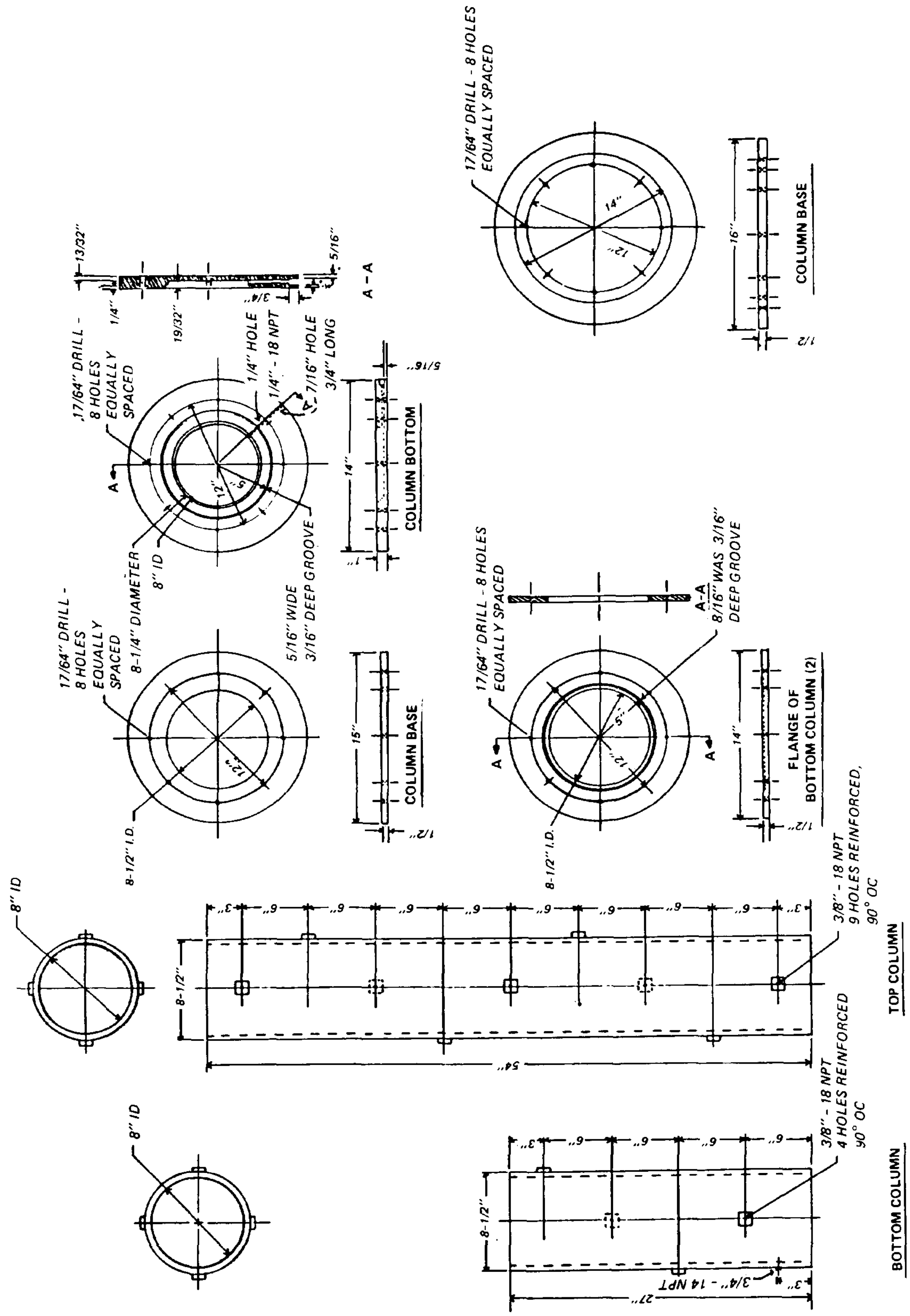


Figure B-2. Plans for top and bottom columns

APPENDIX C

EXAMPLE DESIGN CALCULATIONS FOR RETENTION OF SOLIDS AND INITIAL STORAGE

C-1. General. This appendix presents example calculations for containment area designs for the retention of suspended solids and initial storage. The examples are presented to illustrate the use of field and laboratory data and include designs for sedimentation, weir design, and requirements for initial storage capacity. Only those calculations necessary to illustrate the procedure are included in the examples.

C-2. Example I: Containment Area Design Method for Sediments Exhibiting Flocculent Settling.

a. Project Information.

(1) Each year an average of 300,000 cubic yards of fine-grained channel sediment is dredged from a harbor. A new in-water containment area is being constructed to accommodate the long-term dredged material disposal needs in this harbor. However, the new containment area will not be ready for approximately 2 years. One containment area in the harbor has some remaining storage capacity, but it is not known whether the remaining capacity is sufficient to accommodate the immediate disposal requirements. Design procedures must be followed to determine the residence time needed to meet effluent requirements of 4 grams per litre and the storage volume required for the 300,000 cubic yards of channel sediment. These data will be used to determine if the existing containment area storage capacity is sufficient for the planned dredged material disposal activity. The existing containment area is about 3 miles from the dredging activity.

(2) Records indicate that for the last three dredgings, an 18-inch pipeline dredge was contracted to do the work. The average working time was 17 hours per day, and the dredging rate was 600 cubic yards of in situ channel sediment per hour. The project depth in the harbor is 50 feet.

b. Results of Containment Area Survey. The existing containment area has the following dimensions:

(1) Size: 96 acres.

(2) Shape: length-to-width ratio of about 3.

(3) Volume: 1,548,800 cubic yards (average depth, from surveys, is 10 feet).

(4) Weir length: 24 feet (rectangular weir).

(5) Minimum ponding depth: 2 feet (assumed).

c. Results of Laboratory Tests and Analysis of Data. Sediment and dredging-site water characterization was conducted as described in Chapter 3.

A pilot settling test was conducted, and no interface was observed during the first 4 hours of the test. An 8-inch column test was then run to determine flocculent and compression settling properties. The following data were obtained from the laboratory tests:

(1) Salinity of dredging site water: <1 part per thousand.

(2) Channel sediment in situ water content w : 85 percent.

(3) Specific gravity G_s : 2.69.

(4) Grain size analysis indicates approximately 20 percent of the sediment is coarse grained.

(5) Observed flocculent settling concentrations as a function of depth (see Table C-l).

(6) Percent of initial concentration with time (see Table C-2). This is determined as follows:

(a) Column concentration at the beginning of tests is 132 grams per litre.

(b) Concentration at l-foot level at time = 30 minutes is 46 grams per litre (Table C-l).

(c) Percent of initial concentration = 46 ÷ 132 = 0.35 = 35 percent.

(d) These calculations are repeated for each time and depth to develop Table C-2.

(7) Plot the percent of initial concentration versus the depth profile for each time interval from data given in Table C-2 (see Figure C-1).

(8) Determine concentration as a function of time (15-day settling column data) (see Table C-3).

(9) Plot time versus concentration from data in Table C-3 as shown in Figure C-2.

d. Design Concentration. Compute the design concentration as follows:

(1) The project information is:

(a) Dredge size: 18 inches.

(b) Volume to be dredged: 300,000 cubic yards.

(c) Average operating time: 17 hours per day.

(d) Production: 600 cubic yards per hour.

(2) Estimate the time of dredging activity:

Table C-1
Observed Flocculent Settling Concentrations with Depth, in Grams per Litre*

Time, min	Depth from Top of Settling Column, ft 1	2	3	4	5	6	7
0	132.0	132.0	132.0	132.0	132.0	132	132
30	46.0	99.0	115.0	125.0	128.0	135	146
60	25.0	49.0	72.0	96.0	115.0	128	186
120	14.0	20.0	22.0	55.0	78.0	122	227
180	11.0	14.0	16.0	29.0	75.0	119	
240	6.8	10.2	12.0	18.0	65.0	117	
360	3.6	5.8	7.5	10.0	37.0	115	
600	2.8	2.9	3.9	4.4	14.0	114	
720	1.01	1.6	1.9	3.1	4.5	110	
1,020	0.90	1.4	1.7	2.4	3.2	106	
1,260	0.83	1.14	1.2	1.4	1.7	105	
1,500	0.74	0.96	0.99	1.1	1.2	92	
1,740	0.63	0.73	0.81	0.85	0.94	90	

* Note: Although a 6-foot test depth is recommended, an 8-foot depth was used in this test.

Table C-2

Percent of Initial Concentration with Time*

Time T, min	Depth from Top of Settling Column, ft 1	2	3
0	100.0	100.0	100.0
30	35.0	75.0	87.0
60	19.0	37.0	55.0
120	11.0	15.0	17.0
180	8.0	11.0	12.0
240	5.0	8.0	9.0
360	3.0	4.0	6.0
600	2.0	2.2	3.0
720	1.0	1.2	1.4

* Note: Initial suspended solids concentration = 132 grams per litre.

Table C-3

Concentration of Settled Solids as a Function of Time

Time days	Concentration g/ℓ
1	190
2	217
3	230
4	237
5	240
6	242
7	244
9	249
10	247
15	256

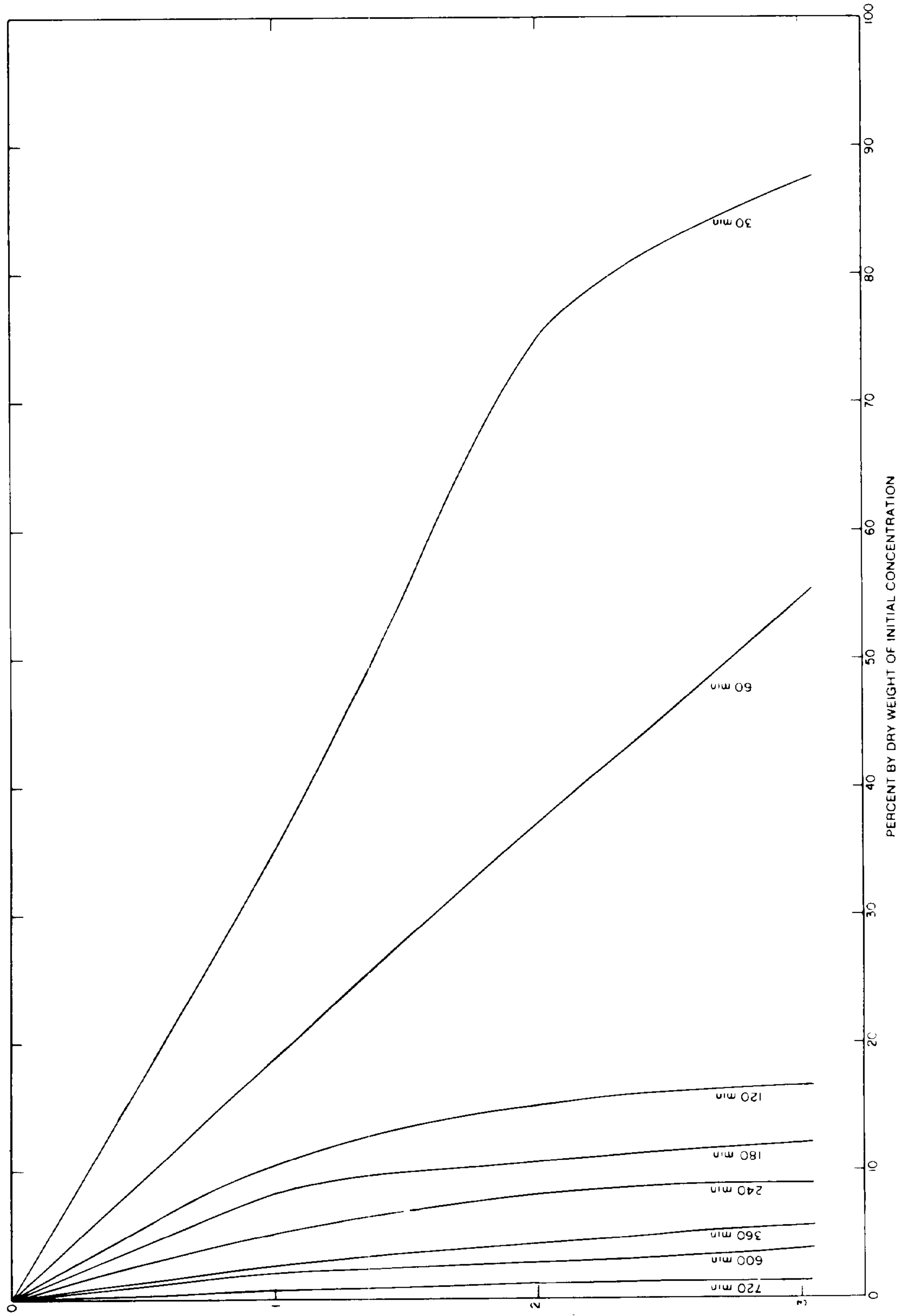


Figure C-1. Percent of initial concentration versus depth profile

$$\frac{300{,}000\ \text{yd}^3}{600\ \text{yd}^3/\text{hr}} = 500\ \text{hr}$$

$$\frac{500\ \text{hr}}{17\ \text{hr/day}} = 29.4 \quad 30\ \text{days}$$

(3) Average time for initial dredged material consolidation is:

$$\frac{30\ \text{days}}{2} = 15\ \text{days}$$

(4) Design solids concentration C_d is the concentration shown in Figure C-2 at 15 days:

$$C_d = 253\ \text{grams per litre}$$

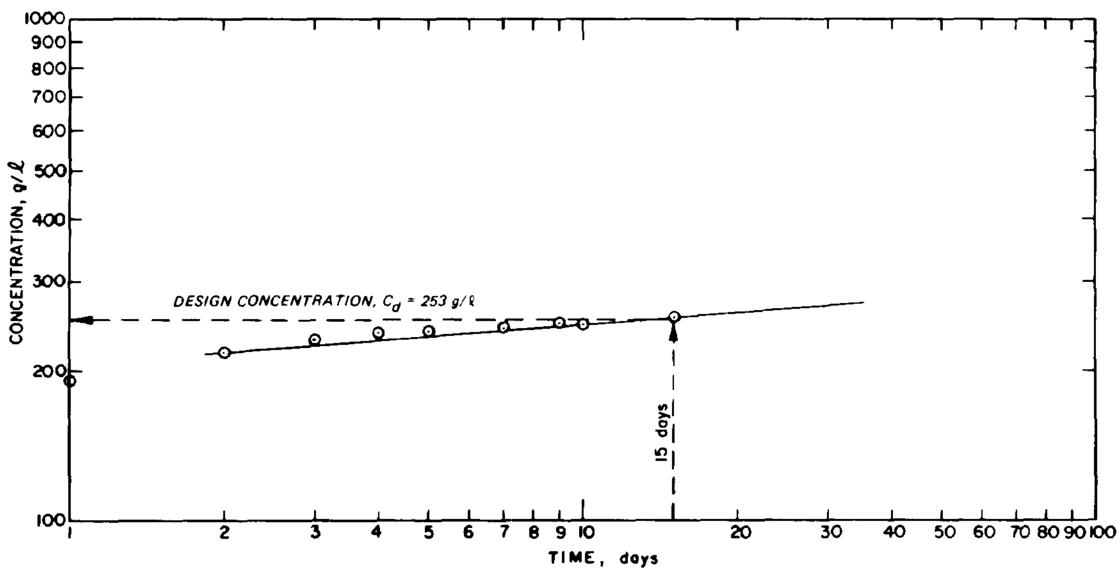


Figure C-2. Time versus concentration

e. Volume Required for Dredged Material. Estimate the volume required for dredged material as follows:

(1) Compute the average void ratio e_o using Equation 4-2:

$$e_o = \frac{G_s \gamma_w}{C_d} - 1$$

where $G_s = 2.69$, $\gamma_w = 1{,}000$ grams per litre, and $C_d = 253$ grams per litre. Thus,

$$e_o = \frac{2.69(1{,}000)}{253} - 1$$

$$e_o = 9.63$$

(2) Laboratory tests indicate that 20 percent of the sediment is coarse-grained material; therefore, the volume of coarse-grained material V_{sd} is

$$V_{sd} = 300{,}000(0.20) = 60{,}000 \text{ cubic yards}$$

and the volume of fine-grained material V_i is:

$$V_i = 300{,}000 - 60{,}000 = 240{,}000 \text{ cubic yards}$$

(3) Compute the volume of fine-grained channel sediments after disposal in the containment area using Equation 4-3:

$$V_f = V_i \left[\frac{e_o - e_i}{1 + e_i} + 1\right]$$

$$e_i = \frac{wG_s}{S_D}$$

$$= \frac{(85/100)(2.69)}{1.00}$$

$$e_i = 2.29$$

$$V_i = 240{,}000 \text{ cubic yards}$$

$$V_f = \left[\frac{9.63 - 2.29}{1 + 2.29} + 1\right](240{,}000)$$

$$V_f = 775{,}440 \text{ cubic yards}$$

(4) Estimate the total volume required in the containment area using Equation 4-4:

$$V = V_f + V_{sd}$$

$$V_{sd} = 60{,}000 \text{ cubic yards}$$

$$V = 775{,}440 + 60{,}000$$

$$V = 835{,}440 \text{ cubic yards}$$

(5) Determine the maximum height of dredged material. Foundation conditions limit dike heights to 10 feet. A ponding depth of 2 feet is assumed using Equation (4-4b):

$$H_{dm_{(max)}} = H_{dk_{(max)}} - H_{pd} - H_{fb}$$

$$H_{dm_{(max)}} = 10 \text{ feet} - 2 \text{ feet} - 2 \text{ feet}$$

$$H_{dm_{(max)}} = 6 \text{ feet}$$

(6) The minimum surface area that could be used must be compared to the available surface area of 96 acres. Using Equation 4-4c:

$$A_{ds_{(min)}} = \frac{V}{H_{dm_{(max)}}}$$

$$A_{ds_{(min)}} = \frac{835{,}440 \text{ yd}^3}{6 \text{ ft}} \times \frac{27 \text{ ft}^3}{\text{yd}^3}$$

$$A_{ds_{(min)}} = 3{,}759{,}480 \text{ ft} = \text{approximately } 86 \text{ acres}$$

Since the minimum required surface area is less than the available 96 acres, the dredged material can physically be stored during the dredging operation.

f. Residence Time Required for Sedimentation. The design residence time is computed as in the following example:

(1) Calculate removal percentages for the assumed ponding depth of 2 feet. Calculating the total area down to a depth of 2 feet from Figure C-l gives an area of 200 (scale units), Calculating the area to the right of the 30-minute time line down to a depth of 2 feet gives 124 (scale units). These areas could also have been determined by planimetering the plot. Compute removal percentages as follows (see Equation 4-7):

$$R = \frac{124}{200} \times 100 = 62$$

For a settling time of 30 minutes, 62 percent of the suspended solids are removed from the water column above the 2-foot depth.

(2) The calculations illustrated in step (1) are repeated for each time, and the results are tabulated in Table C-4.

(3) Plot the data in Table C-4 as shown in Figure C-3.

(4) Determine the mean residence time required to meet the 4-grams-per-litre effluent suspended solids requirements.

$$\text{Required Solids Removal} = \frac{C_i - C_{eff}}{C_i}$$

Table C-4

Removal Percentages as Function of Settling Time

Time, min	Removal, percentage
30	62.0
60	81.0
120	90.2
180	93.1
240	95.5
360	97.0
600	98.4
720	99.3

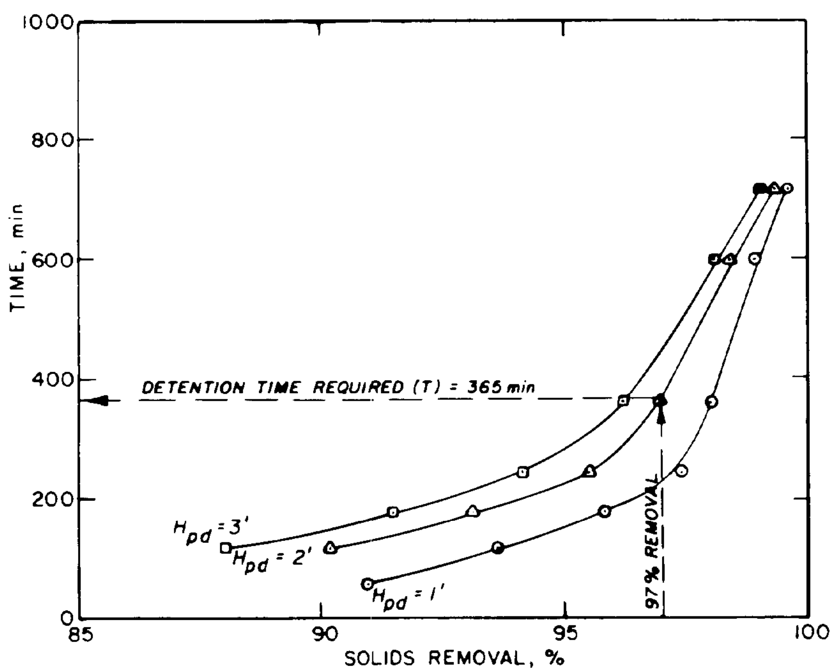


Figure C-3. Solids removal versus time

$$= \frac{132 - 4}{132} = 0.97 \text{ or } 97 \text{ percent}$$

(5) From Figure C-3, T = 365 minutes.

(6) No specific data on hydraulic efficiency exist for this site. Therefore, the hydraulic efficiency correction factor will be estimated using Equation 4-14.

$$\frac{T_d}{T} = 0.9 \left[1 - \exp\left(-0.3 \frac{L}{W}\right)\right]$$

$$= 0.9 \left\{1 - \exp\left[-0.3 (3)\right]\right\}$$

$$= 0.53$$

$$\text{HECF} = \frac{T}{d}$$

$$= \frac{1}{0.53}$$

$$= 1.87$$

$$T = \text{HECF} (T_d)$$

$$= 1.87 (365)$$

$$= 683 \text{ min}$$

The required theoretical or volumetric retention time equals 683 minutes or 11.4 hours.

g. Design Surface Area Required for Flocculent Sedimentation. Compute this value using Equation 4-13 as follows:

$$Q_i = \frac{\left(\frac{18 \text{ in.}}{12}\right)^2 \pi}{4} \times 15 \text{ ft/sec}$$

$$= 26.5 \text{ ft}^3\text{/sec}$$

$$A_{df} = \frac{T\,Q_i}{H_{pd}\,(12.1)}$$

$$= \frac{11.4\ (26.5)}{2\ (12.1)}$$

$$= 12 \text{ acres}$$

h. Design Surface Area. Since both the A_{ds} and A_{df} are smaller than the available 96 acres, use 96 acres as the design surface area A_d .

$$A_d = 96 \text{ acres} \times 43{,}560 \text{ ft}^2/\text{acre}$$

$$A_d = 4{,}181{,}760 \text{ ft}^2$$

i. Thickness of Dredged Material Layer. Determine the thickness of the dredged material layer from:

$$H_{dm} = \frac{V}{A_d}$$

$$= \frac{835{,}440 \text{ yd}^3 \times 27}{4{,}181{,}760 \text{ ft}^2}$$

$$H_{dm} = 5.4 \text{ ft}$$

j. Required Containment Area Depth (Dike Height). The required containment area depth is determined from:

$$H_{dk} = H_{dm} + H_{pd} + H_{fb}$$

$$= 5.4 + 2 + 2$$

$$H_{dk} = 9.4 \text{ feet}$$

D = 9.4 feet is less than the maximum allowable dike height of 10 feet.

k. Weir Length.

(1) The existing effective weir length L_e equals the weir crest length L for rectangular weirs:

$$L_e = 24 \text{ feet}$$

$$Q_i = 26.5 \text{ cubic feet per second}$$

$$H_{pd} = 2 \text{ feet}$$

Using Figure 4-7 from the main text, a 2-foot ponding depth at the weir requires an effective weir length of approximately 60 feet. The existing 24-foot weir length is therefore inadequate, and additional weir length should be provided.

(2) The remaining volume of 1,548,800 cubic yards in the existing containment area is sufficient to accommodate disposal of the 300,000 cubic yards of maintenance channel sediment into the basin under a continuous disposal operation. Since the required basin depth is less than the existing depth, no upgrading will be necessary to accommodate the first dredging operation.

C-3. Example II: Containment Area Design Method for Sediments Exhibiting Zone Settling.

a. Project Information. Fine-grained maintenance dredged material is scheduled to be dredged from a harbor maintained to a project depth of 50 feet. Channel surveys indicate that 500,000 cubic feet of channel sediment must be dredged. All available disposal areas are filled near the dredging activity, but an available tract of 80 acres is available for a new site 2 miles from the dredging project. An evaluation of the foundation conditions indicate that the maximum allowable dike height is 15 feet. The containment area must be designed to accommodate initial storage requirements while meeting effluent suspended solids levels of 75 milligrams per litre. In the past, the largest dredge contracted for the maintenance dredging has been a 24-inch pipeline dredge. This is the largest size dredge located in the area.

b. Results of Laboratory Tests. Sediment and dredging site water characterization was conducted as described in Chapter 3. A pilot settling test was conducted, and an interface was observed within a few hours. A column settling test for zone settling was then conducted as described in Chapter 3. Flocculent settling data were collected above the interface. The test was also continued for 15 days for purposes of evaluating initial storage requirements. The following data were obtained from the laboratory tests:

(1) Salinity: 15 parts per thousand.

(2) Channel sediment in situ water content w : 92.3 percent.

(3) Specific gravity G_s : 2.71.

(4) Depth to suspended solids interface as a function of time for a series of zone settling tests (see Table C-5).

(5) Concentration of settled material as a function of time data (15-day settling column data) (see Table C-6).

(6) Concentration of settled solids versus time curve (see Figure C-4).

(7) Representative samples of channel sediments tested in the laboratory indicate that 15 percent of the sediment is coarse-grained material (> No. 200 sieve).

$$V_{sd} = 500{,}000(0.15) = 75{,}000 \text{ cubic yards}$$

$$V_i = 500{,}000 - 75{,}000 = 425{,}000 \text{ cubic yards}$$

(8) Suspended solids concentration data for port samples taken above the interface for the flocculent test (Table C-7).

(9) Concentration profile diagram plotted from data in Table C-7 (Figure C-5). The initial supernatant suspended solids concentration C_o was assumed equal to the highest concentration of the first port samples taken,

Table C-5

Depth to Solids Interface (Feet) as a Function of Settling Time (Hours) at C_i = 150 grams per litre

Time, hr	Depth, ft
0	0
0.25	0.050
0.50	0.090
0.75	0.170
1.0	0.230
2.0	0.420
3.0	0.475
4.0	0.505
5.0	0.530
6.0	0.553
7.0	0.565
8.0	0.575
10.0	0.595
20.0	0.655
30.0	0.690

* From plot of depth versus time V_s = 0.24 feet per hour.

Table C-6

Concentration of Settled Solids
as a Function of Time*

Time Days	Concentration g/ℓ
1	192
2	215
3	219
4	140
5	251
6	272
8	280
10	290
15	320

* See Figure C-3.

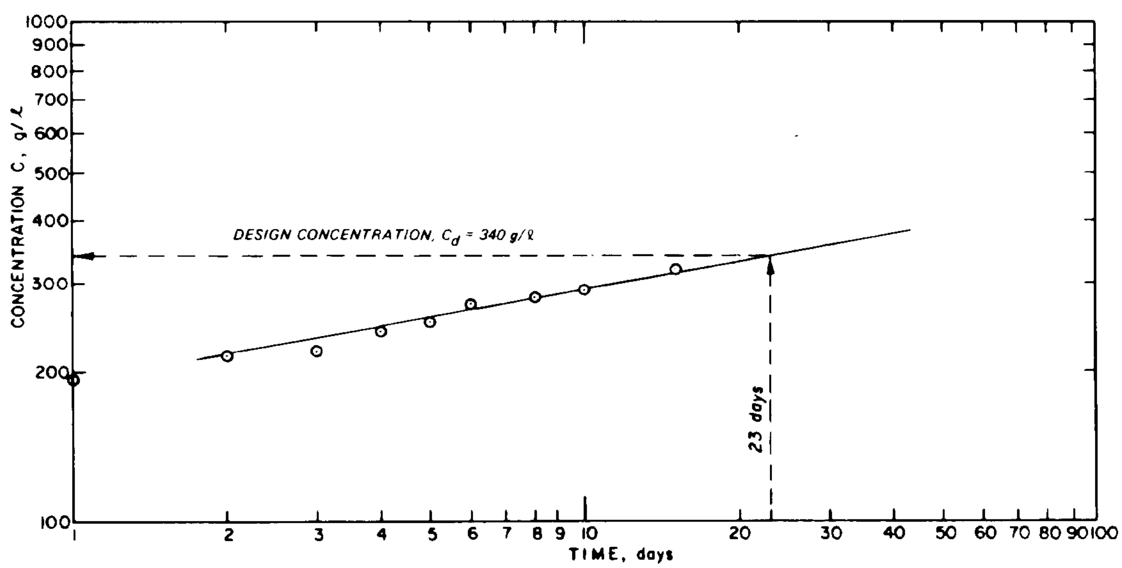


Figure C-4. Concentration of settled solids versus time

Table C-7

Observed Flocculent Settling Data

Sample Extraction Time t (hr)	Depth of Sample Extraction z (ft)	Total Suspended Solids, C (mg/ℓ)	Fraction of Initial, Ø (percent)
3	0.2	93	55
3	1.0	169	100
7	1.0	100	59
7	2.0	105	62
14	1.0	45	27
14	2.0	43	25
14	3.0	50	30
24	1.0	19	11
24	2.0	18	11
24	3.0	20	12
48	1.0	15	9
48	2.0	7	4
48	3.0	14	8

169 milligrams per litre. The concentration profile diagram was therefore constructed using 169 milligrams per litre as Ø = 100 percent.

c. Design Concentration. Compute this value as follows:

(1) The project information is as follows:

(a) Dredge size: 24 inches.

(b) Volume to be dredged: 500,000 cubic yards.

(2) Good records are available from past years of maintenance dredging in this harbor. They show that each time a 24-inch dredge was used, the dredge operated an average of 12 hours per day and dredged an average of 900 cubic yards per hour.

(3) Estimate the time of dredging activity:

$$\frac{500{,}000 \text{ yd}^3}{900 \text{ yd}^3/\text{hour}} = 556 \text{ hours}$$

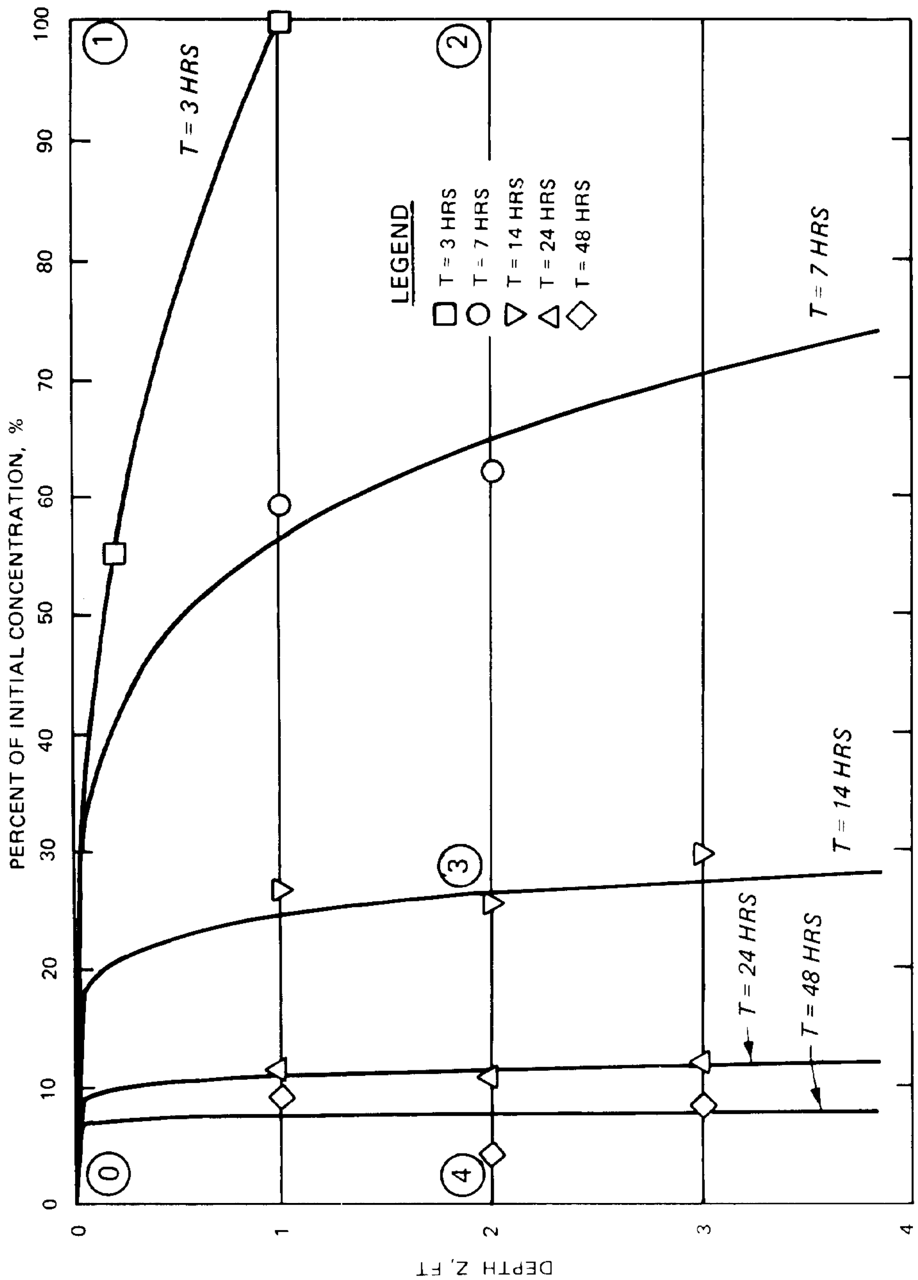


Figure C-5. Suspended solids concentration profile diagram where operating time per day = 12 hours. Thus,

$$\frac{556 \text{ hours}}{12 \text{ hours/day}} = 46 \text{ days}$$

(4) Average time for dredged material consolidation:

$$\frac{46 \text{ days}}{2} = 23 \text{ days}$$

(5) Design concentration is the solids concentration of settled solids shown in Figure C-4 at 23 days:

C_d = 340 grams per litre or 21.1 pounds per cubic feet

d. Volume Required for Dredged Material. This volume is estimated as follows:

(1) Compute the average void ratio using Equation 4-2:

$$e_o = \frac{G_s \gamma_w}{\gamma_d} - 1$$

G_s = 2.71

γ_w _ 1,000 grams per litre

γ_d = 340 grams per litre = design concentration C_d
(See Figure C-4)

$$e_o = \frac{2.71(1,000)}{340} - 1$$

e_o = 6.97

(2) Compute the volume of fine-grained channel sediments after disposal in containment area using Equation 4-3:

$$V_f = V_i \quad \frac{e_o - e_i}{1 + e_i} + 1$$

where, using Equation 4-1, $e_i = \dfrac{wG_s}{S_D}$

$$e_i = \frac{\left(\frac{92.3}{100}\right)\left(2.71\right)}{1.00}$$

$$e_i = 2.5$$

$$V_i = 425{,}000 \text{ cubic yards}$$

$$V_f = \left(\frac{6.97 - 2.50}{1 + 2.50}\right) + 1\ (425{,}000)$$

$$= 967{,}785 \text{ cubic yards}$$

(3) Estimate the volume required by dredged material in containment area using Equation 4-4:

$$v = V_f + V_{sd}$$

$$V_{sd} = 75{,}000 \text{ cubic yards}$$

$$V = 967{,}785 + 75{,}000$$

$$= 1{,}042{,}785 \text{ cubic yards}$$

e. Maximum Possible Thickness of Dredged Material at End of Disposal Operation.

(1) Because of foundation problems, dike heights are limited to 15 feet. Therefore, the disposal area must be increased to accommodate the storage requirements. Use Equation 4-4b to determine the allowable dredged material height:

$$H_{dm_{(max)}} = H_{dk_{(max)}} - H_{pd} - H_{fb}$$

$$H_{dk_{(max)}} = 15 \text{ feet}$$

$$H_{pd} = 2 \text{ feet}$$

$$H_{fb} = 2 \text{ feet}$$

$$H_{dm_{(max)}} = 15 - 2 - 2$$

$$H_{dm_{(max)}} = 11 \text{ feet}$$

(2) Compute the minimum possible surface area using Equation 4-4c:

$$A_{ds} = \frac{V}{H_{d_{(max)}}}$$

$$A_{ds} = \frac{1{,}042{,}785\ \text{yd}^3 \times \dfrac{27\ \text{ft}^3}{\text{yd}^3}}{11\ \text{ft}}$$

$$A_{ds} = 2{,}559{,}563\ \text{ft}^2$$

$$A_{ds} = 59\ \text{acres}$$

Since this value is less than the 80-acre tract available, the dredged material can be physically stored.

f. Minimum Area Required for Zone Sedimentation. This value is computed as follows:

(1) From data in Table C-5, V_s = 0.24 feet per hour.

(2) Compute the area requirement using Equation 4-5:

$$A_z = \frac{Q_i\ (3600)}{V_s}$$

$$Q_i = A_p V_d$$

$$V_p = 15\ \text{ft/sec}$$

$$Q_i = \frac{\left(\dfrac{24\ \text{in.}}{12}\right)^2 \pi}{4} \times 15\ \text{ft/sec}$$

$$= 47.12\ \text{ft}^3/\text{sec}$$

$$A_z = \frac{47.12\ (3600)}{0.24}$$

$$= 706{,}800\ \text{ft}^2$$

$$A_z = \frac{706{,}800}{43{,}560} = 16.22\ \text{acres}$$

(3) Increase the area by a factor of 1.87 (from Equation 4-14) to account for hydraulic inefficiencies (assuming the containment area can be constructed with a length-to-width ratio of approximately 3):

$$A_{dz} = 1.87(16.22 \text{ acres})$$

$$A_{dz} = 30.3 \text{ acres}$$

Thus, the minimum area required for effective zone settling is 30.3 or approximately 30 acres. This is less than the 80 acres available at the site.

g. Retention Time for Suspended Solids Removal.

(1) A relationship of suspended solids remaining versus retention time was developed using the laboratory data in Figure C-5. Ratios of suspended solids removed as a function of time were determined graphically using the step-by-step procedure described in Chapter 4. The lower horizontal boundary for the determined areas corresponded to the minimum average ponding depth of 2 feet. An example calculation for removal ratio for the concentration profile at T = 14 hours and ponding depth of 2 feet using Equation 4-9 is as follows:

$$R_{14} = \frac{\text{Area right of the profile}}{\text{Area total}} = \frac{\text{Area } 1{,}230}{\text{Area } 1{,}240} = 0.78$$

The areas were determined by planimeter. The portion remaining at T = 14 hours is found using Equation 4-10 as follows:

$$P_{14} = 1 - R_{14} = 1 - 0.78 = 0.22$$

The concentration of suspended solids remaining is found using Equation 4-11 as follows:

$$C_{14} = P_{14}\,C_o = 0.22 \text{ (169 milligrams per litre)} = 37 \text{ milligrams per litre}$$

Values at other times were determined in a similar manner. The data were arranged in Table C-8. A curve was fitted to the data for total suspended solids versus retention time and is shown in Figure C-6.

Table C-8

Percentage of Initial Concentration and Suspended Solids Concentrations versus Time, Ponding Depth of 2 Feet

Sample Extraction Time, t (hr)	Removal Percentage R_t	Remaining Percentage P_t	Suspended Solids (mg/ℓ)
3	14	86	145
7	47	53	90
14	78	22	37
24	90	10	17
48	94	6	10

(2) Since the final site configuration is not known beforehand, an appropriate value should be selected from Table 4-1 for the resuspension factor. The minimum ponding depth of 2 feet required by the site design is used. A resuspension factor of 1.5 was selected corresponding to an available area <100 acres and ponding depth of 2 feet.

(3) The value of effluent suspended solids of 75 milligrams per litre must be met at the point of discharge and considers anticipated resuspension. The corresponding value for total suspended solids concentration under quiescent settling conditions is determined using Equation 4-12 as follows:

$$C_{col} = \frac{C_{eff}}{RF} = \frac{75 \text{ mg}/\ell}{1.5} = 50 \text{ mg}/\ell$$

(4) The required configuration of the disposal area must correspond to a retention time that will allow the necessary sedimentation. Using Figure C-6, 50 milligrams per litre corresponds to a field mean retention time of 10 hours. To determine the required disposal site geometry, the theoretical retention time should be used. The hydraulic efficiency correction factor was calculated from Equation 4-14 to be 1.87 for an L/W of 3. The theoretical retention time was calculated using Equation 4-8 as follows:

$$T = T_d \text{ (HECF)} = 10 \text{ (1.87)} = 18.7 \text{ hours}$$

(5) The disposal area configuration can now be determined using data on the anticipated flow rate and the theoretical retention time. Since the dredging equipment available in the project area is capable of flow rates up to 47 cubic feet per second, the high value should be assumed. The ponded area required is calculated using Equation 4-13 as follows:

$$A_{df} = \frac{T\ Q_i}{H_{pd}\ (12.1)}$$

$$= \frac{18.7\ (47)}{2\ (12.1)}$$

$$= 36 \text{ acres}$$

The disposal site should therefore encompass approximately 36 acres of ponded surface area if the dredge selected for the project has an effective flow rate not greater than 47 cubic feet per second. In this case, the surface area of 36 acres required to meet the water quality standard is greater than the minimum surface area of 30 acres required for effective zone settling. However, the area required for storage, 59 acres, is the controlling surface area. The design surface area A_d is therefore 59 acres.

h. Determination of Disposal Area Geometry. From previous calculation, the minimum design area is 59 acres as required for initial storage. This corresponds to the following values as previously calculated:

$$H_{dm} = 11 \text{ feet}$$

H_{pd} = 2 feet

H_{fb} = 2 feet

A_d = 59 acres

i. Design for Weir.

(1) The design parameters are:

Q_i = 47 cubic feet per second

H_{pd} = 2 feet

(2) Using Figure 4-7, approximately 55 feet of effective weir length is required.

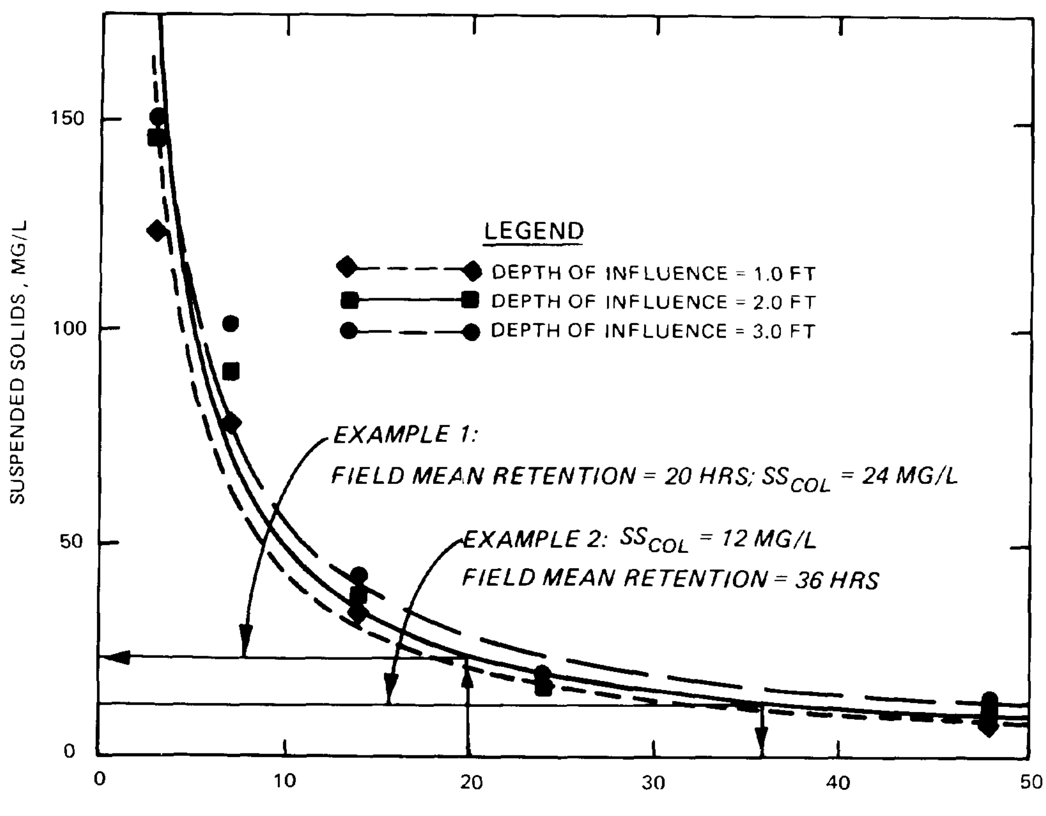


Figure C-6. Plot of supernatant suspended solids concentration versus time from column settling tests

APPENDIX D

DREDGED MATERIAL CONSOLIDATION TEST PROCEDURES

D-1. General. The accuracy of any calculation of the consolidation behavior of fine-grained dredged material is only as good as the soil parameters used. It is therefore very important that the necessary time and resources be allocated to field sample testing and interpretation of the results. Procedures for obtaining sediment samples are found in Chapter 2. This appendix describes methods of consolidation testing, recommended oedometer test procedures for dredged material, and test data interpretation.

D-2. Consolidation Testing. There are essentially three methods of conducting consolidation tests on fine-grained dredged material. They are the self-weight settling test, the controlled rate of strain test, and the oedometer test. Each of these methods has its advantages and disadvantages, and a combination is usually desirable.

a. Self-weight Settling Test. The self-weight settling test is advantageous in determining the void ratio-effective stress relationship at very low levels of effective stress. However, to cover the range of stresses encountered during the consolidation of a prototype dredged fill deposit, the settling column height must equal that of the prototype. If the settling column height equals that of the dredged fill layer, then the time required to complete the test could be on the order of years for typical layers. This is not practical in most situations; so for efficiency, the settling test should be supplemented with one of the other type tests for the higher effective stresses.

b. Controlled Rate of Strain Test. A large-strain controlled rate of strain device specifically for the purpose of testing fine-grained dredged material is now under development. When such a device is available, it is recommended that it be routinely used to define consolidation properties at the high void ratios common to dredged fill.

c. Oedometer Test. The most common type of consolidation testing currently available is the oedometer test. The apparatus required by this test is found in all well-equipped soils laboratories, and the test has been used successfully on numerous dredged materials. Disadvantages of the test include:

(1) The fact that void ratio-effective stress relationships at very low (<0.005 tons per square foot) levels of effective stress are generally not possible.

(2) The fact that the time required between load increments may sometimes be 2 weeks or more.

(3) The fact that large strains during a given load increment add to the uncertainties of test data analysis for coefficients of consolidation and permeabilities.

(4) The question of whether a thin oedometer sample with no initial excess pore pressure when subjected to a sudden load increment reacts the same as an underconsolidated thick sample whose excess pore pressure is slowly decreased. Regardless of the disadvantages, the fact that it is the most common and readily available test is an advantage that makes the oedometer test the most attractive for dredged material today.

D-3. Recommended Oedometer Test Procedure.

a. Oedometer testing of very soft dredged fill materials is accomplished essentially as specified in EM 1110-2-1906 for stiffer soils. The major difference is in the initial sample preparation and the size of the load increments. The majority of dredged fill samples will be in the form of a heavy liquid rather than a mass capable of being handled and trimmed.

b. Before testing begins, both accurate and buoyant weights of the top porous stone and other items between the sample and dial gage stem should be determined because this will be a major part of the seating load. The force exerted by the dial indicator spring must also be determined for the range of readings initially expected because this will constitute the remainder of the seating load and will be considered the first consolidating load applied to the sample. The dial gage force is determined using a common scale or balance. Samples are prepared for testing by placing a saturated bottom porous stone, filter paper, and consolidometer ring on the scale and then recording their weights. Without removing this apparatus from the scale, material is placed in the ring with a spatula. The material is placed and spread carefully to avoid trapping any air within the specimen. After slightly overfilling the ring with material, the excess is screeded with a straightedge, with care being taken not to permit excess material to fall onto the scale. After a level surface flush with the top of the ring is obtained, the ring top is wiped clean and a final weight recorded.

c. The ring with bottom stone is next assembled with the remainder of the consolidometer apparatus. Care must be taken not to jar or otherwise disturb the sample during this process. Once the consolidometer is ready, it is placed on the loading platform, and assembly is completed. As soon as the seating load is placed, the water level in the consolidometer should be brought level with the top of the top porous stone and held there through at least the first three load increments or until the difference in the actual weight and buoyant weight of the seating load is insignificant. Thereafter, the level of the water is not important so long as the sample is kept inundated.

d. Since some consolidation will normally occur very rapidly when the seating load is placed, it is important that this first load is placed very quickly, to include the dial gage. If all induced settlement is not accounted for, later calculations may be inconsistent. It may be necessary to use a table level or some other measuring device to check the height of the top of the porous stone above the sample ring at some time during this first load increment. Of course, the thickness of the top porous stone and filter paper must have been previously measured. In this way, a reconciliation between deformation recorded by the dial gage and actual deformation can be made.

e. After the sample has been subjected to the seating load, dial gage readings are taken at times 0.1, 0.2, 0.5, 1.0, 2.0, 4.0, 8.0, 15.0, and 30.0 minutes; 1, 2, 4, 8, and 24 hours; and daily thereafter until primary consolidation is complete as determined by the time-consolidation curve. The first series of readings is valid for determination of the first point of the e-log-σ' curve and may be used in coefficient of consolidation or permeability determinations if the seating load is placed quickly and in a manner so as not to induce extraneous excess pore pressures.

f. Consolidation of the sample is continued according to the following recommended loading schedule: 0.005, 0.01, 0.025, 0.05, 0.10, 0.25, 0.50, and 1.00 tons per square foot. Exactly what the first load increment will equal depends on the weight of the top porous stone, loading column, and dial gage force. To keep the dial gage force relatively constant throughout testing, the dial gage may have to be reset periodically. If so, it should be reset just before the next load increment is placed and not during a load increment. If consolidation behavior at loads much greater than about 1.0 ton per square foot is required, it is recommended that samples which have been preconsolidated to 0.5 ton per square foot be used, since most typical dredged fill samples will have undergone more than 50-percent strain by the time the above loading schedule is completed. Experience has shown that extrapolation of the e-log-σ' curve produced from the recommended loading schedule to lower void ratios should yield reasonably accurate results, providing that the void ratios through the extrapolated range are greater than about 1.0.

g. When primary consolidation Is completed under the final load of the schedule, the difference between the tops of the top porous stone and the top of the sample ring should again be determined by a table level or other measuring device as a second check on final sample height as determined from dial gage readings. This check is considered important, since the dial gage will probably have been reset several times during the loading schedule. Before the dial gage is removed, the sample should be unloaded and allowed to rebound under the seating load and dial gage force only. When the sample is fully rebounded, a final dial gage reading is made, and the sample is removed for water content and weight of solids measurements.

h. The preceding recommended test procedure is not meant to replace the more comprehensive treatment of EM 1110-2-1906 or other soils testing manuals. Its purpose is merely to point out where the conventional procedure must be modified or supplemented to handle extremely soft dredged fill material. A final recommendation is that a specific gravity of solids test always be accomplished for the actual material consolidated, since calculations are very sensitive to this value and typical estimated values may lead to significant error.

D-4. Calculation of Permeability. Since the conditions of the oedometer test correspond very closely with those assumed in small strain consolidation theory when data are analyzed for each load increment, there is probably no advantage in using the more complicated finite strain theory in deducing permeability. Then the expression can be written:

$$k = \frac{T_u \bar{H}^2 \gamma_w \bar{a}_v}{(1 + \bar{e})t} \tag{D-1}$$

where

k = coefficient of permeability, centimetres per second

T_u = time factor for specified percent consolidation

$\bar{H}$ = effective specimen thickness, centimetres

a_v = coefficient of compressibility, square centimetres per gram

e = void ratio

t = time required to reach specified percent consolidation, second

The bar indicates average values during the load increment. If 50-percent consolidation is assumed to occur simultaneously with 50-percent settlement, the equation can be written:

$$k = \frac{0.197 \bar{H}^2 \alpha_w \bar{a}_v}{(1 + \bar{e})t_{50}} \tag{D-2}$$

where t_{50} is the time required for 50-percent settlement from the compression-time curve for the particular load increment. The values for k are then plotted versus e , and a smooth curve drawn through the points.

APPENDIX E

JAR TEST PROCEDURES FOR CHEMICAL CLARIFICATION

E-1. General.

a. Laboratory Jar Tests. Jar tests have been used to evaluate the effectiveness of various coagulants and flocculants under a variety of operating conditions for water treatment. The procedures and evaluation process (item 4) and (item 20) have been adapted to dredged material (item 29). However, conducting jar tests and interpreting the results to determine design parameters are not simple tasks because there are many variables that can affect the tests. Only experience can assist in applying the following jar test procedures to a specific project. Additional information (item 22) is available on equipment requirements and the importance of flocculent type, flocculent concentration, flocculent addition methods, temperature, mixing and test equipment, and intensity and duration of mixing on the jar tests results.

b. Jar Test Uses. Jar tests are used in these procedures to provide information on the most effective flocculant, optimum dosage, optimum feed concentration, effects of dosage on removal efficiencies, effects of concentration of influent suspension on removal efficiencies, effects of mixing conditions, and effects of settling time.

(1) The general approach used in these procedures is as follows:

(a) Using site-specific information on the sediment, dredging operation, containment areas, and effluent requirement, select mixing conditions, suspension concentration, settling time, and polymers for testing.

(b) Prepare stock suspension of sediment.

(c) Test a small number (four to six) of polymers that have performed well on similar dredged material. The tests should be run on 2-grams-per-litre suspensions, which is a typical concentration for effluent from a well-designed containment area for freshwater sediments containing clays. If good removals are obtained at low dosages (10 milligrams per litre or less), then select the most cost-effective polymer. If good removals are not obtained, examine the polymer under improved mixing and settling conditions and test the performance of other flocculants.

(d) After selecting a polymer and its optimum dosage, examine the effect of polymer feed concentration over the range of 1 to 30 grams per litre, typical concentrations used in the field, at the optimum dosage.

(e) Determine dosage requirements for the expected range of turbidity and suspended solids concentration to be treated at the primary weir.

(f) Examine the effects of the range of possible mixing conditions on the required dosage of flocculant for a typical suspension.

(g) Examine the effects of settling time on the removal of suspended solids and turbidity from a suspension of average concentration, using the selected dosage and likely mixing conditions.

(2) The purpose of the approach described is to select an effective polymer for a suspension of a standard concentration, 2 grams per litre, which is a typical effluent solids concentration. In this manner, the effectiveness and dosage requirements of various polymers are easy to compare. The other test variables are set to simulate anticipated field conditions. After a polymer is selected, other variables are examined: polymer feed concentration, solids concentration of suspension to be treated, mixing, and settling time. The approach may be changed to fit the needs and conditions of the specific study.

(3) The details of each test typically are modified to satisfy the constraints and conditions of the project and test. This procedure generally requires judgment from experience with jar tests and chemical treatment. Detailed procedures are found in the following paragraphs.

E-2. Selection of Test Conditions.

a. Mixing Intensity and Duration. Prior to testing, the mixing intensity and duration for the jar tests should be selected based on project conditions. Assuming that mechanical mixing will not be used in the treatment system, the amount of mixing should be based on the available head between the two containment areas, i. e., the difference between the water surfaces of the two areas that can be maintained throughout the project (see Figure E-I). The depth of the secondary area must be sufficient to provide 2 to 3 feet of storage and 2 to 3 feet of ponding for good settling. Preferably, 2 to 3 feet of head should be available for mixing. The object is to convert the head into mixing energy in the culvert(s) joining the two containment areas. The amount of head loss is a function of flow rate, culvert diameter, and length.

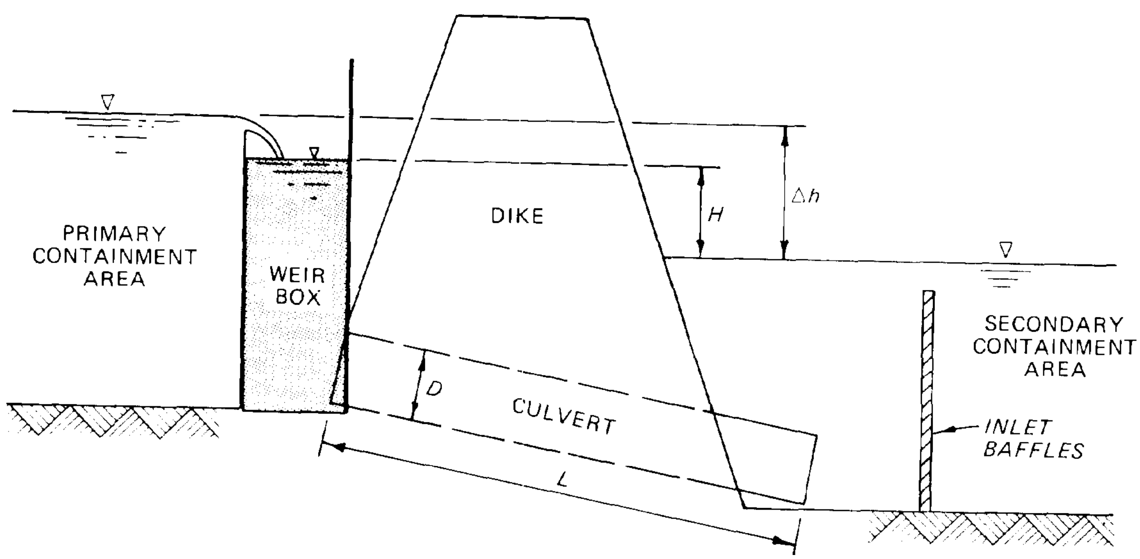


Figure E-1. Example weir mixing system

Table E-I presents typical mixing values for good culvert mixing designs under a variety of conditions assuming a maximum of five culverts and a maximum culvert length of 100 feet. The net mixing Gt is the product of the mean velocity gradient (intensity) and the duration. The mixing intensity in terms of the mean velocity gradient G_{-1} for the design conditions in Table E-l varied from about 250 to 500 set . The effectiveness of polymers increased as the mixing Gt increased to about 30,000.

(1) The designer may select a Gt value from Table E-l for an example with similar flow and mixing head, but preferably the designer should calculate the head loss, mixing intensity, and duration for the existing or designed culvert according to the following procedure for pipe flow (item 31). Assuming a submerged inlet and outlet and corrugated metal pipe,

$$H = \left(1.5 + \frac{Lf}{D}\right) \frac{v^2}{2g} \tag{E-1}$$

where

H = head loss, feet
L = culvert length, feet
f = friction factor
= $185\ n^2/D^{1/3}$ (n = Manning's coefficient, 0.024 for corrugated metal pipes)
D = culvert diameter, feet
v = maximum velocity through culvert, feet per second
= $4\ Q_{max}/\pi D^2$
Q_{max} = maximum flow rate, cubic feet per second
g = gravity, 32.2 feet per second^2

Alternate methods and sources for fraction factor and Manning's coefficient are available in Hydraulics Design Criteria 224-1/2 to 224/1/4. The mean velocity gradient G can be calculated as follows:

$$G = \sqrt{\frac{\gamma_s f \bar{v}^3}{2gD\mu_s}} \tag{E-2}$$

where

G = mean velocity gradient, second^{-1}
γ_s = specific weight, 62.4 pounds per cubic foot
$\bar{v}$ = average velocity, feet per second
μ_s = absolute viscosity, 2.36×10^{-5} pounds per second per square foot at 60° F

The duration t of the mixing in seconds is determined by

$$t = \frac{L}{\bar{v}} \tag{E-3}$$

Table E-1

Design Mixing Values (Gt)

Flow cfs	Available Head, ft 2	3	4	5	6
5	8,200	9,800	11,300	12,200	12,900
8	7,800	9,300	10,800	11,600	12,300
12	7,500	9,000	10,400	11,200	11,900
16	7,200	8,700	10,000	10,800	11,500
21	7,000	8,400	9,700	10,500	11,100
27	6,800	8,200	9,500	10,200	10,800
36	6,600	7,900	9,100	9,800	10,400
47	6,400	7,600	8,800	9,500	10,100
60	6,200	7,400	8,500	9,200	9,800
74	6,000	7,200	8,300	8,900	9,500
106	5,700	6,800	7,900	8,500	9,000

The mixing increases with increases in head loss, culvert length, and duration and with decreases in culvert diameter. Long, multiple, small-diameter, corrugated culverts provide the best mixing conditions. Good mixing requires a Gt of about 30,000, though a Gt of about 8,000 provides adequate mixing.

(2) An alternative to using long, small-diameter, corrugated culverts to effectively convert the available head into mixing would be to install static mixers in the culvert. Static mixers are fixed obstructions that, when placed in a culvert, efficiently increase the turbulence produced by the flow. The mixers increase the head loss without using smaller diameter or longer culverts. When using these devices, care must be taken to accurately determine the head loss to ensure that good mixing is provided while not exceeding the available head.

(3) After determining G and t for field conditions, use the same G and t for rapid mixing conditions in the laboratory jar test. If the G is greater than the G available on the jar test apparatus, mix at maximum speed and increase the duration to obtain the same Gt . The relationship between G and revolutions per minute of a jar test apparatus is shown in Figure E-2. For slow mixing, mix at 20 revolutions per minute (G = 10 seconds^{-1}) for

300 seconds to simulate the exit loss conditions as the water dissipates its kinetic energy upon entering the secondary cell.

b. Suspension Concentration. The next step is to predict the average solids concentrations and turbidity of the suspension to be treated at the primary weir. This can be estimated from past records of dredging at the site or flocculent settling tests. Procedures for containment area design considering both flocculent and zone settling are found herein. The results of flocculent settling tests, when available, should be used to determine the suspension concentration.

c. Settling Time for Flocculated Material. The next variable to establish is settling time. Flocculated (chemically treated) material settles at a rate of about 0.25 feet per minute. The required ponding depth for good settling is about 2 to 3 feet; therefore, a minimum of 10 minutes is needed for settling. Also, due to basin inefficiencies, some of the water will reach the secondary weir in 10 to 20 percent of the theoretical residence time. For secondary containment areas, this may be as short as 10 to 20 minutes, though the mean residence time may be about 50 minutes. Based on this information, the settling time in the jar test should be set at 10 minutes. The effect of settling time on suspended solids removal can be evaluated in the jar test procedures.

d. Selection of Polymers for Testing. The final consideration before starting the jar tests is the selection of polymers to be tested. To simplify the operation of feeding and dispersing the polymer at the project, a low viscosity liquid polymer should be used. Some polymers effective on dredged material are:

Betz	1180
	1190
Calgon	M-503
Hercofloc	815
	849
	863
	876
Magnifloc	573c
	577c
Nalco	7103
	7132

Polymer manufacturers may be able to suggest others. The manufacturers can also recommend maximum polymer feed concentrations. Polymer selected for testing should be nontoxic, nonhazardous, and unreactive. Polymer manufacturers can provide detailed information on the properties of their products. Also, the US Environmental Protection Agency has approved many polymers for use on potable water at the desired dosages. Very little of an applied dosage is expected to be discharged from the containment area since the polymer adsorbs on the solids and settles in the containment area. Therefore,

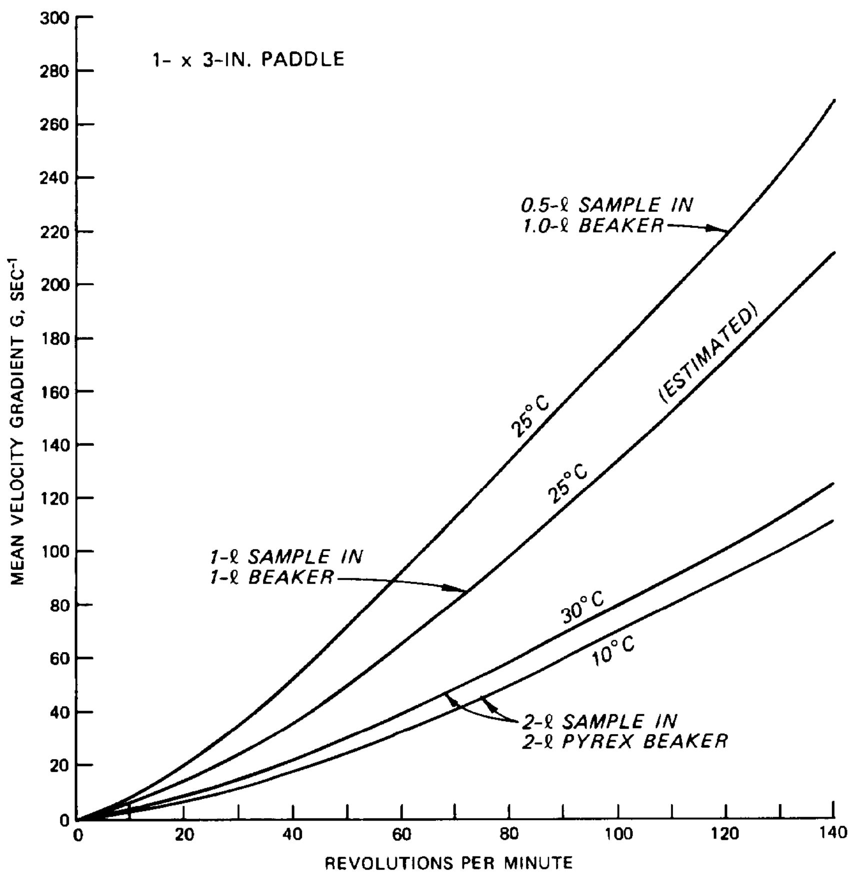


Figure E-2. Velocity gradient G calibration curves for jar test apparatus

polymers should not be detrimental to the quality of the receiving waters. Polymers do not increase the long-term release of contaminants or nutrients from treated dredged material (item 35).

E-3. Suspension Preparation. Dredged material that is discharged over the weir is composed of only the finest fraction of the sediment. In many cases, this material has been suspended and mixed in the primary containment area for several days while the coarser material settled. Therefore, to obtain representative suspensions for testing, the following procedure is recommended:

a. Thoroughly mix each sediment sample to ensure homogeneity. Then, blend equal portions of each sample to form a representative composite of the sediment. Grain size analysis and soil classification may be performed on this material to characterize the mixture and to compare it with previous characterizations of the sediment.

b. If the sediment mixture contains more than 10 percent (dry weight basis) coarse-grained (>No. 200 sieve) material, the material should be sieved though a standard US series No. 200 sieve. The fines can be washed through the sieve using water from the bottom of the water column at the dredging site. If this water is unavailable, tap water may be used in its place, but the salinity of the suspension of fines (<No. 200 sieve) must be adjusted to naturally occurring salinity of the bottom waters at the project site.

c. Prepare a supply of 2.0-grams-per-litre suspensions by diluting a well-mixed portion of the slurry of fines with water from the dredging site or with tap water adjusted with salt to the same salinity. Suspensions at other concentrations would be prepared in the same manner.

E-4. Jar Test Procedures. Having established the test variables, the designer is ready to start the laboratory jar test procedures. Care must be exercised in the tests to ensure that each sample is handled uniformly. The tests must be performed in a standard manner to evaluate the results. The following variables must be controlled: identical test equipment and setup, suspension preparation, sample temperature, polymer feed concentration and age, polymer dosage, sample premix time and intensity, polymer addition method, duration and intensity of rapid mixing, duration and intensity of slow mixing, settling time, sampling method, and laboratory analyses of samples. All of the following procedures described in this section are not necessary for every project. The required tests are dependent on the purpose of the study, and some tests can be eliminated based on past experience of treating dredged material under similar circumstances,

a. Selection of polymer. The laboratory jar test procedures are as follows:

(1) Fill a 1- or 2-litre beaker with a 2.0 grams-per-litre suspension of fine-grained dredged material.

(2) Mix at 100 revolutions per minute and incrementally add polymer at a dosing of 2 milligrams per litre until flocs appear. Note the total dosage applied. (Use a polymer feed concentration of 2 grams per litre or 2 milligrams per millilitre.)

(3) Fill six 1- or 2-litre beakers with a 2.0-grams-per-litre suspension of dredged material and measure the suspended solids concentration and turbidity of the suspension.

(4) Mix at 100 revolutions per minute for 1 minute and then rapidly add the desired polymer dosage to each beaker. Use a range of polymer dosages from 0 milligram per litre to about twice the dosage determined in step (2).

(5) Immediately adjust the mixing to the desired G for rapid mixing as determined earlier. Mix for the desired duration t also determined earlier.

(6) Reduce the mixer speed to a G of 10 seconds^{-1} and slow mix for 300 seconds.

(7) Turn off the mixer and settle for 10 minutes.

(8) Withdraw the samples from the 700-millilitre level of 1-litre beakers and from the 1,400-millilitre level of 2-litre beakers.

(9) Measure the suspended solids concentration and turbidity of the samples. The test data should be recorded on a report form similar to the one shown in Figure E-3. Also record any significant observations such as nature, size, and settling characteristics of the flocs, time of floc formation, and any peculiarities.

(10) Repeat steps (3) through (9) as needed to adequately define the effects of dosage on clarification.

(11) Repeat steps (1) through (10) for the other polymers. A dosage of 10 milligrams per litre should reduce the solids concentrations by 95 percent if the polymer is effective. Examine enough polymers to find at least two effective ones.

(12) Select the most cost-effective polymer that can be easily fed and dispersed.

b. Selection of Polymer Feed Concentration. After selecting the best polymer, the effects of polymer feed concentration and polymer solution age on the removals can be evaluated. Some polymers require great dilution and aging following dilution to maximize their effectiveness. This test is not required if adequate dilution water and solution aging are provided in the design to meet the manufacturer's recommendations. Often, to simplify the treatment system design, these recommendations are not met. The test is performed as follows:

(1) Prepare six fresh solutions of the selected polymer ranging in concentration from about 1 to 40 grams per litre.

(2) Fill six beakers as in step (3) of E-4.a.

(3) Mix at 100 revolutions per minute for 1 minute and then rapidly add the polymer solutions at the effective dosage established earlier and in the same manner.

(4) Continue to follow the procedures outlined in steps (5) through (9) of paragraph E-4.b.

(5) Allow two solutions to age as desired (between 1 hour and 1 day) and repeat steps (2) through (4).

c. Determination of Required Dosage. The dosage requirements of the selected polymer for the anticipated average solids concentration of the primary effluent suspension to be treated at the primary weir should be evaluated. This concentration was determined previously from past records or flocculent settling tests. The procedure is as follows:

(1) Fill six beakers with suspensions at the desired concentration of the fine-grained fraction of dredged material. Measure the suspended solids concentration and turbidity of the suspension.

JAR TEST REPORT FORM

TEST NO. ____________ DATE ____________ SAMPLE SOURCE ____________

COAGULANT ____________ DOSING METHOD ____________ TURBIDITY ________ pH ______

CONC. ____________ SALINITY ____________ SS ________ TEMP ______

JAR NO.		1	2	3	4	5	6
RAPID MIX	RPM						
	TIME, SEC						
SLOW MIX	RPM						
	TIME, SEC						
POLYMER DOSE	mg/l						
	ml						
SAMPLE VOLUME	ml						
SETTLING TIME	MIN						
SUSPENDED SOLIDS AFTER SETTLING	mg/l						
TURBIDITY AFTER SETTLING	NTU'S						

WES FORM 1 DEC 81 2243

Figure E-3. Jar test report form

(2) Mix at 100 revolutions per minute for 1 minute and then rapidly add the desired polymer dosage to each beaker. The range of dosages should be proportional to the solids concentration.

(3) Continue to follow the procedures outlined in steps (5) through (10) of E-4.a. Other suspensions with different solids concentrations may be examined in the same manner to determine the possible range of dosages required for the project and the possible range of effluent quality obtainable under conditions of variable primary effluent solids concentration to be treated.

d. Effects of Mixing. Other mixing conditions can be examined to determine the impact of low flow conditions and to evaluate whether the mixing is adequate. The effects of increasing the mixing by a Gt of 5,000 and 10,000 and of decreasing flow rate by 50, 75, and 90 percent on the polymer dosage requirements can be evaluated as follows:

(1) Calculate the new mixing intensity and duration.

(2) Fill six beakers with a suspension at the anticipated average solids concentration.

(3) Mix at 100 revolutions per minute for 1 minute and then rapidly add the desired polymer dosage to each beaker. Select a range of dosages surrounding the optimum dosage determined in the last set of experiments on the same suspension.

(4) Immediately adjust the mixing to the G value calculated in step E-2.a. for rapid mixing and mix for the calculated duration t .

(5) Follow the procedures outlined in steps (6) through (9) of E-4.a.

e. Effects of Settling Time. The effects of settling time on effluent quality can be examined as follows:

(1) Determine the range of settling time of interest, bearing in mind that the secondary basin will be hydraulically inefficient and the settling conditions will not be quiescent.

(2) Follow the procedures outlined in steps (3) through (9) of paragraph E-4.a., but adjust the settling time and sampling schedule to cover the range determined above.

APPENDIX F

ESTIMATION OF DREDGED MATERIAL CONSOLIDATION BY FINITE STRAIN TECHNIQUE

F-1. General. In this appendix, the technique for estimating consolidation by finite strain techniques is described. Also, the practical problem of a single dredged fill layer deposited on a compressible foundation will be solved for settlement as a function of time by both small strain and linear finite strain theories. The solutions will involve only hand calculations and the appropriate percent consolidation curves given previously in this report.

F-2. Estimation of Consolidation Using Finite Strain Technique.

a. Laboratory Test Data. Consolidation of dredged material due to self weight must be estimated using results from appropriate laboratory tests. The following procedure for hand computation uses standard consolidation (oedometer) laboratory test data. Procedures for these tests are described in Chapter 3 and Appendix D. The laboratory tests yield a relationship between void ratio and effective stress as shown in Figure F-1. An exponential form for the relationship should be determined by curve fitting techniques. The fit should be of the form:

$$e = (e_{oo} - e_{\infty}) \exp(-\lambda\sigma') + e_{\infty} \qquad \text{(F-1)}$$

where e_{oo} is void ratio at zero effective stress and e_{∞} is the void ratio at infinite effective stress. Such a curve is also shown in Figure F-1 where λ, e_{oo}, and e_{∞} were chosen to give the best apparent fit to the test data.

b. Determination of Layer Thicknesses. The void ratio at the end of the sedimentation phase as well as initial thickness of the deposited layer will be determined from column settling tests as described in Chapter 3. The layer thickness in reduced coordinates for each deposited layer should be calculated as follows:

$$\ell = \frac{h}{1 + e_{oo}} \qquad \text{(F-2)}$$

where h is the layer thickness as deposited and e_{oo} is the initial void ratio since the effective stress is assumed initially zero throughout the layer. In a normally consolidated layer or layer having any other than uniform void ratio distribution, ℓ can be calculated to sufficient accuracy by dividing the layer into a number, m, of sublayers and using

$$\ell = \sum_{i=1}^{m} \ell_i = \sum_{i=1}^{m} \frac{h_i}{1 + e_i} \qquad \text{(F-3)}$$

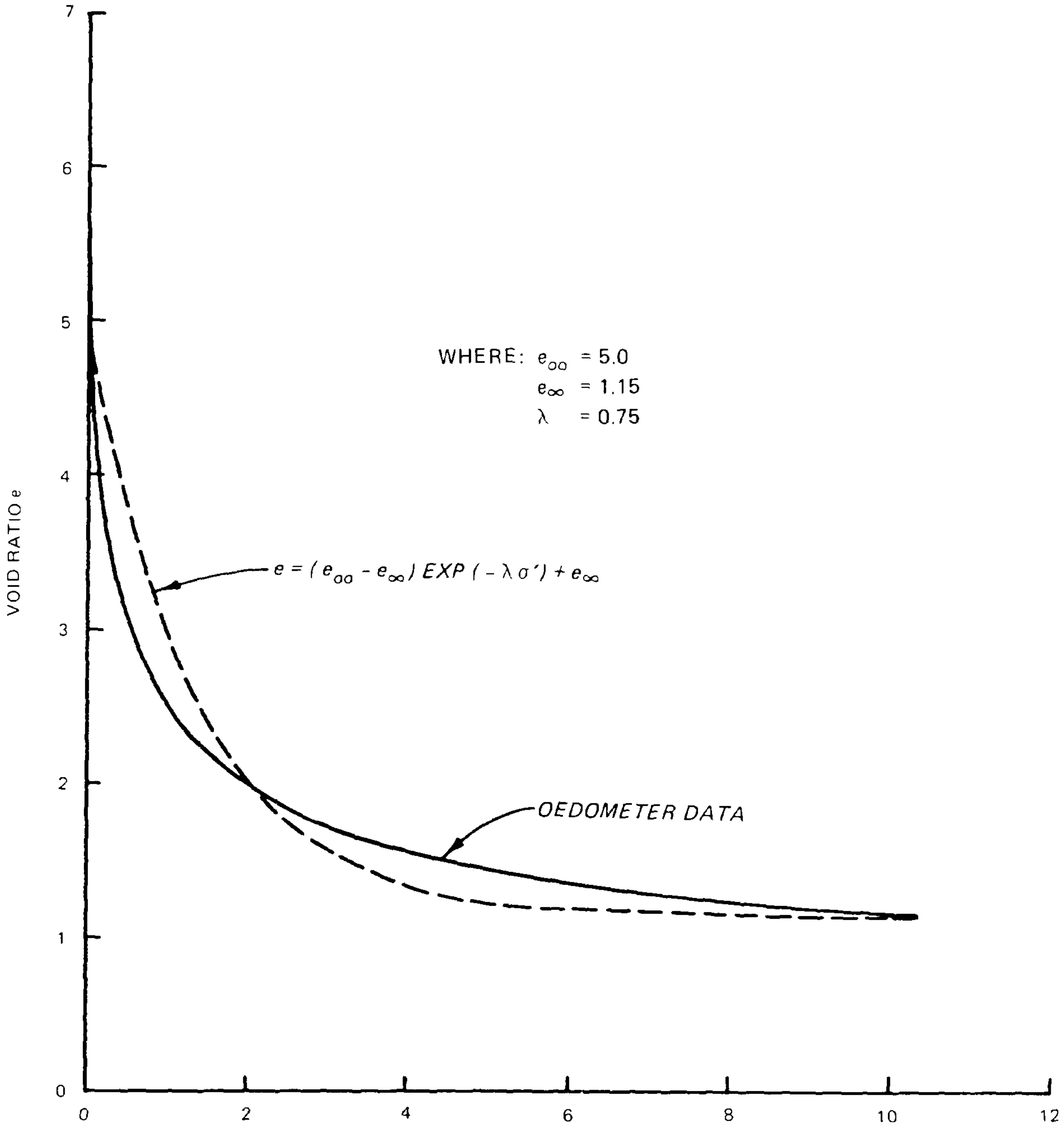


Figure F-1. Exponential void ratio-effective stress relationship compared to oedometer data, 0-12.0 tsf where h_i is the sublayer height and e_i is the average void ratio in the sublayer. The sublayer void ratio is obtained from the $e - \log \sigma'$ curve for the material by considering the effective weight of all material and surcharge above the center of the sublayer or by direct measurement of the saturated water content of the sublayer.

c. Calculation of Ultimate Settlement.

(1) The ultimate settlement of a consolidating fine-grained layer is defined as that which has occurred after all excess pore pressures have dissipated. Within the layer, the material assumes a void ratio distribution due to the buoyant weight of material above plus any surcharge, and this void ratio is related to the effective stress by the material's $e - \log \sigma'$ curve as determined by laboratory testing. Therefore, initial and final void ratio distributions are known or can be calculated.

(2) Ultimate settlement is calculated by dividing the total layer into a number, m , of sublayers such that

$$\delta(\infty) = \sum_{i=1}^{m} \delta_{i,\infty} = \sum_{i=1}^{m} (e_{i,o} - e_{i,\infty})\, \ell_i \qquad \text{(F-4)}$$

where δ is the settlement, ℓ_1 is defined in Equation F-3, and $e_{i,o}$ and $e_{i,\infty}$ are the average initial and final void ratios of each sublayer, respectively. The ultimate average effective stress is then calculated for each sublayer by

$$\sigma_i' = \frac{1}{2} \ell_i (\gamma_s - \gamma_w) + \begin{pmatrix}\text{effective weight}\\ \text{of all sublayers}\\ \text{above it}\end{pmatrix} + (\text{surcharge}) \qquad \text{(F-5)}$$

where the effective weight of each sublayer is $\ell_i \; (\gamma_s - \gamma_w)$. Then, using this average effective stress, an average void ratio is picked from the oedometer test data and substituted into Equation F-4.

d. Calculation of Settlement Versus Time.

(1) The coefficient of consolidation for finite strain, g , should be determined from a plot such as shown in Figure F-2 for the void ratio corresponding to an average effective stress during the consolidation process if the coefficient is relatively constant over the range of expected void ratios. If there is substantial variation in the coefficient of consolidation over the expected range of void ratios, the coefficient can be periodically updated during the calculation to conform to the average void ratio in the layer at the time consolidation is calculated.

(2) The nondimensional time factor for the real time in question is calculated as follows:

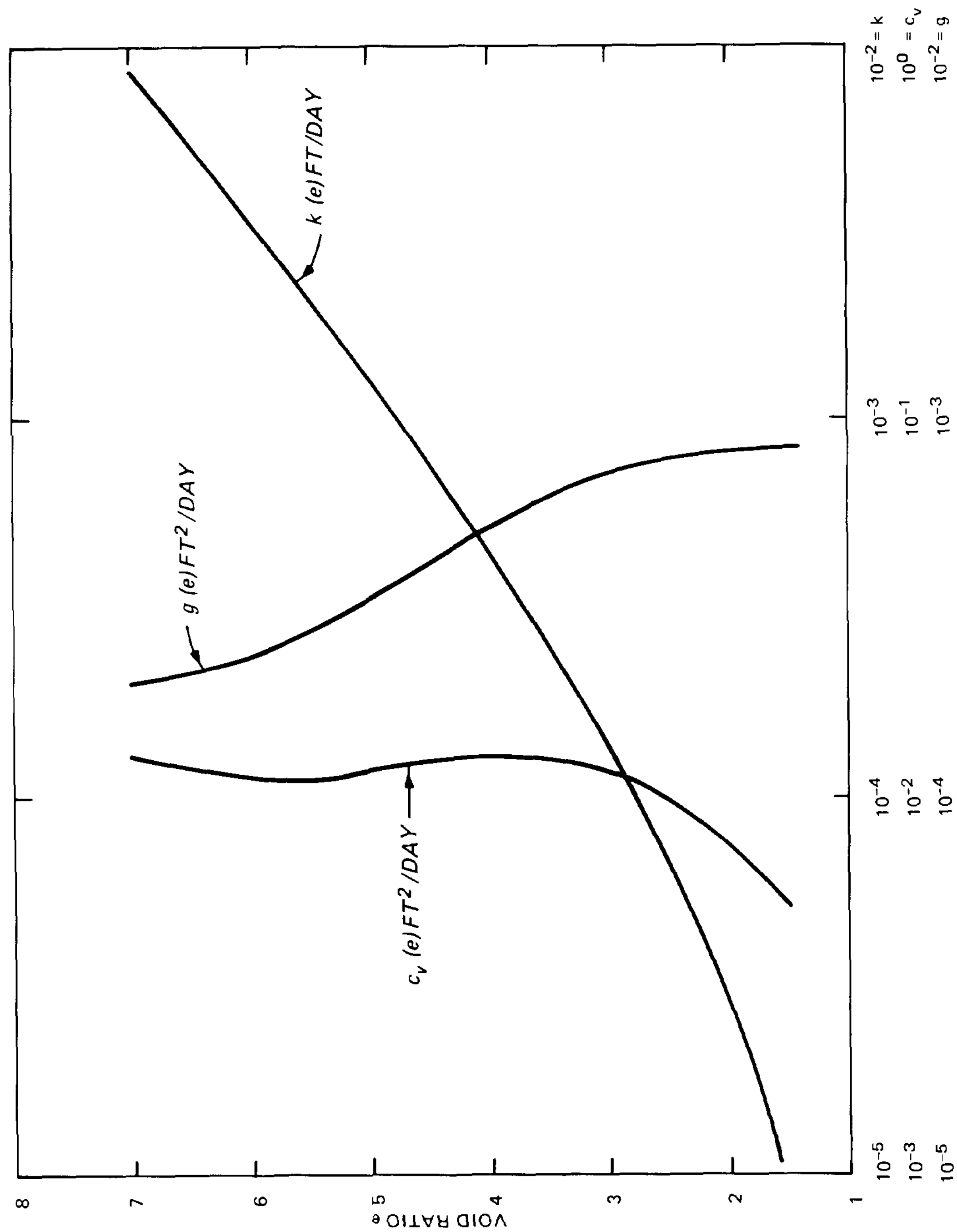


Figure F-2. Typical permeability and coefficients of consolidation as a function of void ratio

$$T_{fs} = \frac{gt}{\ell^2} \qquad \text{(F-6)}$$

(3) Calculate the dimensionless parameter N as follows:

$$N = \lambda \ \ell \ (\gamma_s - \gamma_w) \qquad \text{(F-7)}$$

(4) The percent consolidation, U , is then read from Figure F-3 thru F-6, depending on the value of N and initial conditions and boundary conditions for the calculated time factor.

(5) With the percent consolidation known, settlement is then

$$\delta(T_{fs}) = \delta_\infty \cdot U(T_{fs}) \qquad \text{(F-8)}$$

at the real time t chosen in calculating T_{fs} .

(6) An example of this procedure for a single dredged fill layer deposited on a compressible foundation is solved in F-4 by both a small strain and linear finite strain formulation. In the example, an updated coefficient of consolidation and layer height are used in calculating the dimensionless time factor.

F-3. Empirical Estimate of Settlement Due to Desiccation.

a. Determination of Void Ratio at Saturation and Desiccation Limits.

(1) The void ratio at the saturation limit, e_{SL} , can be determined empirically as follows:

$$e_{SL} = \frac{1.8 LL \ G_s}{100} \qquad \text{(F-9)}$$

where

e_{SL} = void ratio at saturation limit
LL = Atterberg liquid limit of dredged material in percent
G_s = Specific gravity at the dredged material

(2) The void ratio at the desiccation limit can be determined empirically as:

$$e_{DL} = \frac{1.2 \ PL \ G_s}{100} \qquad \text{(F-10)}$$

where

e_{DL} = void ratio of desiccation limit
PL = Atterberg plastic limit of dredged material in percent

b. Calculation of Desiccation Depths.

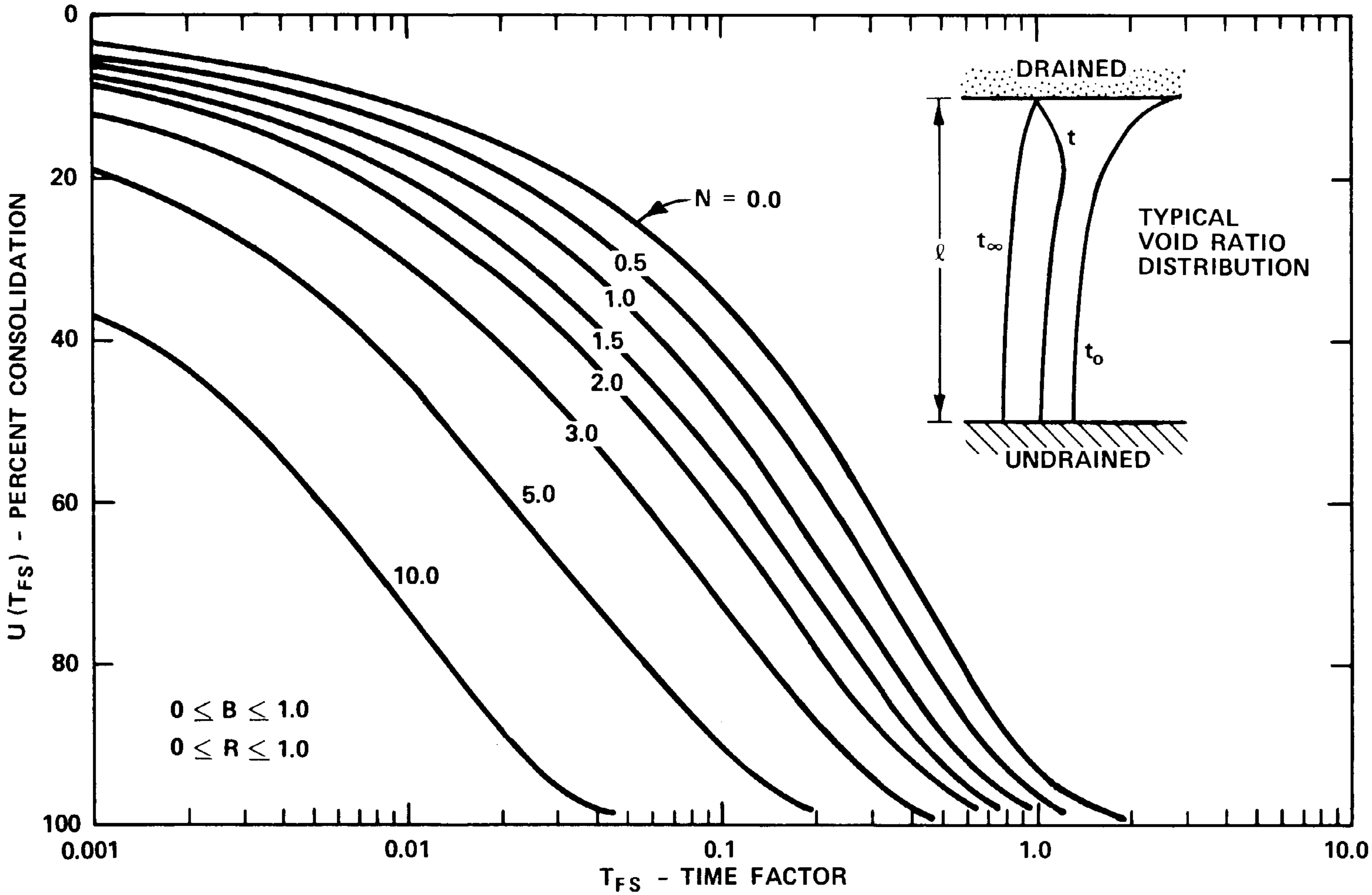


Figure F-3. Degree of consolidation as a function of the time factor for normally consolidated, singly drained layers by linear finite strain theory

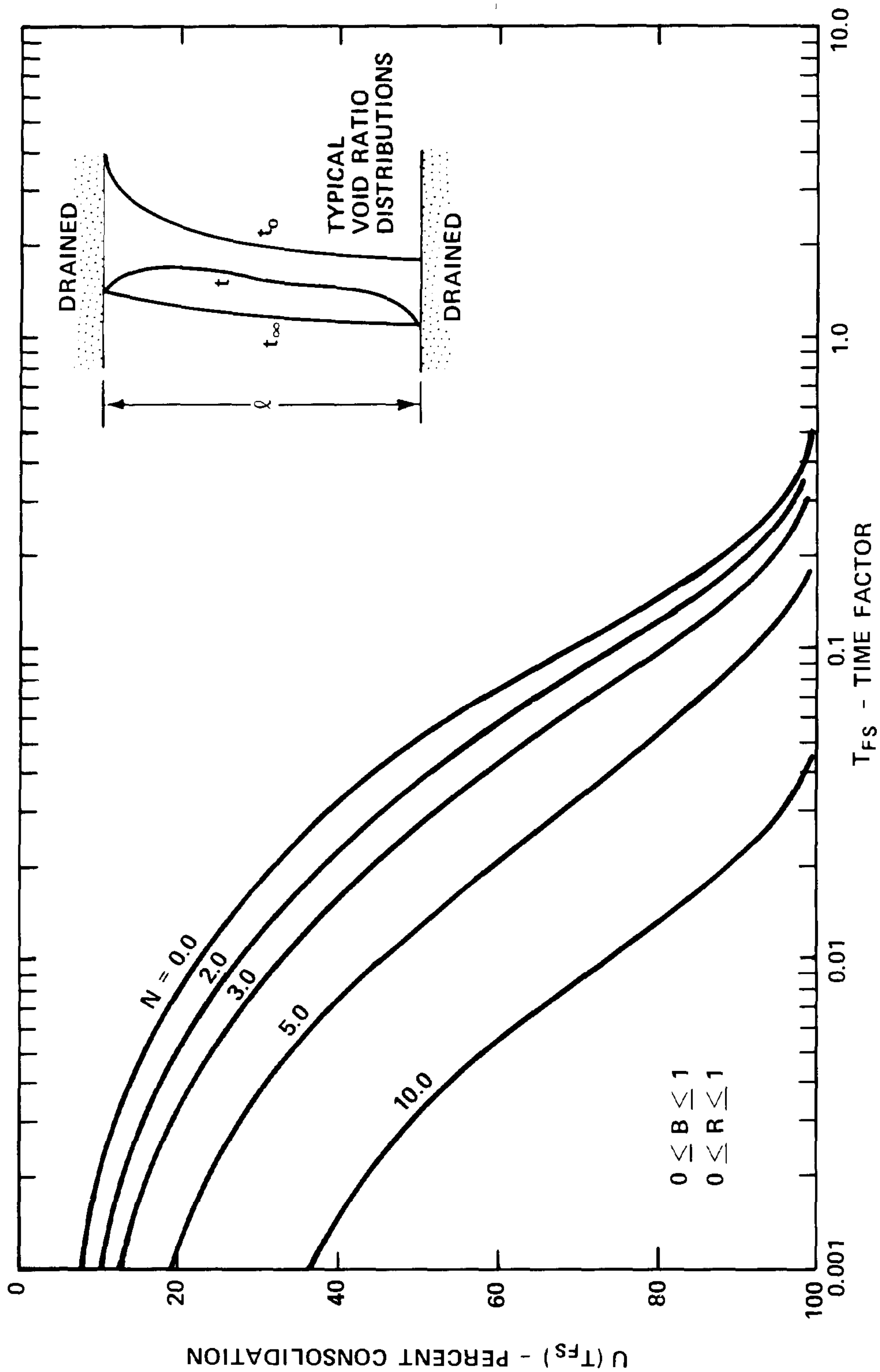


Figure F-4. Degree of consolidation as a function of the time factor for normally consolidated, doubly drained layers by linear finite strain theory

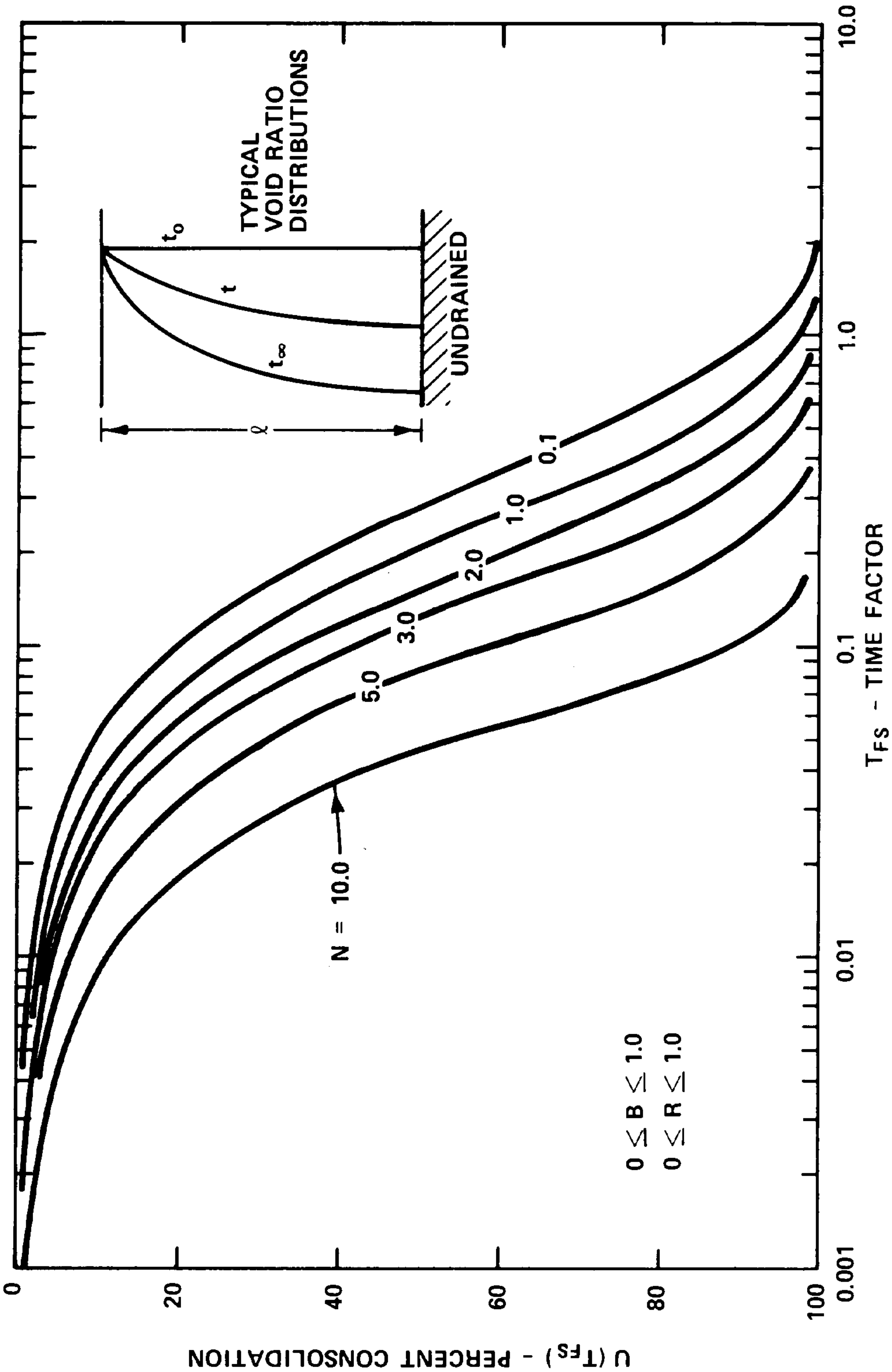


Figure F-5. Degree of consolidation as a function of the time factor for dredged fill, singly drained layers by linear finite strain theory

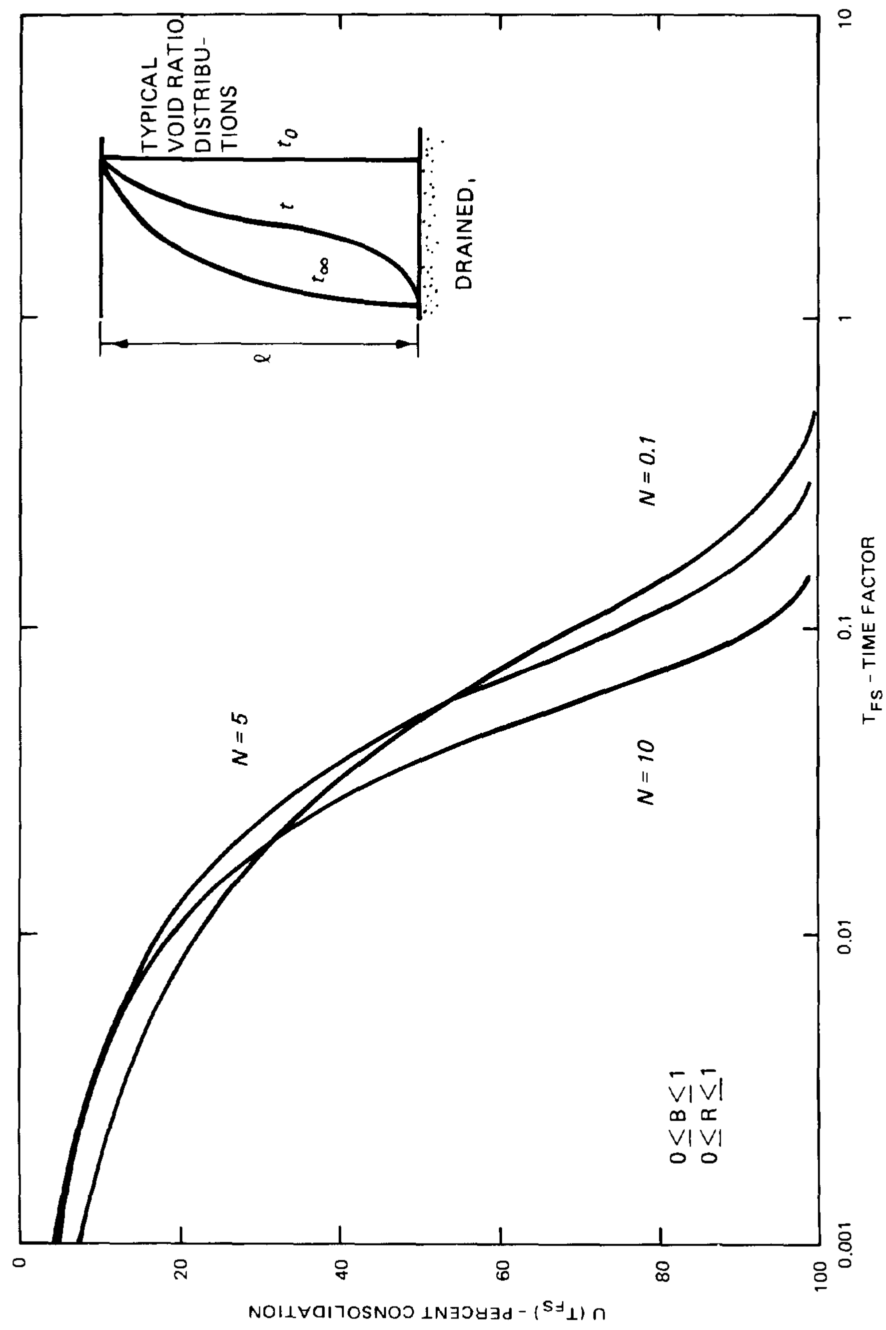


Figure F-6. Degree of consolidation as a function of the time factor for dredged fill, doubly drained layers by linear finite strain theory

(1) As long as the material remains saturated and the free water table is at the surface, the effects of evaporative drying cannot extend deeper than the intersection of the ordinate denoting e_{SL} and the ultimate void ratio distribution curve (See Figure F-7). Thus, the maximum depth to which first-stage drying can occur is

$$h_{1st} = (\ell - z_{SL})(1 + e_{SL}) \qquad \text{(F-11)}$$

where

h_{1st} = maximum depth of first-stage drying

z_{SL} = material coordinate at intersection of e_{SL} and ultimate void ratio distribution curve

While void ratios lower than e_{SL} may exist in the dredged material below z_{SL}, they are due to self-weight consolidation and not surface desiccation during first-stage drying.

(2) The absolute maximum depth to which second-stage drying can occur is the water table depth (which sometimes can be measured in the field) or the intersection of the ordinate denoting e_{DL} with the ultimate void ratio

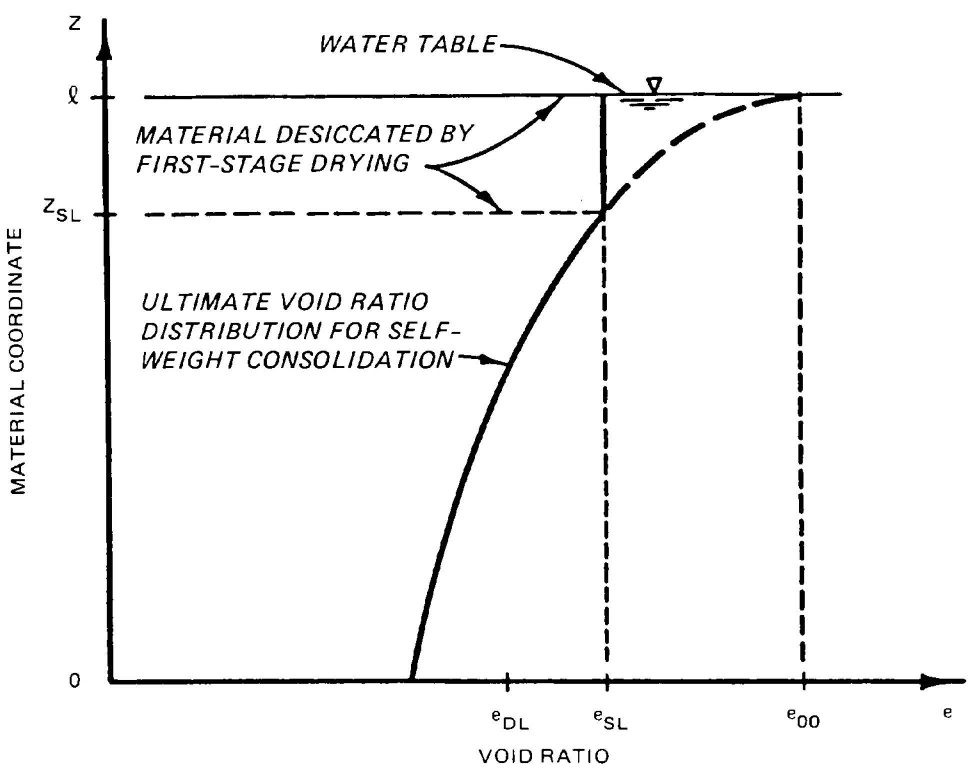


Figure F-7. Maximum depth of material desiccated by first-stage drying

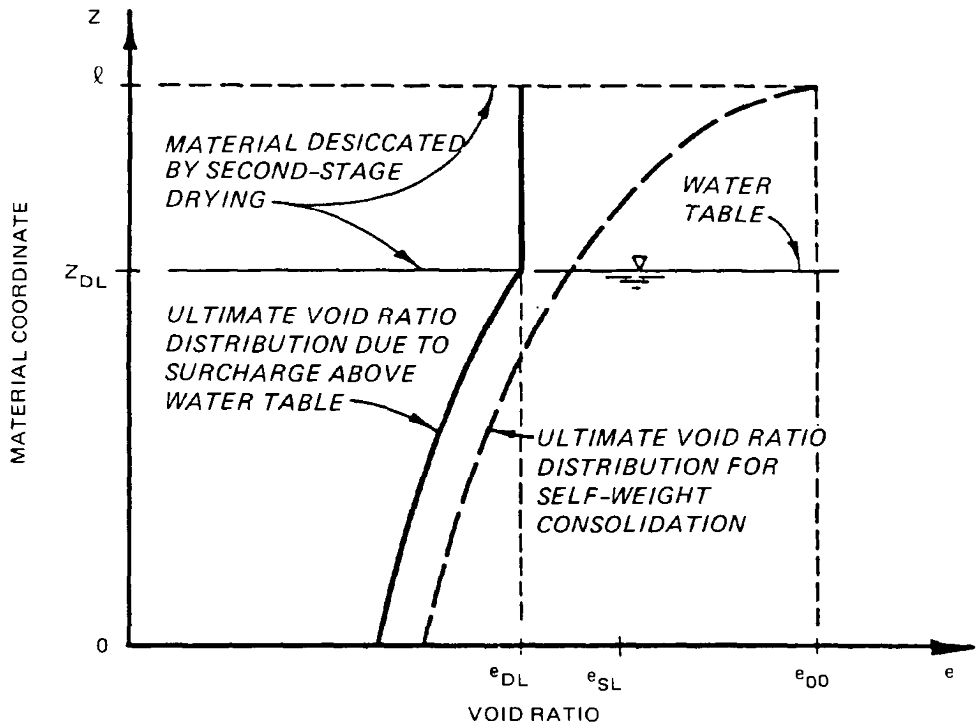


Figure F-8. Maximum depth of material desiccated by second-stage drying

distribution curve that is based on the surcharge induced (See Figure F-8). In equation form

$$h_{2nd} = (\ell - z_{DL})\ (1 + e_{DL}) \tag{F-12}$$

where

h_{2nd} = maximum depth of second-stage drying

z_{DL} = material coordinate at intersection of e_{DL} and ultimate void ratio distribution curve

Again it can be seen that void ratios lower than e_{DL} may exist below z_{DL} due to consolidation effects. It is also important to note that h_{1st} can be larger than h_{2nd} due to the low void ratio of a completely desiccated dredged material. A field indicator of the depth to which second-stage drying can be effective is the depth of cracks in the dredged material. Of course, cracks subjected to periodic rainfall are probably shallower than they would be under constant evaporative conditions.

(3) The preceding two equations form a rational basis for estimating the depths of crust formation in dredged material under first- and second-stage drying. They should be applicable whenever sufficient dredged material is

present to provide an intersection between the ultimate void ratio distribution and the appropriate limiting void ratio, and there is no external influence limiting the water table depth. If insufficient material is present, the entire dredged fill layer may be subjected to first- and second-stage drying processes in turn. If the water table depth is limited, the second-stage drying depth will be similarly limited. Again, the practical maximum depth of second-stage drying is best estimated from the maximum depth of desiccation cracks.

(4) The maximum depth of first-stage drying as expressed in Equation 5-11 should be a realistic measure for most fine-grained soils whose e_{SL} intersects the consolidated void ratio curve above the material coordinate defining the soil's maximum field crust thickness. For those soils with an e_{SL} so low that z_{SL} is greater than z_{DL} when based on the preceding considerations, the z_{SL} should be limited to no greater than z_{DL} .

c. Evaporation Efficiencies. The expression for defining the drying rate during second-stage evaporation will be simply a linear function of the water table depth:

$$C_E = C'_E \left(1 - \frac{h_{wt}}{h_{2nd}}\right) \text{ for } h_{wt} \leqq h_{2nd} \qquad \text{(F-13)}$$

where

C_E = evaporation efficiency

h_{wt} = depth of water table below surface

h_{2nd} = maximum depth of second-stage drying

C'_E = maximum evaporation efficiency for soil type

d. Water Loss and Desiccation Settlement.

(1) The water lost from a dredged material layer during first-stage drying can be written

$$\Delta W' = CS - C'_E \cdot EP + (1 - C_D)\ RF \qquad \text{(F-14)}$$

where

$\Delta W'$ = water lost during first-stage drying
CS = water supplied from lower consolidating material
EP = pan evaporation rate
CD = drainage efficiency
RF = rainfall

Even though some minor cracks may appear in the surface during this stage, the material will remain saturated, and vertical settlement is expected to correspond with water loss or

$$\delta'_D = -\Delta W' \qquad \text{(F-15)}$$

where δ'_D = settlement due to second-stage drying.

(2) Water lost during second-stage drying can be written

$$\Delta W'' = CS - C'_E \; 1 - \frac{h_{wt}}{h_{2nd}} \cdot EP + (1 - C_D)RF \qquad \text{(F-16)}$$

where

$\Delta W''$ = water lost during second-stage drying.

(3) Two things prevent an exact correspondence between water loss and settlement during second-stage drying. First is appearance of an extensive network of cracks that may encompass up to 20 percent of the volume of the dried layer. Second is the probable loss of saturation within the dried material itself. Combining these two occurrences into one factor enables the vertical settlement to be written

$$\delta''_D = -\Delta W'' - 1 - \frac{PS}{100} h_{wt} \qquad \text{(F-17)}$$

where

δ''_D = settlement due to second-stage drying

PS = gross percent saturation of dried crust that includes cracks

In determining the second-stage drying settlement, there are three unknowns and only two equations. Therefore, hand calculation involves an iterative procedure.

(4) The empirical approach as outlined and interaction of consolidation and desiccation are incorporated in the computer solutions described in 5-2.d. Use of the computer solutions is recommended for evaluation of the long-term storage capacity of confined disposal areas.

F-4. Example Consolidation Calculations.

a. Problem Statement. It is required to determine the time rate of surface settlement of a 10.0-foot-thick, fine-grained dredged fill material having a uniform initial void ratio after sedimentation of 7.0. The layer will be deposited on a normally consolidated compressible foundation 10.0 feet thick that overlies an impermeable bedrock. Laboratory oedometer testing of the dredged material resulted in the σ'-e relationship shown in Figure F-l and k-e , c_v-e , and g-e relationships as shown in Chapter 5. Laboratory oedometer testing of the foundation material resulted in the relationships shown in Figures F-9 and F-10. Laboratory testing also revealed specific gravity of solids G_s = 2.75 in the dredged material and G_s = 2.65 in the foundation material.

b. Void Ratio Distributions.

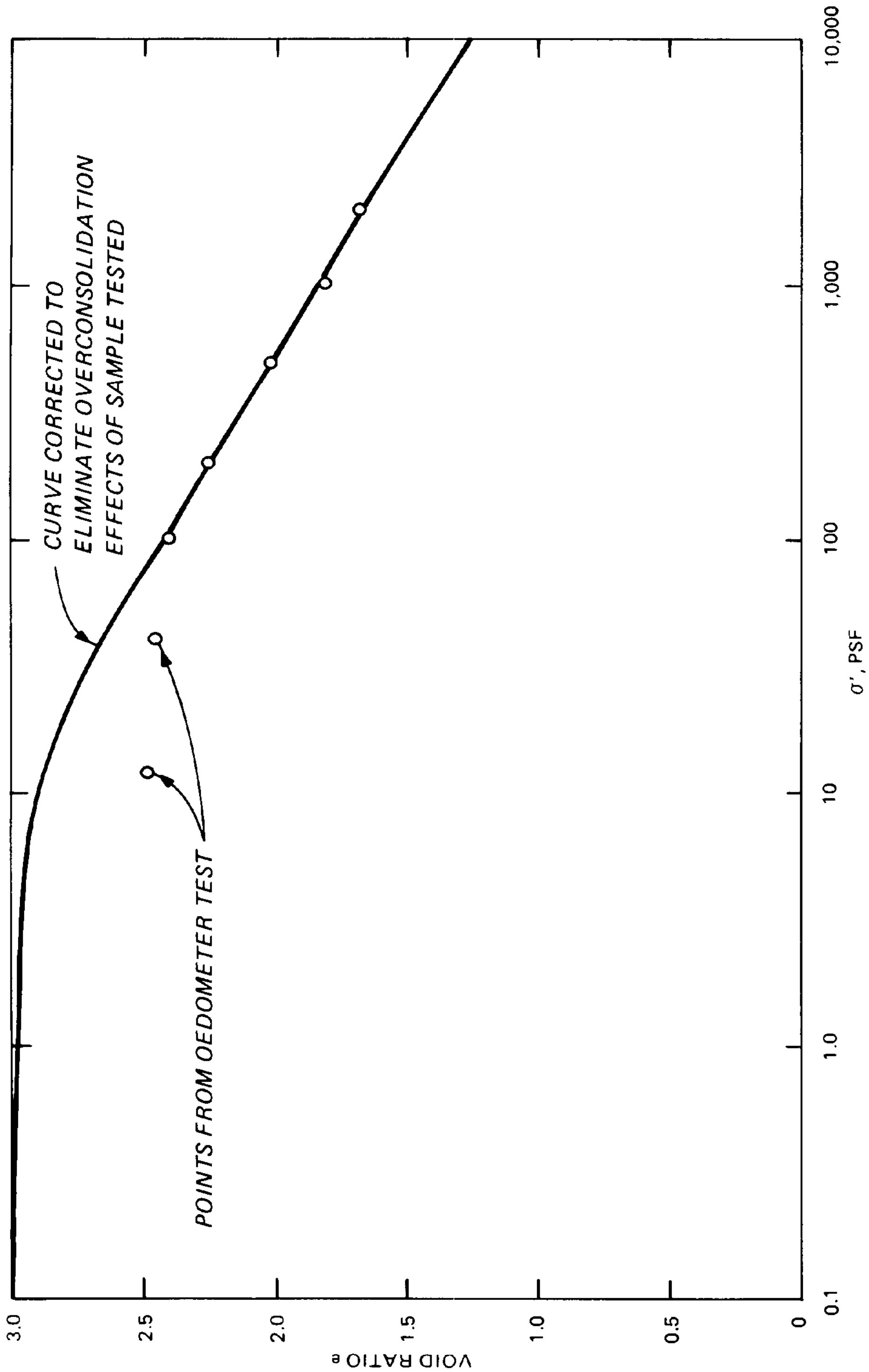


Figure F-9. Relationship between void ratio and effective stress, e-log σ' curve, for compressible foundation for oedometer testing

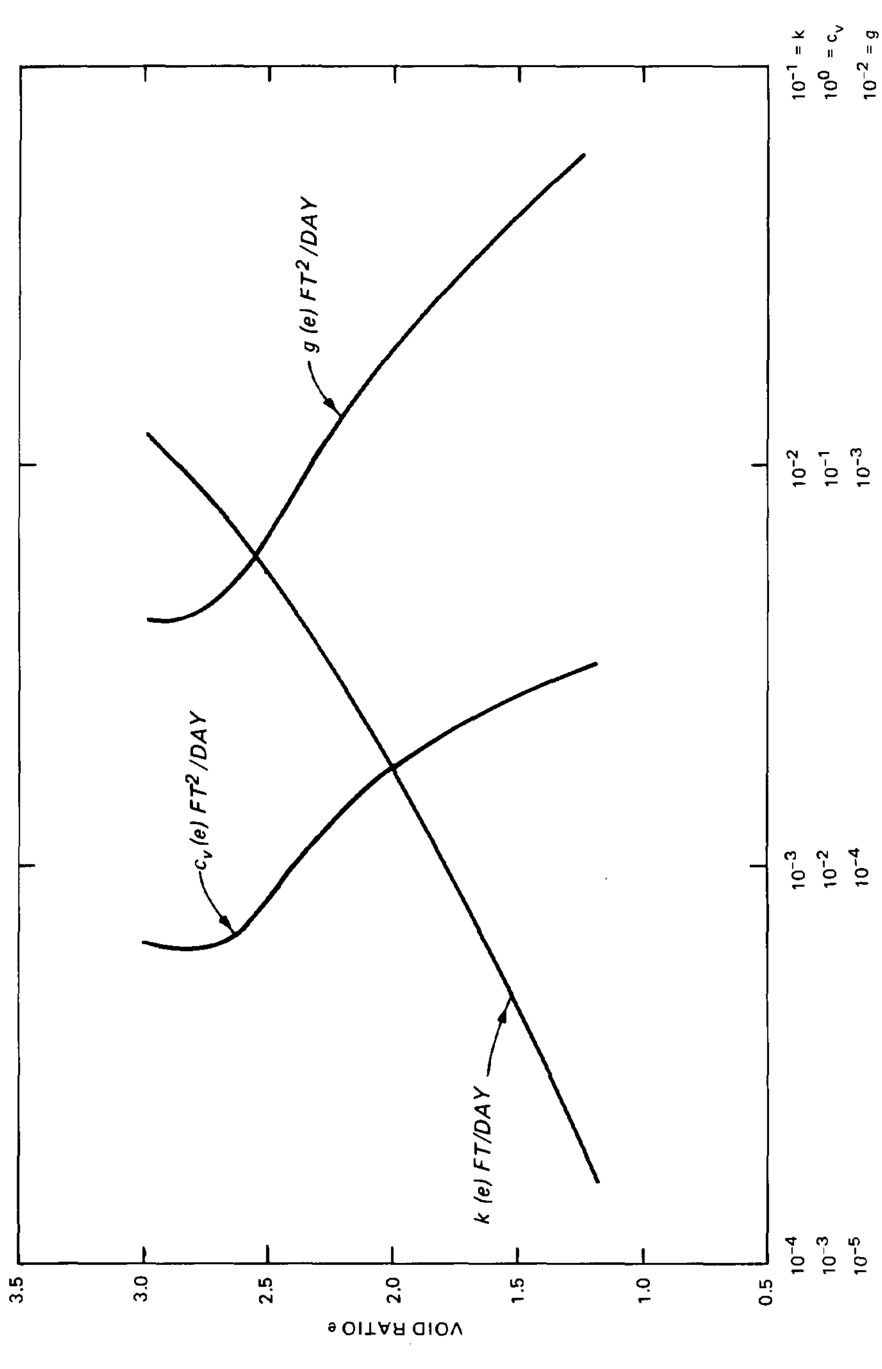


Figure F-10. Permeability and coefficients of consolidation as a function of void ratio for a compressible foundation

Table F-1
Void Ratio Distribution and Ultimate Settlement Calculations*

i	$h_{i,o}$ ft	ℓ_i ft	$\sigma'_{i,o}$ psf	$e_{i,o}$	$\sigma'_{i,\infty}$ psf	$e_{i,\infty}$	$h_{i,\infty}$ ft	$\delta_{i,\infty}$ ft
1	1.0	0.125	0.0	7.00	6.8	6.52	0.94	0.06
2	1.0	0.125	0.0	7.00	20.5	5.93	0.87	0.13
3	1.0	0.125	0.0	7.00	34.1	5.57	0.82	0.18
4	1.0	0.125	0.0	7.00	47.8	5.34	0.79	0.21
5	1.0	0.125	0.0	7.00	61.4	5.14	0.77	0.23
6	1.0	0.125	0.0	7.00	75.1	4.98	0.75	0.25
8	1.0	0.125	0.0	7.00	102.4	4.75	0.72	0.28
9	1.0	0.125	0.0	7.00	116.0	4.65	0.71	0.29
10	1.0	0.125	0.0	7.00	129.7	4.57	0.70	0.30
	Σ = 10.0	Σ = 1.250					Σ = 7.80	Σ = 2.20

Foundation

i	$h_{i,o}$ ft	ℓ_i ft	$\sigma'_{i,o}$ psf	$e_{i,o}$	$\sigma'_{i,\infty}$ psf	$e_{i,\infty}$	$h_{i,\infty}$ ft	$\delta_{i,\infty}$ ft
1	1.0	0.259	13.3	2.86	149.8	2.31	0.86	0.14
2	1.0	0.275	40.9	2.64	177.4	2.26	0.90	0.10
3	1.0	0.286	69.7	2.50	206.2	2.23	0.92	0.08
4	1.0	0.293	99.5	2.41	236.0	2.20	0.94	0.06
5	1.0	0.299	130.0	2.35	266.5	2.17	0.95	0.05
6	1.0	0.305	161.1	2.28	297.6	2.14	0.95	0.04
7	1.0	0.308	192.6	2.25	329.1	2.11	0.95	0.04
8	1.0	0.312	224.6	2.21	361.1	2.09	0.96	0.04
9	1.0	0.314	256.8	2.18	393.3	2.07	0.96	0.03
10	1.0	0.317	289.2	2.15	425.7	2.05	0.90	0.03
	Σ = 10.0	Σ = 2.968					Σ = 9.38	Σ = 0.61

* Symbols are defined in the main text.

(1) For the most accurate calculations, it is necessary to know the distribution of void ratios throughout the consolidating layers both before consolidation begins and after it ends. As an aid in this and later calculations, Table F-I is set up where the layers are subdivided into ten increments each. Entries in the table correspond to average conditions at the center of each sublayer.

(2) Completion of the table is a straightforward exercise for the dredged fill layer. The column for $e_{i,o}$ is given in the problem statement and the initial effective stress $\sigma'_{i,o}$ will always be zero by definition. The sublayer depth in reduced coordinates is calculated directly from Equation F-2.

$$\ell_i = \frac{h_{i,o}}{1 + e_{i,o}} = \frac{1.0}{1 + 7.0} = 0.125 \text{ ft}$$

The ultimate effective stress $\sigma'_{i,\infty}$ column is computed from Equation F-5. Thus

$$\sigma'_{1,\infty} = \frac{1}{2}\ell_1 \, (\gamma_s - \gamma_w) = \frac{0.125}{2} \, (2.75 - 1.0)62.4 = 6.8 \text{ psf}$$

and

$$\sigma'_{2,\infty} = \frac{1}{2}\ell_2 \, (\gamma_s - \gamma_w) + \ell_1(\gamma_s - \gamma_w) = 20.5 \text{ psf}$$

The final void ratio $e_{i,\infty}$ is read from the laboratory oedometer test curve. The usual e-$\log \sigma'$ curve is more accurate for this purpose than the curve given in Figure F-3. The final sublayer height $h_{i,\infty}$ is also calculated by substitution into Equation 5-2:

$$h_{1,\infty} = \ell_1(1 + e_{1,\infty}) = 0.125(1 + 6.52) = 0.94 \text{ ft}$$

(3) Completion of the table for the compressible foundation layer is not quite as simple since the initial void ratio is not usually known. However, it can be calculated given its e-$\log \sigma'$ curve in the normally consolidated state as shown in Figure F-9. An iterative process is required. First assume an initial void ratio for the first layer, $e_{1,o}$. Based on this void ratio, calculate ℓ from Equation F-2. Thus, assuming $e_{1,o} = 3.0$

$$\ell_i = \frac{h_1}{1 + e_{1,o}} = \frac{1.0}{1 + 3.0} = 0.250 \text{ ft}$$

Using this value of ℓ_1, $\sigma'_{1,o}$ is calculated from Equation F-5 as

$$\sigma'_{1,o} = \frac{1}{2}\ell_1(\gamma_s - \gamma_w) = \frac{0.250}{2}(2.65 - 1.0)62.4 = 12.9 \text{ psf}$$

Based on this value of $\sigma'_{1,o}$, a new estimate of $e_{1,0}$ is made from Figure F-9 and the process repeated until no further iterations are required. (Usually three iterations are required for an accuracy ±0.01 in the void ratio.) Using the total effective weight of the first layer, a first estimate of the void ratio in the second layer is made from Figure F-1 and its true average void ratio determined as was done with the first sublayer. The first estimate of each following sublayer is based on the effective weight of those above it.

(4) Once the initial void ratios and effective stresses have been determined throughout the compressible foundation, the final void ratios and effective stresses are easily found. The final effective stress $\sigma'_{1,\infty}$ is its initial value plus the effective weight of the dredged fill layer. Thus if

$$\sigma'_{\text{dredged fill}} = \ell_{d.f.}(\gamma_s - \gamma_w) = 136.5 \text{ psf}$$

then

$$\sigma'_{i,\infty} = \sigma'_{i,o} + 136.5$$

for the foundation. The final sublayer void ratio can then be read from the e-log σ' curve, and the final sublayer height $h_{i,\infty}$ can be calculated from Equation F-3.

c. Ultimate Settlement. Ultimate settlements for the compressible layers are calculated directly from Equation F-4 or from the difference in the sum of the sublayer heights initially and finally. As shown in Table F-l, for the dredged fill, δ_∞ = 2.20 feet, and for the foundation, δ_∞ = 0.61 feet. The fact that ultimate settlement plus total sublayer final heights in the foundation does not equal the initial total sublayer heights is due to round-off errors in the calculations.

d. Settlement as a Function of Time.

(1) A prerequisite to determining settlement as a function of time is the selection of an appropriate coefficient of consolidation during the course of consolidation, and in the case of linear finite strain theory, appropriate values for λ and N .

(2) For the dredged fill layer, a look at Table F-1 shows the void ratio will vary between the extremes 7.00 to 4.57. Figure F-2 is used to determine the appropriate coefficient of consolidation for the average void ratio during consolidation. For the foundation, where the void ratio extremes are 2.86 to 2.05, Figure F-10 is used.

(3) The value of λ must be determined by approximating the laboratory-determined curve with one of the form of Equation F-1. Figure F-12a shows that an appropriate value for the dredged fill is

$$\lambda = 0.026 \text{ cubic foot per pound}$$

and Figure F-12b shows that an appropriate value for the foundation is:

$$\lambda = 0.009 \text{ cubic foot per pound}$$

These curves were fitted in the range of expected void ratios only and should not be used in computations outside those ranges.

(4) All that remains is to calculate the dimensionless time factor from Equation F-6 where H = 10.0 feet initially for both layers with appropriate constants. By small strain theory, Figure F-11 is used to determine percent consolidation. Curve type I is used for the foundation and type III for the dredged fill. By linear finite strain theory, Figure F-3 is used for the foundation and Figure F-5 for the dredged fill. The calculations are organized in Table F-2 and results plotted in Figures F-13 and F-14.

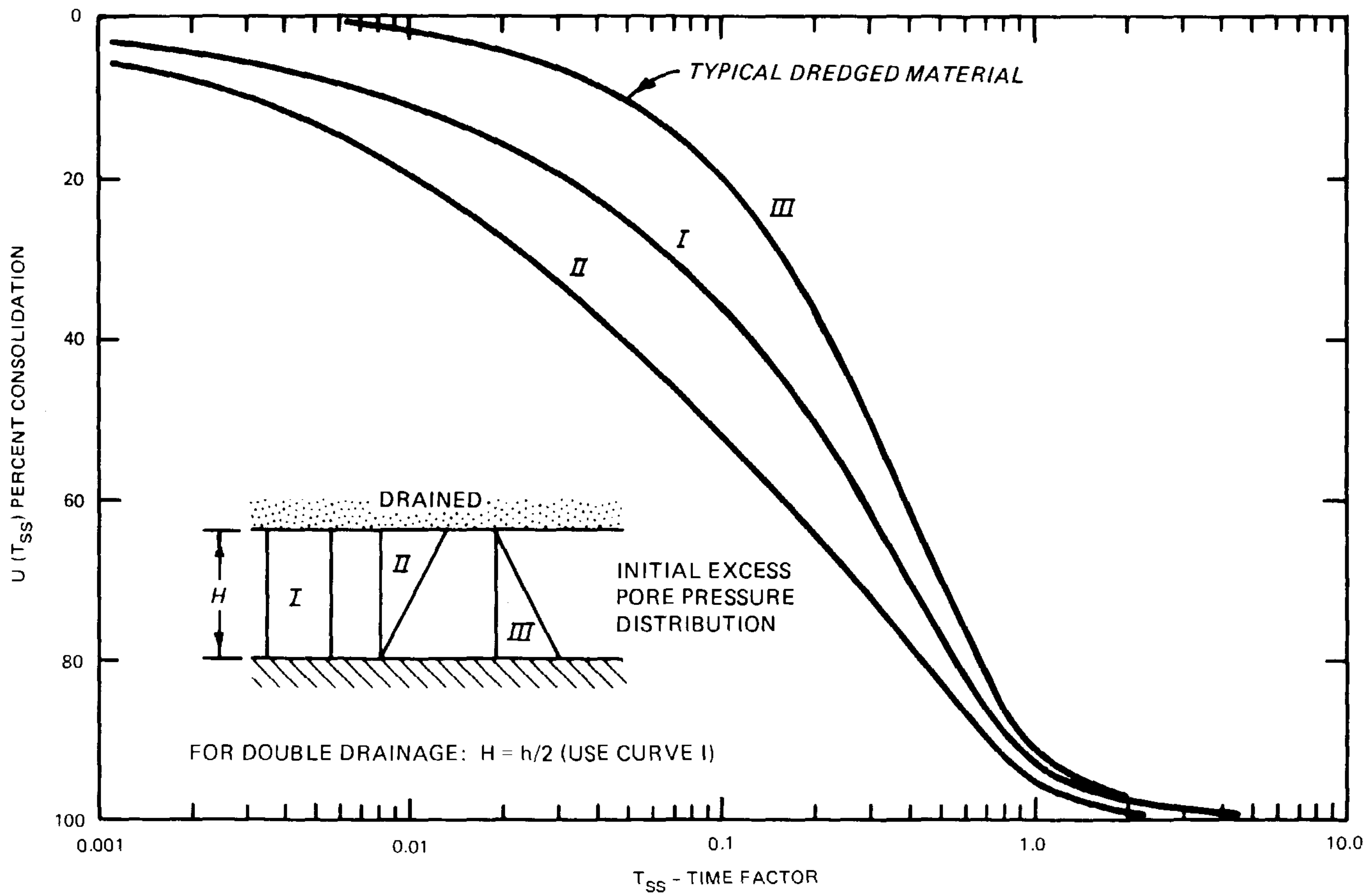
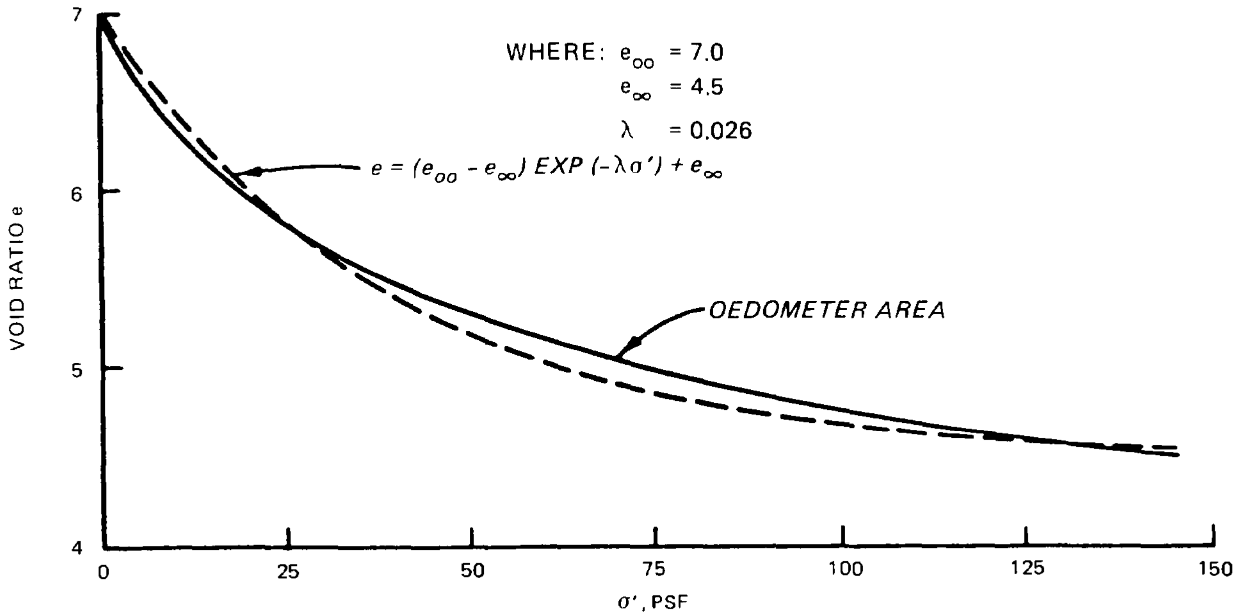
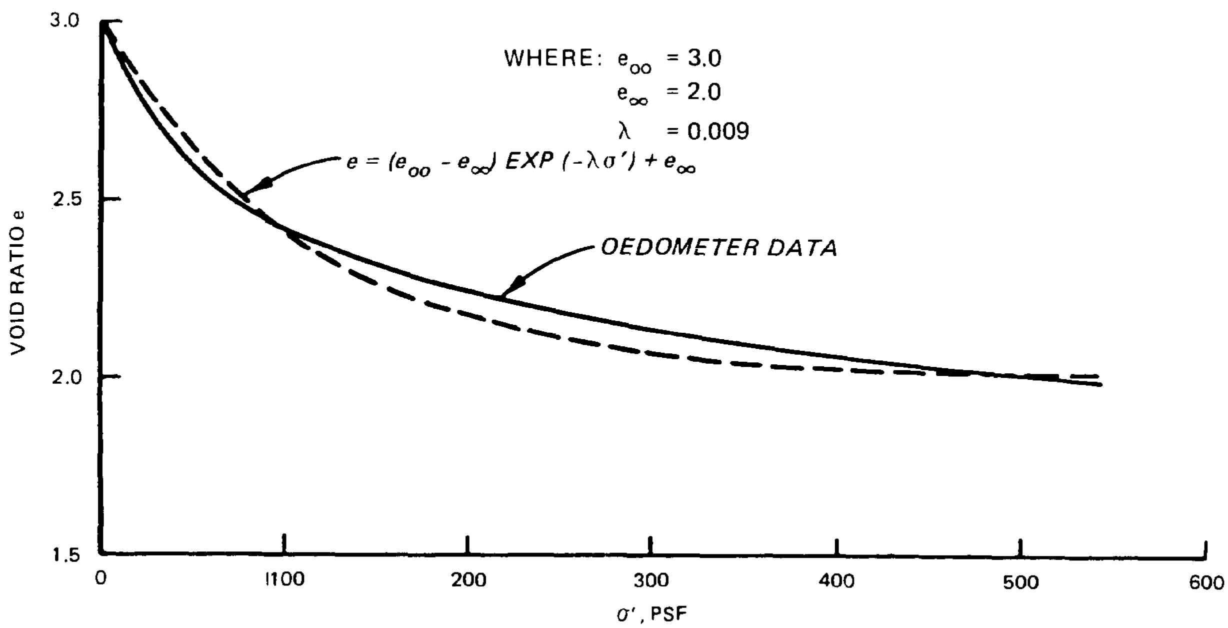


Figure F-11. Degree of consolidation as a function of the time factor for various initial conditions by small strain theory



a. DREDGED FILL



b. FOUNDATION

Figure F-12. Exponential void ratio-effective stress relationship fitted to oedometer data

Table F-2

Percent Consolidation and Settlement Calculations

				Small Strain Theory						Linear Finite Strain Theory		
t days	$\bar{e}$	$\bar{H}$ ft	c_v ft^2/day	T	U %	δ ft	$\bar{e}$	$\bar{H}$ ft	g ft^2/day	T	U %	δ ft
						Dredged Fill						
500	6.8	9.75	1.25×10^{-2}	0.066	14	0.31	6.4	9.25	2.16×10^{-4}	0.069	33	0.73
1,000	6.5	9.38	1.20×10^{-2}	0.136	26	0.57	5.9	8.63	2.410×10^{-4}	0.154	64	1.41
1,500	6.3	9.13	1.17×10^{-2}	0.211	39	0.86	5.5	8.13	2.73×10^{-4}	0.262	85	1.87
2,000	6.1	8.88	1.15×10^{-2}	0.292	50	1.10	5.3	7.87	2.96×10^{-4}	0.379	94	2.07
2,500	5.9	8.63	1.14×10^{-2}	0.383	60	1.32	5.3	7.87	2.96×10^{-4}	0.474	97	2.13
3,000	5.8	8.50	1.13×10^{-2}	0.469	68	1.50	5.3	7.87	2.96×10^{-4}	0.57	99	2.18
3,500	5.7	8.38	1.13×10^{-2}	0.56	74	1.63	5.3	7.87	2.96×10^{-4}	0.66	100	2.20
4,000	5.6	8.25	1.13×10^{-2}	0.66	80	1.76					100	2.20
4,500	5.5	8.13	1.13×10^{-2}	0.77	85	1.87					100	2.20
5,000	5.4	8.00	1.14×10^{-2}	0.89	89	1.96					100	2.20
						Foundation						
500	2.30	9.79	1.15×10^{-2}	0.060	28	0.17	2.25	9.65	1.19×10^{-3}	0.068	62	0.38
1,000	2.30	9.79	1.15×10^{-2}	0.120	40	0.24	2.20	9.50	1.30×10^{-3}	0.148	78	0.48
1,500	2.25	9.65	1.24×10^{-2}	0.200	51	0.31	2.20	9.50	1.30×10^{-3}	0.221	87	0.53
2,000	2.25	9.65	1.24×10^{-2}	0.266	58	0.35	2.15	9.35	1.45×10^{-3}	0.329	93	0.57
2,500	2.25	0.65	1.24×10^{-2}	0.333	65	0.40	2.15	9.35	1.45×10^{-3}	0.412	96	0.59
3,000	2.20	9.50	1.32×10^{-2}	0.439	73	0.45	2.15	9.35	1.45×10^{-3}	0.494	98	0.60
3,500	2.20	9.50	1.32×10^{-2}	0.51	77	0.47	2.15	9.35	1.45×10^{-3}	0.58	99	0.60
4,000	2.20	9.50	1.32×10^{-2}	0.59	81	0.49	2.15	9.35	1.45×10^{-3}	0.66	100	0.61
4,500	2.20	9.50	1.32×10^{-2}	0.66	84	0.51					100	0.61
5,000	2.20	9.50	1.32×10^{-2}	0.73	87	0.53					100	0.61

Dredged material: δ_∞ = 2.20 ft ; N = 3.55

Foundation: δ_∞ = 0.61 ft ; N = 2.75

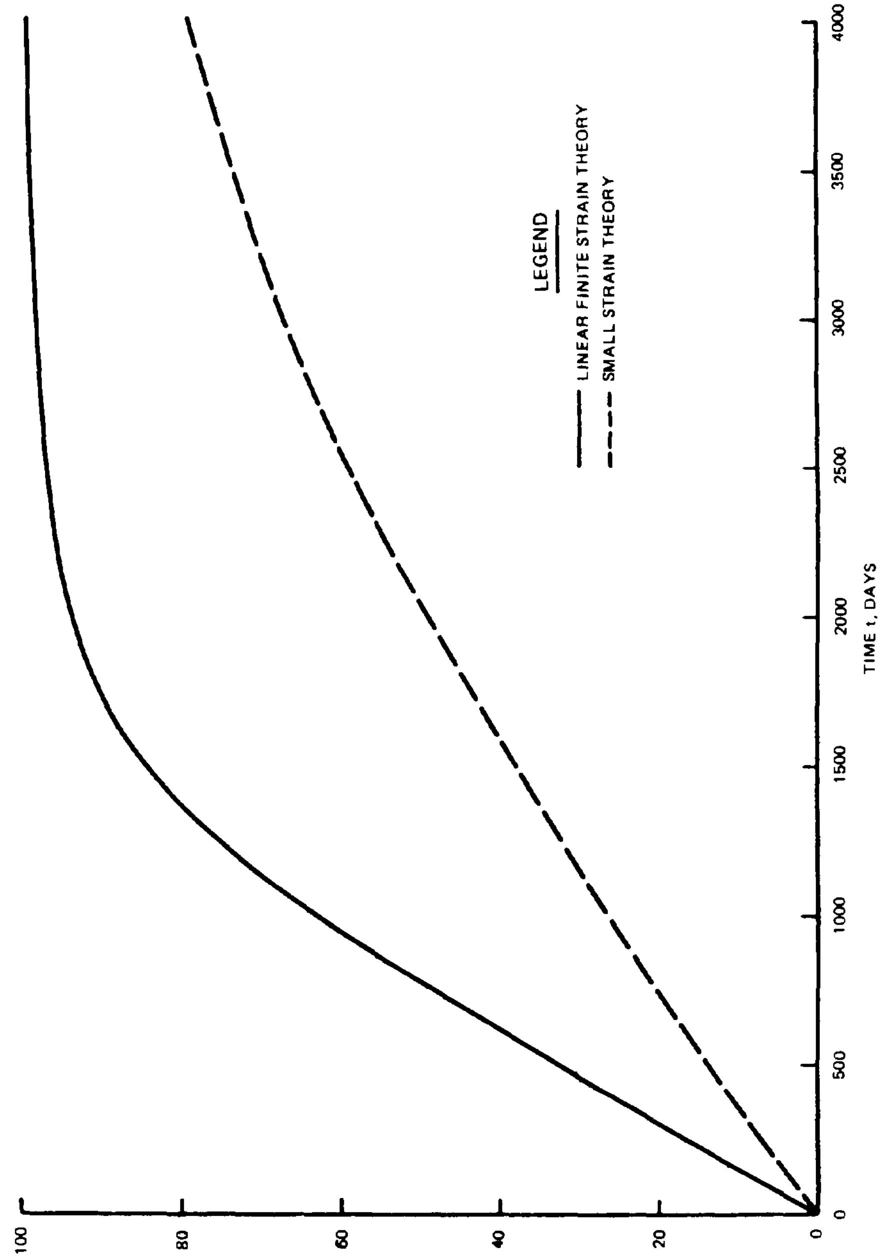


Figure F-13. Comparison of consolidation percentages in the dredged fill layer as a function of time

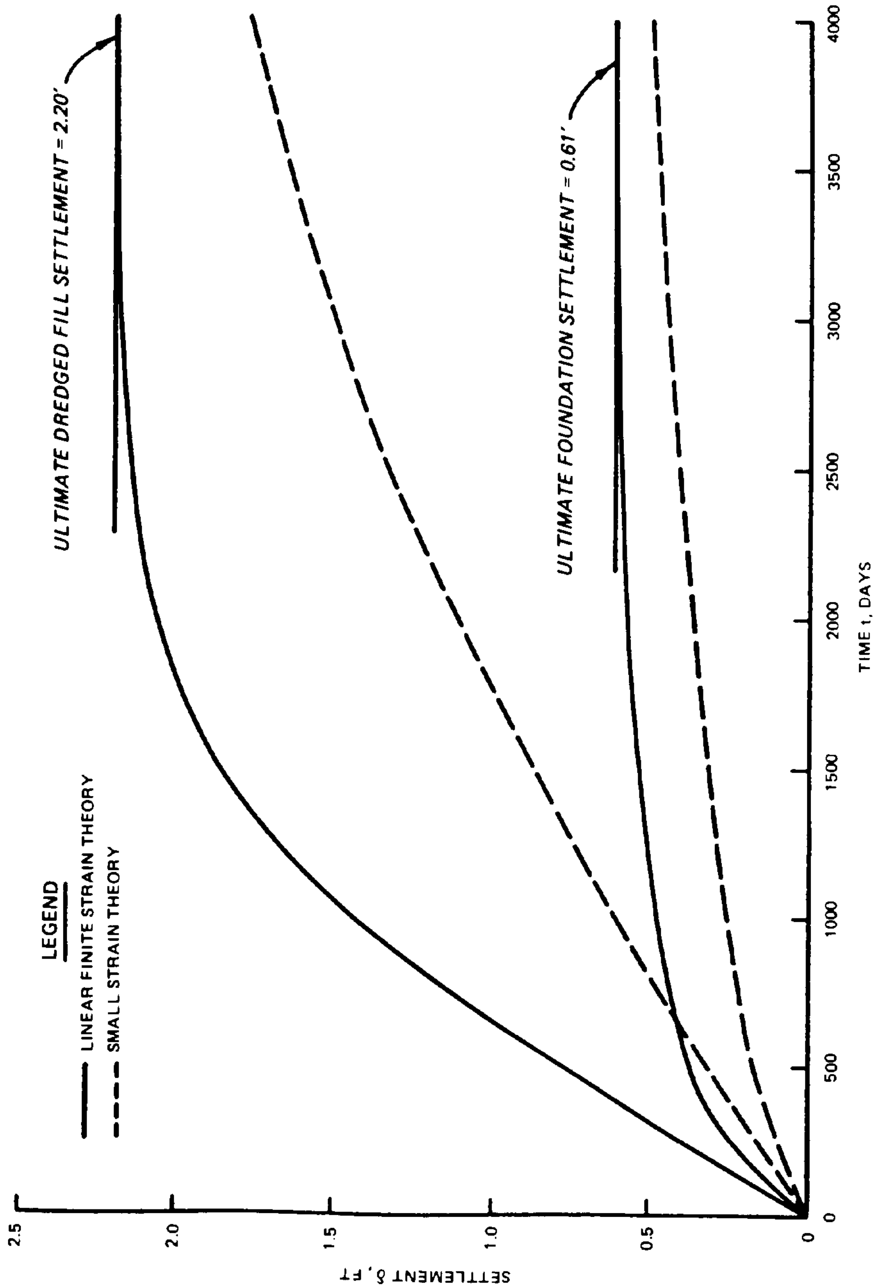


Figure F-14. Comparison of settlement predictions by small strain and linear finite strain theories

APPENDIX G

PROCEDURES AND EXAMPLE CALCULATIONS FOR DESIGN OF A CHEMICAL CLARIFICATION SYSTEM

G-1. Design Procedures.

a. Polymer Feed System.

(1) This design assumes that a low- to medium-viscosity liquid polymer is being used to minimize handling, pumping, and dilution problems. In most cases, the simplest system (shown in Figure G-l) is adequate. Polymer manufacturers should be able to inform the designer if this system is adequate. The experiments on polymer feed concentrations and aging should also indicate its adequacy. If the viscosity of the polymer is high or if low polymer feed concentrations are needed, systems like those shown in Figures G-2 and G-3 should be used. If the polymer requires aging prior to being fed, the two-tank system should be used. These systems are suitable for all but the smallest projects. Polymers requiring predilution should be avoided in systems like those in Figures G-2 and G-3 because they increase the equipment and operating labor requirements.

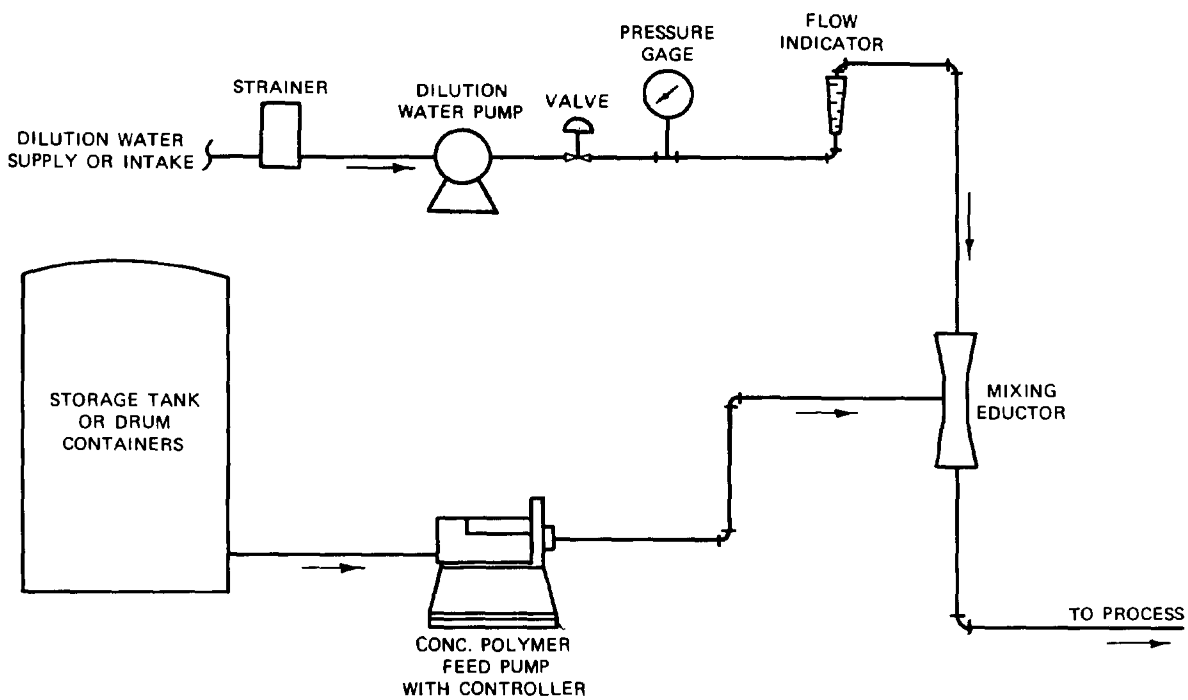


Figure G-1. Schematic of a simple liquid polymer feed system

(2) The polymer can be stored at the site in the delivery containers, either 55-gallon drums or bulk shipping tanks. The polymer can be fed directly from these containers or transferred to a polymer feed tank.

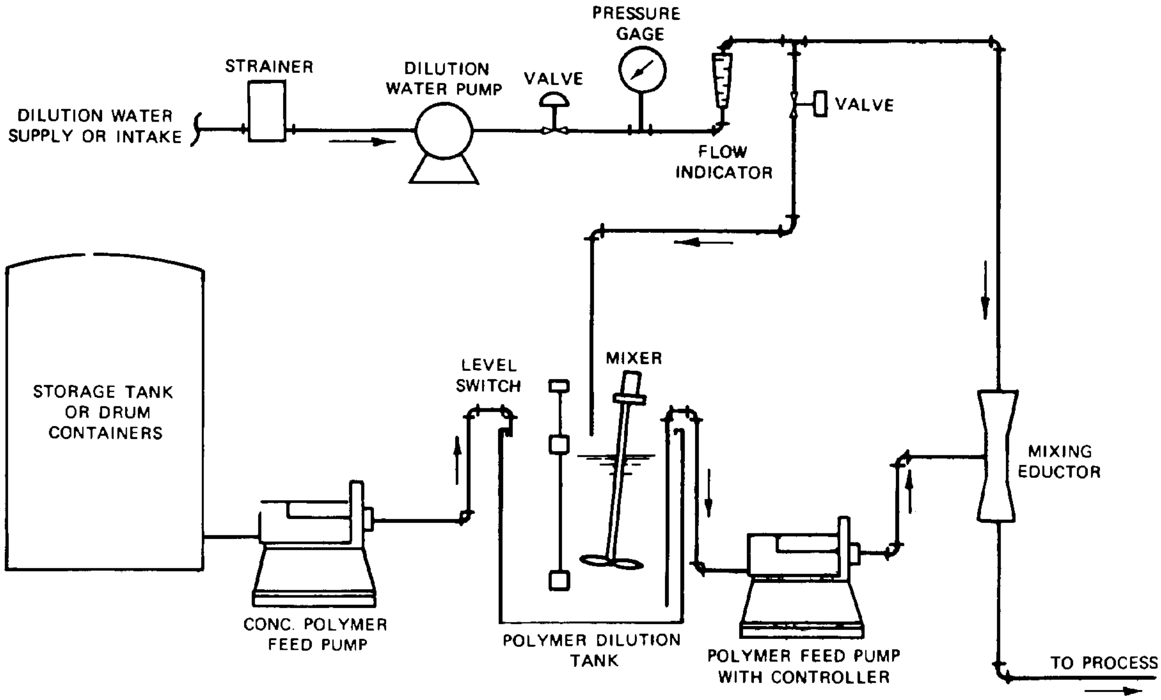


Figure G-2. Schematic of single-tank liquid polymer feed system

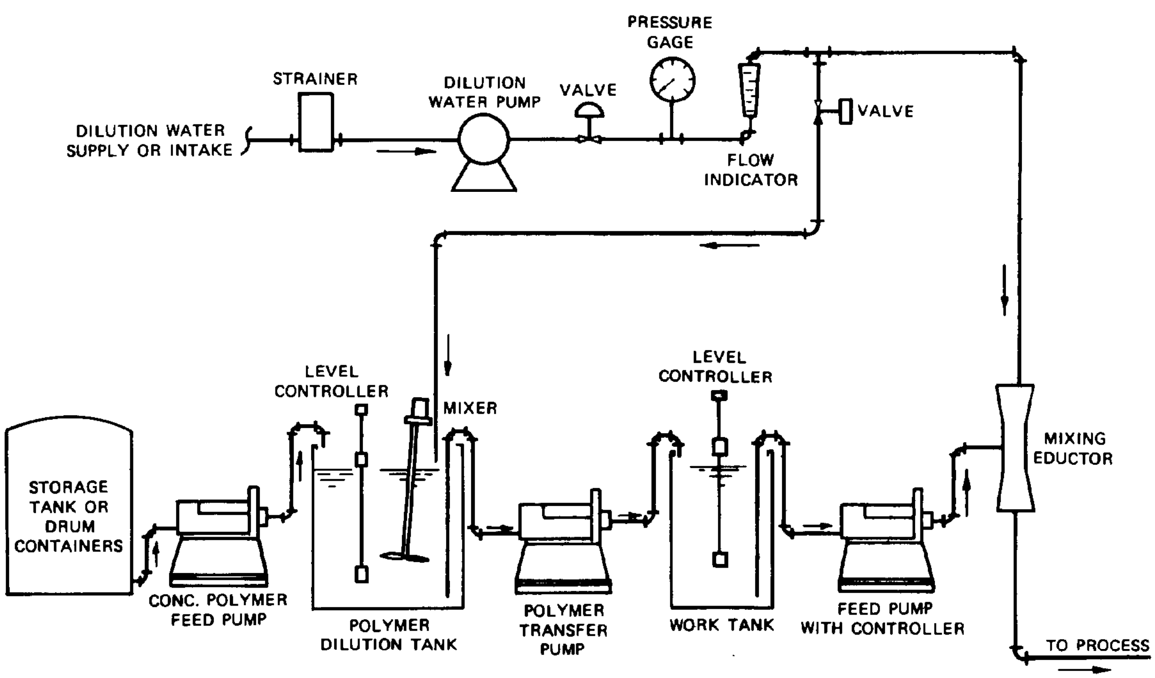


Figure G-3. Schematic of a two-tank liquid polymer feed system

Provisions should be made to guard against freezing. The feed tank may need to be heated or stored in a heated shelter to lower the viscosity and facilitate pumping on cold days. The size of the feed tanks and storage facilities is project dependent.

(3) The volume of polymer required for the project is calculated as follows:

Total Volume of Inflow, ℓ
= (Volume to Be Dredged, yd^3) x (In Situ Sediment Conc., g/ℓ)
x 764.4 ℓ/yd^3 ÷ (Dredged Material Slurry Conc., g/ℓ) (G-1)

Total Volume of Settled Material, ℓ
= (Total Volume of Inflow, ℓ) x(Influent Slurry Conc., g/ℓ)
÷ (Conc. of Settled Material, g/ℓ) (G-2)

If the concentration of settled material is unknown, it is generally conservative to let

Total Volume of Settled Material, ℓ
= 2 x (Volume to Be Dredged, yd^3) x 764.4 ℓ/yd^3 (G-3)

Then,
Total Volume to Be Treated, ℓ = (Total Volume of Inflow, ℓ)
- (Total Volume of Settled Material, ℓ) (G-4)

Total Volume of Polymer Required, gal
= (Required Dosage, mg/ℓ) x (Total Volume to Be Treated, ℓ)
÷ (Specific Weight of Polymer, kg/R)
÷ 10^6 mg/kg ÷ 3.785 ℓ/gal (G-5)

Total Poundage of Polymer, lb
= (Total Volume of Polymer, gal) x 3.785 ℓ/gal
x (Specific Weight of Polymer, kg/R) x 2.205 lb/kg (G-6)

(4) Concentrated polymer solutions should be fed using a positive displacement pump. The pump speed should be regulated by either a manual or automatic controller. The pump should be capable of discharging a wide range of flows to handle the possible range of required polymer dosages and flow rates of water to be treated. The pump capacity should be at least twice the maximum anticipated polymer feed rate or four times the average feed rate. The minimum pumping rate must be less than 10 percent of the average anticipated polymer feed rate to handle low flow conditions. The average polymer feed rate is

Avg. Feed Rate, ml/sec
= (Avg. Flow Rate, cfs) x (Avg. Required Dosage, mg/ℓ)
x 28.31 ℓ/ft^3 ÷ (Specific Weight of Polymer, g/ml)
÷ 1,000 mg/g (4-30)

The polymer pump flow capabilities should range from about

Pump Range, ml/set = (0.1 to 4) x (Avg. Feed Rate, ml/sec) (G-7)

Two polymer pumps operated in parallel may be required to provide the desired range of feed rates.

(5) If the polymer requires a tank for predilution, as in Figures G-2 and G-3, the polymer should be diluted by a factor of 10 or 20 in the tank. The polymer feed rate would then increase by this same factor.

(6) The polymer feed tanks and dilution tanks should be large enough to feed polymer for 1 to 2 days under average conditions. The average daily concentrated polymer feed volume is

$$\text{Daily Volume, gal/day} = (\text{Avg. Feed Rate, ml/set}) \times 86{,}400 \text{ set/day} \div 3{,}785 \text{ ml/gal} \quad \text{(G-8)}$$

(7) The polymer must be diluted to aid feeding and dispersion. The amount of dilution required can be determined from the manufacturer or experimentally. As a practical limitation, the dilution factor should not exceed 200 under average conditions due to excessive requirements for water at higher dilutions.

(8) Supernatant from the containment area, preferably treated supernatant from the secondary cell, can be used for dilution water. However, if the polymer is to be prediluted in a tank, water of good quality should be used to minimize deposition of material in the tank and to maintain the effectiveness of the polymer. The dilution water can be collected from a screened intake suspended near the surface at a place free of debris, resuspended material, and settled material.

(9) The dilution water may be pumped by any water pump. The pump capacity should be about 200 times the average polymer feed rate of concentrated polymer. A controller is not needed to regulate the dilution water flow rate since maximizing the dilution aids in dispersion. The polymer and dilution water may be mixed in-line using a mixing eductor.

(10) Any injection system can be used so long as it distributes the polymer uniformly throughout the water to be treated. It may consist of a single nozzle or a perforated diffuser pipe running along the weir crest. The system should be as maintenance-free as possible. Fine spray nozzles should be avoided because suspended material from the dilution water may clog them.

(11) The feed lines may be constructed of rubber hoses or PVC pipe. They must be designed to carry the design flows of the viscous polymer solution at low temperature. Provisions must be made to prevent freezing, particularly when the system is not operating.

b. Mixing System. The weir box and discharge culvert(s) should, if possible, be designed to provide adequate mixing. A 2-foot drop between the water surface of the first basin and the second basin is sufficient energy for mixing if efficiently used. Mechanical mixers should be considered if sufficient energy is unavailable. The design of mechanical mixing systems has been presented in item 22 and will not be duplicated here.

(1) Weir. The weir should be designed to collect supernatant from the primary containment area and to disperse the polymer thoroughly. The weir box does not provide efficient mixing, and, therefore, it is undesirable to lose all the energy of the water by a free fall into the weir box. The system should provide a small drop into the weir box and high head loss through the discharge culvert(s) between the primary and secondary containment areas.

(a) The weir box should be designed to prevent leakage; the bottom of the box should be sealed. Only one section of the box needs to be adjustable to the bottom of the box; this would minimize leakage. Weir boards with tongue and groove joints would also decrease leakage. The weir box should be submergible without the weir boards floating from their positions, All sections of the weir should be level and at the same elevation. An example is shown in Figure G-4.

(b) The height of the weir crest should be adjustable to stop the flow when the flow is too low to treat or to maintain the flow in order to keep treating when the dredge has stopped. The depth of flow over the weir must be controlled by increments of 1 to 2 inches to maintain a fairly constant flow rate. The weir must also be able to stop the flow when the treatment system is down for maintenance or repair. The simplest mode of operation would be to stop the flow over the weir by adding weir boards when the flow rate is low, and then to remove the added weir boards and resume operation after the elevation of the water surface returns to its height at average flow.

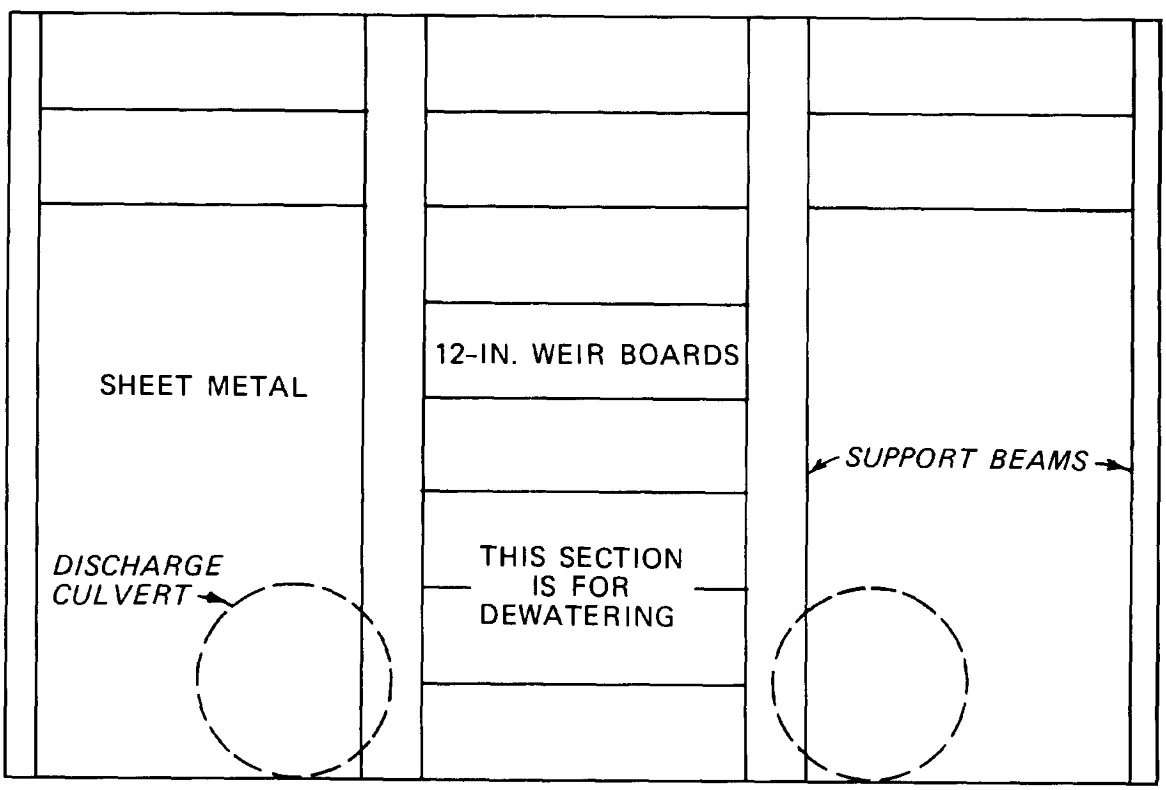


Figure G-4. Frontal view of a weir

(2) Discharge culvert. The discharge culvert(s) must be designed to provide the required mixing and to discharge the design flow rate safely. The design procedure presented here determines the length, diameter, and number of culverts that maximize mixing within the constraints of most projects. Frictional head loss provides the mixing and increases with increasing culvert length and decreasing diameter. Multiple culverts increase the duration of mixing but decrease the intensity of the mixing. Static mixers may be used in-line to increase the head loss of a culvert without increasing its length or decreasing its diameter. The use and design of static mixers will not be discussed herein, but information on their use is available from their manufacturers.

(a) The design approach is to size the culvert(s) for the maximum flow rate and the minimum available head and then to calculate the available mixing under average flow conditions. The maximum flow rate is assumed to be the average dredge flow rate with continuous, 24-hours-per-day production. The designer should also consider other possible sources of inflow. The average flow at the weir is assumed to be the product of average dredge flow rate and fractional production time ratio (generally about 0.75 or 18 hours-per-day). In this manner, the culvert(s) will be able to safely discharge the design flow. It is important to estimate the flow rates fairly accurately in order to properly size the culvert(s). Undersizing can result in overtopping the dikes or in forcing the dredge to operate intermittently. Oversizing can result in inadequate mixing. The amount of mixing can be compared with the mixing requirements determined experimentally to evaluate the design. If inadequate, the designer may wish to change the containment area design to provide a greater head for mixing. The required head can be determined using the design equations.

(b) The design procedure is as follows (see Figure G-5 for an example weir mixing system). Assume that the maximum flow rate is the average dredge flow rate with continuous production, 24 hours per day. Assume a 0.5-foot drop into the weir box under maximum flow. Determine the difference in elevation Δh, in feet, between the water surface of the basins at their highest operating levels from the design. Let H, in feet, $= \Delta h - 0.5$ where H is the maximum permissible head loss through the culvert at maximum flow. Assuming a submerged inlet and outlet and a corrugated metal culvert (though less head loss and better mixing for low flows would be realized if the outlet were not submerged), then

$$H = \left[1.5 + \frac{Lf}{D}\right] \frac{v^2}{2g} \qquad \text{(G-9)}$$

where

f = friction factor

D = culvert diameter, feet

Select the range of culvert lengths from containment area layouts. Let

Q = maximum flow rate, cubic feet per second

N_c = number of parallel culverts

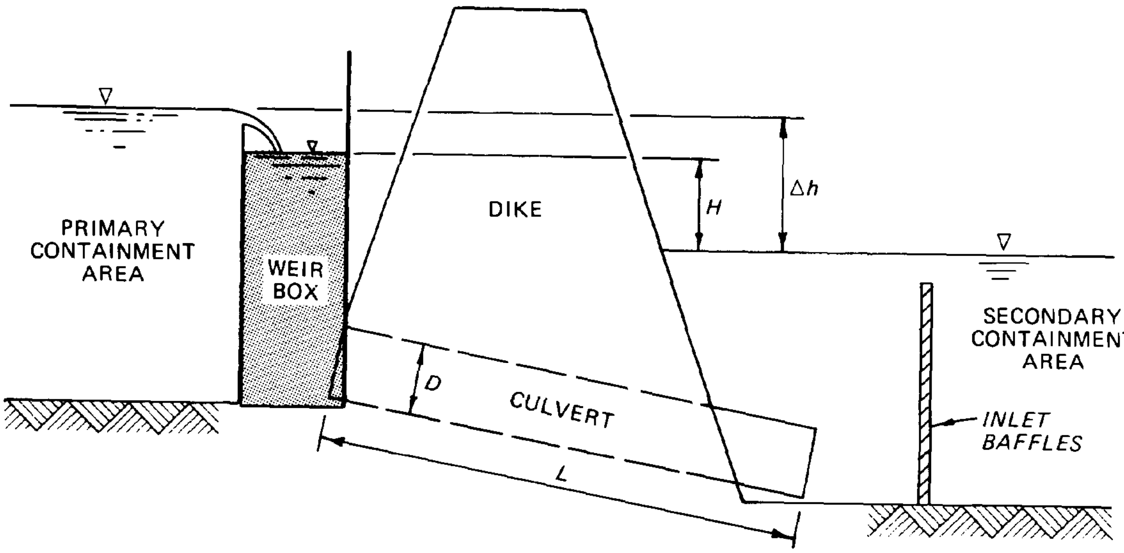


Figure G-5. Example weir mixing system

Then

$$H = \left[1.5 + \frac{185(0.025)^2 L}{D^{4/3}}\right] \frac{8Q^2}{g\pi^2 N_c^2 D^4} \qquad \text{(G-10)}$$

Rearranging the above equation gives

$$D = \left[\frac{8Q^2 \; 1.5D^{4/3} + 185(0.025)^2 L}{g\pi^2 H N^2}\right]^{3/16} \qquad \text{(G-11)}$$

This equation converges to the minimum diameter in three or four iterations by using 2 feet for the initial D and then substituting the calculated D for the next iteration. Solve the above equation using the minimum and maximum culvert length based on the containment area layout for up to five culverts. For each number of culverts, choose the largest commercially available diameter between the calculated diameters for the minimum and maximum culvert lengths. If there are not any commercial sizes between these diameters, select the next larger commercial size and the maximum length. Calculate the culvert length for the selected commercial sizes.

$$L = \left[\frac{g\pi^2 H N^2 D^4}{8Q^2} - 1.5\right] \left[\frac{D^{4/3}}{185(0.025)^2}\right] \qquad \text{(G-12)}$$

Calculate $\bar{v}$ and f for the selected sizes at average flow.

$$\bar{v} = 4\ \bar{Q}/\pi D^2 \qquad \text{(G-13)}$$

$$f = \frac{185\ (0.025)^2}{D^{1/3}}$$

where

$\bar{v}$ = mean velocity at average flow, feet per second
$\bar{Q}$ = average flow rate, cubic feet per second
f = friction factor

Calculate the mixing Gt of each design at average flow.

$$Gt = \sqrt{\frac{\gamma_s f \bar{v} L^2}{2g\mu_s D}} \qquad \text{(G-14)}$$

where

Gt = mixing effort

γ_s = specific weight, 62.4 pounds per cubic foot

μ_s = absolute viscosity, 2.36 x 10^{-5} pounds per second per square foot at 60" F

Calculate the head loss at average flow and the maximum carrying capacity of the culvert at a head of Δh to determine the limits of the design. Select the best overall design based on mixing, cost, operating flexibility, etc.

c. Secondary Containment Area.

(1) Design approach. The secondary area must be designed to provide adequate residence time for good settling and sufficient volume for storage of settled material. The total volume of the cell is the sum of the ponded volume and storage volume. The required ponded volume is a function of the hydraulic efficiency of the cell and the flow rate. The storage volume depends on the solids concentration entering the basin, the depth of the cell, the total volume to be treated, the flow rate, and the mud pumping schedule.

(2) Ponded volume. Effective settling requires a ponded depth of 2 to 3 feet and a minimum of 20 minutes of detention. Due to short-circuiting, the mean residence time should be at least 60 minutes and the theoretical residence time of the ponded volume should be at least 150 minutes. The shape of the cell should have a length-to-width ratio of at least 3:1 to reduce short-circuiting.

(3) Storage volume. The settling properties of flocculated dredged material resulting from chemical clarifications have not been well defined. Solids concentration or density profiles have been measured at only one field site. The settled material was very fluid and, as such, did not clog the inlet culvert, even though settled material accumulated near the inlet to a depth I foot higher than the top of the culvert. The kinetic energy of the inflow was capable of keeping the inlet clear of material. Resuspended material settled rapidly in the basin. The concentration of settled material at

the interface between the supernatant and settled layer was 50 grams per litre, and the concentration increased with increasing depth at a rate of 25 grams per litre per foot. Therefore, deeper basins stored more material in a given volume due to compaction. The concentration of the material increased rapidly upon dewatering.

(4) Storage requirements estimation. Knowing the average available depth of the secondary basin, the total storage requirements can be estimated as follows:

(a) The total mass of material to be stored M or pumped from the secondary area, M , is

$$M, g = (\text{Primary effluent conc.} - \text{Secondary effluent conc., } g/\ell) \times (\text{Volume to be treated, } \ell) \tag{G-15}$$

(b) The average concentration of settled material C_s is

$$C_s, g/\ell = [2 \times 50\ g/\ell + 25\ g/\ell\text{-ft} \times (\text{Average depth of storage, ft})] \div 2 \tag{G-16}$$

(c) Total volume of settled treated material V_s is

$$V_s, \ell = (M, g) \div (C_s, g/\ell)$$

$$V_s, ft^3 = (V_s, \ell) \div 28.32\ \ell/ft^3 \tag{G-17}$$

(d) The maximum area required A_s is

$$A_s, \text{acre} = (V_s, ft^3) \div (\text{Average depth of storage, ft}) \div 43{,}560\ ft^2/\text{acre} \quad (4\text{-}42) \tag{G-17b}$$

(5) Ponded area. The required volume V_p and area for ponding A_p is

$$V_P = (\text{Average flow rate, cfs}) \times 9{,}000 \text{ sec} \tag{G-18}$$

$$A_p = V_p \div (\text{Average depth of ponding, ft}) \tag{G-19}$$

(6) Design area. The containment area should be designed to have a total depth of the sum of the ponded depth and the depth of storage. The area of the cell should be the larger of the areas required for ponding and for storage. If the area required for storage is greater than the area required for ponding, the depth of ponding can be reduced to a minimum depth of 2 to 3 feet, thereby increasing the available depth of storage. If the area for storage is still greater, the only way to reduce the area requirements further would be to decrease the required storage volume by transferring settled treated material from the basin to the primary containment area. In the overall basin design, it is important to use the greatest practical depth and to optimize its use to provide good mixing through the discharge culvert, ponding for good settling, and storage for treated material. To minimize the size of the secondary area and to maximize the energy available for mixing, the secondary area should be used only for temporary storage, except for small one-time projects. Therefore, the settled treated material should be regularly removed from the basin. This approach would also facilitate dewatering and recurring use of the area for chemical treatment.

(7) Mud pumping. If the settled material is to be pumped, the required pumping rate would be

$$\text{Mass Pumping Rate, g/day} = (\text{Influent conc., g}/\ell - \text{Effluent conc., g}/\ell) \times (\text{Average flow rate, cfs}) \times 28.31\ \ell/\text{ft}^3 \times (\text{Seconds of production per day}) \quad \text{(G-20)}$$

$$\text{Volumetric Pumping Rate, ft}^3\text{/day} = (\text{Mass Pumping Rate, g/day}) \div [20 \times 50\ \text{g}/\ell + 25\ \text{g}/\ell\text{-ft} \times (\text{Average depth of storage, ft})] \times 2 \quad \text{(G-21)}$$

(8) Inlet baffles. The inlet hydraulics of the secondary area can have a significant effect on settling performance. Inlet baffles as shown in Figure 4-18 can reduce the effects of short-circuiting and turbulent flow and assist in distributing the flow laterally. The baffles should be placed about one culvert diameter directly in front of the inlet. The baffle should be at least two diameters wide and may be either slotted or solid. Slotted baffles are better and may be made of 4- by 4-inch wooden posts spaced several inches apart. The main purpose of the inlet baffles is to dissipate the kinetic energy of the incoming water and reduce the velocity of the flow toward the weir.

(9) Effect on dewatering. Design of the secondary area must consider dewatering of the primary area. If the primary area is to be dewatered using the primary weir box to drain the water, the elevation of the surface of the water or stored material in the secondary area must be lower than the final elevation of the stored material to be attained during dewatering. The elevation difference should be at least 2 feet if the drainage is to be treated. The point is demonstrated in Figure G-6.

(10) Alternatives. There are several alternatives that can be used to provide for dewatering:

(a) The secondary area can be constructed at a lower elevation.

(b) The settled, treated material stored in the secondary area can be dewatered and thereby consolidated first.

(c) The material can be pumped out of the secondary area.

(d) The water can be pumped out of the primary area.

(e) A special drainage structure can be constructed to drain the primary cell.

(f) A channel can be cut through the settled material in the secondary area to permit drainage through the basin. The best approach depends on site- and project-specific considerations. The effect of treatment on dewatering of the primary area is just one example showing that the designer should consider the entire disposal operation when designing the treatment system. Treatment should not be added to a disposal operation as an afterthought.

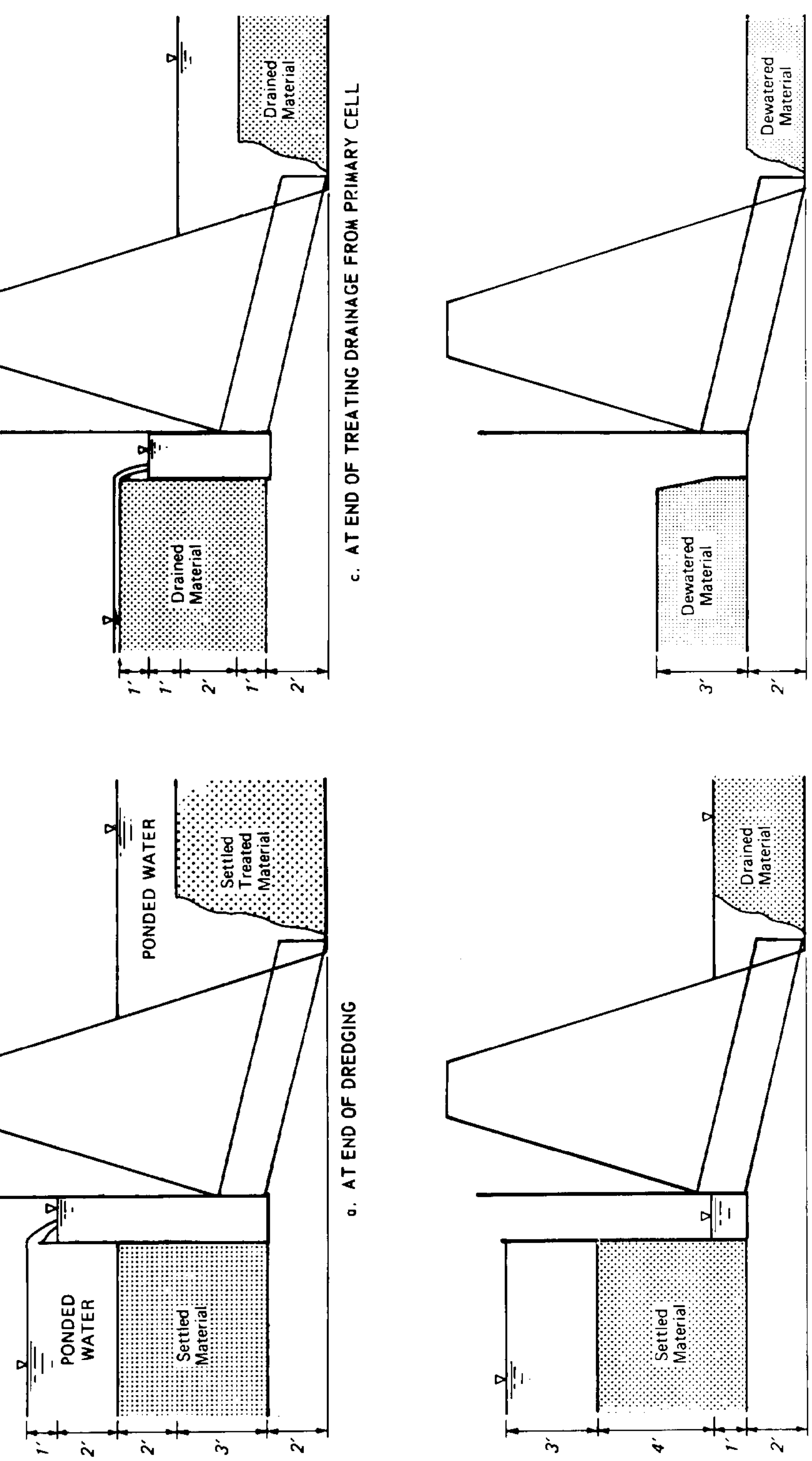


Figure G-6. Example elevations in containment areas during dewatering sequence

d. General Design Considerations.

(1) Shelter. A building should be provided to house the equipment and to furnish shelter for the operators. An 8- by 12-foot portable building is sufficient unless the polymer storage tank and dilution tanks must be housed.

(2) Equipment. The equipment should be simple, rugged, heavy-duty, continuous-duty, and low-maintenance. Backup equipment must be provided for all essential components.

(3) Safety. Good lighting must be provided for the entire work area. The weir must be furnished with a walkway and railings. Provisions should be made for safe, simple adjustments of the weir boarding.

e. Operating Guidelines. Prior to the start of the project, an operator's manual and treatment log book should be prepared to minimize problems during the operation of the treatment system. The operator's manual should contain the maintenance schedule, procedures for operating each piece of equipment and the weir, and the procedures for adjusting the polymer dosage. The treatment log book should be used to keep a complete record of the treatment operation. The record should include hours of operation, flow rate, polymer dosage, influent and effluent turbidity, basin depths, depth of settled treated material, maintenance actions, problems, and significant observations.

(1) Maintenance schedule. The maintenance schedule and operating procedures for the equipment are dependent on the equipment selection and should be developed specifically for the selected pieces. To set the polymer dosage, it is first necessary to calibrate the polymer pump. The polymer flow rate should be measured for the range of controller settings. Next, based on the laboratory results, a table should be prepared that gives the required dosage as a function of influent turbidity. Then, a table of controller settings should be prepared for a variety of dosages and flow rates. At low flow rates, there is less mixing, and the polymer is less effective. Therefore, higher dosages are often required at low flow. If a relationship between mixing and required dosage was developed in the laboratory, the relationship should be converted to relate flow rate and dosage so the operator can readily adjust the dosage. The required dosages must be verified during the start of operation, and the values in the tables must be adjusted accordingly. After verification, the operator would only have to measure the influent turbidity and flow rate to determine the controller setting for the polymer pump.

(2) Field dosage verification. During verification of the required dosages, the effectiveness of a particular dosage can be evaluated immediately by grabbing a sample of treated suspension from the end of the discharge culvert connecting the two containment areas and running a column settling test on the sample. If the supernatant is clear after 10 minutes of settling, the dosage should be decreased until the supernatant is slightly cloudy. Better clarification will be achieved in the settling basin, where the material can flocculate. This is especially true when the system has been operating continuously for a long period. After selecting a dosage, the effluent turbidity should be monitored to determine whether the dosage should be adjusted further. The dosage should be minimized to reduce chemical costs, but the

effluent quality should not be allowed to deteriorate beyond the effluent requirements.

(3) Flow measurement.

(a) The flow rate can be estimated by measuring the weir length and the depth of water flowing over the weir crest, as described in Chapter 4.

(b) A table relating the depth of flow over the weir h and the flow rate Q should be generated and included in the operator's manual. The weir length should be measured, not taken from design drawings, to ensure accuracy. With this table, the operator would easily be able to estimate the flow rate by measuring the depth of flow without performing any difficult computations or requiring additional information. The operator should measure the depths at several locations along the weir crest and average the resulting flow rates to determine the overall flow rate. This method would minimize the estimating errors caused by an unlevel or uneven weir crest.

(c) The weir crest may become submerged at flow greater than 20 percent above the average. The actual flow rate that submerges the weir is dependent on the weir length and culvert design. The flow rate over submerged weirs is controlled by the discharge capacity of the culvert.

(4) Weir operation.

(a) The weir must be properly operated to maintain good mixing conditions. The weir crest must be kept sufficiently high to maintain the required difference in elevation between the water surfaces of the two containment areas. The weir should also be used to maintain the required flow rate for good mixing. When the flow decreases below the minimum rate for good mixing, the operator should either lower the weir crest by 1 or 2 inches to increase the flow to its average rate, or raise the weir crest sufficiently to stop the flow.

(b) The minimum flow rate is based on the experimentally determined minimum acceptable mixing Gt for effective treatment. The minimum flow can be determined as follows:

$$Q_{min} = Q_{avg} {\frac{Gt_{min}}{Gt_{avg}}}^{2} \qquad \text{(G-22)}$$

An example computation is given below:

Given: Average flow = 25 cubic feet per second
Gt of average flow = 9,000
Minimum acceptable Gt = 6,000

The minimum allowable flow is

$$Q_{min} = 25 \text{ cubic feet per second } {\frac{6,000}{9,000}}^{2}$$

= 11.1 cubic feet per second

(c) In general, the weir crest should be operated at the highest practical elevation, and the primary containment area should be allowed to fill to this elevation before any water is discharged over the weir and treatment is started. This would maximize the depth and provide the best conditions for mixing, settling, and storage. Maintaining the maximum ponded depth in the primary area also minimizes the turbidity of the discharge to be treated and therefore reduces the required polymer dosage.

(5) Other considerations.

(a) General operation. During the project, the primary and secondary effluent turbidities and the flow rate should be measured at least six times per day, and the polymer flow rate should be adjusted as needed. Each piece of equipment should be inspected regularly, particularly the water intake, injection rig, and pumps. The fuel and chemical levels should also be checked as required. Regular maintenance must be performed throughout the project. The buildup of settled treated material should be followed, and the material should be pumped out of the basin as the storage volume is depleted.

(b) Leakage. The operator should try to eliminate leakage through the weir when the treatment system is turned off. The flow rate of the leakage is too low to treat, but after a couple of days of downtime, the leakage can completely exchange the contents of the secondary area if left unchecked. Since it is untreated, the effluent quality will deteriorate markedly.

(c) Dewatering. At the end of the project, the treatment system can be used to treat the drainage from the primary containment area during dewatering. The elevation of the interface of the settled material in the primary area must be greater than the elevation of the water surface of the secondary area. Therefore, the secondary area must be dewatered first to compact the settled treated material and to provide the depth required to treat the drainage at the lower weir height. It is possible that treated material may need to be pumped from the secondary area before the primary area can be dewatered through the weir.

G-2. Polymer Feed System Design Example. Given the following project information and laboratory results, the design would proceed as follows:

a. Project Information:

In situ sediment volume	200,000 cubic yards
In situ sediment conc.	900 grams per litre
Specific gravity of sediment	2.68
Dredged material slurry conc.	150 grams per litre
Dredge discharge pipe size	14 inches
Production time	100 hours per week
Avg. conc. of settled material	400 grams per litre
Mean daily temperature	50° F

Laboratory Results:

Selected polymer	low viscosity liquid
Specific weight of polymer	1.0 kilograms per litre
Required dosage at average flow and turbidity	10 milligrams per litre
Polymer feed concentration	20 grams per litre

b. Polymer Requirements:

$$\text{Volume of Inflow, } \ell = 200{,}000\ \text{yd}^3 \times 900\ \text{g}/\ell \times 764.4\ \ell/\text{yd}^3 \div 150\ \text{g}/\ell = 9.17 \times 10^8\ \ell \quad \text{(G-23)}$$

$$\text{Volume of Settled Material, } \ell = 9.17 \times 10^8\ \ell \times 150\ \text{g}/\ell \div 400\ \text{g}/\ell = 3.44 \times 10^8\ \ell \quad \text{(G-24)}$$

$$\text{Volume to be Treated, } \ell = 9.17 \times 10^8\ \ell - 3.44 \times 10^8\ \ell = 5.733 \times 10^8\ \ell \quad \text{(G-25)}$$

$$\text{Volume of Polymer Required, gal} = 10\ \text{mg}/\ell \times 5{,}733 \times 10^8\ \ell \div 1.10\ \text{kg}/\ell \div 10^6\ \text{mg/kg} \div 3.785\ \ell/\text{gal} = 1{,}380\ \text{gal} \quad \text{(G-26)}$$

$$\text{Pounds of Polymer Required, lb} = 1{,}380\ \text{gal} \times 3.785\ \ell/\text{gal} \times 1.10\ \text{kg}/\ell \times 2.205\ \text{lb/kg} = 12{,}640\ \text{lb} \quad \text{(G-27)}$$

c. Storage. Since less than 2,000 gallons of polymer is required, drums should be used for storage instead of a bulk tank. The drums may be stored outside, since they are not expected to freeze during the project. However, barrel warmers should be used to aid in transferring the polymer to the feed tank due to the cool temperature. A hand pump or a small electric positive displacement pump should be used for the transfer from storage.

d. Polymer Pump. The feed system shown in Figure G-l should be used, since the selected polymer is a liquid of low viscosity requiring a 50-fold dilution. The average polymer flow rate is

$$\text{Avg. Dredge Flow Rate} = 15\ \text{fps} \times \pi/4 \times (14\ \text{in.} \div 12\ \text{in./ft})^2 = 16.04\ \text{cfs}$$

$$\text{Avg. Polymer Flow Rate} = 16.04\ \text{fps} \times 10\ \text{mg}/\ell \times 28.31\ \ell/\text{ft}^3 \div 1.10\ \text{g/ml} \div 1{,}000\ \text{mg/g} = 4.13\ \text{ml/set} = 0.065\ \text{gpm or } 94.2\ \text{gpd}$$

The polymer pump capacity should be about four times the average rate or 0.25 gallon per minute. The pump should be able to pump as low a flow as 0.4 millilitre per second or 0.0065 gallon per minute.

e. Polymer feed tank. The polymer feed tank should be sized to hold a 2-day supply of polymer. The tank should be kept in a heated shelter with the pumping equipment.

Tank Volume = 94.2 gpd x 2 days
x (0.8, the production efficiency)
= 150 gal

f. Dilution Water Pump. To reduce the polymer feed concentration below 20 grams per litre, the dilution factor must be 55. At average polymer flow rate, the required dilution water flow rate would be 3.6 gallons per minute. The dilution water pump capacity should be twice this rate to dilute higher polymer flow adequately. Therefore, the dilution water flow rate should be

Dilution Water Pump Rate = [(1.1 x 1,000 g/ℓ) ÷ 20 g/ℓ]
x 2 x 0.0654 gpm = 7.20 gpm

The pump must deliver this flow rate and produce high pressure (60 pounds per square inch) to force the viscous polymer solution through the eductor, feed lines, and injector.

g. Feed Lines. The size of the feed lines should be determined by head loss analysis for pipe flow. This subject is discussed in any fluid mechanics textbook or hydraulics handbook. The pipe diameter is dependent on the viscosity, flow rate, length of line, minor losses, and losses through the eductor and injector. One-inch inside diameter (ID) rubber hose or PVC pipe should be used for this example. The head loss would be less than 30 pounds per square inch.

G-3. Example Culvert Design. Given an l&inch-diameter dredge pipeline, a minimum head difference of 3 feet between the primary and secondary cells, and a range of culvert lengths between 50 and 100 feet based on the containment area design, the culvert design would proceed as follows:

a. Q_{max} = 15 fps x π(18 in./12 in./ft)^2 ÷ 4
= 26.5 cfs

Q_{ave} = 26.5 cfs x (Production ratio, 0.75)
= 19.9 cfs

b. Δh = 3 ft
H = 3 ft - 0.5 ft = 2.5 ft

c. Using equation G-11, the calculated minimum diameters for the following lengths and numbers of culverts are:

N	L, ft	D, ft	D, in.
1	50	2.23	26.8
1	100	2.44	29.3

2	50	1.67	20.0
2	100	1.85	22.2
3	50	1.42	17.0
3	100	1.57	18.8
4	50	1.26	15.1
4	100	1.41	16.9
5	50	1.15	13.8
5	100	1.29	15.5

d. Using Equation G-12, the selected commercial sizes and calculated lengths are:

N	D, in.	L, ft
1	27	54.1
2	21	69.3
3	18	73.3
4	18	100.0
5	15	83.0

e. Using Equation G-13, the friction factor and velocity at average flow are:

N	D, in.	$\bar{v}$, fps	ft
1	27	5.00	0.0882
2	21	4.14	0.0959
3	18	3.75	0.1010
4	18	2.82	0.1010
5	15	3.24	0.1073

f. Using Equation G-14, mixing at average flow:

N	D, in.	L, ft	G (sec^{-1})	t (sec)	Gt
1	27	54.1	449	10.8	4,855
2	21	69.3	400	16.7	6,690
3	18	73.3	382	19.5	7,470
4	18	100.0	249	35.5	8,830
5	15	83.0	346	25.6	8,870

g. Head loss at average flow:

$$H = 1.41 \text{ feet}$$

h. Flow through a completely submerged weir:

$$Q = 29.0 \text{ cubic feet per second}$$

1. Generally, a Gt of about 8,000 provides adequate mixing for chemical treatment. In this example, either three la-inch-diameter, 73-foot-long culverts; four 18-inch-diameter, 100-foot-long culverts; or five 15-inch-diameter, 83-foot-long culverts could be used. However, four la-inch-diameter culverts would be the best design, since it would provide considerably more mixing than three culverts and about the same mixing as five culverts. Also, this design would provide better mixing at lower flow rates.

G-4. Design Example. Given the following project information, the settling basin size would be determined as follows:

a. Project Information.

Primary effluent solids conc.	2 grams per litre
Secondary effluent solids conc.	50 milligrams per litre
Volume to be treated (as determined in the polymer feed system design)	5×10^8 litres
Depth of basin	6 feet
Average flow rate	16 cubic feet per second

b. Volume of Settled Treated Material. Assuming a ponded depth of 3 feet,

From Equation G-15, mass of settled material = (2 - 0.05) g/ℓ x 5 x 10^8 ℓ
= 9.75 x 10^8 g

From Equation G-16, avg. conc. of settled material
= [(2 x 50 g/ℓ) + (25 g/ℓ-ft x 3 ft)] ÷ 2
= 88 g/ℓ

From Equation G-17, volume of settled material
= 9.75 x 10^8 g ÷ 88 g/ℓ
= 1.11 x 10^7
= 3.91 x 10^5 ft^3 or 9.0 acre-ft

c. Required Area Based on Storage.

From Equation G-18,
Area = 9.0 acre-ft ÷ 3 ft
= 3.0 acres

d. Volume of Ponding.

From Equation G-19,
Ponded volume = 16 cfs x 9,000 sec
= 1.44 x 10^5 ft^3 or 3.3 acre-ft

e. Required Area Based on Ponding.

From Equation G-20,
Area = 3.3 acre-ft ÷ 3 ft
= 1.1 acres

f. Second Trial. The areas based on storage and ponding are quite different. Therefore, the ponded depth should be decreased to reduce the area required for storage.

Using a ponded depth of 2 feet and, therefore a storage depth of 4 feet,
From Equation G-15,
Avg. conc. of settled material
= [(2 x 50 g/ℓ) + (25 g/ℓ-ft x 4 ft)] ÷ 2 = 100 g/ℓ

From Equation G-17,
Volume of settled material = 9.75 × 10^8 g ÷ 100
= 9.75 × 10^6 ℓ
= 3.45 × 10^5 ft^3
= 7.9 acre-ft

From Equation G-17b
Area for storage = 7.9 acre-ft ÷ 4 ft
= 1.98 acres

From Equation G-18,
Ponded volume = 16 cfs x 9,000 sec
= 1.44×10^5 ft^3
= 3.3 acre-ft

From Equation G-19,
Area for ponding = 3.3 acre-ft ÷ 2 ft
= 1.65 acres

g. Final Design. The two areas in the second trial are similar, indicating a better design. Therefore, the secondary cell should have the following characteristics:

Volume	12 acre-ft or 5.2×10^5 ft^3
Area	2 acres
Depth	6 feet
Storage depth	4 feet
Ponded depth	2 feet

G-5. Mud Pumping.

a. The area and depth of the basin can be reduced further if the basin is not used for storage, that is, if the settled material is pumped out regularly. The size could be reduced to about an area of 1.0 acre and a depth of 5 feet. With a shallow storage depth, the solids concentration of the settled material would be about 60 grams per litre.

b. The mud pumping rate, assuming 16 hours of production per day would be:

From Equation G-20,
Solids Pumping Rate = (2.0 - 0.05) g/ℓ
x 28.31 ℓ/ft^3 x 16 cfs
x 16 hr/day x 3,600 sec/hr
= 5.09×10^7 g/day

From Equation G-21,
Volumetric Pumping Rate = 5.09×10^7 g/day ÷ 60 g/ℓ
= 8.5×10^5 ℓ/day
= 0.347 cfs or 156 gpm

APPENDIX H

MONTHLY STANDARD CLASS A PAN EVAPORATION FOR THE CONTINENTAL UNITED STATES

THE FOLLOWING EVAPORATION CHARTS
ARE BASED ON US WEATHER SERVICE DATA

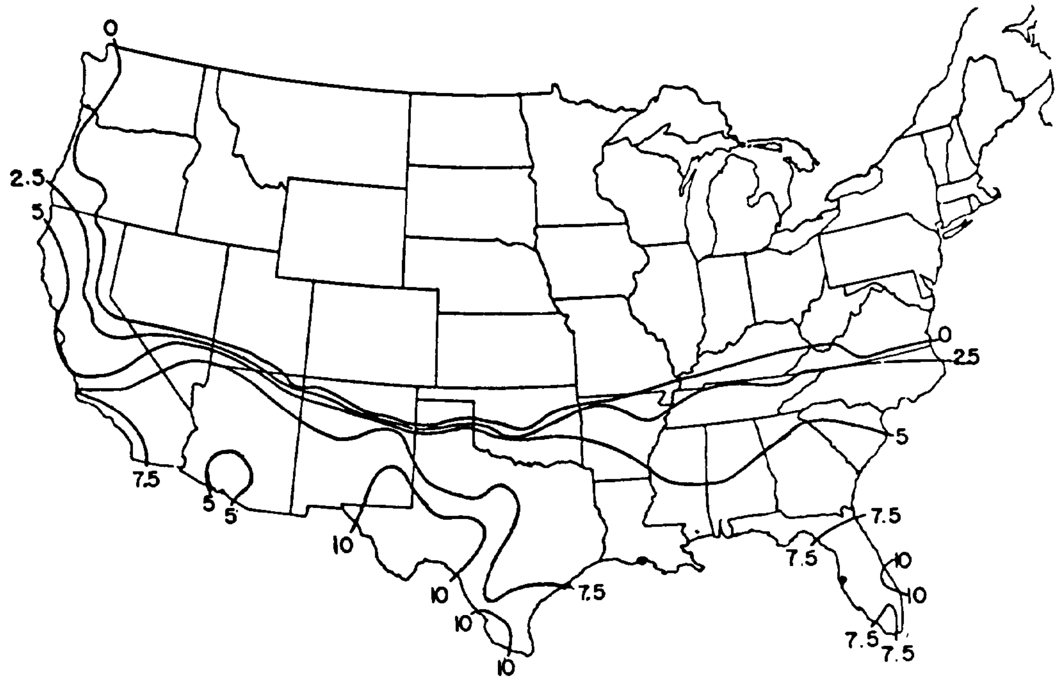


Figure H-l. Average pan evaporation, in centimetres, for the continental United States for the month of January based on data taken from 1931 to 1960

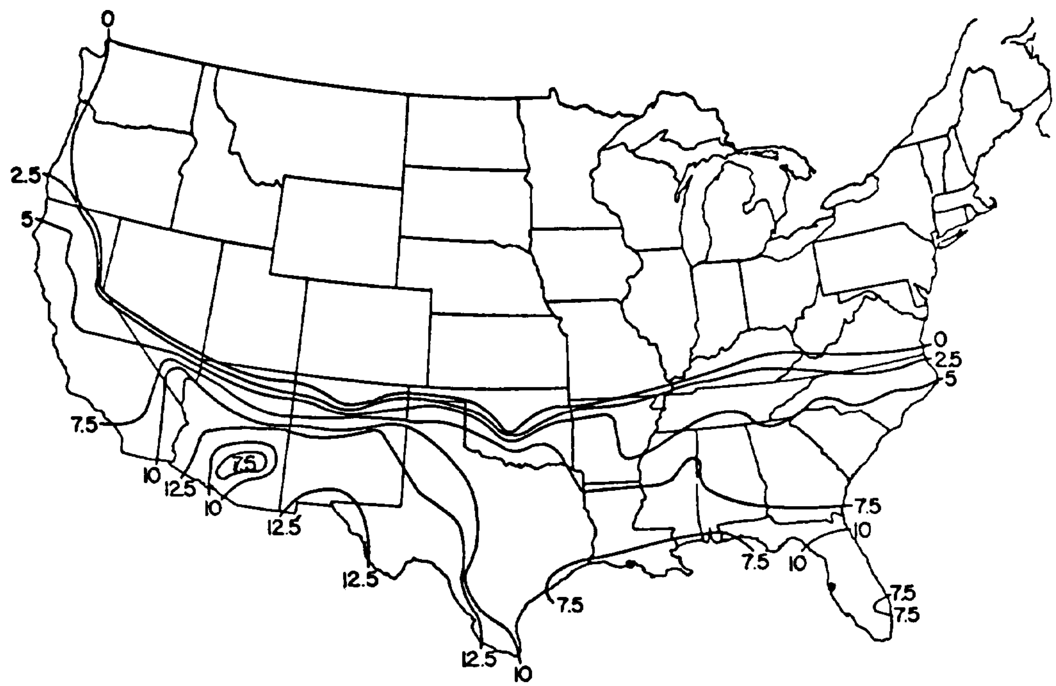


Figure H-2. Average pan evaporation, in centimetres, for the continental United States for the month of February based on data taken from 1931 to 1960

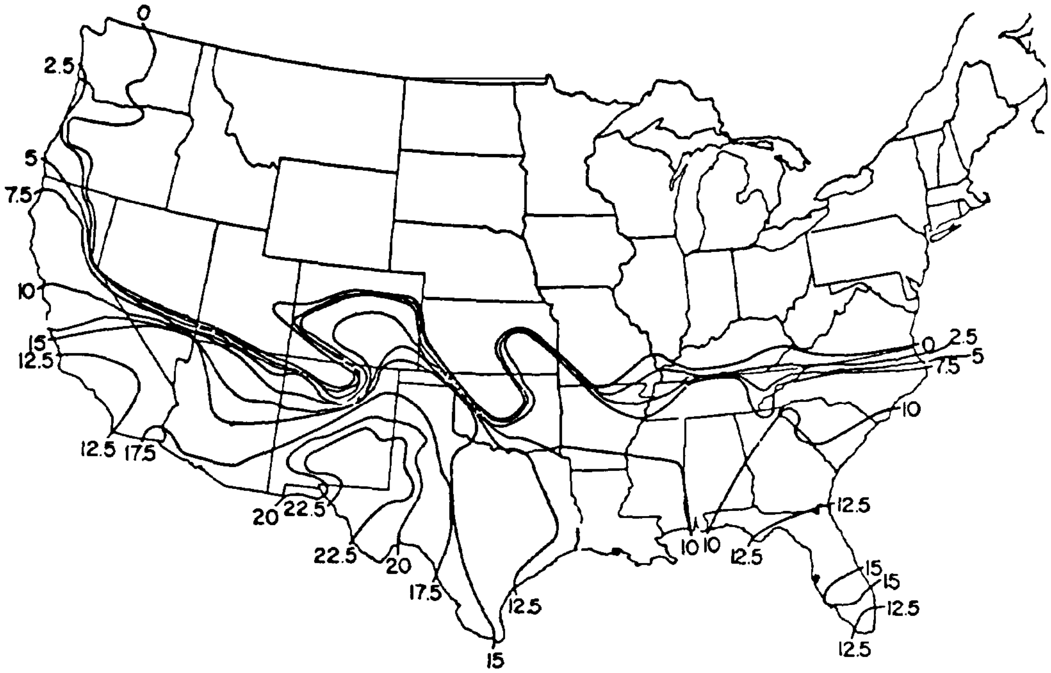


Figure H-3. Average pan evaporation, in centimetres, for the continental United States for the month of March based on data taken from 1931 to 1960

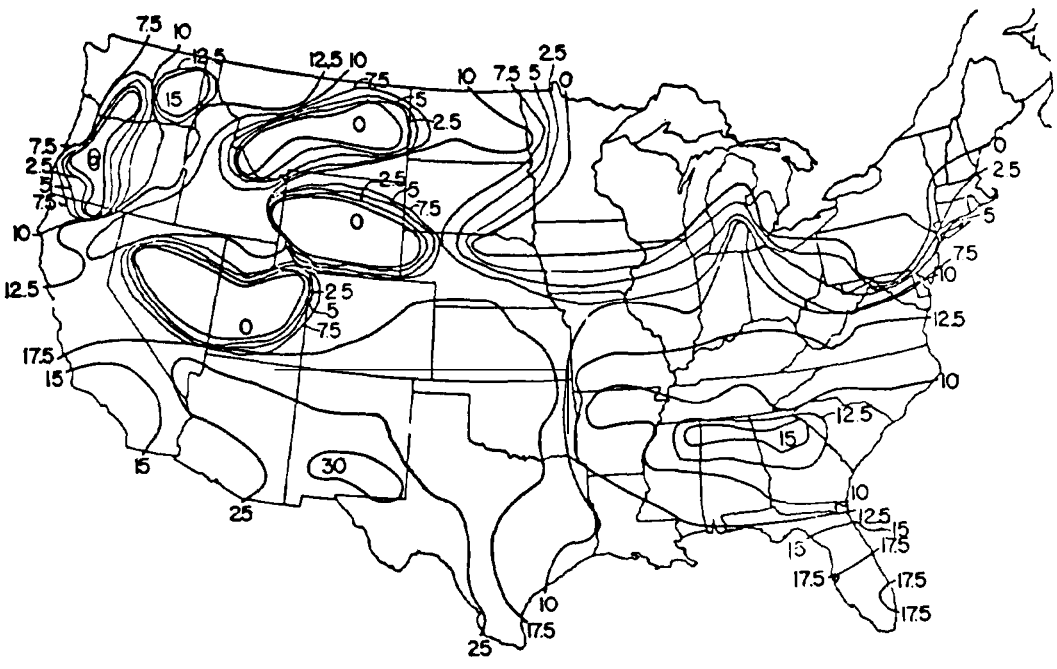


Figure H-4. Average pan evaporation, in centimetres, for the continental United States for the month of April based on data taken from 1931 to 1960

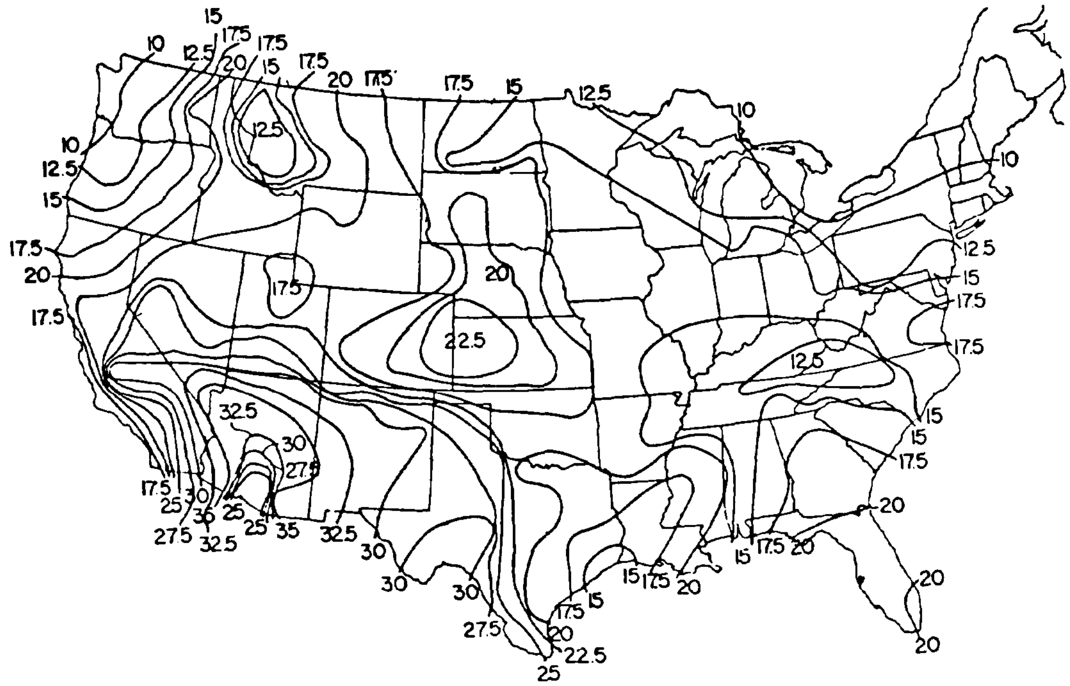


Figure H-5. Average pan evaporation, in centimetres, for the continental United States for the month of May based on data taken from 1931 to 1960

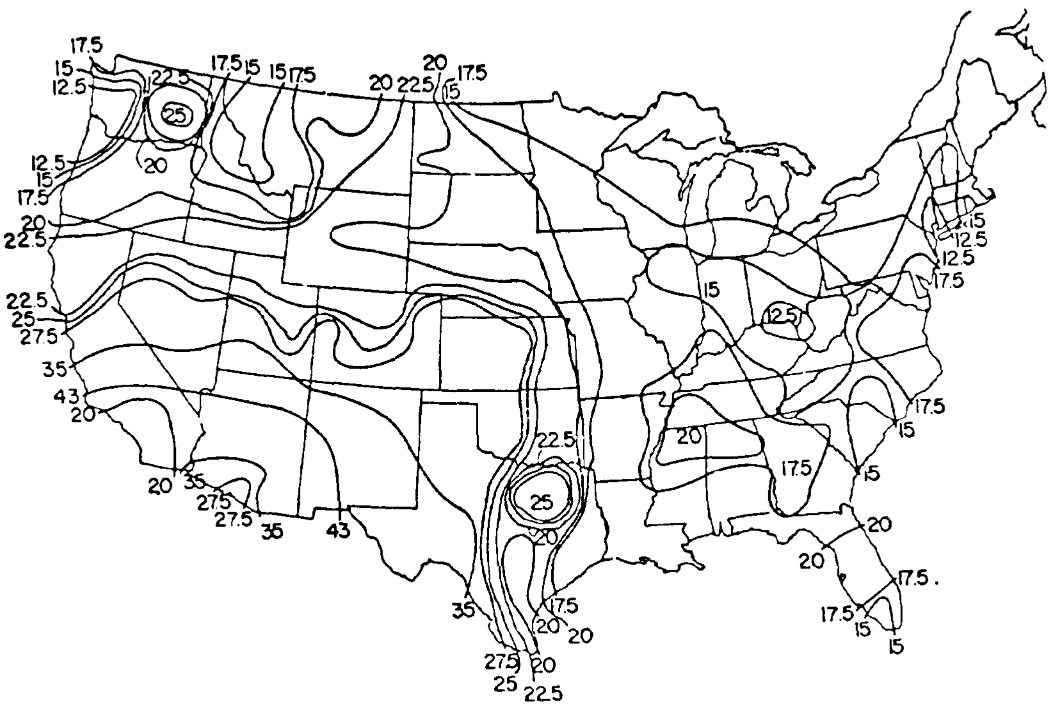


Figure H-6. Average pan evaporation, in centimetres, for the continental United States for the month of June based on data taken from 1931 to 1960

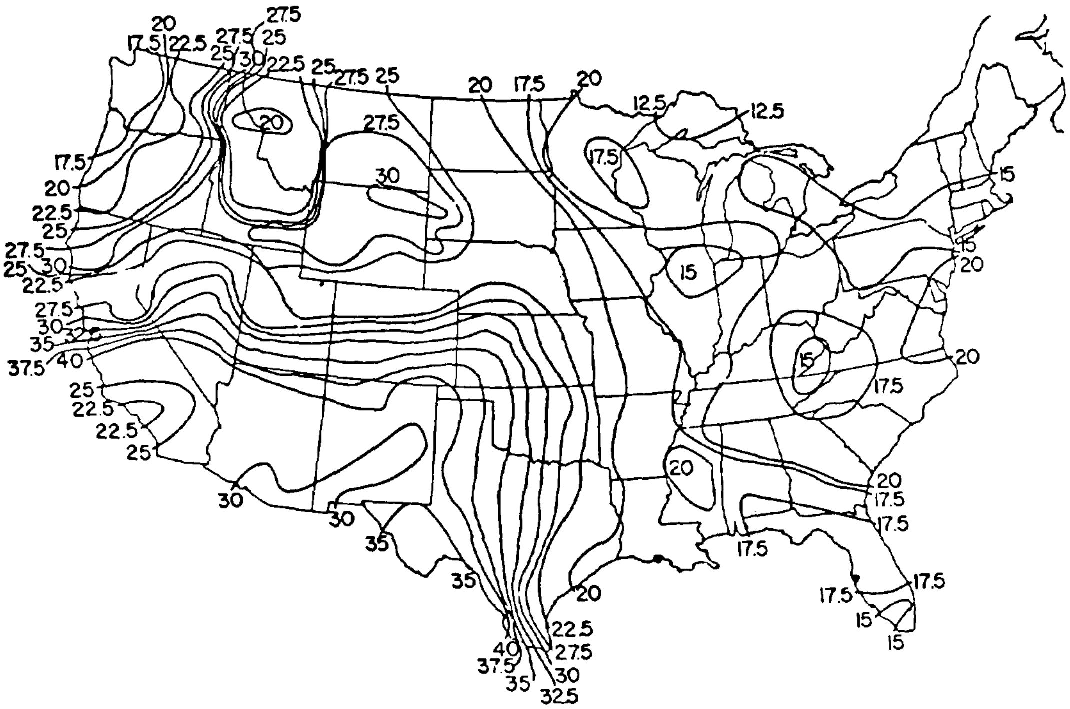


Figure H-7. Average pan evaporation, in centimetres, for the continental United States for the month of July based on data taken from 1931 to 1960

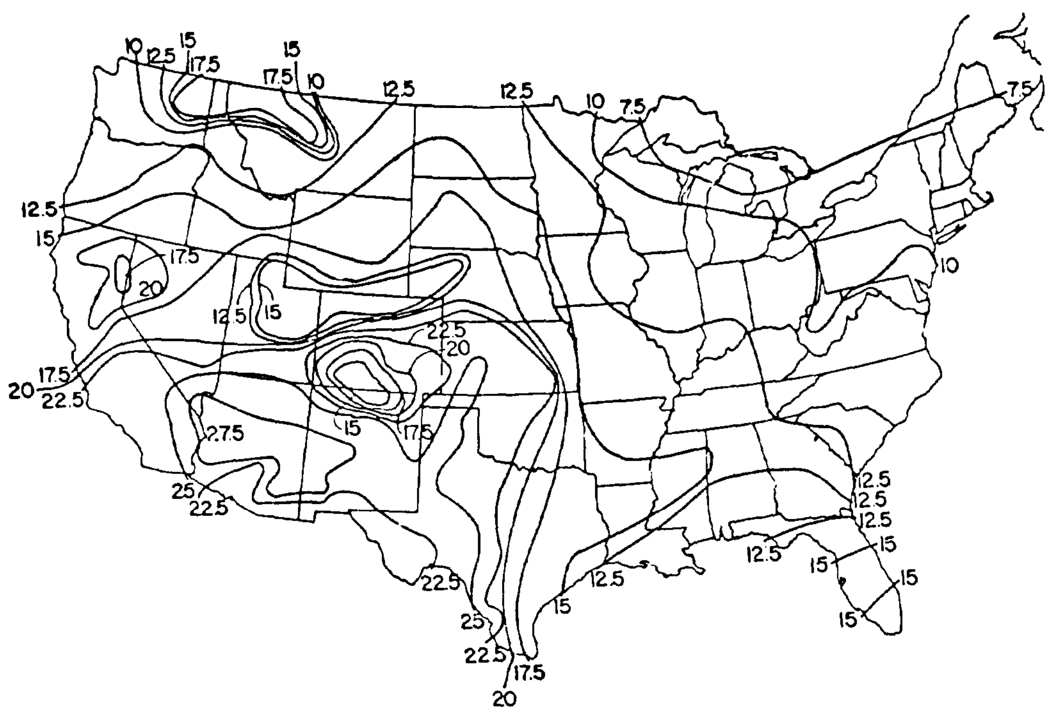


Figure H-8. Average pan evaporation, in centimetres, for the continental United States for the month of August based on data taken from 1931 to 1960

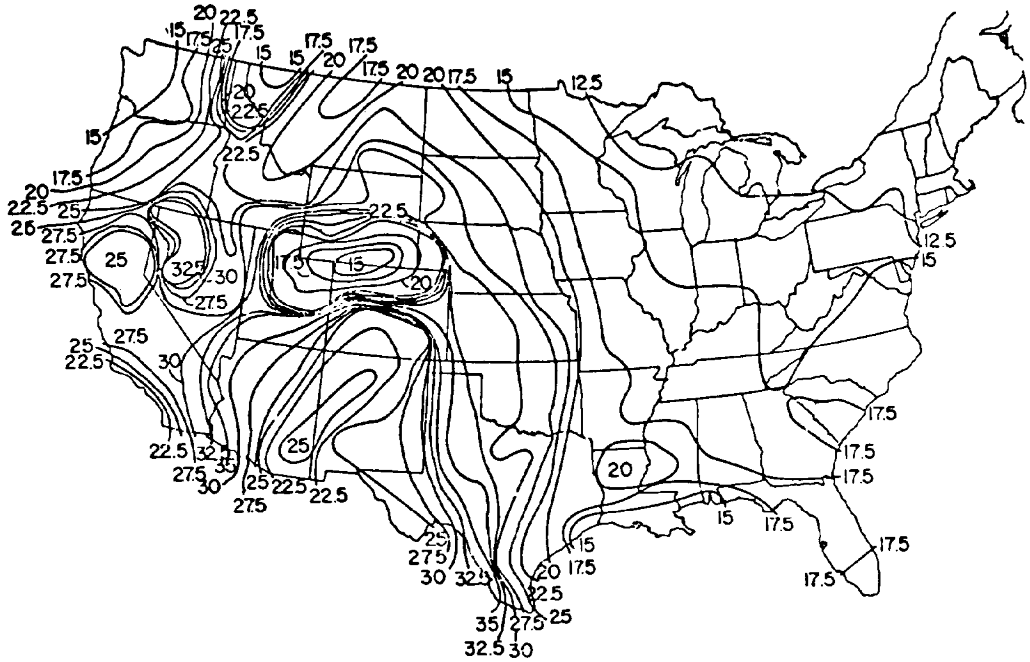


Figure H-9. Average pan evaporation, in centimetres, for the continental United States for the month of September based on data taken from 1931 to 1960

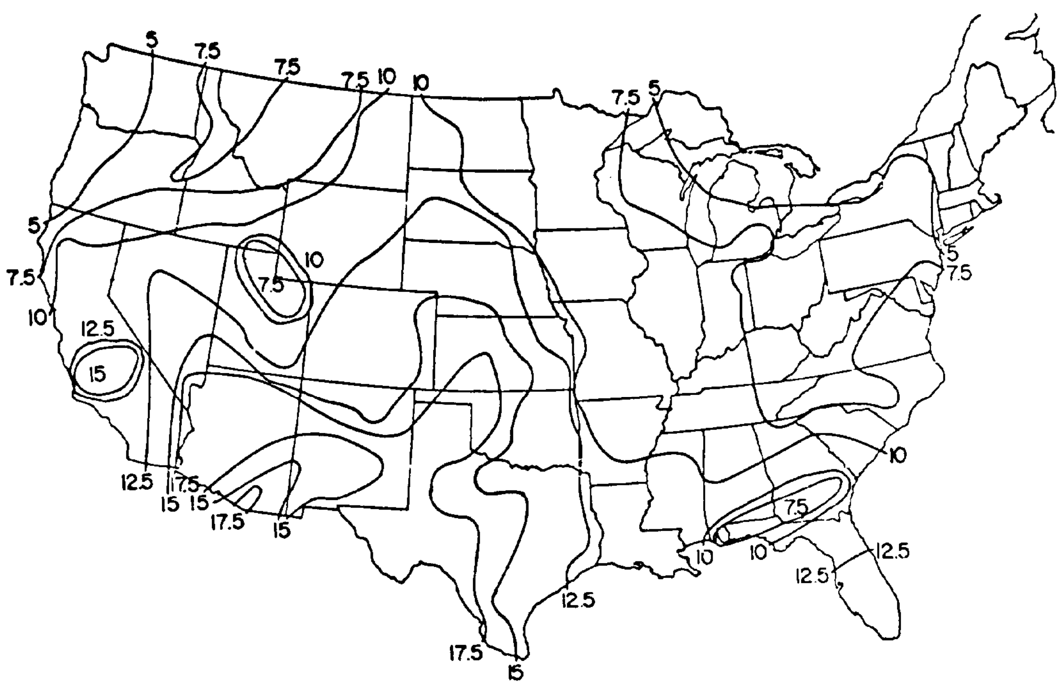


Figure H-10. Average pan evaporation, in centimetres, for the continental United States for the month of October based on data taken from 1931 to 1960

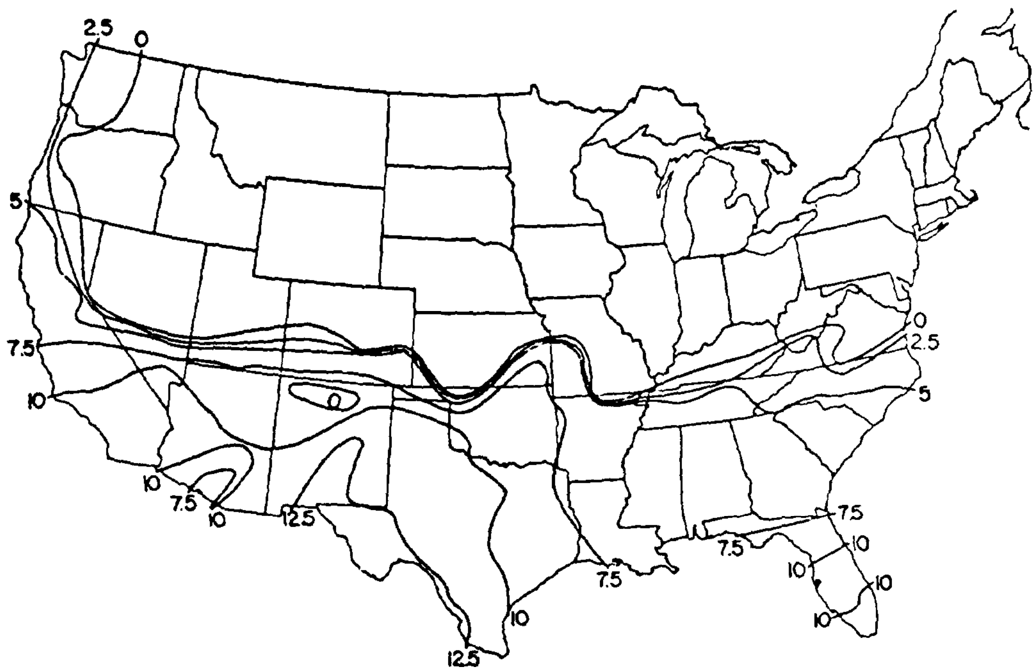


Figure H-11. Average pan evaporation, in centimetres, for the continental United States for the month of November based on data taken from 1931 to 1960

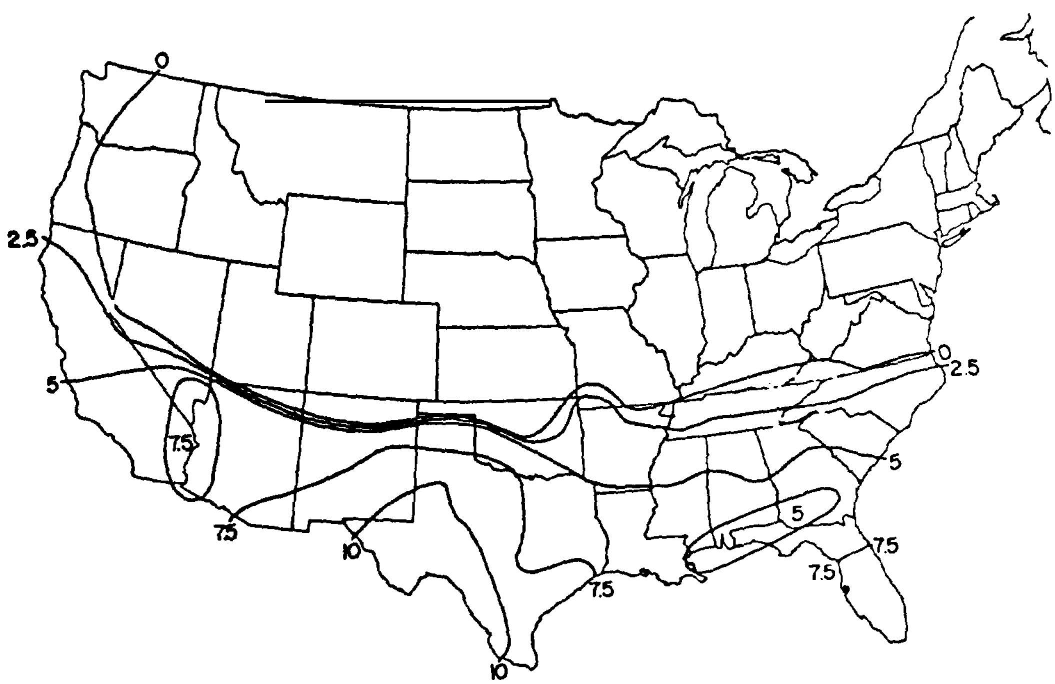


Figure H-12. Average pan evaporation, in centimetres, for the continental United States for the month of December based on data taken from 1931 to 1960

APPENDIX I

PROCEDURES FOR SELECTING EQUIPMENT FOR DEWATERING OPERATIONS

I-1. General Procedures. In order to predict whether or not draglines and other equipment can operate successfully on perimeter dikes, on interior berms composed of dewatered dredged material, or inside disposal sites, criteria have been developed relating vehicle ground pressure, with or without mats, and rating cone index (RCI) of the supporting soil, as shown in Figure T-1. The RCI can be obtained rapidly in the field by one or two technicians by hand-pushing a small cone penetrometer through the soil and determining the resistance to penetration. (Under some conditions, field penetration resistance data for remolded material must also be determined.) The critical layer RCI is the lower of the 0- to 6-inch or 6- to 12-inch layer resistance values encountered in the field; for, if the dragline (or other type of vehicle or equipment) breaks through these layers, soil strength usually decreases even further, and the vehicle will become immobilized. Caution should be exercised when selecting a vehicle whose ground pressure just equals that obtained from Figure I-1 for the available RCI, because of possible undetected soft spots in the area or possible vehicle operation errors that could cause immobilization. WES Technical Report D-77-7 (item 37) should be consulted for more exact procedures.

I-2. Effects of Trenching. Once the dragline has moved onto the interior berms to continue the periodic trench deepening operation, criteria are also available, as shown in Figure I-2, to predict the rate at which trenching operations may be conducted. In this figure, which shows linear trenching in feet per hour versus RCI, the RCI is for the soil supporting the dragline. The relationships in Figure I-2 are, at this stage, based on limited data. However, in the absence of better data, they may be used for approximate preliminary estimates of expected behavior.

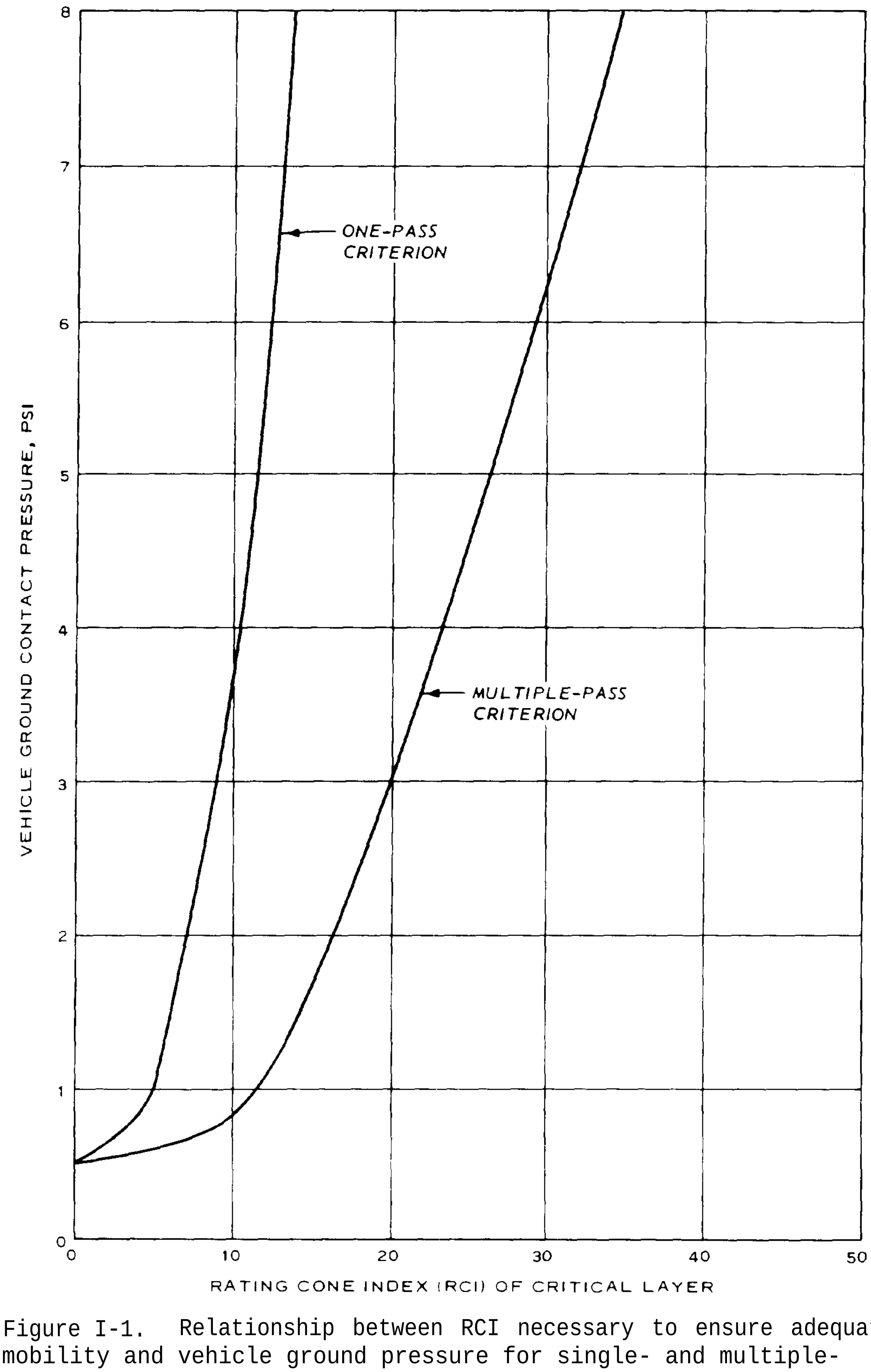


Figure I-1. Relationship between RCI necessary to ensure adequate mobility and vehicle ground pressure for single- and multiple-pass operations in confined dredged material disposal areas

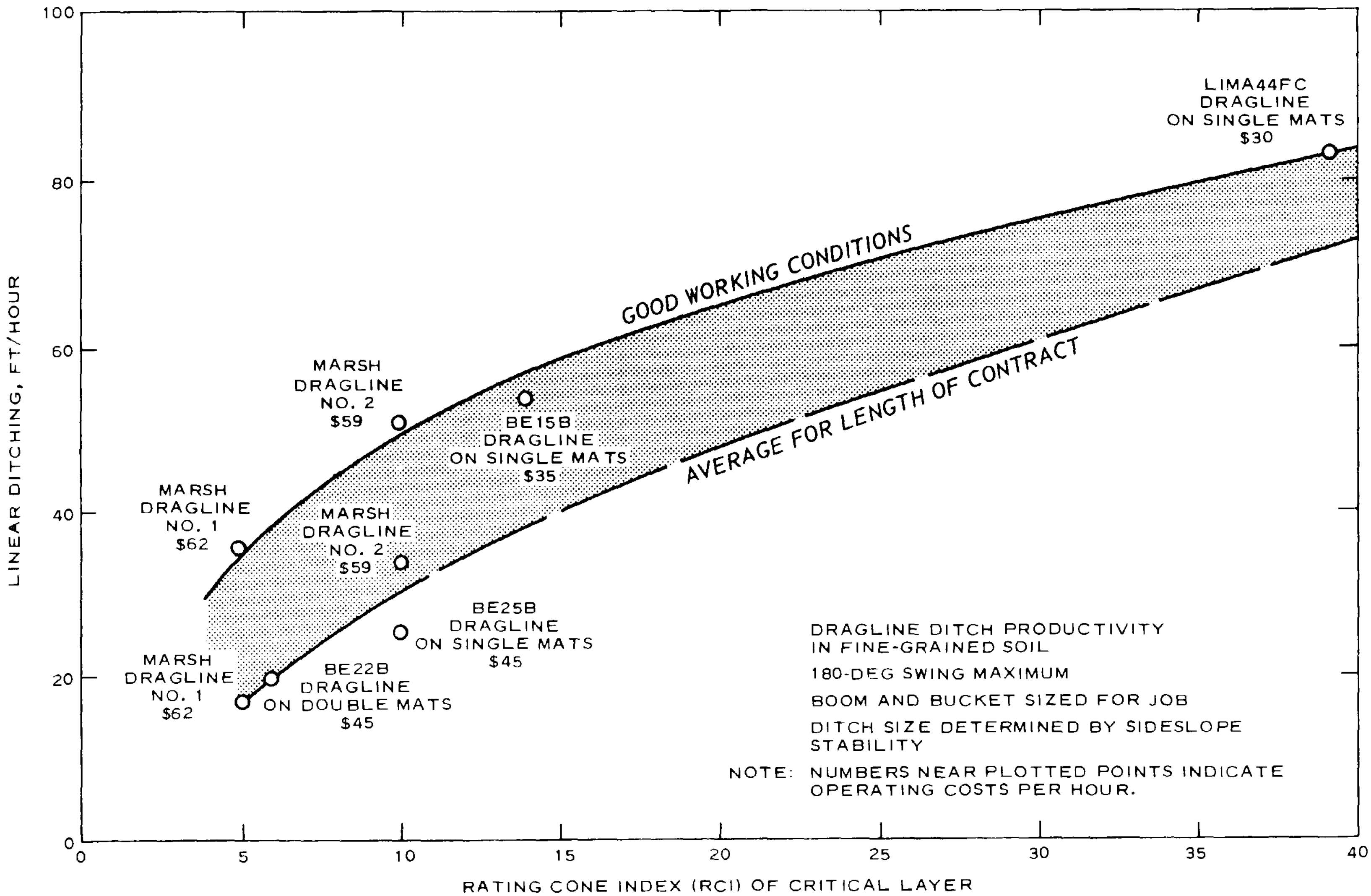


Figure I-2. Relationship between RCI of confined disposal area surface crust and linear trenching rate obtainable by dragline equipment

APPENDIX J

DYE TRACER TECHNIQUE TO ESTIMATE MEAN RESIDENCE TIME AND HYDRAULIC EFFICIENCY

J-1. Fluorescent Dyes.

a. General. Determination of retention time of ponded water is an important aspect of containment area design for retention of solids. Dye tracer studies may be undertaken to provide retention time data for better operation or management of existing dredged material containment areas. Various artificial tracers have been used to generate inflow and settling data characteristics. Radioactive tracers are effective; however, their use involves troublesome special handling and safety precautions. Commercially produced fluorescent dyes are easier and safer to handle and have been used extensively in inflow studies. Fluorescent materials used in tracing are unique in that they efficiently convert absorbed light into emitted light with a separate characteristic spectrum. Using the proper light source and filters, a fluorometer can measure small amounts of fluorescent material in a sample, Thus, when a fluorescent dye is mixed with a given parcel of water, that parcel may be identified and traced through a water system. The mean residence time and the amount of mixing of the water parcel in the system can be quantified by measuring the time variation of dye concentrations of the water leaving the system.

b. Physical-Chemical Considerations. For a given fluorescent dye, the interaction of the dye with surrounding environmental conditions should be considered. Use of a dye in nature's water normally is not affected by chemical changes. However, if the dye were to be used in waters having high chloride concentrations, the dye loss could be significant. Photochemical decay of dye concentration must also be considered when planning a dye tracer study. Factors influencing photochemical decay are light intensity, cloud cover, water turbidity, and water column depth. Other physical-chemical impacts on dyes are related pH, temperature, and salinity. Under acidic conditions, adsorption occurs more strongly, resulting in a reduction in fluorescence. A general rule of thumb on temperature impacts is that fluorescence decreases 5 percent for every 2° C increase in temperature. Tests have shown that dye decay occurs at a slower rate under saline conditions (7.02 metres sodium chloride solution) (item 30). Additional guidance for designing dye tracer studies and details of physical-chemical effects on dyes are found in items 1, 28, 11, 36, 30, 39, 38, and 8.

c. Dye Types. Fluorescent dyes have been used since the early 1900's. Several have been developed and used with varying degrees of success in the tracing of surface and ground waters. Smart and Laidlaw (item 30) evaluated eight dyes: Fluorescein, Rhodamine B, Rhodamine WT, Sulpho Rhodamine B, Lissamine FF, Pyramine, Amino G Acid, and Photine CU. Rhodamine B is stable in sunlight, but it is readily adsorbed to sediments in water. Rhodamine WT was developed specifically for water tracing and is recommended for such routine use.

J-2. Measurement Techniques.

a. Theory of Operation. Unlike sophisticated and complex analytical laboratory spectrofluorometers, filter fluorometers are relatively simple instruments. Basically, filter fluorometers are composed of six parts: a light (excitation energy) source, a primary or "excitation" filter, a sample compartment, a secondary or "emittance" filter, a photomultiplier, and a readout device.

(1) When a fluorescent material is placed in a fluorometer, that spectral portion of the light source that coincides with the peak of the known excitation spectrum of the test material is allowed to pass through the primary filter to the sample chamber. This energy is absorbed by the fluorescent material, causing electrons to be excited to higher energy levels. In returning to its ground state, the fluorescent material emits light that is always at a longer wavelength and lower frequency than the light that was absorbed. It is this property that is the basis of fluorometry, the existence of a unique pair of excitation and emission spectra for different fluorescent materials. Finally, only a certain band of the emitted light, different from that used for excitation, is passed through the secondary filter to the photomultiplier, where a readout device indicates the relative intensity of the light reaching it. Thus, with different light sources and filter combinations, the fluorometer can discriminate between different fluorescent materials,

(2) The selection of light sources and filters is crucial since they determine the sensitivity and selectivity of the analysis. Fluorometer manufacturers recommend and supply lamps and filters for most applications, including Rhodamine WT applications.

(3) Two types of fluorometers are in common field use today. The standard instrument used in water tracing by many groups, including the USGS (Item 38), has been the Turner Model III manufactured by G. K. Turner Associates. Turner Designs has capitalized on recent advances in electronics and optics and developed a fluorometer, the Model 10 series, that is better adapted to field use than the Turner Model III, but is also more expensive.

b. Field Use. Once a fluorometer is calibrated, it must be decided where and how field samples will be analyzed--in situ or in a laboratory, continuously or discretely. During in situ analysis, the operation of the fluorometer in flow-through mode (where water from a given discharge point in the containment area is pumped continuously through the sample chamber in the fluorometer) is advantageous over its operation in cuvette mode (where a discrete sample is analyzed). Specifically, in situ flow-through analysis allows the homogeneity of fluorescence in the discharge to be easily observed, and eliminates the need for handling individual samples. Also, during in situ flow-through analysis, a strip chart recorder can be attached to the fluorometer, simplifying data collection by providing a continuous record of the fluorescence measured. During laboratory analysis, however, the flow-through system is seldom used, since discrete samples are homogeneous and usually lack the volume needed to fill the system. Instead, the fluorometer is operated in cuvette mode, where only a small portion of a sample is required for analysis.

(1) Each method of analysis also has its inherent problems. Laboratory analysis requires that discrete samples be collected, bottled, labeled, stored in the field, and then transported to the laboratory; this introduces many opportunities for samples to be lost through mislabeling, misplacement, or breakage. Also, the frequency of sampling may be insufficient to clearly define the changes in dye concentration as a function of time.

(2) In situ analysis, on the other hand, is usually performed under adverse environmental conditions--often at a fast pace, in a cramped and unsteady work space, or in less than ideal weather conditions. Thus, it is more likely that an error will occur during in situ analysis than during analysis in the controlled environment of a laboratory. It is also usually necessary to compute and apply many more temperature correction factors to fluorescence values during in situ analysis than during a laboratory analysis, since the samples to be analyzed in situ have not had a chance to reach a common temperature. This also increases the chances for error during analysis. In addition, in situ analysis is usually final. That is, if questions are raised about the validity of a measurement after the analysis, no sample is available for verification. In situ analysis may not be used when significant turbidity interference occurs.

(3) To minimize the risk involved in relying on either method alone, a combination of the two may be employed-- a preliminary in situ analysis to help guide the sampling effort and a final laboratory analysis to ensure accurate results for quantitative analysis.

(4) Regardless of when and where fluorometric analysis takes place, several general precautionary measures should be taken to ensure that the analysis is reliable.

(a) The fluorometer should be accurately calibrated.

(b) Sample contamination should be avoided by rinsing or flushing the sample chamber between readings.

(c) The fluorometer operator should have experience with the instrument that is used. Experience can be gained through practice prior to the analysis.

(d) Sample temperatures should be observed and recorded during analysis to determine the necessary fluorescence correction factors.

(e) All information used to determine concentration units should be recorded (i.e., scale and meter or dial deflection).

(f) The calibration should be checked on a regular basis (every hour or so). This is especially important if the fluorometer is powered by a battery. When the battery is drained, readings are no longer accurate.

(5) For flow-through analysis in particular, all connections between the sampling hose, fluorometer, and pump must be tight to prevent air bubbles from entering the sample chamber. Air bubbles may also be introduced by a leaky pump seal. Thus, it is recommended that the pump be connected to the system

so that water is drawn up through the fluorometer to the pump. A screen placed at the intake end of the sampling hose will prevent sand and pebbles from altering the optics of the system, since they may scratch the glass in the sample chamber as they travel through the system.

(6) When analyzing samples in cuvette mode, the optics of the system may be distorted by scratches or smudges on the cuvette, making it necessary to wipe the cuvette clean prior to its insertion in the sample chamber. Once the cuvette is inside the warm sample chamber, a reading must be made quickly to prevent warming of the sample or condensation forming on the cuvette. Warming of the sample would cause a reduction in fluorescence, whereas condensation would distort the system optics.

(7) A person who has handled dye should never touch the fluorometer, or should use rubber gloves to handle dye and then discard them. Extremely small traces of dye on cuvettes or sample tubes can cause extremely large errors.

J-3. Sampling.

a. Sampling Equipment. The basic equipment needed to perform a dye tracer study includes the following:

(1) Fluorometers and accessories (filters, spare lamps, recorders, cuvettes, and sample holders). A spare fluorometer should be included if available, since the entire field study centers around its operation.

(2) Standard dye solutions for calibrating the fluorometers.

(3) Generators or 12-volt deep-cycle marine batteries (with charger) to power fluorometers and pumps, if the dye concentration is to be monitored continuously.

(4) Sampling equipment--pump and hoses, automatic sampler or discrete sampler (e.g., a Van Dorn sampler), bottles, labels, waterproof markers.

(5) Temperature-measuring device for measuring sample temperatures, if the temperature of the samples being analyzed will vary significantly.

(6) Dye, dilution vessels, and injection equipment (e.g., bucket, pump, and hoses).

(7) Description and dimensions of the containment area and surveying equipment to measure dimensions of the containment area.

(8) Equipment and records to determine the flow rate of the effluent from the containment area (e.g., production records, dredge discharge rate, weir length, depth of flow over the weir, and head above the weir).

(9) Miscellaneous equipment (e.g., life jackets, tool kits).

(10) Data forms.

Additional equipment might include cameras, radios, rope, and lights. All equipment should be checked for proper performance prior to transporting to the field.

b. Preparatory Tasks.

(1) Prior to conducting the dye tracer study, the average discharge rate at all points of discharge from the containment area should be measured or estimated. Equipment should be prepared, calibrated, and installed to measure or estimate the discharge rate during the dye tracer study. If production records are to be used to estimate the discharge, the discharge should be correlated to production. The average discharge rate, $\bar{Q}$, is equal to the sum of the average discharge rate at each discharge point, $\bar{q}$.

(2) A survey of the containment area should be performed to determine the area, depth, and volume of ponding, V_p, at the site for determination of the theoretical residence time, T. The volume can be estimated from as-built or design drawings of the site, but the depth of fill and ponding should be verified in the field if an accurate estimate of the hydraulic efficiency is to be determined from the dye tracer study. The ponded volume is needed to estimate dye requirements. An accurate determination of the volume is not needed to determine only the mean residence time.

(3) Using the average discharge rate and the ponded volume, the theoretical residence time of the site should be computed to plan the duration of the dye tracer study and to determine the hydraulic efficiency.

$$T = V_p/\bar{Q} \quad \text{(J-1)}$$

(4) The background fluorescence should be measured at the site. Background fluorescence is the sum of all contributions to fluorescence by materials other than the fluorescent dye. The best method to determine the background fluorescence is to measure the fluorescence of the discharge from the site several times prior to addition of dye at the inlet. If the background fluorescence is expected to be variable, the fluorescence of supernatant from the influent should be measured before and during the dye tracer study. The fluorescence of the water at the dredging site should not be used as the background fluorescence since some of the sediment that is mixed with the site water may remain suspended and exhibit fluorescence. Similarly, the sediment may release or adsorb fluorescent materials that would alter the fluorescence of the site water.

(5) The effect of turbidity on the measurement of fluorescence should be examined to determine whether the discharge samples should be filtered prior to measuring their fluorescence. Turbidity will reduce the fluorescence by absorbing and scattering the light from the fluorometer lamp. Filtering is necessary only when samples are highly turbid or when the turbidity varies significantly. The effect of turbidity can be tested very simply. A sample of the discharge is divided in half, and a small amount of dye is added to one of the portions. The fluorometer is blanked or zeroed on the portion without dye in it, and the fluorescence of the portion containing dye is measured. Next, both samples are filtered or centrifuged to remove the turbidity. The process is then repeated using the filtrates or supernatants--blanking the

fluorometer on the portion without dye in it and measuring the fluorescence of the portion containing dye. If the measured fluorescence of the sample without turbidity differed from the measured fluorescence of the sample with turbidity, then it is evident that turbidity affected the analysis. Alternatively, distilled water could be used as the blank when the turbidity or the background fluorescence is expected to vary significantly during the study.

c. Dye Dosage Requirements.

(1) Dye is usually released instantaneously as a slug in studies performed to measure the mean residence time or hydraulic efficiency of a basin. The dye marks a small parcel of water that disperses as the parcel passes through the basin. Ideally, the dispersion in a settling basin is kept very low, and the parcel moves as a slug through the basin by plug flow. In practice, the net flow-through velocity is very low, sufficiently low that the parcel would move by plug flow in the absence of external forces. However, containment areas are subject to wind forces that transform the basins into partially mixed basins where the velocities induced by wind are much greater than the net flow-through velocity. Consequently, the flow through the basin more closely represents completely mixed conditions than plug flow conditions. Therefore, the dye requirements are determined based on the assumption that the dye is completely mixed in the basin rather than longitudinally dispersed.

(2) A typical dye tracer curve for a dredged material containment area, shown in Figure J-1, shows a residence time distribution that is characteristic of a partially mixed basin. Dye appears quickly at the discharge point at time t_i, and then shortly thereafter the peak concentration is discharged at time t_p. After the peak concentration reaches the discharge point, the dye concentration quickly decreases to about 30 to 60 percent of the peak concentration, depending on the wind and the theoretical residence time of the basin. The dye concentration then gradually decreases until all of the dye is finally discharged of time t_f. The mean residence time and theoretical residence time are shown in the figure as $\bar{t}$ and T, respectively. The residence time distribution indicates that some of the water short-circuits to the discharge point before the dye is completely mixed throughout the containment area. However, the dye becomes well mixed soon after the peak concentration is discharged, and then the dye concentration decreases gradually (instead of rapidly as it did before being completely mixed) to zero.

(3) Before determining the dye dosage requirements for a study, a standard calibration curve should be developed for the dye and the fluorometers to be used. This consists of plotting the fluorometer response for at least five known concentrations of dye. The design dye concentration is based on the ability to measure the dye concentration accurately for the length of the study, while not exceeding the maximum fluorometer response or excessively coloring the water.

(4) The dye dosage requirements are based on achieving an initial concentration of 30 parts per billion in a completely mixed basin. This concentration of Rhodamine WT corresponds to 30 percent of the full scale deflection of many commonly used fluorometers. With this quantity of dye, the peak

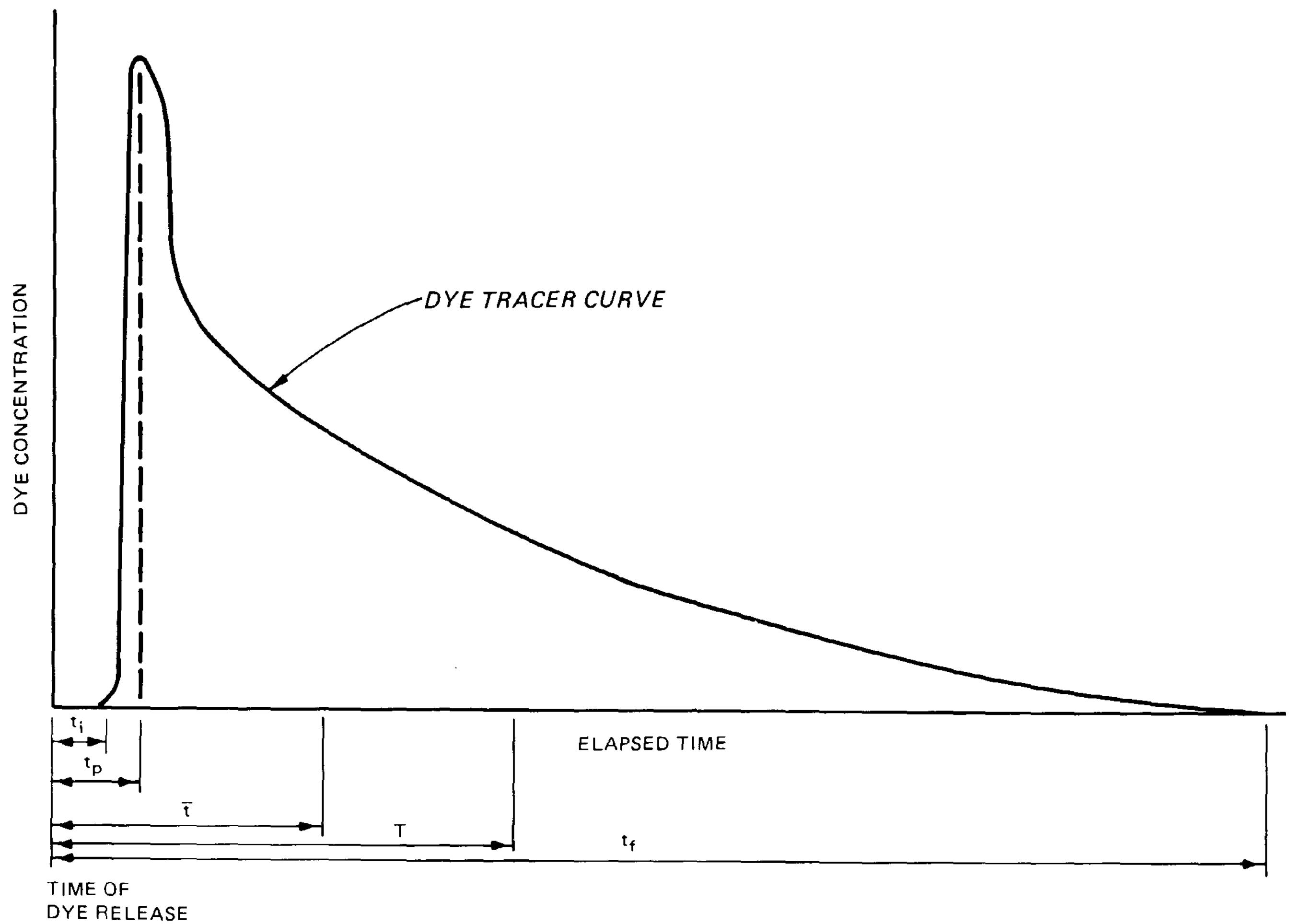


Figure J-1. A typical plot of the residence time distribution for dredged material containment areas concentration will generally be less than 100 parts per billion (or 100 percent of the maximum fluorometer response) except for very small containment areas (<15 acres) or for areas with very bad channeling and short-circuiting. Since the peak concentration may exceed the capacity of the fluorometer, discrete samples should be taken during the period when the peak concentration is being discharged. These samples may be diluted to measure the peak concentration.

(5) The dye dosage requirements are computed as follows:

$$\text{Dye Dosage, lb} = 0.00272\ (C_o\text{, ppb})\ (V_p\text{, acre-ft}) \qquad \text{(J-2a)}$$

$$= 6.24 \times 10^{-8}\ (C_o\text{, ppb})\ (V_p\text{, cu ft}) \qquad \text{(J-2b)}$$

$$= 2.21 \times 10^{-9}\ (C_o\text{, ppb})\ (V_P\text{, litres}) \qquad \text{(J-2c)}$$

where

C_o = desired dye concentration (generally 30 parts per billion for Rhodamine WT)

V_p = ponded volume

Dye Dosage = quantity in pounds of pure dye to be added to containment area

(6) Fluorescent dyes are not generally produced at 100 percent strength. Rhodamine WT is typically distributed at 20 percent dye by weight. Consequently, the quantity of manufacturer stock dye would be five times as large as computed in Equation J-2.

$$\text{Stock Dye Dosage} = \frac{\text{Dye Dosage}}{\text{Stock Concentration}} \qquad \text{(J-3)}$$

where the stock concentration is the fractional dye content by weight.

(7) The volume of stock dye required can be computed as follows:

$$\text{Volume of Stock Dye} = \frac{\text{Stock Dye Dosage}}{\text{Specific Weight}} \qquad \text{(J-4)}$$

The specific weight of liquid Rhodamine WT dye at a concentration of 20 percent by weight is about 1.19.

d. Dye Addition. The dye should be added to the influent stream in liquid form in a quantity and manner that is easy to manage. If the dye comes in solid form, it should be dissolved prior to adding it. Solid dye is easier to transport, but it is often inconvenient to dissolve at field locations. The dye may be diluted to a volume that will ensure good mixing with the influent stream, but the quantity should not be so large that it takes more than about 5 or 10 minutes to add the dye. The dye may be pumped into the influent pipe or poured into the influent jet or pool. Greater dilutions should be used to ensure good mixing if the dye is to be poured into the influent. Care must be taken that the dye is distributed so that it flows into the containment area in the same manner that the influent does.

e. Sampling Procedures.

(1) Sampling should be conducted at all points of discharge from the containment area.

(2) The dye concentration may be measured continuously at the discharge, or discrete samples may be collected throughout the test. Discrete samples must be taken when turbidity interference occurs, since the samples must be filtered or centrifuged. Discrete samples should be taken when the dye is being measured continuously to provide a backup in the case of equipment malfunction and to verify the results of the continuous monitor.

(3) The sampling frequency should be scheduled to observe any significant change in dye concentration (about 5 to 10 percent of the peak dye concentration). Sampling should be more frequent near the start of the test, when dye starts to exit from the containment area, and when the peak dye concentration passes the discharge points. About 40 carefully spaced samples should clearly define the residence time distribution or dye tracer curve.

(4) The sampling duration should be sufficiently long to permit the dye concentration to decrease to 10 percent of the peak concentration or less. For planning purposes, the duration should be at least about 2.5 times the theoretical residence time.

(5) The flow rate at all points of discharge from the containment area should be measured. If the flow rate varies significantly (more than 20 percent of average), the flow rate should be measured periodically throughout the test. Production records may be used to provide an indication of the variability of the flow rate. The flow rate over weirs may be estimated by measuring the depth of flow over the weir and the length of the weir crest and applying the weir formula for sharp-crested weirs:

$$Q = 3.3\ LH^{3/2} \tag{J-5}$$

or

$$Q = 2.6\ Lh^{3/2} \tag{J-6}$$

where:

Q = flow rate, cubic feet per second
L = weir crest length, feet
H = static head above weir crest, feet
h = depth of flow above weir crest, feet

J-4. Data Analysis.

a. Data Reduction. The data should be tabulated in the following form:

Sample	Time from Dye Addition	Flow Rate	Dye Concentration Above Background	Time Interval
i	t_i	Q_i	C_i	Δt_i

Column 1 is the number of the sample, i . If the dye concentration was monitored continuously, discrete points on the dye concentration curve may be used as samples. Column 2 is the time, t_i , that elapsed between the time that the dye was added to the influent and the sample was taken from the effluent.

Column 3 is the flow rate, Q_i, at the time that the sample was taken. The flow rate is needed only when the flow rate is not constant during the test. Column 4 is the dye concentration of the sample discounted for the background fluorescence, C_i; that is:

$$C_i = C_{si} - C_{bi} \qquad (J\text{-}7)$$

where

C_i = dye concentration discounted for background fluorescence of sample i

C_{si} = measured fluorescence of sample i

C_{bi} = background fluorescence at time t_i

If the background fluorescence does not vary, C_{bi} would be a constant and may be eliminated from the expression for calculating C_i if the fluorometer is blanked or zeroed with the site water. Column 5 is the interval of time, Δt_i, over which the sample is representative of the results. The value of this interval is one-half of the interval between the times when the samples immediately preceding and following the sample of interest were taken.

$$\Delta t_i = \frac{t_{i+1} - t_{i-1}}{2} \qquad (J\text{-}8)$$

where

Δt_i = time interval over which sample i is representative

t_{i+1} = time when the following sample was taken

t_{i-1} = time when the preceding sample was taken

A data table is produced for each point of discharge.

b. Determination of Mean Residence Time.

(1) After generating the data tables, the mean residence time is computed as follows:

$$\bar{t} = \frac{\sum_{i=0}^{n} t_i\, C_i\, Q_i\, \Delta t_i}{\sum_{i=0}^{n} C_i\, Q_i\, \Delta t_i} \qquad (J\text{-}9)$$

where:

$\bar{t}$ = mean residence time
n = total number of samples

(2) If the flow rate is nearly constant throughout the test, the equation may be simplified to:

$$\bar{t} = \frac{\sum_{i=0}^{n} t_i \, C_i \, \Delta t_i}{\sum_{i=0}^{n} C_i \, \Delta t_i} \qquad \text{(J-10)}$$

(3) If the sampling interval is constant (i.e., Δt_i = constant) but the flow rate is not constant, the equation may be simplified to:

$$\bar{t} = \frac{\sum_{i=0}^{n} t_i \, C_i \, q_i}{\sum_{i=0}^{n} C_i \, q_i} \qquad \text{(J-11)}$$

(4) If both the sampling interval and the flow rate are constant, the equation may be simplified to:

$$\bar{t} = \frac{\sum_{i=0}^{n} t_i \, C_i}{\sum_{i=0}^{n} C_i} \qquad \text{(J-12)}$$

c. Determination of Hydraulic Efficiency.

(1) The hydraulic efficiency is the ratio of the mean residence time to the theoretical residence time where:

$$\text{Hydraulic Efficiency} = \frac{\bar{t}}{T} \qquad \text{(J-13)}$$

(2) The correction factor for containment area volume requirements is equal to the reciprocal of the hydraulic efficiency. This correction is applied by multiplying the volume by the correction factor.

$$\text{Hydraulic Efficiency Correction Factor for Volume Requirements} = \frac{1}{\text{Hydraulic Efficiency}} \tag{J-14}$$