DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
441 G St. NW
Washington, DC 20314-1000

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Manual
No. EM 1110-2-2107

1 August 2022

Engineering and Design
DESIGN OF HYDRAULIC STEEL STRUCTURES

1. Purpose. This manual prescribes guidance for the design of Hydraulic Steel Structures (HSS) by load and resistance factor design (LRFD). This includes design of new structures, replacement, rehabilitation, and repair. Mechanical and electrical design considerations are addressed in EM 1110-2-2610. Commentary is provided in Appendix B. Due to the U.S. Army Corps of Engineers' (USACE) unique structures and their associated risks, criteria in this manual may exceed industry requirements. The criteria in this manual are minimum USACE requirements. These criteria may be exceeded by the designer to enhance resiliency as risk and economics dictate.

1. Applicability. This manual applies to Headquarters, U.S. Army Corps of Engineers (HQUSACE) elements, major subordinate commands, districts, laboratories, and field operating activities with responsibility for design of civil works projects.

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

FOR THE COMMANDER:

6 Appendixes
(See Table of Contents)

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Chapter 1
Introduction

1.1 Purpose. This manual prescribes guidance for the design of Hydraulic Steel Structures (HSS) by load and resistance factor design (LRFD). This includes design of new structures, replacement, rehabilitation, and repair. Mechanical and electrical design considerations are addressed in EM 1110-2-2610. Commentary is provided in Appendix B. Due to the U.S. Army Corps of Engineers' (USACE) unique structures and their associated risks, criteria in this manual may exceed industry requirements. The criteria in this manual are minimum USACE requirements. These criteria may be exceeded by the designer to enhance resiliency as risk and economics dictate.

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1.4 References. References are listed in Appendix A.

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

1.6 Delegated USACE Design Approval for Dam and Levee Projects.

1.6.1 Risk Assessment. Guidance in this manual supersedes all other previously published USACE guidance for the design of HSS. Justifications for deviations and waivers from mandatory design standards in this manual must include a risk assessment. All deviations and waivers must be clearly identified in the decision documents and design reports and must be deliberately called out within the review plan as a specific charge for the review.

1.6.2 Approval and Documentation. All proposed deviations and waivers from these mandatory design standards, including rationale, must be documented in a memorandum approved by the respective District and Division Dam Safety Officer or Levee Safety Officer and concurred by the Dam Safety Oversight Group (DSOG) or Levee Safety Oversight Group (LSOG), whichever is appropriate. The DSOG or LSOG will ensure the appropriate USACE Community of Practice leader(s) or their designated representatives are included in the concurrence process. Review documentation will account for all decisions and rationale for deviations and waivers, including the memorandum documenting approval and concurrence.

1.7 Background. Types of HSS. ER 1110-2-8157 provides a definition of HSS. Typical HSS are lock gates, Tainter gates, Tainter and other types of valves, bulkheads and stoplogs, vertical lift gates, components of hydroelectric and pumping plants, closure gates for levee systems, wicket gates, penstocks, needle dams, crest gates, roller gates, outlet works gates, and lifting beams. Standard manufacturer's design of small size, low head, and low risk are exempted from these requirements. HSS may be subject to submergence, wave action, hydraulic hammer, cavitation, impact, corrosion, vibration due to defective seals, friction from seals and bearings, and severe climatic conditions.

1.8 Design Policy.

1.8.1 Hydraulic Steel Structures Design. Engineering responsibilities for the design of HSS are prescribed by ER 1110-2-8157. HSS must be designed according to this manual and American Institute of Steel Construction (AISC) Specification for Structural Steel Buildings, ANSI/AISC 360-16, except as specified herein. Welding requirements must follow American Association of State Highway and Transportation Officials (AASHTO)/American Welding Society (AWS) D1.5M/D1.5:2015 or AWS D1.1/D1.1M:2015 (AWS) except as specified herein.

1.8.2 Discussion. This manual provides the minimum requirements necessary to provide for safety, reliability, and required performance of HSS and was written to replace the now expired Engineer Technical Letter (ETL) 1110-2-584. The Engineer, meeting the qualifications as defined by ER 1110-2-8157, must perform or oversee the performance of the design, fabrication, and installation of HSS. Coordination with Operations Divisions, project personnel, or end users is required during all design phases.

1.9 Requirement Criterion. The terms "will" or "must" denotes a mandatory requirement for compliance with this manual. The term "should" indicates a strong preference for a given criterion. The term "may" indicates a criterion that is usable. Other suitably documented, verified, and approved criterion may also be used so long as it is in a manner consistent with this design manual.

1.10 Reuse of Existing Designs. All newly fabricated HSS must be designed according to this manual. Existing designs used for new work must be modified to meet all provisions of this manual.

1.11 Design Guidance. For non-carbon HSS, use the loads, load factors, and load combinations in this manual. Use AISC Design Guide 27 for design of stainless steel structures. For design of aluminum structures, the current Aluminum Association Aluminum Design Manual is used. For aluminum use LRFD of building structures in the referenced manual.

Chapter 2
Types of Hydraulic Steel Structures

2.1 General. USACE designs, constructs, and operates many types of projects that control or regulate water. The primary project purposes are flood storage (reservoirs), navigation, and flood risk management. USACE projects sometimes include hydropower and water supply as secondary purposes. Each project uses HSS (gates) to control the flow of water in various ways. This chapter provides a brief overview of the types and uses of various HSS. See the detailed design requirements for each HSS type in the HSS type specific chapter.

2.2 Project Types.

2.2.1 Navigation. Navigation projects permit navigation between different water levels upstream and downstream of the dam. The navigation lift height varies considerably in different geographic areas. In coastal areas, lifts are often 10 ft (3 m) or less. Along the Mississippi and Ohio Rivers, lifts of 15 to 25 ft (5.6 to 7.6 m) are common. In more hilly or mountainous areas, especially in the Pacific Northwest, lift heights can reach 100 ft (30 m). Lock gates open and close to pass navigation traffic.

2.2.2 Flood Storage. Flood storage dams provide volume in the reservoir to store incoming flows to prevent downstream flooding. They use HSS to retain water during high inflows, release water after the high inflows to recover flood storage capacity, and regulate low flows for downstream water supply or water quality. Reservoir levels may be kept low to provide flood storage capacity while others will remain at near-full hydrostatic head. Levels are specific to the function and operation of the project.

2.2.3 Hydropower. Hydropower may be included as a secondary function of flood storage or for navigation dams. HSS on projects with hydropower are similar to those on other dams. Typically, there are HSS to control flows through the turbines and bulkheads to allow turbine maintenance. Reservoir levels on hydropower dams are generally kept high to maximize power generation.

2.2.4 Levee Systems. Levee system projects include levees or floodwalls surrounding a populated area. Openings are built through the structures to permit vehicle, railroad, or pedestrian access. HSS are provided to close the openings during flood conditions. They consist of sliding, hinged, and rolling configurations suitable for the different heights and widths of the openings. These HSS are seldom operated but must perform reliably during flood events. Alternately, openings are provided for waterway transportation, drainage, and tidal flow. The gates used for these applications are similar to gates used for other purposes.

2.2.5 Water Supply and Recreation. Water supply and recreation are generally not the primary function of USACE projects; however, they may be included as part of a mixed-use reservoir to include flood storage and hydropower. Pool levels, which fluctuate based on need, must be balanced with these other uses.

2.3 HSS Types.

2.3.1 Miter Gate. Miter gates are commonly used as lock gates and are also used in levee closures. A miter gate consists of two individual leaves. When closed and retaining pool, the miter leaves form a shallow three-hinged arch, with the arch pointing in the upstream direction. When open, the leaves rotate into recesses in the lock walls. The arch action is an efficient way to span the opening and a relatively small force is required for operation. See Chapter 9 for miter gate design requirements. Figure 2.1 shows a typical lock miter gate.

2.3.2 Tainter Gates. Tainter gates are primarily used as spillway crest gates and to a lesser extent, as lock gates. Pool is typically applied to upstream side of the skin plate toward the trunnion pin. The shape of a Tainter gate is shaped as a sector of a cylinder with a horizontal axis. The advantage of Tainter gates is that they can be opened and closed under large differential heads. The curved shape of the gate provides favorable hydraulic discharge characteristics and hydraulic operating load. See Chapter 10 for Tainter gate spillway and Chapter 11 for lock gate design requirements. Figure 2.4 shows a typical Tainter gate.

2.3.3 Tainter Valves. Tainter valves are used for lock emptying and filling. Tainter valves behave similarly to Tainter gates but water is typically placed rib side of the skin plate. See Chapter 12 for Tainter valve design requirements.

2.3.4 Lift Gates. Lift gate types are submersible or overhead and can be single or multiple leaf. Submersible gates are lowered to pass navigation or debris over the top of the gate. Overhead gates are raised to pass navigation or debris below the bottom of the gate. Multiple leaf gates can be configured to lower the upper section to pass navigation or debris or to pass flows between sections. Figure 2.2 shows a lock submersible lift gate. See Chapter 13 for lift gate design requirements.

2.3.5 Closure Gates for Levee Systems. A wide variety of gate types is used to close openings in levees and floodwalls. This is largely due to the various widths, heights, and slopes of the openings (roadways are often not flat). Other considerations may include how fast the gate must be operated (small streams can flood very quickly compared to major rivers) and what type of equipment is required to close the gate. The most common closure gates include swing gates, miter gates, rolling gates, trolley gates, and stoplogs. See Chapter 14 for design requirements.

2.3.5.1. Swing Gates. Swing gates are mounted on hinges and swung open and closed like a door. For wider openings, there can be dual swing gates that close against a removable post located at the center of the opening.

2.3.5.2. Miter Gates. Miter gates for flood closures are similar to miter gates for locks but usually much shallower. They provide an efficient arch action to span wider closures.

2.3.5.3. Rolling Gates. Rolling gates have wheels so they can be rolled open and closed. This requires a track for the wheels across the opening.

2.3.5.4. Trolley Gates. Trolley gates are similar to rolling gates, but the gate is suspended from an overhead track. This eliminates tracks across the roadway but requires an overhead support structure.

2.3.5.5. Stoplogs. Stoplogs provide the simplest closure gate configuration. Beams are stacked into slots on each side of the opening. For wider openings, removable posts can be used, however, the posts must be adequately supported. Chapter 14 describes various closure gates in more detail.

2.3.6 Bulkheads and Stoplogs. Bulkheads and stoplogs are HSS used to permit dewatering of sections of a project to permit maintenance, repairs, or emergency closures. A bulkhead consists of a single fabricated structure, whereas stoplogs consist of several shallower structures stacked to form a bulkhead system. See Chapter 15 for design requirements.

2.3.6.1. Bulkhead. Bulkheads are used where a single piece can be operated by the available lifting equipment, as such, they are typically used for narrower and shallower openings. Construction is similar to that of a lift gate. A simply framed bulkhead is shown in Figure 2.5.

2.3.6.2. Stoplog. A stoplog can consist of a single member, such as a tube or I-beam, or can be fabricated from plates or rolled sections into built-up beams or trusses. A typical truss type stoplog is shown in Figure 2.6.

2.3.7 Sector Gates. Sector gates are used for very low lift navigation gates, such as those encountered in coastal areas. These gates consist of two leaves joined at the center of the lock that rotate into recesses in the lock wall when opened. Each leaf is shaped as a sector of a cylinder with a vertical axis. The advantage of sector gates is that they can be opened and closed under small differential heads or with flow through the lock. This can eliminate the need for a separate filling and emptying system for the lock, providing a major cost savings. Figure 2.3 shows a typical sector gate.

2.3.8 Other HSS Types. The types of HSS described above are the most common types used in USACE projects, and they are further discussed in Chapters 9–16. However, this list is not comprehensive. There are many other types of HSS.

2.3.8.1. Prefabricated Gates. Prefabricated gates include slide, sluice, and flap gates. They can be purchased directly from gate suppliers and come in a variety of sizes that are suitable for a wide range of heads. Prefabricated flap gates are used to permit flows in one direction only, such as to provide interior drainage to an area inside a levee.

2.3.8.2. Dam Crest Gates. Dam crest gates come in many shapes. Only Tainter and lift gates are discussed in detail in this manual.

2.3.8.3. Roller Gates. Roller gates are used on some older, low head navigation dams. These are shaped like a hollow tube and are operated by rolling up a toothed track that is mounted on the dam pier.

2.3.8.4. Wicket Gates. Wicket gates are used on some dams where the gates are closed for only parts of a season. Wickets fold down on the top of the dam when not in use and are propped up into position when in use. Wicket gates are also used to control flows to hydropower turbines.

2.3.8.5. Other Types of HSS. Bear trap, clamshell, inflatable or hinged crest gates, other valves including Butterfly, Jet Flow, Howell-Bunger, Stony Gate, and Flap Gates, and other types of HSS are also used on some locks, dams, hydropower, and water control projects.

2.4 Design Requirements. Chapters 3 through 8 of this manual contain basic HSS design requirements that apply to all types of HSS. Chapters 9–14 provide geometry, details, and other information about specific types of HSS. Some of the information in those chapters may also be useful in the design of other types of HSS. The best source for geometry and detailing information on less common HSS types is usually the design and construction records from previous projects. However, such information should be modified to comply with current design requirements.

Figure 2.1. Miter Gate

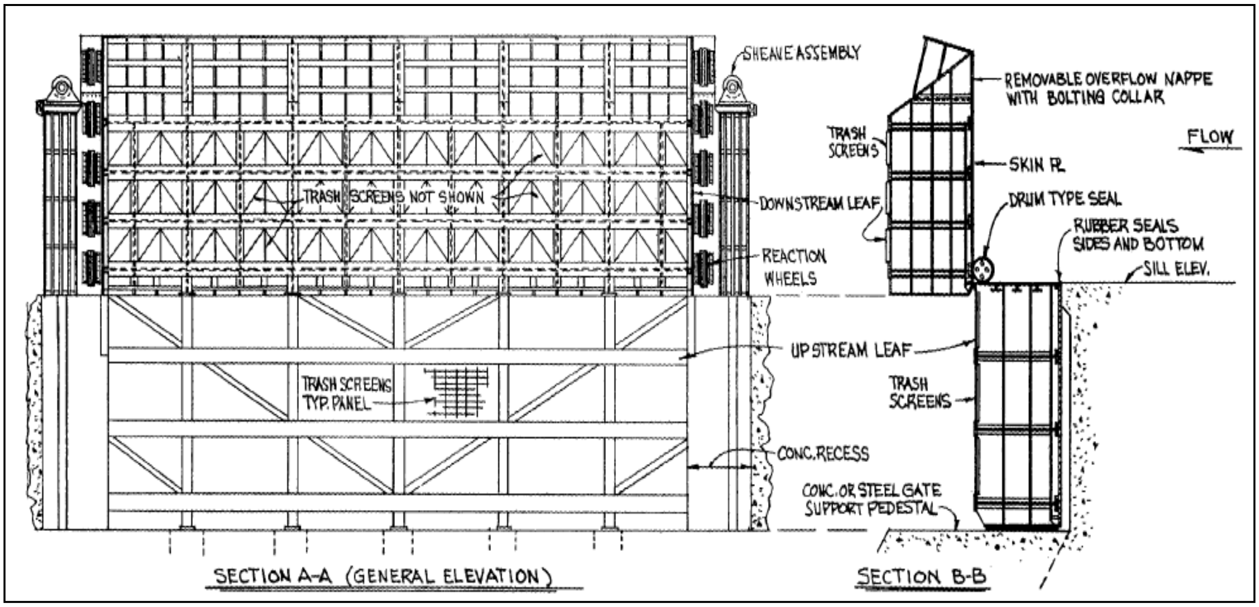


Figure 2.2. Submersible Lift Gate

Figure 2.3. Sector Gate

Figure 2.4. Tainter Gate


Figure 2.5. One-Piece Bulkhead

Figure 2.6. Bulkhead Formed from Stacked Stoplogs

Chapter 3
Design Considerations

3.1 Purpose. This chapter identifies the general considerations for the design of HSS. Specific requirements are described in other chapters.

3.2 Design Philosophy. HSS will be designed for the specified limit states to achieve the objectives of constructability, safety, and serviceability—with consideration for inspectability and economy—throughout the service life of the structure. The guidance in this manual is intended to provide ductile structures and to prevent brittle behavior. Redundant structures should be used when practical and economical.

3.2.1 Failure Modes. HSS design must consider all possible modes of failure. A probable failure mode analysis (PFMA) should be performed on HSS where life safety or significant economic loss would occur in the event of a failure. The PFMA should consider failure modes associated with the HSS, its mechanical and electrical systems, and its interface with other structural systems. See ER 1110-2-1156 for more information on performing a PFMA. Possible failure modes include, but are not limited to:

3.2.1.1. General yielding or excessive plastic deformation;

3.2.1.2. Local and global instability;

3.2.1.3. Fatigue damage;

3.2.1.4. Fracture;

3.2.1.5. Excessive elastic deformation; and

3.2.1.6. Damage from excessive vibration.

3.2.2 Limit States. HSS must be designed to satisfy all applicable limit states. A limit state is a controlling condition in which a structural system or component becomes unfit for its intended purpose. Limit States applicable to HSS are listed and described below.

3.2.2.1. Strength. The Strength Limit State ensures safety against yield, net-section fracture, and stability failure (local or global) during the intended life of the structure.

3.2.2.2. Serviceability. The Serviceability Limit State ensures that the HSS will meet all operational requirements by imposing limits on stress, deformation, and cracking.

3.2.2.3. Fatigue. The Fatigue Limit State avoids crack initiation due to repeated stress cycles so that serviceability of the HSS is maintained and fracture is prevented.

3.2.2.4. Fracture. The Fracture Limit State ensures that fracture will not occur under given design conditions.

3.2.3 Critical and Normal Structures.

3.2.3.1. HSS are designated as critical or normal. Critical structures are those where failure could result in the potential for one or more loss of life. Loss of life could result directly from breach or indirectly from flooding damage to a lifeline facility. An existing risk assessment for a dam or levee can help inform the potential for loss of life determination. In absence of a risk assessment, hazard potential can be used.

3.2.3.2. Guidance on the determination of hazard potential is provided in ER 1110-2-1806 and ER 1110-2-1156. Examples of classification of structures according to probable loss of life are provided in Appendix H of EM 1110-2-2100. In some cases, extreme economic or environmental loss may be justification for the designation of a structure as critical. HSS not meeting the definition of critical are normal structures.

3.2.4 Service Life. The requirements of this manual are based on an HSS service life of 100 years. HSS service life is the length of time a project will remain in use to provide its intended function.

3.2.5 Reliability.

3.2.5.1. Reliability Targets. Strength requirements in this manual are intended to achieve an acceptable level of reliability over the service life. The reliability targets for HSS designed according to this EM are shown in Table 3.1. These reliability targets account for consequences of failure and for the environment and operating conditions of the HSS over the service life.

3.2.5.2. Application. The guidance for selection of loads and load factors in Chapter 4 of this manual, combined with capacity and resistance factors from AISC, is intended to result in structures that provide the target reliability. For simplicity of application, the load factors provided in this manual were developed for single load path structures. Therefore, they also can be safely applied to redundant structures.

Table 3.1
Target Reliability for 100-Year Service Life, β

	Normal	Critical
Redundant Load Path	3.0	3.5
Single load Path	3.5	4.0

3.2.6 Constructability and Quality Assurance. Consider the effects of construction, including fabrication and erection, on the design. Constructability issues include deflection, strength, and stability of HSS or their components during stages of construction. Design HSS so that fabrication and erection can be performed without undue difficulty or distress and that construction force effects, including residual effects, are within tolerable limits. When the Engineer has assumed a sequence of construction to prevent certain stresses, it will be defined in the contract documents. See Chapter 8 for guidance on constructability and quality assurance.

3.3 Loads.

3.3.1 General. All loads to which an HSS is subjected will be considered in the design according to Chapter 4. Loads with a negligible impact on the design may be ignored.

3.3.2 Cyclic and Static Loading. Cyclically loaded structures are those that repeatedly undergo significant changes in stresses during operation. Statically (non-cyclically) loaded structures are defined as those that do not repeatedly undergo significant changes in stresses in the process of operating. Statically loaded structures may experience changes in applied stresses from loading and unloading during the normal use of the structure, but those changes only occur for a relatively small number of operating cycles throughout the service life of the structure.

3.3.3 Load Combinations. Loads are combined to produce maximum effects for a given limit state under the varying load frequencies. Several load combinations are defined, and varying load factors are applied to achieve a consistent level of reliability. Loads are combined using principal and companion action loads as described in Chapter C2 of American Society of Civil Engineering (ASCE) 7-22 and the following paragraphs.

3.3.3.1. Principal and Companion Action Loads. A load used in combination with other loads can be defined as a principal load or as a companion load. The maximum combined load occurs when one load—the principal action—is at its extreme value, while the other loads—the companion actions—are at values that are expected, while the principal action is at its extreme value. Definitions are in the following paragraphs.

3.3.3.2. Principal Load. A principal load is a specified variable load or rare load that dominates in a given load combination. Principal loads are selected as described in paragraph 4.3.3. For LRFD, a principal load factor is applied to the principal load in a load combination to account for the variability of the load and the load pattern for the analysis of its effects.

3.3.3.3. Companion Load. A companion load is a specified variable load that accompanies the principal load in a given load combination. For LRFD, a companion load factor is applied to a companion load in a load combination to account for uncertainty in the magnitude of the companion load acting simultaneously with the factored principal load. Companion loads are typically usual loads. For strength load combinations, unless otherwise specified in paragraph 4.3.5, temporary and dynamic companion loads must have a minimum 10-year return period. This is the maximum return period of the usual load category defined in paragraph 3.3.4.1. For hydrostatic loadings, this is the normal operating condition (pool).

3.3.4 Probability of Loading. Loads can be separated into categories based on their probability of occurrence. Loads with less probability of occurrence can have different design requirements to achieve the same reliability. Loads are categorized as usual, unusual, and extreme based on average annual return periods (Tr) or annual exceedance probability (AEP). The probability of loading associated with the usual, unusual, and extreme load categories are described below and are illustrated in Figure 3.1.

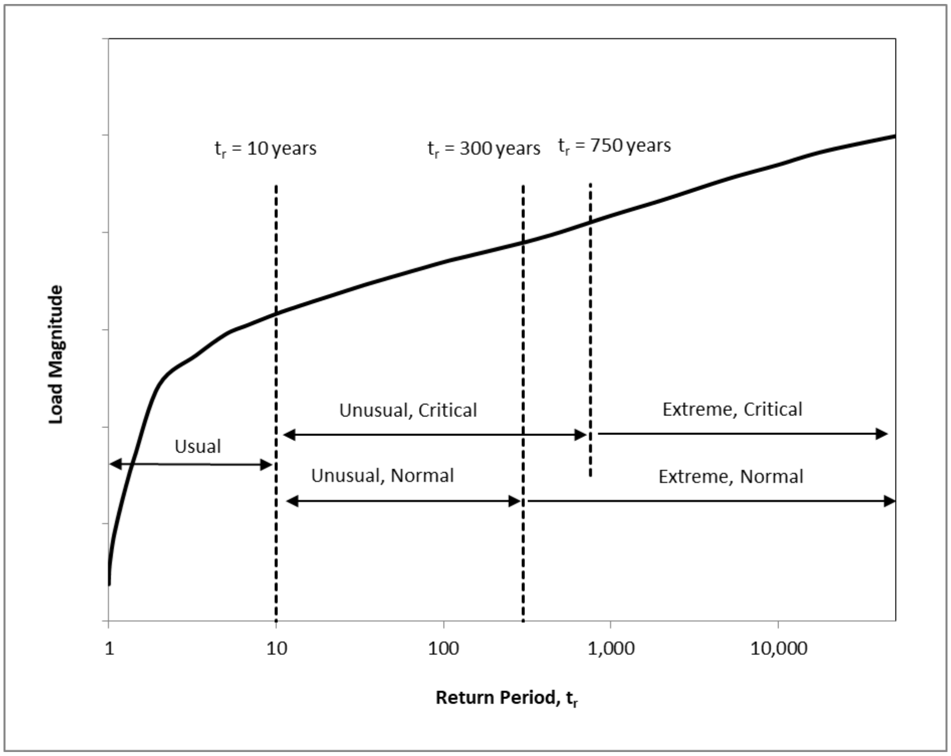


Figure 3.1. Load Category Versus Return Period

3.3.4.1. Usual. The usual load category represents daily or frequent operational conditions for which highly reliable performance is required. The design criteria for the usual load category applies to load cases with the predominant load (or combined loads) having a mean return period (Tr) less than or equal to 10 years (AEP of 0.10). Under this category, deflections are minimized, steel remains within the elastic state, and fatigue cracking is avoided.

3.3.4.2. Unusual. The unusual load category represents infrequent operational conditions that can be reasonably expected to occur within the service life of the project for which a defined level of performance is required. The design criteria for the unusual load category applies to load cases with predominant loads expected to have a return period (Tr) of greater than 10 years (AEP of 0.10) and less than or equal to 750 years (AEP of 0.0013) for critical structures and less than or equal to 300 years (AEP of 0.0033) for normal structures. Under this category, deflections meet operational requirements and steel generally remains within the elastic state.

3.3.4.3. Extreme. The extreme load category represents possible conditions that are unlikely to occur within the service life of the project. The design criteria for extreme load cases are applicable if the predominant load (or combined loads) has a return period (Tr) of greater than 750 years (AEP of 0.0013) for critical structures and greater than 300 years (AEP of 0.0033) for normal structures. Under this category, the structure is expected to withstand the loading, however, some permanent deformation or yielding may result as long as failure critical members remain intact.

3.3.5 Load Duration. Loads on structures vary with time. This affects how the loads are combined in load case combinations and to some extent the performance requirements for the load. Loads can be grouped into the following categories based on duration:

3.3.5.1. Permanent loads (Lp) are continuous loads, such as dead loads. Permanent loads are usual loads.

3.3.5.2. Temporary (intermittent static) loads (Lt) are loads with durations from several minutes to several months, such as flood loads and operation live loads.

3.3.5.3. Dynamic (impulse) loads (Ld) are loads with durations of seconds or less, such as vessel and ice impact, earthquake, wave, and turbulent water flow. Response of HSS structures to these loads may be dynamic, however, for design purposes the response is usually considered to be static for most HSS. Because of the short duration, it is extremely unlikely that more than one dynamic load exists at any given time.

3.4 Materials.

3.4.1 General. Materials are selected to provide the required strength, ductility, and other properties such that the design may be constructed to meet all applicable limit states and performance requirements. Specified material properties may include yield and tensile strengths, ductility, fracture toughness, and corrosion resistance. Material certification reports will be reviewed to ensure material requirements are met.

3.4.2 Material Selection. American Society of Testing and Materials (ASTM) A709 is the preferred structural steel specification used to ensure the appropriate limit states and performance criteria are met. ASTM F1325 Grade A325 is the preferred structural fastener specification. Weld metal is selected based on strength and toughness requirements and material types. Other materials used for attachments, appurtenances, and connections, are selected on a project-by-project basis.

3.5 Member Types. HSS are comprised of various member types, each with specific functions and performance requirements.

3.5.1 Primary and Secondary Members Primary members are the main load-carrying members that provide load path from point of load application to the supports. Secondary members (e.g., bracing, intercostal, and diaphragm) provide support or stability to the primary members. Secondary members also include attachments or other appurtenances. Failure of a secondary member is generally localized, but can lead to a global failure if it leads to failure of the primary member. Impacts from secondary members on primary members must be considered in the design. Some members may serve as both primary and secondary members depending on loading conditions and system response.

3.5.2 Redundancy. Redundancy is a structural condition where there are more elements of support than necessary for strength or stability and loads can be redistributed throughout the structure to avoid collapse. Non-redundant members are identified by the Engineer through analysis and/or judgment.

3.5.2.1. Failure Critical Members.

3.5.2.1.1 A Failure Critical Member is any member for which failure would cause collapse, partial collapse, or loss of functionality of the structure. All Failure Critical Members in an HSS should be identified by the Engineer. The selection of loads and load factors in Chapter 4 is intended to be adequate for design of failure critical members.

3.5.2.1.2 Consider these measures when designing Failure Critical Members:

• Define additional levels of scrutiny throughout the design and fabrication.

• Require additional material testing and nondestructive testing (NDT) requirements.

• Specify additional testing to assure the required performance is achieved and improved reliability of connections is obtained.

3.5.2.2. Fracture Critical Members. Fracture Critical Members (FCM) are a subset of Failure Critical Members and are defined in ER 1110-2-8157. All FCM in an HSS must be identified by the Engineer and labeled in the project plans. Appropriate materials and fabrication requirements will be included in the project specifications.

3.6 Analysis. Individual HSS members are sized to meet the performance requirements under all applicable limit states for the specified load combinations. The load effects in each member are determined through an understanding of the distribution of loads throughout the structure and application of proper analysis techniques. Proper analysis techniques account for the overall response of the HSS as a system and not just individual members or components.

3.6.1 System Response to Loads. An understanding of the load paths, how the applied loads are distributed throughout the various members and into the supports, and the interaction among HSS members is essential to adequate design of HSS.

3.6.2 Simplified Analysis. Guidance to develop simplified analysis models for miter and Tainter gates is provided in Chapters 9 and 10. The simplified analysis methods of Chapter 9 may be applied to the skin plate design of vertical lift gates, stoplogs, bulkheads, and levee closures. The simplified analysis methods of Chapter 10 may be applied to lock Tainter gates and Tainter valves. These models are typically 2D and conducive to hand analysis or simplified truss or frame models.

3.6.3 Advanced Analysis.

3.6.3.1. Techniques. Advanced analysis techniques, such as finite element methods, may be employed where complex loading conditions or member configurations exist or where verification of simplified analyses is desired. When advanced analysis techniques are used, documentation and verification is required.

3.6.3.2. Documentation. Documentation consists of a complete description of the analysis model. This includes element types and descriptions, meshing techniques, boundary conditions and nodal connectivity, adequacy of simplifications of assumed behavior, loads, and load application. It also includes any other information necessary to adequately describe the analysis and results. The documentation should include all assumptions used in the analysis, how the assumptions differ from reality, and why the differences are acceptable.

3.6.3.3. Verification. Verification of advanced analysis used where complex loading conditions or member configurations exist should include hand calculations or independent model analysis. The verification model should be in addition to a detailed check of the original model. Hand calculations may include a comparison of loads and reactions and analysis of simple 2D models. The independent model analysis is subject to the same documentation requirements apply and should also include some hand calculations.

3.7 <u>Corrosion Control</u>. Corrosion is controlled through proper material selection, coating specifications, detailing, and cathodic protection systems. Corrosion control helps prevent deterioration to ensure reliability is maintained through the service life of the HSS. See Chapter 7 for guidance on corrosion control.

3.8 <u>Inspection and Maintenance</u>. See Chapter 7 for guidance on designing for inspection and maintenance. Inspection and Maintenance are considered in the design of HSS by providing good access for conducting inspections and maintenance, specifying proper materials and corrosion protection, employing good detailing practices, and designing components for easy replacement.

3.9 <u>Plans and Specifications</u>. The HSS design is communicated to the fabricator through the plans and specifications. Include all information necessary to fabricate the HSS in these documents.

3.10 <u>Fabrication and Erection</u>. Fabrication and erection processes are considered in the design. Design should consider additional stresses imposed during fabrication and erection and should include proper detailing to accommodate the processes. See Chapter 8 for details.

3.11 <u>Deviations from Prescribed Design</u>. Where special conditions exist, proposed modifications to the design requirements must be submitted to CECW-EC.

Chapter 4
Design

4.1 Design Basis. This chapter provides the basis for design of HSS and rules for assigning load factors and developing load cases. General guidance for selection of nominal loads is provided. General load factors and load combinations are defined in section 4.3. Loads and load cases for specific structure types are described in Chapters 9–16. All HSS members and connections must satisfy Equation 4.1 for each limit state, unless otherwise specified. The basic safety check in LRFD may be expressed mathematically as:

$$\sum \gamma_i Q_{ni} \leq \alpha \varphi R_n \qquad \text{(Equation 4.1)}$$

Where:

γi = load factors that account for variability in loads to which they are assigned

Q_{ni} = nominal (code-specified) load effects

α = performance factor

ϕ = resistance factor that reflects the uncertainty in the resistance for the particular limit state and, in a relative sense, the consequence of attaining the limit state. These are provided in AISC, except the resistance factor for forged and cast materials, which is 0.7.

R_n = nominal resistance as specified in AISC

4.1.1 Performance Factor for HSS. For LRFD of HSS, resistance factors of AISC are multiplied by a performance factor, α. The factor is applied to account for uncertainty due to lesser inspection ability or more corrosive environments. Even with this factor, corrosion must be accounted for in the design as described in paragraph 4.1.2. The value of α is 1.0 except for the following structures where α must be 0.90.

4.1.1.1. For HSS where inspection and maintenance are difficult because the HSS is normally submerged and removal causes disruption of a larger project. Examples of this type of HSS include Tainter valves and leaves of vertical lift gates that are normally submerged and infrequently dewatered.

4.1.1.2. For HSS in brackish water or seawater.

4.1.2 Design for Corrosion. Load and resistance factors in this manual do not account for significant loss of section due to corrosion. This must be accounted for in the design. The performance factor, α, is intended to account for minor section loss in structures difficult to maintain or in very corrosive environments. But the expectation is that it will at some time be serviced and inspected. In instances where routine painting is not considered likely before significant section loss occurs during the service life of the HSS, a cross section that accounts for anticipated loss must be used.

4.1.3 Strength Limit State. Structures must have design strengths at all sections that are at least equal to the required strengths calculated for all combinations of factored loads. The required strength of structural components must be determined by structural analysis using appropriate factored load combinations. Each relevant strength limit must be considered.

4.1.4 Serviceability Limit State. The overall structure and the individual members and connections must be checked for serviceability. The following limit states must be considered in design for serviceability.

4.1.4.1. Deformation in the structural members and supports due to service loads must not impair the operability or performance of the HSS.

4.1.4.2. Vibrations (e.g., the seals, equipment, or movable supports) must not impair the operability of the HSS.

4.1.4.3. Loss of section or impacts to connections and moving parts that do not impact strength but may impair serviceability or operability of the structure during its service life must be limited by designing to minimize corrosion potential.

4.1.5 Fatigue Limit State. The fatigue limit state must be satisfied by selecting fatigue resistant details and stress range limits as defined in the AASHTO or AISC design specifications for the projected loading cycles expected over the service life of the HSS. The applied stress range due to fatigue loading will be as defined in Chapter 5.

4.1.6 Fracture Limit State. The fracture limit state must be satisfied by limiting the potential for fracture as prescribed in Chapter 5. Additional requirements must be imposed on FCMs as defined within Chapters 4, 5, and 8.

4.2 Loads. Apply loads as specified in this section and in combination with other loads as prescribed in section 4.3.

4.2.1 D, Dead Load. Dead load includes the total weight of an HSS and its permanent attachments and equipment. Dead load is computed based on the nominal cross-section of the members and weight of all attachments and appurtenances, all fasteners and welds, and any coating system. Weight of the coating system is based on the specified thickness of the coating system used. Dead load is treated as a usual load.

4.2.2 G, Gravity Loads. Gravity loads consist of mud (including silt), debris, and atmospheric ice. All of these loads are highly site dependent. Both mud and debris are more likely on navigation gates and spillway gates that have tailwater present on the gate much of the time.

4.2.2.1. Mud. Mud (M) loads are based on site conditions and past experience, except that a minimum 1 in. thick layer of mud must be assumed in all areas where silt can accumulate.

4.2.2.2. Debris. Debris is made up of floating logs or other materials that can collect on HSS.

4.2.2.3. Atmospheric Ice. Atmospheric ice loads are determined using guidance from ASCE 7-22. Other ice gravity loads are determined based on site conditions from spray, seal leakage, overwash, etc. Gravity loads are considered permanent loads.

4.2.3 Hydraulic Loads. Hydraulic loads consist of all loads due to water including weight of water, wave action, and the effects of flowing water, such as flow induced vibrations, constricted flow, and water hammer.

4.2.3.1. Hs, Hydrostatic Loads. Hydrostatic load is differential hydrostatic pressure applied to HSS including lateral forces, weight of water above or in a structure, and uplift. Hydrostatic loads may act as permanent, temporary, or a combination of both loads, depending on the geometry of the structure, hydrologic characteristics of the water body and operational procedures when control structures are present. For load case combinations it is treated as a temporary load, even if the water level is mostly constant.

4.2.3.1.1 Serviceability. For hydrostatic loads as the principal load for serviceability load cases, the water elevation and differential head are selected to meet structural and project serviceability objectives.

4.2.3.1.2 Principal Hydrostatic Loads. For design of hydrostatic loads as the principal loads in a load combination (Hs_{pr}), the design water levels must create the maximum hydrostatic loading caused by a differential head that is geometrically and hydrologically possible. Some considerations:

- The maximum hydrostatic loading may occur at water levels that are not necessarily the largest possible differential head.

- Usually, the maximum hydrostatic load is limited by the height of the structure and other factors. But the maximum hydrostatic load may be from water levels that exceed the top of a structure.

- In some cases, hydrostatic loading may be experienced from differential head across a structure in either direction.

- Determination of the maximum differential head condition should be made in consultation with the project's hydraulic engineers.

4.2.3.1.3 Companion Hydrostatic Loads. When Hs is a companion load (Hs_c) values to be used for design are normal operating conditions as defined in paragraph 3.3.3.3.

4.2.3.1.4 Fatigue. For fatigue cases, Hs will be determined as described in Chapter 5.

4.2.3.1.5 Specific Guidance. Additional guidance on selection of water levels for computing hydrostatic loads is found in Chapters 9–16.

4.2.3.2. Hd, Hydrodynamic Loads. Dynamic loads are created by thrust from vessels (prop wash and temporal head), downdrag, inertial resistance, overtopping impingement, and hydraulic shock (water hammer). Guidance on selection of hydrodynamic loads is provided in Chapters 9–16.

4.2.3.2.1 Flow Induced Vibrations. Typically, HSS are not designed to resist flow induced vibrations for strength or fatigue. Flow induced vibrations can be controlled or minimized through proper detailing of seals and control of gate operation (e.g., select gate openings that minimize vibration) where vibration is the result of seal detailing. This topic has been extensively researched. Some information for design is provided in the gate specific chapters of this manual. See Hart and Hite (1979) and Thang (1990) for basic information. For other sources of flow induced vibration, hydraulic analysis or modeling may be required to minimize hydrodynamic loading. Consult with the hydraulic engineer for modeling requirements.

4.2.3.2.2 Overtopping. Overtopping must be considered for HSS where overtopping may occur either through intentional design or from high pool events. Aside from increased differential hydrostatic head from overtopping, forces from overflowing water impinging on HSS members must be considered. Design HSS to withstand this event (e.g., in order to maintain pool until the event has passed).

4.2.3.3. Hw, Wave Loads. Wave loads must be considered for all HSS subjected to significant wind and fetch and will be added to the hydrostatic load from the corresponding water elevation. When determining wave-loading effects, the load category will reflect the water surface elevation and wind conditions that produce the maximum effects. Wave loads are computed as described in EM 1110-2-1100. Wind events used to generate wave loads must account for the location of the structure and characteristics of the hydraulic loading.

4.2.3.3.1 Wave as Principal Load. For HSS on reservoirs and other locations with permanent water loading, the design will consider a case with wave loads as principal loads (Hw_{pr}). These waves are generated by extreme wind events with minimum return periods as defined in paragraph 4.3.4.1. When the wind and water elevation are independent, the wave loads are combined with a companion hydrostatic load, Hs.

4.2.3.3.2 Wave as Companion Load. For other load cases with independent pool and wind-wave events where wave loads are the companion loads (Hw_c), design wave loads must be determined as defined in Chapter 3.

4.2.3.3.3 Coastal Waves. For coastal situations with correlation between surge and wave, annual exceedance of combined loads must be computed by a coastal hydraulic engineer using a coupled analysis of the water elevation and wave heights. The surge level and wave force computed as a function of annual probability of exceedance will be provided by the hydraulic engineer.

4.2.4 T, Self-Straining. Self-straining loads are the cumulative effect of self-straining forces. These effects arise from contraction or expansion due to environmental or operational temperature changes, shrinkage, moisture changes, creep in component materials, movement caused by differential settlement, uneven supports, or combinations thereof. Thermal movement self-straining loads are according to ASCE 7-22. Many HSS types are largely unrestrained and self-straining loads are considered to be insignificant. Uneven support conditions in miter gates can lead to self-straining loads. Uneven support forces that cause additional tension loads in miter gate anchorages are called prying. See Chapter 9.

4.2.5 W, Wind. Principal and companion wind loads must be applied according to ASCE 7-22. Wind loads for critical structures must be calculated using criteria for category IV structures if a failure from the wind load would result in consequences that meet the definition of a critical structure. Wind Loads for all other structures may be calculated using criteria for category II structures. Wind loads for serviceability cases must be computed using the serviceability wind loads from ASCE 7-22.

4.2.6 IM, BI, Impact.

4.2.6.1. Impact loads are caused by floating debris and ice (IM), and barge impacts (BI). For LRFD design, this load is assumed to be an extreme principal load (IM_X or BI_X). It may also be applied as a companion load (IM_c or BI_c). These loads may be correlated to flood and surge water levels. These loads need not be applied to the design of skin plates.

4.2.6.2. IM, Debris Impact. Floating debris and ice must be applied to the HSS members exposed to ice and debris at or above the water elevation to produce maximum effects in each member. Recommended loads are provided in Chapters 9–16 but designers must account for local site conditions to determine design loads. Impacts should be determined from an assessment of probable debris and from past experience. The magnitude of ice loads should take into consideration available local records of ice conditions. For more detailed methods for computing ice forces, see EM 1110-2-1612.

4.2.6.3. BI, Barge Impact

4.2.6.3.1 Barge impact is caused by aberrant vessels or barges moved by wind or current, or by accidents from powered barge tows and vessels entering or exiting locks or passing through gated control structures. The type and size of vessels, barges, and barge tows that may impact HSS are site dependent and should be determined by the engineer after careful research of local conditions. For coastal locations, barges may be moved many miles during high surge and wind events.

4.2.6.3.2 Chapters 9–16 of this manual provide minimum design barge impact loads and their application from previous USACE HSS guidance. HSS located where failure may result in loss of life due to uncontrolled release of water or high economic or environmental consequences may require higher design barge impact load values. HSS located where failure from barge impact would have low consequences may be able to justify using the minimum design values.

4.2.6.3.3 Because of their large potential effect on the design of HSS, design barge impact loads should be determined through risk assessment. EM 1110-2-3402 provides guidance for calculating magnitude and probability of barge impact loads that can be used in the assessment. Guidance for performing risk assessment is provided below.

4.2.6.3.4 Risk assessments for determining barge impact loads will account for costs, probability of an impact and of the magnitude of the resulting load, and consequences. The assessment can be scaled to the size and complexity of the project. The USACE Risk Management Center (RMC) is responsible for the development, dissemination, and interpretation of methodology guidance for use in conducting risk-informed evaluation of new project designs or modifications to existing projects. Methodology contained in USACE – U.S. Bureau of Reclamation (USBR) Best Practices in Dam and Levee Safety Risk Analysis, ER 1110-2-1156, and other USACE guidance can be used to perform the risk assessment.

4.2.6.3.5 Design barge impact loads will be confirmed by approval through the technical review process, project stakeholders, USACE centers such as the RMC and the USACE Inland Navigation Design Center (INDC), and CECW-EC.

4.2.7 IX, Thermally Expanding Ice. This load is applied at sites where continuous ice cover can or does form adjacent to HSS. Thermally expanding ice is a temporary load defined as 5,000 lbs/ft across HSS members exposed to continuous ice sheets. The thermally expanding ice load must be applied at or within 1 foot (ft) below the water elevation to produce maximum effects in each member. This load is a principal load assumed to be extreme (IX_X).

4.2.8 EQ, Earthquake. See section 4.4.

4.2.9 F, Friction. Opening and closing gates develops friction forces caused by the movement. Friction is developed at points of rotation, such as trunnions, pintles, pins, and along seals that rub on seal plates. A load factor is applied to the friction coefficient to account for uncertainty in the amount of friction that will develop. Nominal friction coefficients are provided in Chapters 9–16.

4.2.1 L, Live Load. Live loads are vertical load from personnel, equipment, vehicles, or temporary storage on operating surfaces and walkways. The live load depends on the intended use of the operating surface or walkway and the equipment or vehicles expected to access it. A minimum live load of 100 psf will be used. Live load is factored according to ASCE 7-22. Chapters 9, 13, and 16 contain example load combinations with live load that can be applied to any HSS with an operating surface or walkway.

4.2.2 Q, Operational Loads. Operational loads on gates are applied as follows:

4.2.2.1. Usual Conditions. Under the normal conditions the forces from operational equipment (acting through hydraulic cylinders, wire ropes, gate arms, or other mechanisms) are generally reactions from dead load, gravity loads, friction, and other forces that resist movement. Some equipment, such as hydraulic cylinders, can apply loads directly to the HSS, such as when continuing the closing motion to gates that are already closed. These loads are applied as companion loads to other principal loads.

4.2.2.2. Abnormal Gate Operation. Unusual load conditions are caused by abnormal gate operation for gates operated by two or more mechanisms. Abnormal gate operation includes unbalanced hoists and unbalanced operation. Dead, gravity, and friction loads are applied with only one operator moving the gate. The resulting force in the operating mechanism is a reaction.

4.2.2.3. Gate Jammed.

4.2.2.3.1 General. Loads caused by a jammed gate may be subject to the full capacity (maximum load output) of the machinery. The machinery maximum output is the maximum load output from the driver (such as the motor, hydraulic cylinder, or actuator) carried through the machinery drive train and applied to the structure. Where load limiting devices are used, the machinery maximum output is the maximum output the machinery can deliver to the structure under the load restriction provided by the load limiting device safe limit. Gate machinery loads must be coordinated with the project mechanical engineer.

4.2.2.3.2 Application. The loads from the full capacity of the machinery are applied as principal loads. The gate is modeled as fixed in locations of assumed jams. The probability of the gate operating at the full capacity of the machinery must be determined in order to determine the loading category. Normally, the full jammed condition is very unlikely, and this load can be considered to be in the extreme load category with unknown return period. A load factor from Condition 3 from paragraph 4.3.3 applied for this case. If the maximum output is limited to values that may be more frequently reached during operation, this may be an unusual or usual load and load factors from Condition 2 applied.

4.3 Load Factors and Load Combinations.

4.3.1 Load Factors. Load factors for each load and loading condition are defined for each HSS type in Chapters 9–16. The load combinations defined in paragraph 4.3.2 must be used to satisfy Equation 4.1.

4.3.2 Equations. Specific design equations are provided for HSS types in Chapters 9–16. The general equation for combining loads for the strength limit state is:

$$U = \Sigma\gamma_p\, Lp + \gamma_{pr}\, L_{pr} + \Sigma\, \gamma_c\, Lt_c + \gamma_c\, Ld_c \qquad \text{(Equation 4.2)}$$

Where:

U = Factored Applied Load

γ_p = Load Factor Applied to Permanent Loads

Lp = Permanent Loads

γ_{pr} = Load Factor Applied to Principal Loads

L_{pr} = Principal Load. The principal loads are normally extreme, X, but may be unusual, N, or usual, U, if the maximum loads meet the definitions of those load categories in paragraph 3.3.4.

Lt = Temporary Loads

Ld = Dynamic Loads

$_c$ = Designates companion loads. See paragraph 3.3.3.3.

γ_c = Load Factor Applied to Companion Loads

4.3.2.1. Lp, Lt, and Ld are further defined in Chapter 3. Ld is not included as a companion load when the principal load is a dynamic load.

4.3.2.2. For principal action loads that are correlated, such as hydrostatic and wave forces from storm created surge and wave, the applied principal action load must be determined from the combined, correlated load. The combined load will not be combined with other temporary or dynamic loads.

4.3.3 Permanent Load Factors (γ_p). Permanent loads on HSS consist of dead loads and gravity loads. Maximum and minimum load factors must be applied to provide the greatest effect.

D, when combined with other loads, γ_p =1.2 or 0.9

D, when used alone is a principal load, γ_{pr} =1.4

G, γ_p = 1.6 or 0

4.3.4 Principal Load Factors (γ_{pr}). Principal load factors are used to provide low probability of failure. Normally, extreme loads are used for design loads but for some sites the maximum principal load on a structure may occur with a return period in the unusual or usual load category. Except as where defined in paragraphs 4.3.4.4 and 4.3.4.5 below, or shown in Chapters 9–16, load factors, γ_{pr}, applied to principal loads are as follows:

4.3.4.1. Principal Load Condition 1. Maximum loading is not limited by the geometry of the structure or other physical factors. The return period of the load can be estimated. Examples include wind loads and most wave loads. Nominal loads for design are based on return periods that provide very low probability of exceedance. Minimum design return periods are greater than or equal to 3,000 years for normal structures and to 10,000 years for critical structures.

$$\gamma_{pr} = 1.2$$

4.3.4.2. Principal Load Condition 2. The maximum loading that can be applied is limited by the geometry of the structure or other physical factors. The return period of the load can be calculated or estimated. Examples are differential hydrostatic loading limited by the height of a HSS or operating loads limited by the capacity of the machinery.

4.3.4.2.1 Principal Load Condition 2 (Extreme). The load factor below is applied unless the load has a return period that meets the conditions of principal load condition 1, or unless otherwise stated in paragraph 4.2. Return periods in the extreme range are less than 3,000 years but greater than 300 years for normal structures and less than 10,000 years but greater than 750 years for critical structures.

$$\gamma_{pr} = 1.3$$

4.3.4.2.2 Principal Load Condition 2 (Unusual). The load is limited at a maximum value with a return period in the unusual range of less than 300 years but greater than 10 years for normal structures and less than 750 years but greater than 10 years for critical structures.

$$\gamma_{pr} = 1.4$$

4.3.4.2.3 Principal Load Condition 2 (Usual). The load is limited at a maximum value with a return period in the usual range of less than 10 years.

$$\gamma_{pr} = 1.5$$

4.3.4.3. Principal Load Condition 3. The return period of the load is unknown. Design loads are considered to be of very low expected probability of exceedance (extreme). Examples are impact loads, thermal expansion of ice, barge impact, and many hydrodynamic loads. The load factor below is intended to account for the uncertainty in the knowledge of these loads.

$$\gamma_{pr} = 1.3$$

4.3.4.4. Loads Derived from ASCE 7-22 (Live Load, Self-Straining, Wind).

$$L,\ \gamma_{pr} = 1.6$$

$$T,\ \gamma_{pr} = 1.2\ (\text{min})$$

$$W,\ \gamma_{pr} = 1.0$$

4.3.4.5. Barge Impact.

$$BI,\ \gamma_{pr} = 1.3$$

4.3.5 Companion Load Factors (γ_c). Companion load factors depend on the source of the load. For live load, self-straining, and wind companion load factors are taken from load combinations in ASCE 7-22. For friction, nominal values in Chapters 9–16 typically represent values above expected amounts for components in good condition, but at less than ultimate possible values. For this reason, a load factor of 1.4 is applied to friction coefficients. For all other temporary and dynamic companion loads with a minimum return period of 10 years as specified in paragraph 3.3.3.3 or as specified for Earthquake in paragraph 4.2.7, the companion load factor is 1.0.

F, γ_c = 1.4

L, γ_c = 1.0 (ASCE 7-22)

T, γ_c = 1.0 (ASCE 7-22)

W, γ_c = 0.5 (ASCE 7-22)

All other Lt and Ld, γ_c =1.0

4.3.6 Fatigue Load Factors. See section 5.1.

4.3.7 Load Factors Used for Design. Load factors used for design of HSS are summarized in Table 4.1.

Table 4.1
Minimum Load Factors

Limit State		Serviceability and Fatigue[5]	Strength			
Load Category		**Usual and Unusual**	**Permanent and Companion**	**Principal Load Factors, γ_{pr}** Usual	Unusual	Extreme
Return Period – Critical		< 750	< 10	< 10	10–750	> 750
Return Period – Normal		< 300	< 10	< 10	10–300	> 300
Permanent Loads, L_P			γ_p			
Dead	D	1.0	1.2[1], 0.9[2]	1.4	NA	NA
Gravity (Mud/Ice)	G	1.0	1.6[1], 0[2]	NA	NA	NA
Temporary Loads, L_T			γ_c			
Hydrostatic	Hs	1.0	1.0	1.5[3]	1.4[3]	1.3[7]
Ice, Thermal Expansion	IX	1.0	1.0	NA	NA	1.3[7]
Operating Equipment	Q	1.0	1.0	1.5[3]	1.4[3]	1.3[7]
Live Load	L	1.0	1.0[4]	NA	1.6[4]	NA
Self-Straining	T	1.0	1.0[4]	1.2[4]		
Gate Operation Friction	F	1.0	1.4	NA	NA	NA
Dynamic Loads, L_D			γ_c			
Hydrodynamic	Hd	1.0	1.0	NA	NA	1.3[7]
Wave	Hw	1.0	1.0	NA	NA	1.2[7]
Debris/Floating Ice	IM	1.0	1.0	NA	NA	1.3[7]
Barge Impact	BI	1.0	1.0	NA	NA	1.3
Wind	W	1.0	0.5[4]	NA	NA	1.0[4]
Earthquake	EQ	NA	NA	NA	1.5	1.0 or 1.25[6]

Notes:

1. Applied when loads add to the predominant load effect.
2. Applied when loads subtract from the predominant load effect.
3. Usual or unusual loads used as principal loads for strength design when they are the maximum possible loads.
4. From ASCE 7-22. Where other standards are referenced, load cases and load factors from those standards will be used for design when those loads are primary loads. See load descriptions for details.
5. Load factors for finite fatigue life are shown. Load factors for infinite fatigue life are 2.0 for all loads. See section 5.1.1.
6. For site specific earthquake the load factor is 1.0. Otherwise, the higher load factor is used. See section 4.4.
7. Typical design load factors shown. See paragraph 4.3.4 for selection of principal load factors.

4.4 Earthquake.

4.4.1 General. In developing earthquake loads, two different earthquakes, as defined in ER 1110-2-1806, need to be considered when designing for serviceability and strength. The Operating Basis Earthquake (OBE) is an unusual load and the Maximum Design Earthquake (MDE) is an extreme load. For critical features (defined in ER 1110-2-1806), the MDE is the same as the Maximum Credible Earthquake (MCE). Ground motions and performance requirements for the OBE and MDE will be according to ER 1110-2-1806. Risk-informed design may require additional seismic loading at different earthquake return periods.

4.4.2 Earthquake Loads. Earthquake loads are of low probability of occurrence and short duration. Since they are principal loads, they are combined with companion loads according to paragraph 4.4.7 when developing load combinations. The other static loads typically included with earthquake for HSS are Dead (D), Gravity (G), and Hydrostatic (Hs).

4.4.3 Simplified Screening. The simplified screening method can used for design of HSS when earthquake is not the controlling load case. It should not be used for final design of HSS when earthquake load is the controlling load case. Final design of HSS must use either response-spectrum or time-history methods.

4.4.4 Determination of Ground Motions.

4.4.4.1. U.S. Geological Survey (USGS) – National Seismic Hazard Model (NSHM) Ground Motions. If no ground motions have been developed specifically for the project site, the OBE and MDE can be obtained from published USGS spectral acceleration maps and USGS web-based seismic hazard analysis tools (latest version). In some geographic areas, ground motions may only be available for site class B/C (shear wave velocity of 760 m/s). If this is not appropriate for the project site, adjustments to the ground motion values can be made using the adjustment factors in ASCE 7-22.

4.4.4.2. Site Specific Ground Motions. According to ER 1110-2-1806, site-specific studies are conducted for all projects located in regions of high to moderate seismicity for which earthquake loading controls the design. When the seismic load case is a governing case, it will be necessary to develop response spectra and/or earthquake time histories. Guidelines for development of design response spectra can be found in EM 1110-2-6050, and guidelines for the development of site-specific time-histories can be found in EM 1110-2-6051.

4.4.5 Earthquake-Generated Inertial Forces. In addition to the inertia of the HSS itself, water above the ground surface, and adjacent to or surrounding the structure, will increase inertial forces acting on the structure during an earthquake. The displaced structure moves the surrounding water, thereby causing hydrodynamic pressure to act on the structure. For simplified screening analysis with standard or site-specific ground motions, the hydrodynamic pressure can be approximated by using the Westergaard method (Westergaard 1933) shown in Equation 4.3 and Figure 4.1.

$$p = \frac{7}{8}\gamma_w a_c \sqrt{Hy} \qquad \text{(Equation 4.3)}$$

Where:

p = lateral pressure at a distance y below the pool surface

γ_w = unit weight of water

a_c = maximum acceleration at the HSS supports in the upstream/downstream direction (as fraction of gravitation acceleration, g). See paragraph 4.4.6.

H = pool depth to dam foundation

y = distance below the pool surface

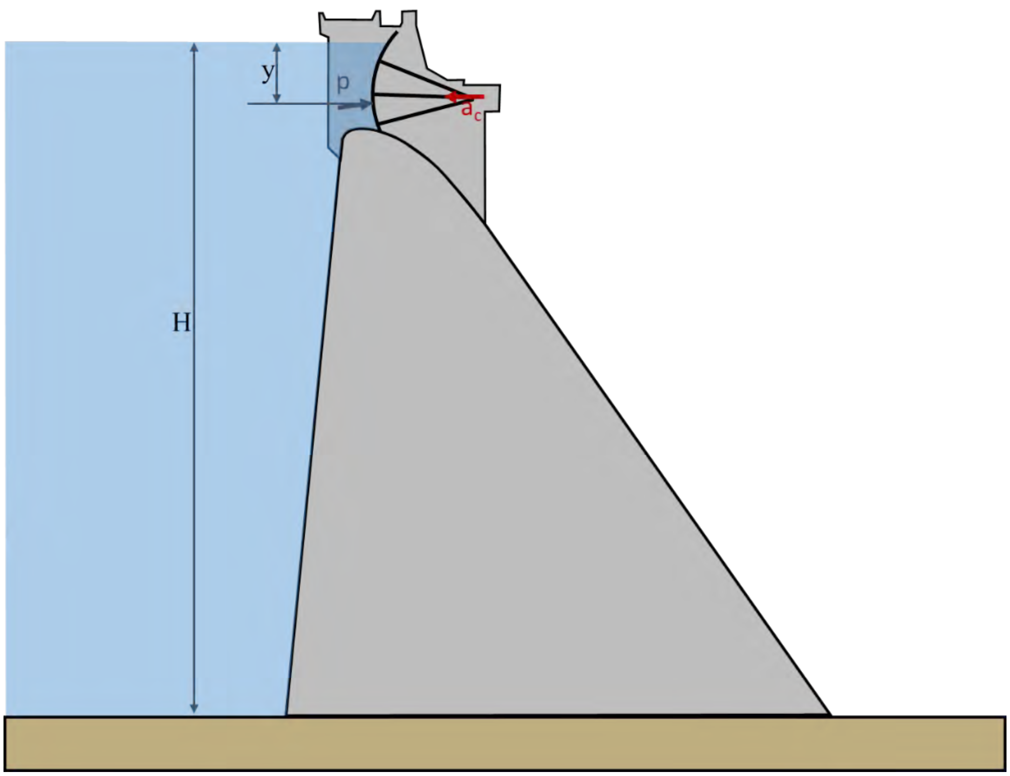


Figure 4.1. Parameters for Equation 4.4

4.4.6 HSS Support Acceleration (a_c).

4.4.6.1. Maximum Acceleration. Maximum acceleration at the HSS support (a_c) may not be equal to the maximum ground acceleration, depending on the height of the structure.

4.4.6.1.1 Near Ground Height. For HSS located near the ground (or in very stiff structures) the peak ground acceleration is appropriate for use as a_c. For example, miter gates that are connected to a lock wall that is very stiff in the upstream/downstream direction will not experience significant amplification of the ground motions. In this case use a_c = PGA.

4.4.6.1.2 Above Ground Height. When the HSS is located at some height in a flexible structure, such as gates located near the top of tall dams, the response of the structure cannot be neglected when determining the acceleration of the HSS. There are several acceptable methods for determining a_c when the structural response is significant.

4.4.6.2. Acceleration.

4.4.6.2.1 Acceleration at the location of the HSS can be determined directly from a finite element model using a time history or response spectrum analysis. Alternatively, a general equation for a_c is given in Equation 4.4 (an explanation of the variables are given in paragraphs 4.4.6.3 to 4.4.6.6), which assumes that the first mode response of the dam (in the upstream/downstream direction) controls the amplification. The response of the higher modes can be represented by ground motion, and vertical motions are neglected. This equation also conservatively assumes that the peak ground acceleration occurs at the same instant as the peak structural response.

$$a_c = C\left[\left(S_A(T_1,\zeta)\tilde{\Gamma} - PGA\right)\varphi(z) + PGA\right] \quad \text{(Equation 4.4)}$$

Where:

C = Pseudo Static Correction Factor = 0.75

$S_A(T_1, \zeta)$ = Spectral acceleration at the period and damping of the structural system

$\tilde{\Gamma}$ = Parameter calculated from the geometry of the structure supporting the HSS; depends on the distribution of the mass within the structure as well as the mode shape

$\varphi(z)$ = Normalized mode shape of the structure supporting the HSS associated with the period T_1, where z is the location of the HSS within the structure measured vertically from the base

PGA = Peak Ground Acceleration

4.4.6.2.2 In Equation 4.4, $S_A(T_1, \zeta)$ is the spectral acceleration at the first mode of vibration in the upstream to downstream direction and damping of ζ; $\varphi(z)$ is the mode shape for the first mode of vibration as a function of height; $\tilde{\Gamma}$ is a scale factor which depends on the geometry of the structure and the height of the water on the structure. The term $\varphi(z)$ is normalized to equal zero at z=0 (base of structure) and equal to 1.0 at the top of the structure.

4.4.6.3. Use of Equation 4.4 will require determination of the period of the structure, the mode shape, and $\tilde{\Gamma}$. The period and mode shape can be calculated from simplified formulas or finite element modeling, but must include the effects of the water in the method used. If the structure is a standard gravity dam configuration with a pier approximately 60 ft tall, Chopra and Tan (1989) give correlations to determine the period and mode shape.

4.4.6.4. Once the mode shape is known, $\tilde{\Gamma}$ $(\tilde{L}_1/\tilde{M}_1)$ can be calculated using the methods given in Chopra and Tan (1989). The spectral acceleration can be determined at the site using site specific seismic hazard studies if available or from published USGS spectral acceleration maps and USGS web based Seismic Hazard Analysis Tools.

4.4.6.5. As a conservative first approximation, $S_A(T_1, \zeta)$ can be assumed equal to the maximum of the design response spectrum at 5% damping (as shown in the example response spectrum in Figure 4.2), $\tilde{\Gamma}$ can be assumed equal to 2.8, and the mode shape can be approximated with Equation 4.5. In Equation 4.5 z is the height of the HSS support from the base of the structure and H_s is the full height of the structure supporting the HSS.

4.4.6.6. Substituting Equation 4.5 into Equation 4.4 results in Equation 4.6. This approximation assumes that the dam is a concrete gravity dam with a height to width ratio greater than 1h:1w, where the gate pier is relatively short in relation to the full height of the dam. For stiffer structures, the amplification may be overestimated by the assumed value of $\tilde{\Gamma}$ and the mode shape. For very short/stiff structures, such as lock walls evaluated in the upstream downstream direction, PGA may be used for a_c. For intermediate stiffness structures, a value $\tilde{\Gamma}$ of 1.5 may be used (Table 4.2).

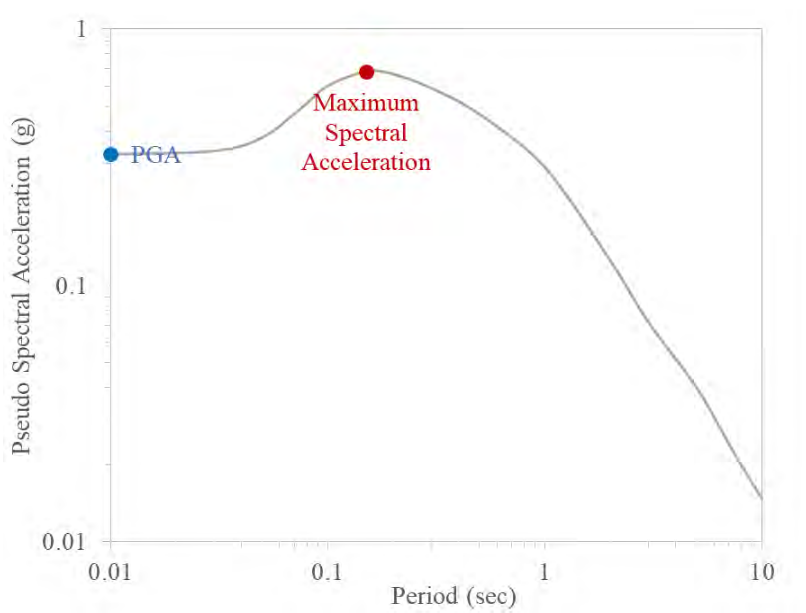


Figure 4.2. Example Site Response Spectrum

Table 4.2
a_c vs Height to Width

Height to Width Ratio	ac
>1h:1.5w	Equation 4.7 using $\tilde{\Gamma}$ = 2.8
1h:1.5w–1h:3w	Equation 4.7 using $\tilde{\Gamma}$ = 1.5
<1h:3w	PGA

$$\varphi(z) = 23.41\sin^2\left(\frac{2\pi z}{32.18H_s} + 0.0122\right) \qquad \text{(Equation 4.5)}$$

$$a_c = \left[S_A(T_1,\zeta)\tilde{\Gamma} - PGA\right]23.41\sin^2\left(\frac{2\pi z}{32.18H_s} + 0.0122\right) + PGA \qquad \text{(Equation 4.6)}$$

Where:

z = the height of the HSS above the dam foundation and H_s is the full dam height

4.4.6.7. If the use of Equation 4.5, either using the simplified assumptions or by calculating the inputs, results in the seismic load case governing the design, a dynamic analysis should be considered to avoid over-designing the steel structure. The dynamic analysis may be a time history or response spectrum analysis depending on the detail of seismic loading information developed for the site.

4.4.6.8. It is possible to create a model that contains the HSS structure as well as the supporting structure and foundation to calculate the loads on the HSS directly. It is also possible to create two models; one to calculate the loads on the HSS and a separate model of only the HSS for use in the design. In these models, multiple components of horizontal motion and vertical ground motion can be included if it is considered significant. Analysis of the concrete structure should be done according to EMs 1110-2-6053 and 1110-2-6051. Appendix D provides information in addition to the guidance provided below.

4.4.7 Load Factors. Equations for Earthquake load are as follows. Variable names are as previously defined. See paragraph 4.2.3.1 for description of the hydrostatic load to be combined with earthquake:

4.4.7.1. For standard and site-specific OBE ground motion analysis:

$$U = \Sigma\gamma_p\ Lp + 1.5\ EQ + \gamma_c\ Lt_c \qquad \text{(Equation 4.7)}$$

4.4.7.2. For standard MDE ground motion analysis:

$$U = \Sigma\gamma_p\ Lp + 1.25\ EQ + \gamma_c\ Lt_c \qquad \text{(Equation 4.8)}$$

4.4.7.3. For site specific MDE and MCE ground motion analysis:

$$U = \Sigma\gamma_p\ Lp + 1.0\ EQ + 1.0\ Lt \qquad \text{(Equation 4.9)}$$

Chapter 5
Fatigue and Fracture

5.1 Design for Fatigue. All cyclically loaded HSS must be designed for the fatigue limit state.

5.1.1 Stress Life. The stress life procedures, as defined in AISC 360 and AASHTO, must be used to design for fatigue. Either reference is acceptable for design. Two fatigue stress limits will be considered, finite life and infinite life (infinite life is called indefinite life in AISC 360 but they are the same). These two stress range limits are represented by the S-N (Stress Range-Number of Stress Cycles) curves in Figure 5.1. The sloping curves represent finite life, and the horizontal curve represents infinite life.

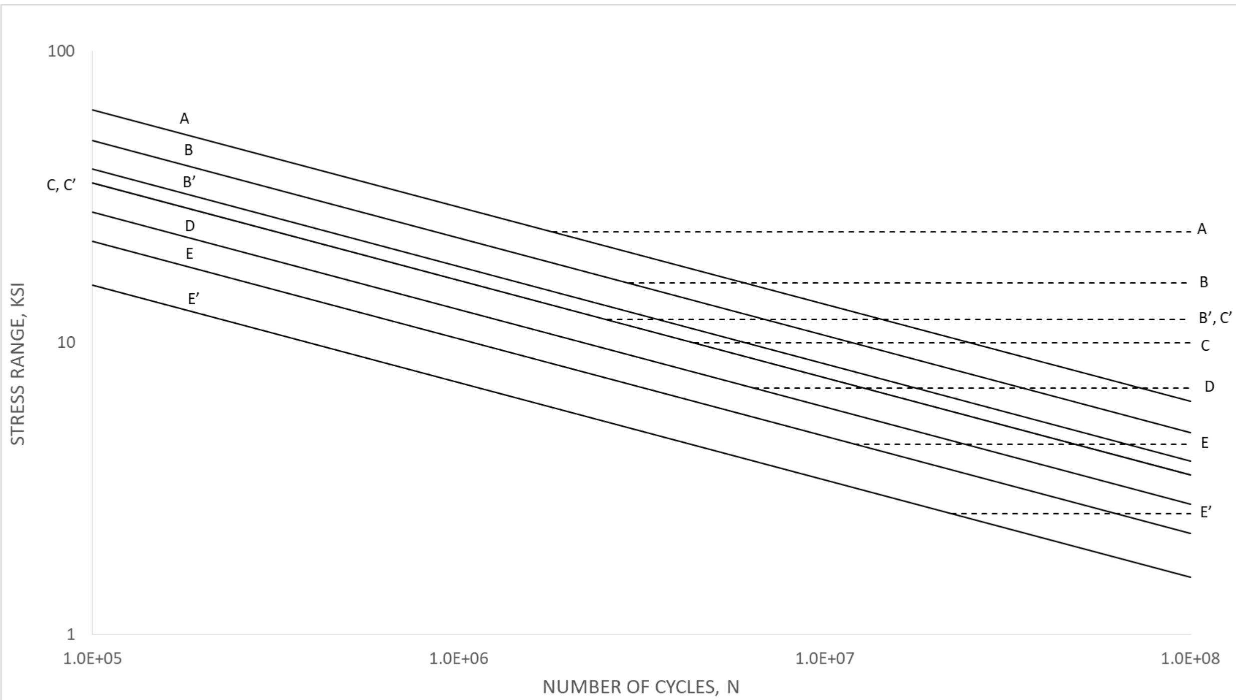


Figure 5.1. S-N Curve

5.1.1.1. Finite Life. Finite life implies fatigue damage will occur under a given combination of stress range and number of stress cycles. A design is acceptable when the combined stress range and number of stress cycles exceeds the fatigue strength for the detail category selected. Therefore, a load factor of 1.0 is applied to the loads used to calculate fatigue stress when evaluating this fatigue stress limit.

5.1.1.2. Infinite Life.

5.1.1.2.1 Infinite life implies that no damage will occur if the stress ranges are less than the Constant Amplitude Fatigue Limit (CAFL) for the detail selected. If a sufficient number of stress cycles exceeds the CAFL, damage occurs, and behavior reverts to finite life.

5.1.1.2.2 To ensure stress ranges do not exceed the CAFL when evaluating infinite life, apply a load factor of 2.0 to the stress magnitude defined in paragraph 5.1.3. Alternately, the stress range from the maximum operational load expected can be used with a load factor of 1.0. The lesser of these factored loads can be used for design. For stress ranges due to hydrostatic load, the maximum operational live load can be limited to that due to load with a 100-year average annual return period. For hydrodynamic loading, use the maximum stress range determined from fluid dynamics modeling.

5.1.2 Fatigue Details. Details should be selected to ensure that fatigue cracking will not occur over the life of the structure.

5.1.2.1. Selecting Fatigue Detail Categories. The detail category defined in AISC and AASHTO that best matches the selected connection detail in terms of geometry, configuration, and degree of stress concentration, should be used in determining design limit states. Fatigue resistant details of Categories A through C are generally preferred. Details classified as Categories D through E should be avoided where possible.

5.1.2.2. Fatigue Detail Category Improvement. Where fatigue is governed by weld toe cracking, fatigue resistance can be improved through weld toe improvements after welding including ultrasonic impact treatment (UIT), heat treatment, grinding, weld toe remelting using gas tungsten arc welding (GTAW) or plasma arc dressing, hammer peening, and shot peening.

5.1.3 Selecting Stress Ranges. Stress ranges are calculated for live load stresses only and are the difference in minimum and maximum values over one stress cycle. Where compression and tension exist within one cycle, the absolute values of compression and tension stresses are added to compute the total stress cycle. Details are fatigue prone only when subjected to a net tension stress. If the live load tension stress is less than half of the dead load compressive stress occurring simultaneously with the tension stress, then a fatigue life check is not necessary (considering a factor of safety equal to 2.0).

5.1.3.1. Stress Determination. Stresses are determined through an appropriate level of analysis to properly consider load path, load distribution, and boundary conditions. In many cases, stresses may be determined using the simplified load models provided in Chapters 9 and 10. Alternatively, stresses may be determined by using instrumentation of HSS with a similar configuration and operation as the HSS being designed, in conjunction with more refined analyses.

5.1.3.2. Stress Magnitude. The magnitude of stress assumed for each stress cycle is the average annual maximum stress that can occur (e.g., the stress in a member caused by the hydrostatic head with a one-year return period). This magnitude of stress is assumed to occur for the number of load cycles expected during the design life. Alternatively, the statistical variation in loading and resulting stresses can be determined and an equivalent fatigue loading computed using Miner's Rule or other appropriate averaging techniques.

5.1.4 Selecting Number of Cycles. The number of stress cycles considered in fatigue design must be based on the stress range selected and must consider multiple stress ranges occurring during a load cycle.

5.1.5 Distortion Induced Fatigue. Distortion induced fatigue must be addressed through proper detailing or through fatigue design. Where fatigue design is used, stresses must be determined through refined analysis and the fatigue detail category will be based on the assumed resistance of the connection.

5.2 Design for Fracture. The fracture limit state must be considered in design of HSS to prevent brittle fracture. Design for fracture must include minimizing stress concentrations, minimizing constraint, and specifying proper material properties. Stress concentrations are controlled through proper detailing and fabrication specifications. Detailing practices for fracture control follow practices used in fatigue design.

5.2.1 Fracture Critical Member Determination. The Engineer will identify all FCM present in the design. Any rational engineering analysis can be used to determine member redundancy as long as it adequately represents the redistribution of load to the structure from the member being analyzed. Load combinations for all applicable limit states will be evaluated in the analysis.

5.2.1.1. Strength Limit State. For the strength limit state, all applicable load combinations will be evaluated with a load factor of 1.0 applied to each load. A resistance factor of 1.0 will be applied to the applicable strength limit. However, in no case should the resulting stresses exceed 90% of yield.

5.2.1.2. Serviceability Limit State. The serviceability limit state will be evaluated where the HSS must function after damage has occurred.

5.2.1.3. Refined Analyses. If refined analysis demonstrates that a structure has adequate strength and stability sufficient to avoid partial or total collapse in the presence of a fractured member (by redistribution of forces or structural redundancy), the member assumed to have fractured may be considered as redundant and would not be determined an FCM (see AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members). Refined analyses that account for plastic redistribution of forces, plastic mechanisms, large displacement theory, or other advanced techniques require CECW-CE approval.

5.2.2 Fracture Control Plan. FCMs and/or components must be designed and fabricated according to the AASHTO/AWS Fracture Control Plan (FCP). See Chapter 8 for guidance on FCP development. Alternatively, the Engineer may establish the FCP using fracture mechanics as a basis and including hydrogen control and other welding controls that limit potential for cracking to an acceptable level.

5.2.3 Constraint. Constraint must be avoided at all connections through proper detailing and fabrication unless it can be shown that failure of those connections will not have an adverse effect on the overall safety and performance of the structure.

5.2.4 Fracture Mechanics. In lieu of the guidance for the design of fracture, the fracture limit state can be addressed directly through fracture mechanics. See EM 1110-2-6054 for guidance on use of fracture mechanics through the application of the Fitness for Service (FFS) procedure.

Chapter 6
Connections and Detailing

6.1 General. Design of Connections. All connections must be detailed by the Engineer. Any deviation from details originally specified must be reviewed and approved by the Engineer. Utilize details that result in safe economical fabrication, erection, and maintenance. The fabrication of connection details must be consistent with the assumptions used in the design analysis of the structure and must be capable of transferring the required forces between connected members.

6.1.1 Design Considerations. Connections must be designed to transfer the required forces obtained from the structural analysis and must maintain sufficient ductility and rotation capacity to satisfy the particular design assumptions. Connection designs must consider stress concentrations, eccentricities, field splices, imposed restraints (fixity), and fatigue resistance.

6.1.1.1. Constraint. Constraint should be minimized in connections by using copes to avoid intersecting welds, limiting weld sizes to that which satisfies the limit states considered, and avoiding stiff and constrained connections at intersecting members. Environmental conditions (welding outdoors in wind and in cold temperatures), economics (bolting vs. welding), and accessibility (equipment and personnel) must also be considered especially when designing or designating field connections.

6.1.1.2. Stress Concentrations. Avoid abrupt transitions in thickness or width, sharp corners, notches, and other details which may cause stress concentrations and have adverse effects on performance.

6.1.1.3. Eccentricities. The design of connections must account for effects of eccentricity.

6.1.1.4. Splices. Splices should be avoided if possible but should be located in uncongested areas of low or moderate stress when required. Show location and accompanying details for splices on the drawings. Optional splices may be proposed by the fabricator or erector but must be reviewed and approved by the Engineer. When large or complex structures are being designed and constructability is a concern, representative fabricators and erectors should be contacted for discussion on limitations regarding fabrication and construction.

6.1.1.5. Fatigue. Connections must be designed to minimize the possibility of fatigue damage by using proper detailing practices and designing for fatigue. Corrosion-induced fatigue is controlled with a well-designed and maintained corrosion protection system. In general, connections that include tensile stress should be detailed as fatigue resistant details to minimize stress concentration, even if fatigue loading is not present.

6.1.2 Welded Connections. Most HSS are constructed using welded connections. Intersecting, overlapping, and intermittent welds should be avoided. Complete joint penetration welds with backing bars should have the backing bars removed and the resulting joint ground flush. Conduct NDT on the completed welds according to Chapter 8.

6.1.2.1. Welding Codes. Cyclically loaded members and fracture critical members must be designed according to AWS D1.5 and should be used for fabrication of all HSS. AWS D1.1 may be used on redundant, non-cyclically loaded HSS where fatigue and fracture are not design considerations.

6.1.2.2. Weld Metal. Weld metal must be selected to satisfy the connection design with consideration for required toughness, strength, and base metal specifications. When welding to high-strength steel, use of undermatching weld metal strength should be considered to limit residual stresses.

6.1.2.3. Fracture Control Plan. See Chapter 8 for guidance on FCP development.

6.1.2.4. Fracture Critical Welds. All welds to FCMs will be considered fracture critical. The Engineer will ensure that all FCMs are identified on contract drawings and that all Fracture Critical Welds (FCW) are identified in shop drawings.

6.1.3 Bolting. Fully tensioned high-strength bolts must be used for all HSS structural applications. For corrosion protection, it is recommended to prime coat the faying surfaces of the bolted connection. Slip-critical connections must be used where slip of the connection may inhibit the operability of the HSS and for cyclically loaded HSS. If using slip critical connections with a vinyl paint system, faying surfaces primed with vinyl primer do not qualify as a slip critical faying surface. If a qualified primer is used, be aware that vinyl paint does not adhere to the primer since it is inorganic zinc paint.

6.1.3.1. Overspray Considerations. Care must be taken not to overspray past the faying surface perimeter or to mask the faying surface so that the vinyl paint system is not compromised. While clean mill scale surfaces qualify as a Class A surface and blast-cleaned steel surfaces qualify as a Class B surface, it is recommended to prime connection faying surfaces for HSS structures for corrosion protection.

6.1.3.2. Bolt Proportioning. Bolts must be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts.

6.1.3.3. Bolting Procedures. Bolting procedures must be specified to ensure that proper tensioning is demonstrated through testing as required in Research Council on Structural Connections (RCSC) S348, Specification for Structural Joints Using High-Strength Bolts.

6.1.3.4. High-Strength Bolts. High-strength bolts used in the fabrication and erection of structural connections in HSS must be ASTM F3125 Grade A325, ASTM F3125 Grade A490, or ASTM 3148 bolts with compatible nuts and washers. ASTM A307 bolts or graded bolts (Society of Automotive Engineering J429 Grade 5 or Grade 8) may be used for non-structural applications but must not be used for structural connections according to AISC and RCSC S348, Specification for Structural Joints Using High-Strength Bolts.

6.1.3.5. Bolt Corrosion Protection.

6.1.3.5.1 Plain. Plain bolts are uncoated bolts. Uncoated bolts will corrode in a submerged environment. For typical HSS applications, all bolts should be painted or coated. Bolts are painted after assembly.

6.1.3.5.2 Galvanized. Galvanized bolts typically come in two varieties: hot-dipped and mechanically galvanized. Since the thickness of coating and resulting thread tolerances are different between the two types they cannot be interchanged, for example, mechanically galvanized bolts must be used with mechanically galvanized nuts and vice versa). Some high-strength bolts, such as ASTM F3125 grade A490, cannot be galvanized since the structural integrity of the bolt is compromised by hydrogen embrittlement. If painting galvanized bolts, there is a preference towards using mechanically galvanized bolts with lubricant only on the internal threads and one face of the nut as opposed to hot-dipped galvanized nuts where the whole nut is typically dipped in wax lubricant and it is difficult to successfully remove all the wax properly for paint preparation.

- Normally Submerged Conditions. Unpainted galvanized bolts are not recommended for normally submerged conditions because unprotected galvanized steel will rapidly sacrifice the zinc coating in submerged environments. Galvanized bolts may be used in infrequently submerged conditions or in noncritical locations, such as seals and fenders that can be accessed for replacement. The Engineer may consider a nonstandard coating system that has shown acceptable performance over a limited time for noncritical connections.

- Other Coated Bolts. Other types of proprietary coatings are available including those for use on high-strength bolts that cannot be galvanized due to hydrogen embrittlement.

6.1.3.5.3 Stainless Steel Bolts. Designers are often tempted to use stainless steel bolts since they offer great resistance to corrosion and could ideally be reused if something has to be taken apart. When relatively new, both stainless steel and painted/galvanized carbon steel bolts can be unbolted but after time, especially in submerged environments, both stainless and plain carbon bolts freeze stuck and typically have to be cut in order to be removed.

6.1.3.5.4 Galvanic Corrosion. Numerous hydraulic submerged carbon steel structures that used stainless steel bolts have suffered damaging galvanic corrosion due to dissimilar metals. Efforts can be made to isolate the metals, but it gets more difficult with the conductivity of water. Overall, it is not recommended to use stainless steel bolts with plain carbon steel structures.

6.1.3.5.5 Painted Bolts. Painted galvanized bolts are recommended for the majority of HSS connections due to easier, less expensive fabrication and better final coating system quality (i.e., it is difficult to successfully blast all plain bolt and nut surfaces properly). Painted galvanized bolts are also useful for seal connections or other connections that cannot be blasted after bolt installation.

6.1.3.6. Bearing Connections. Bearing connections are connections in which a snug-tightened joint or pretensioned joint with bolts transmits shear loads.

6.1.3.6.1 Limit States. Design limit states are the shear strength of the bolts and the bearing strength of the connected materials. Bearing connections rely on bearing between the bolt and joining plate material to transfer load. Bearing connections may either be made with snug-tightened bolts or with fully pretensioned bolts.

6.1.3.6.2 Capacity. Bearing capacity will be dictated by the thickness of the base metal being joined, the size and grade of the bolts being used in the connection, and the presence or absence of threads in the shear plane. The Engineer should ensure that all bearing connections are properly designed and detailed to assure that threads are excluded from the shear plane where appropriate. Bearing connections must not be used where fatigue or cyclic loads are applied to the connection.

6.1.3.7. Pretensioned Connections.

6.1.3.7.1 Use. Fully pretensioned connections must be used for all structural connections on HSS. Structural connections are those connections that transmit primary live and dead load through the structure. Pretensioning bolts reduces the possibility of movement within a connection when it is subjected to loads, particularly dynamic or vibration loading. Pretensioned joints transmit shear and/or tensile loads through the bolts and through the bearing of the bolts against the connected material. As a result, the faying surfaces does not require special preparation.

6.1.3.7.2 Coatings. The surfaces being joined in a pretensioned connection may be uncoated, coated, or galvanized in any manner since the slip resistance is not considered to resist applied loads. The coating affects the ability of the connection to be pretensioned. Soft coatings, such as vinyl paint or a sealing gasket, may prevent the connection from being pretensioned. Pretensioned installation involves the inelastic elongation of the portion of the threaded length between the nut and the thread run out. Faying surfaces must be prepared according to Unified Facilities Guide Specifications (UFGS) 09 97 02 Painting: Hydraulic Structures. When using a vinyl paint system primer on faying surfaces, see discussion in Commentary for paragraph 6.1.3.

6.1.3.8. Slip-Critical Connections. Slip-critical connections transfer shear loads or shear loads in combination with tensile loads through the clamping force between two properly prepared faying surfaces of a connection. Loads are resisted by friction between and without displacement at the faying surfaces. Slip-critical connections require both proper faying surface preparation and proper installation of pretensioned bolts.

6.1.3.8.1 Slip-critical connections must be used for all cyclically loaded connections and for all connections where movement of the connection under applied load is considered detrimental to structural performance (e.g., lifting lugs, dogging brackets, or gudgeon anchorages). Slip-critical connections are also required for all connections that are to be designed according to AISC or AASHTO as a fatigue category B connection. Slip-critical connections must be used in the following instances for connections involving shear or combined shear and tension:

• Connections that are subject to fatigue or load with reversal of the loading direction,

• Connections that use oversized holes,

• Connections that use slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot, and

• Connections in which slip at the faying surfaces would be detrimental to the performance of the structure.

6.1.3.8.2 Faying Surface Preparation. Faying surfaces must be properly prepared to provide frictional slip resistance according to the RCSC S348 and AISC 360. Proper preparation of faying surfaces requires the use of abrasive techniques or the application of appropriate paint coatings designed to create slip resistance. The Engineer should consider the effects of corrosion on faying surfaces as a function of the HSS's exposure or submergence and should specify the use of protective coatings or sealing methods required to prevent corrosion or pack rust from forming, resulting in reduced slip resistance and potential failed connections.

6.1.3.9. Fitted Bolts and Fitted Connections. Fitted bolts are often used in existing structures to create a slip-critical connection, or a connection where movement between parts is detrimental to performance of the structure. Drawings will often refer to "Fitted Bolts," "Body Fit Bolts," "Turned Bolts," or "Match Drilled Bolts." In all instances, the holes are drilled and reamed in the connection to the diameter of the bolt, or the bolt is turned to the diameter of the hole.

6.1.3.9.1 Clearances. Clearances range from hundredths to thousands of an inch depending on the criticality of location and the degree of movement permitted in the connection. These fits are often referred to as "Force Fit," "Press Fit," or "Freeze Fit" and are symbolized as LN (Location Interference Fit) or FN (Force or Shrink Fit). Unlike slip-critical connections designed under AISC or RCSC guidance using clamping force and friction, fitted connections are designed to ensure that shear is occurring in all bolts equally on the application of load.

6.1.3.9.2 Design. There is no AISC code guidance regarding the design of a fitted connection. Clearance between the bolt hole and the fitted bolt must be determined based on materials being joined, degree of movement permitted in the connection, and ease of installation. Fitted connections designed for new construction should consider the use of ASTM A449 bar stock, which is equivalent in material properties to that of ASTM F3125 A325 bolts. Consultation with an experienced bolting engineer, typically a mechanical engineer, is recommended.

6.1.3.10. Other Bolts. For nonstructural applications, use of ASTM A307 bolts or snug-tight high-strength bolts is allowed, provided requirements of AISC are followed.

6.2 Detailing for Performance. All HSS must be detailed to provide acceptable performance including prevention of fatigue and fracture, to provide concentric connections, proper weld access, copes, corner clips, and constructible tolerances, and to avoid points of stress concentration.

6.2.1 Detailing to Minimize Residual Stress. The Engineer should design HSS and HSS details to minimize residual stress, restraint, and constraint. Fabrication requirements, including weld sequence and inspection requirements, must be specified for thick plate weldments or highly constrained weldments that will include large tensile residual stresses. Stress relieving should be considered for large weldments, girders with thick plates, and high restraint.

6.2.2 Detailing for Fatigue Resistance. The Engineer must select fatigue resistant details as described in Chapter 5 for all cyclically loaded HSS, including those subject to vibration and high cycle fatigue, to minimize the potential for crack initiation and propagation.

6.2.3 Detailing for Fracture Resistance. To reduce the potential for cracking, the Engineer must select fracture resistant details for all HSS to avoid restraint and constraint.

6.2.4 Corrosion Control. Detailing for corrosion control includes providing for sufficient drainage and sealing of all connections.

6.2.4.1. Drainage. The Engineer should ensure that properly sized and located drain holes are provided. Minimum size welds that comply with AISC and AWS criteria are required for seal welds. Sufficient room should be provided through copes and access holes to accommodate wrapping of welds.

6.2.4.2. Other Considerations. In non-submerged conditions or where the detail will not be exposed to a significant amount of moisture, welds may be terminated short of copes. In bolted connections, fully pretensioned bolts should be specified and a sufficient number of bolts should be provided to ensure that moisture is excluded from the connection.

6.3 Detailing for Fabrication. All HSS must be detailed to accommodate fabrication processes. Individual members should be fabricated before final assembly of the components into the global structure. There should be sufficient access for welding, sandblasting, painting and other coatings, and inspection equipment and provisions (such as access hatches and safety railing) to provide an inspector access to frequently inspected areas.

6.3.1 Distortion Control. Provide for proper fit-up and weld sequencing, and avoid warping. Weld sequencing is critical to controlling residual stress during fabrication. Individual pieces (e.g., complete joint penetration welds in girder flanges) need to have welds completed before assembling the pieces into the final structure. In general, welds should be sequenced to minimize constraint during the fabrication process. Before fabrication, the fabricator should develop (and the Engineer should review) a weld sequencing plan. For complex and critical components, specify a mockup (see Chapter 8 for details).

6.3.2 Support and Restraint. An adequate amount of support and restraint should be provided during fabrication to ensure the member remains straight. The member should not be overly restrained, as previously discussed, since this can introduce large residual stresses. Rather than introducing very large localized residual stresses, this restraint should spread the residual stresses globally across the member.

6.3.3 Access for Nondestructive Testing. Access for NDT should be taken into consideration during development of design and shop drawings.

6.3.4 Bolting Access. If a bolted connection will be used, sufficient access should be provided to allow for installation and tightening of the bolts.

Chapter 7
Designing for Operations and Maintenance

7.1 Operability. Design for a high degree of operational reliability. Consider all operating scenarios for the HSS over the life of the structure and ensure each is addressed in the design.

7.1.1 Designing for the Serviceability Limit State. This includes limiting sidesway and binding, limiting deflections, minimizing fatigue cracking potential, and minimizing corrosion potential.

7.1.2 Minimize Loads that Impair Operation. These loads include ice and debris. Incorporate deicing systems, such as heating systems and air bubbler systems, where atmospheric ice is expected during operation. Minimize or accommodate debris by incorporating debris barriers or allowing passage of debris while minimizing damage potential from debris passage. Incorporate debris removal access when debris is expected to accumulate on HSS members.

7.1.3 Coordination. Coordinate with Mechanical and Electrical engineers to ensure structural, mechanical, and electrical systems are compatible. Coordinate with end users to ensure their operational needs are met. Coordinate the HSS Operation and Maintenance Manual with all concerned individuals.

7.1.4 Interchangeability. Consider interchangeable parts within a project and within a waterway system.

7.1.5 Long-Term Monitoring. Consider addition of sensors for structural health monitoring and to identify problems.

7.1.6 Lifting Connections. Design lifting connections for safe and quick connect and disconnect operability.

7.2 Maintenance. Avoid structural systems that complicate maintenance. Inaccessible cavities and corners should be avoided. Provisions must be made for lubrication of all moving parts as required by mechanical design. Consider maintenance contributions to life-cycle costs. For components with short service lives, consider connections that can be readily removed for efficient replacement and maintenance.

7.2.1 Corrosion Control. Structural steel must have long life coating systems or cathodic protection. Member thickness may be increased a minimum of 1/8 in. beyond what is required by design for those members that will be difficult to access for maintenance. The minimum steel thickness of any member must be 3/8 in.

7.2.1.1. Corrosion Mechanisms. Design for corrosion control must account for the mechanisms that cause corrosion. Mechanisms include general, localized or pitting, crevice, mechanical, and galvanic corrosion.

7.2.1.2. Material Selection. Materials must be selected for corrosion control based on exposure conditions and on the provided corrosion protection system while satisfying all design limit states. Considerations must be given for compatibility with connecting materials and cost. Weathering steel must not be used for submerged conditions.

7.2.1.3. Coatings. HSS must be protected from corrosion by applying a protective coating system or using cathodic protection. Members must be proportioned to accommodate access for future coatings, coating repairs, and maintenance of cathodic protection systems.

7.2.1.4. Cathodic Protection. For HSS, or portions of HSS that are usually submerged, cathodic protection should be considered to supplement paint coatings.

7.2.1.5. Galvanic Corrosion Considerations. Contact between dissimilar materials should be prevented by using coatings, isolation devices, or other methods that will ensure contact is avoided.

7.2.1.6. Detailing. Design details to ensure satisfactory operation and to minimize maintenance include:

7.2.1.6.1 Drain Hole Locations. Place drain holes in any horizontal, near horizontal, or surface where water can be trapped. The holes must be placed in locations where the structural integrity is not compromised. The cut edges of holes must meet the surface finish requirements of AWS.

7.2.1.6.2 Seal Configurations. Configure seals to minimize gate vibrations.

7.2.1.6.3 Seal Welds. Place seal welds so that water cannot be trapped between the connected plates. Consideration must be given to minimum weld sizes per AISC and AWS. Caulking may be used in lieu of seal welding where seal welds are undesirable.

7.2.1.6.4 Crevices. Grind slag, weld splatter, or any other deposits off the steel; these are areas that form crevices that can trap water.

7.2.1.6.5 Edges and Corners. Break or grind sharp corners or edges to a minimum radius to allow paint or coating to properly cover the surface.

7.2.1.6.6 Enclosed Spaces. Avoid designs with enclosed spaces. If such spaces cannot be avoided, make them large enough for maintenance work and painting or provide sustainable cathodic protection.

7.2.1.6.7 Corrosion Resistant Materials. Consider using corrosion-resistant metal for areas that will be inaccessible for replacement.

7.2.1.6.8 Ponding. Avoid crevices and areas where water may pond.

7.2.2 Access for Maintenance and Inspection. Members will be located and proportioned to provide workers and equipment sufficient access to perform maintenance and repairs. Access will also be provided for routine inspections.

7.2.2.1. Dewatering. Provisions must be made for maintenance and inspection dewatering.

7.2.2.2. Lifting Attachments. Lifting attachments include dogging devices, lifting lugs, lifting eyes, and other devices to accommodate fabrication, shipping, moving, maintenance, and repairs. These attachments must be designed by the Engineer during the design phase. If the need for additional attachments is identified during construction or operation, the Engineer must design them or review and approve the design if designed by someone else. See Chapter 15 for guidance on the design of lifting beams.

Chapter 8
Fabrication

8.1 Fabrication Responsibilities. The fabrication of HSS is unique in that the structures are frequently not fabricated similar to buildings or bridges. While a fabricator may be familiar with applicable building or bridge code requirements, welding procedures, and quality assurance (QA) techniques, the requirements necessary for the fabrication of HSS are more specialized. As a result, the responsibility for fabrication oversight is more involved and detailed for HSS than many other USACE projects.

8.1.1 Designer Responsibilities. The designer must be familiar with the fabrication responsibilities associated with each structure as described in ER 1110-2-8157, Responsibility for Hydraulic Steel Structures.

8.1.2 Engineer Responsibilities. The Engineer is responsible for the following tasks during the fabrication phase of HSS: shop drawing and submittal review and approval; construction site visits; review of Value Engineering (VE) proposals and contract modifications; consultation on plans and specifications interpretation; final inspection of completed structures; and use of guide specifications.

8.2 Use of Guide Specifications. New HSS should be fabricated according to UFGS 055920, Fabrication of Hydraulic Steel Structures. The guide specification should be amended as appropriate. The guide specification incorporates several key requirements dictated in ER 1110-2-8157. These requirements include: use of a fracture control plan (FCP); designation of fracture critical members; designation of fracture critical material (steel with required toughness); mandatory minimum nondestructive testing of fracture critical welds; and submittals required under AWS D1.5.

8.2.1 Responsibility of the Engineer. As described in ER 1110-2-8157 and AWS D1.5, the Engineer is responsible for reviewing and approving all submittals related to fabrication of the HSS. These submittals include: shop drawings, fracture control plan, procedure qualification records, weld procedure specifications, weld tracking log, welding repairs, delivery and shipping plan, and control dimensions. Use UFGS 055920 to ensure that the proper submittals are being requested and reviewed by the Engineer. Review all submittals for Engineer approval.

8.2.2 Fracture and Fatigue Requirements. Fracture and fatigue requirements within UFGS 055920 generally follow requirements of AWS D1.5, Bridge Welding Code. This includes fabrication practices that minimize fracture and fatigue including proper welding procedures, removal of temporary and tack welds, prohibited welded joint configurations, and removal of weld backing bars.

8.2.3 Fracture Critical Members. FCMs must meet the welding and testing requirements of AWS D1.5. All welds to FCMs are considered FCW according to AWS D1.5. In addition, all FCMs identified on HSS must be fabricated from material that possesses toughness to minimize the initiation of fracture. The relevant components of AWS D1.5 have been incorporated into UFGS 055920 to ensure that FCMs are properly fabricated. AWS D1.1 does not address the fabrication of FCMs or the requirements of an FCP and thus its use is discouraged.

8.2.4 FCP Responsibilities. According to ER 1110-2-8157 and AWS D1.5, the guide specification requires the contractor to develop and submit an FCP. An FCP addresses the fabricators quality control requirements associated with FCMs and how the fabricator and steel erector will handle, cut, weld, bolt, assemble, and finish fracture critical components of the HSS according to AWS D1.5 Clause 12, "Fracture Critical Requirements."

8.2.5 FCP Content. The FCP addresses consumable requirements including storage and handling, diffusible hydrogen control, control of electrode exposure, and shielding requirements. The FCP also addresses: welding procedure specification requirements, fabricator certification requirements, thermal cutting and prepping requirements, repair of base metal, straightening, repair welding, record keeping, and general handling and storage requirements for fracture critical material, including the use of protective slings to minimize base metal damage that can lead to stress concentrations and potential fracture.

8.2.6 Special Welding Provisions. Special welding provisions include the prohibition of tack and temporary welds and the incorporation of seal welds. HSS are unique structures that are primarily submerged in water. To prevent corrosion "seal welds" are often required. "Seal welds" is a general term used for any weld with a primary purpose of providing a specific degree of tightness against leakage versus transferring structural loads. As discussed above, AWS D1.5 prohibits the use of seal welds on many bridge connections to minimize the initiation of fracture and fatigue.

8.2.7 Seal Welds. UFGS 055920 specifically addresses seal weld requirements. Ensure all welds are made with the same level of quality and receive the same level of inspection and testing.

8.2.8 Bolting Requirements Including Specifying and Testing Slip-Critical Connections to Meet AASHTO/AISC Fatigue Category B. According to AWS D1.5 and UFGS 055920, each FCM on an HSS must be identified on the plans. All members not identified as FCMs are assumed to be non-FCMs. Non-FCMs are subject to a different/lesser level of quality control than FCMs. It is important for those performing QA on HSS to understand the requirements associated with FCMs.

8.3 <u>Fabrication Shop Certification</u>. According to AWS D1.5 Clause 12, all fabricators performing fracture critical work must be certified under the AISC quality certification program for Major Steel Bridges. According to AISC certification requirements, fabricators must be certified either: IBR, Certified Bridge Fabricator – Intermediate; ABR, Certified Bridge Fabricator – Advanced; or HYD, Fabricators of Hydraulic Steel Structures. Utilization of IBR, ABR, or HYD certification ensures that the fabricator is familiar with the requirements of AWS D1.5 Clause 12, including performing fabrication according to an FCP.

8.3.1 HYD Certification. The HYD certification was created by AISC and USACE to ensure that fabricators capable of performing the fabrication of HSS, but who do not intend to fabricate bridges between each AISC audit cycle, maintain the capabilities of performing work according to AWS D1.5 Clause 12 and the associated FCP. Note that the FCP and fracture critical requirements specified in ER 1110-2-8157 are not included in these alternate certification requirements. Use of alternate certification requirements will require the Engineer to address all FCP requirements in the specifications.

8.3.2 Other Considerations. In addition to certification, the fabrication shop must be capable of handling and fabricating HSS components. Size limitations, crane capacity, and shipping access will dictate the size of HSS components a shop can fabricate. A restriction in component size may result in additional shop or field splices. The size and capabilities of the fabrication shop and steel erector, along with the weight and size restrictions along the shipping route to the project site, must be considered when designing HSS components.

8.4 <u>Welding</u>.

8.4.1 Welding Codes. AWS D1.5 should be used for fabrication of all new HSS.

8.4.2 Acceptance Criteria. AWS D1.5 provides visual weld inspection acceptance criteria and ultrasonic inspection acceptance criteria for both tension and compression welds. Contract drawings must identify all FCM as well as all tension members. The labeling of FCM and tension members is critical to define which ultrasonic inspection criteria will be used by the fabricator. The Engineer should also consider adding testing notes to the tail of critical welds to avoid confusion during fabrication.

8.4.3 Welding Procedure Specifications. Welding Procedure Specifications (WPS) must be reviewed and approved by the Engineer for all welds on an HSS regardless of prequalification. AWS D1.5 only permits pre-qualified welds for ASTM A709 steel joined with approved low hydrogen shielded metal arc welding (SMAW) electrodes less than 90 ksi.

8.4.3.1. Prequalification Record. To maintain records and compliance with ER 1110-2-8157, UFGS 055920 requires that all welds performed on HSS utilizing weld processes other than SMAW be qualified by testing as per AWS D1.5. To qualify a WPS by testing, a Prequalification Record (PQR) is generated based on the test results (NDT and destructive testing) of the proposed WPS.

8.4.3.2. Witnessing the PQR. According to AWS D1.5, the Engineer or designated representative must be present to witness the welding and destructive testing of the welding procedure being qualified. The WPS generated from the PQR must be reviewed and approved by the Engineer. Use of materials other than ASTM A709 require Engineer approval and a PQR and WPS to be generated for each combination of base metals (ASTM A709 welded to ASTM A572, ASTM A709 welded to A36, etc.).

8.4.3.3. Non-Standard Materials. There are often components of HSS that are manufactured from material other than ASTM A709 (e.g., steel rub blocks or bearing blocks thicker than 4 in.; the maximum thickness of ASTM A709). In these instances, it is necessary for the Engineer to define the materials that may be welded per the AWS D1.5 code and ensure the proper PQR and WPS for joining these materials is requested and submitted.

8.4.4 WPS for Repair Welding. In addition to WPS generated and submitted for new fabrication, a WPS is required for all repair welding performed on an HSS during fabrication that is due to a nonconformance. The WPS for repair welding, according to AWS D1.5 Clause 12, will require separate essential variables including increased preheat and post-weld heat treatment or cooling times. Repair welding should be inspected by the Engineer. According to AWS D1.5, all weld repairs must be approved by the Engineer.

8.4.5 Fracture Control Plan. Clause 12 of AWS D1.5 provides the provisions applicable to the fabrication of FCM.

8.4.5.1. Fabricator Requirements. The fabricator is responsible for generating the specific FCP according to Clause 12 as it pertains to the means and methods the fabricator intends to utilize in the fabrication of an HSS.

8.4.5.2. Engineer Requirements. An FCP is required for all HSS according to ER 1110-2-8157 and AWS D1.5. The Engineer must specify in the contract specifications any specific requirements that must be addressed in the FCP. Examples of specific requirements may include the prohibition of specified splices, field welds, or for individual components to be shop assembled before field installation. The Engineer must review the submitted FCP to ensure that the contractor has addressed all fracture critical requirements including base metal, consumable, preheat, handling, and testing requirements.

8.5 Installation of Bolted Structural Connections. Structural bolted connections must be installed per the requirements stipulated in RCSC S348, “Specification for Structural Joints Using High-Strength Bolts.” The Engineer should ensure that adequate clearance is available for installation of all bolted connections as discussed in Chapter 5. The Engineer should verify the dimensions of hydraulic torque wrenches and adequate clearance on both the nut and bolt head sides of the connection.

8.5.1 Installation Sequence. The installation and tensioning sequence for complex bolted connections should be identified on the drawings. Connections are typically tensioned from the most rigid to the least rigid portion of the connection according to RCSC requirements. Drawings and specifications must ensure that all connections are snug-tightened. Connections that are fully pretensioned must be properly labeled. ER 1110-2-8157 requires all structural connections on an HSS be fully pretensioned at a minimum. This ensures that all plies of the connection are in contact before tensioning.

8.5.2 Testing of Bolted Structural Connections. The testing and verification of bolted structural connections must be defined in the specifications. Testing and verification requirements are highlighted in the notes contained in UFGS 055920. This will require the Engineer to edit the specifications to ensure that the testing requirements are moved from the notes section of the specifications to the main text of the specifications.

8.5.3 Verification. The RCSC does not require re-tensioning or the verification of installed bolted assemblies provided that pre-installation verification procedures were followed. As the size of typical HSS connections (plate thickness as well as number of bolts) differs from common building connections, additional tensioning and installation verification of HSS connections is recommended.

8.6 Fabrication Shop Quality Assurance. Quality assurance for fabrication of HSS is a team approach. As outlined in ER 1110-2-8157, the Engineer has a considerable role in performing QA for HSS. As described in section 7.1, the designer of each HSS should be familiar with the fabrication responsibilities associated with each structure according to ER 1110-2-8157. In executing the requirements listed above, it is critical for the Engineer to establish roles and responsibilities closely with construction staff. It is also essential for the Engineer to become part of the QA team for the fabrication and erection of an HSS.

8.6.1 Quality Assurance Plan. The Engineer should develop a QAP to address how QA will be performed. The QAP should include qualifications of personnel (types and amounts) in terms of percentages of details, of testing that will be accomplished, and actions to be taken where fabricator is in noncompliance.

8.6.2 Responsibility for Shop Inspection. According to ER 1110-2-8157 and UFGS 055920, the Engineer will perform periodic site visits to the fabrication facility. It is the responsibility of the Engineer to coordinate these visits with the appropriate USACE construction personnel. At a minimum, the Engineer should be present at "Witness Points" as established in the specifications.

8.6.2.1. Witness Point. Witness points are critical points in the fabrication and assembly of HSS where assembly details, dimensional tolerances, and fabrication details are verified. Witness points are established as hold points to ensure fabrication does not proceed until the Engineer and the USACE Construction Personnel are satisfied that the fabrication is proceeding according to the plans and specifications. At a minimum, witness points must be established for the Initial QA Inspection, Intermediate QA Inspection, and Final Inspection.

8.6.2.2. Engineer Responsibilities. UFGS 055920 provides additional guidance on establishing witness points. The Engineer should address witness points in both the specifications and in the engineering considerations document prepared for construction. Depending on the complexity of HSS and the experience of the construction QA staff, additional fabrication inspection should be anticipated.

8.6.2.3. Bolting Inspector Certification. Consider including certification requirements for bolting inspection personnel in project specifications. See AASHTO-National Steel Bridge Alliance (NSBA) Steel Bridge Collaboration G4.2, Recommendations for the Qualification of Structural Bolting Inspectors for guidelines.

8.6.3 Transport. The Engineer should review the contractor's transport plan for all components fabricated at a shop and delivered to the work site. The Engineer should review the pick plan, the shipping plan, and should inspect the delivered product after transport. The contractor should address transport and handling of the structure in the FCP as referenced above.

8.6.4 Field Fabrication Inspection Requirements. The Engineer should anticipate participating in field fabrication inspections. Field fabrication is typically required for large HSS, particularly both welded and bolted splices of primary members due to shipping restrictions. The Engineer should be present to participate in similar QA inspections of assembled components including splices, installation of hoisting components (wire ropes, chains, etc.), and sacrificial anode installation. Quality assurance must be maintained in the field to ensure that the FCP is adhered to.

Chapter 9
Miter Gates

9.1 Introduction. This chapter provides guidance for the structural design of miter gates used at navigation projects.[1] Miter gates are the most frequently chosen type of gate for navigation locks throughout the world. The structural system of miter gates has a long history dating back to the late 15th century with engineers Leonardo Da Vinci and Bertola da Novate utilizing them on the Navigilio Grande canal in Italy.

9.2 Miter Gate Configuration. A large percentage of the locks in the United States are equipped with double-leaf miter gates that are used for moderate and high-lift locks. These gates are fairly simple in construction, operation (opening and closing) can be accomplished more rapidly than other types of gates, and maintenance costs are generally low. The primary disadvantages of this gate type are that it cannot withstand reverse head, and it cannot be closed during an emergency situation with an appreciable flow through the chamber.

9.2.1 Leaves. The two leaves of a miter gate form a shallow three-hinged arch with the apex facing upstream. When the gate is in the closed position and loaded by hydrostatic head, each leaf is supported by the lock wall on one end and by the other leaf at the center of the lock. Usually, the angle of each leaf in the closed position is 1:3 as would be described in a plan view. This arch shape is very efficient for spanning larger distances between lock walls.

9.2.2 Vertically and Horizontally Framed Gates. Miter gates are framed either horizontally or vertically. Plate 9.3 and Plate 9.4 show miter gate geometry for vertically and horizontally framed gates.

9.2.2.1. Horizontally Framed Gates. The skin plate of a horizontally framed gate is supported by horizontal members that either may be straight girders acting as beam-columns or curved girders acting as circular arches. Each horizontal member is supported by the vertical quoin post at the end near the lock wall and a miter post at the other end. All hydrostatic and other horizontal loads (ice, impact) are transmitted through the girders to the quoin blocks and into the walls.

9.2.2.2. Vertically Framed Gates. A vertically framed gate resists the water pressure by a series of vertical girders. The vertical girders span from the lower horizontal girder near the sill at the bottom to a horizontal girder at the top. The top girder spans between walls similar to a horizontally framed gate. Therefore, arching action only takes place at the top girder level and most of the hydrostatic loading is transferred directly to the sill. Vertically framed gates are used primarily for wide, shallow gates, usually when the height-to-width ratio of a leaf is less than about 1H:2W.

[1] For additional in-depth information on miter gates, see PIANC Report No. 154 "Mitre Gate Design and Operation" and "Lock Gates and Other Closures in Hydraulic Projects" by Daniel & Paulus.

9.2.3 Considerations. Horizontally framed gates are typically selected for most gates where the height-to-width ratio of the leaf exceeds 1H:2W or where the gate height exceeds approximately 36 ft. When gate dimensions meet the gate geometry aforementioned, a decision has to been made whether to select a vertically or horizontally framed gate. Considerations include how the boundary conditions of the gates are supported and how the gate loads are transferred into the lock structure.

9.2.3.1. Load Transfer. In the closed (mitered) position, horizontally framed gates transfer loads horizontally through the quoin blocks. Vertically framed gates transfer load to the gate sill and to the top girder with just one quoin block at the top girder. These two types of load transfer are shown in Figure 9.1. Due to common problems with continuous blocks typically used on horizontally framed gates not reliably transferring load due to wear and misalignment, the vertically framed miter gate often provides a more reliable load path transfer and avoids increased incidental loading on the pintle.

9.2.3.2. Redundancy. An advantage of the horizontally framed gate is that it has redundant horizontal framing members. A disadvantage of the vertically framed gate is that it does not have redundant horizontal members and therefore the top girder is failure critical.

9.2.3.3. Installation. With the simple support conditions, vertically framed miter gates can be properly installed in-the-wet while horizontally framed gates cannot since quoin blocks should be reset with each installation, which requires dewatering. The vertically framed gate is easier support adjustment since only one short quoin block at the top of the gate has to be adjusted and it is usually above the waterline. For these reasons, when gate and lock geometry allow for vertically framed gates, this gate type is usually selected.

9.2.3.4. Replacement Miter Gates. Because many miter gates designed now are replacement gates for existing locks, care must be taken to ensure the new gate can accommodate the existing lock’s geometrical constraints or that the lock can be modified to accept the replacement gate. There have been several instances where original horizontally framed gates have been replaced with vertically framed gates. In such cases, it is important to check the gate sill for stability against increased loads. It may require retrofitting. Top girder width of a vertically framed gate is often larger than a horizontally framed gate so interference with the gate recess must also be considered.

9.2.3.5. Operational Considerations. In the open position, miter gate leaves fit into recesses in the wall. The bottom of the recess should extend below the gate bottom to preclude operating difficulties from silt and debris collection. Enlarged recesses are sometimes used to facilitate the removal of accumulated ice. An air bubbler system on the lock and gate is recommended to help clear ice and debris from gate recesses. Left leaf and right leaf nomenclature is determined by viewing the leaves from the upstream location.

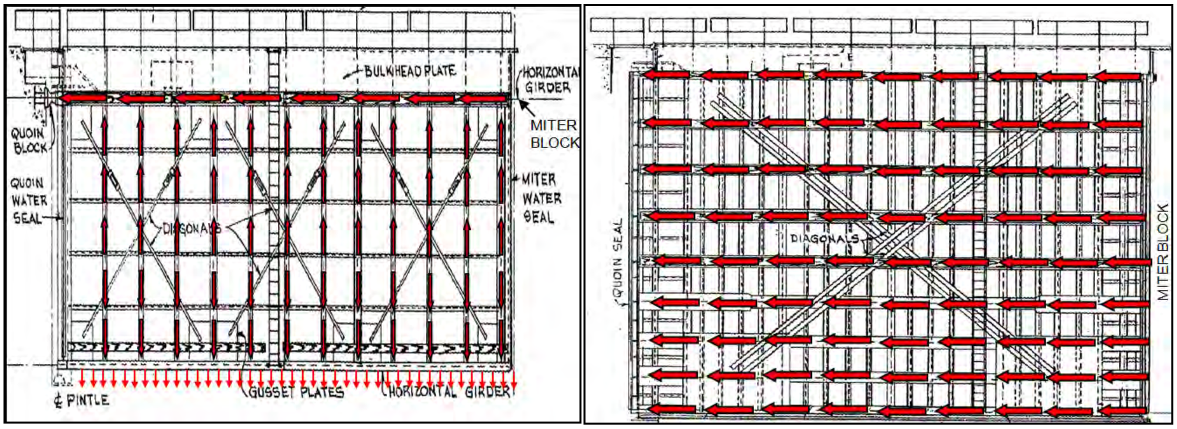


Figure 9.1. Load Transfer in Vertically Frame (Left) and Horizontally Framed (Right) (PIANC Working Group No. 154, 2017)

9.3 Loads and Load Combinations.

9.3.1 Design Requirements. Design requirements are provided in Chapters 3 and 4. This section provides information specific to design of miter gates for strength, fatigue, and serviceability.

9.3.2 Loads. Loads that are applicable to miter gate design include dead load, gravity loads, hydrostatic and hydrodynamic loads, operating loads, barge and other impact loads, ice loads, and earthquake loads. Wave loads are possible, but miter gates are normally installed in locations on water bodies where wave load is not significant. If significant wave loads may be present, see wave load guidance in Chapter 4.

9.3.2.1. D, Dead Load. Dead load is defined in Chapter 4. A load factor of 1.2 is used when dead load adds to load effects and 0.9 when it reduces load effects.

9.3.2.2. G, Gravity Loads. Gravity loads include mud weight (M) and ice weight (C) determined based on site-specific conditions. This load is applied with a load factor of 1.6 when it adds to load effects. When it reduces load effects it is not applied.

9.3.2.3. Hs, Hydrostatic Loads. Hydrostatic loads consist of hydrostatic pressure on the gate considering both upper and lower pools. Hydrostatic loads are described in Chapter 4.

9.3.2.3.1 For strength design of miter gates, Hs consists of hydrostatic pressure on the gate considering both upper and lower pools. For Hs as a principal load, Hspr, Hs is the maximum hydrostatic loading from differential head.

9.3.2.3.2 For companion hydrostatic loads, Hsc is the normal operating condition with a return period of 10 years as defined in paragraph 3.3.3.3.

9.3.2.3.3 For fatigue design of miter gates, the fatigue stress range as described in Chapter 5 will be computed considering load variation due to Hs.

9.3.2.4. Hd, Hydrodynamic Loads.

9.3.2.4.1 For strength design of miter gates, Hd is defined as a temporal head of 1.25 ft to account for prop-wash, lock overfill, and seiches (setup and surges) unless there is testing or evidence to support the need for higher design values. Hd will be applied as a companion load to produce maximum load effects on the gate and anchorages, as shown in the load combinations.

9.3.2.4.2 For fatigue design of miter gates, Hd is defined as the inertial load produced while the gate is moving through the water and will be 30 psf applied to the submerged portion of the gate, unless there is testing or evidence to support the need for higher design values.

9.3.2.5. Hw, Wave Loads. Wave loads are typically not significant as principal loads for miter gates but may be significant as companion loads, Hw_c. As companion loads, they are usual loads computed according to Chapter 3. See Chapter 4 for determining wave loads.

9.3.2.6. Q, Operating Loads.

9.3.2.6.1 For strength design with the operating load applied as a principal load, Q_{pr}, the design operating load is the maximum load that can be exerted by the operating machinery on the gate assuming that it is jammed or stuck (obtained from the mechanical engineer who designed the machinery). See Chapter 4 and EM 1110-2-2610 for further discussion on operational loads. For other load cases the operating equipment forces are included in the gate reactions and not applied as separate loads.

9.3.2.6.2 For fatigue design of anchorages, Q is the reaction at the anchorage from operating forces created by normal gate operation. It includes temporal head, inertial forces, drag and friction, and other forces that combine in the force required to move the gate. See Levine et al. (2019) for discussion of fatigue calculations.

9.3.2.7. BI, Barge Impact.

9.3.2.7.1 Barge impact load should be modeled as a point load as shown in Figure 9.2. The load is applied in the downstream direction to girders above pool level at (a) the miter point (symmetric loading), and (b) anywhere in the girder span at which a single barge may impact (unsymmetrical loading). This location is anywhere in the span of the gate but at least 35 ft from either lock wall. The standard barge width is 35 ft. Both impact locations will be investigated to determine the maximum structural effect.

9.3.2.7.2 The minimum recommended magnitude of design load is 250 kips for unsymmetrical loading and 400 kips for symmetric loading. Gates at locations in which failure may result in loss of life from uncontrolled release of water or high economic or environmental consequences may require higher design loads. See section 4.2.6.3 for additional guidance on selection of design barge impact loads.

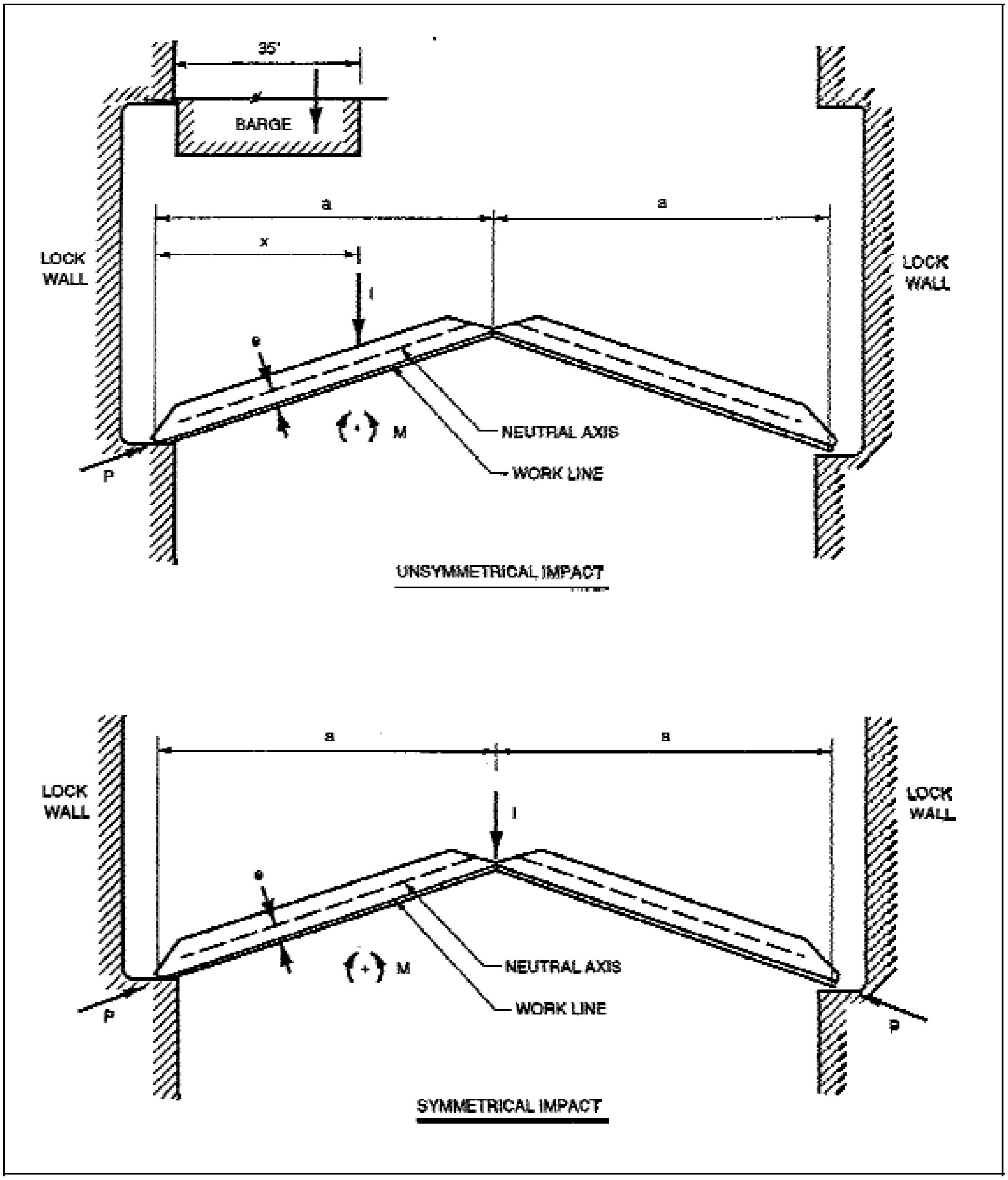


Figure 9.2. Point Load Impact for Miter Gate Girders

9.3.2.8. IX, Thermally Expanding Ice load. Thermally expanding ice is an extreme load specified in Chapter 3. Barge impact load on miter gates usually governs over the thermally expanding ice load.

9.3.2.9. L, Live Loads. Miter gates usually have access ways on top of the gate to cross the lock when the gates are closed. Live loads are defined in Chapter 4.

9.3.2.1. T, Self-Straining (Prying). Forces and reactions in miter gates can be affected by support conditions. Testing of existing gate anchorages has shown that a prying force can develop when the quoin blocks are out of adjustment, the quoin blocks clearance tolerance is too tight, and the embedded and gate quoin blocks come into contact before a gate is fully mitered. This creates an additional reaction force in the anchorages at the top of the gate. See section 9.4.1.14 for more description and illustration.

9.3.2.1.1 Prescriptive guidance for determining design prying forces did not exist at the time this manual was completed. The prying load has been measured on the anchorages at a few sites. At Mel Price Locks on the Mississippi River, the prying force was measured to be 340 kips when the quoin blocks were out of adjustment. A prying load of 100 kips was measured at the Poe Lock in Sault Saint Marie, MI. At Mississippi River Lock 24, prying increased the maximum tensile loads in the anchorage by 150%. This is not a complete list of prying measurements and the USACE INDC should be consulted for more information. For replacement projects, obtaining strain gage data from existing anchorages may help determine to design loads.

9.3.2.1.2 For fatigue calculations, it is unlikely that the blocks will be out of alignment throughout the service life of a project. Some past designs have assumed that prying loads were present over 15% of the service life.

9.3.2.2. EQ, Earthquake Design Loads. See Chapter 4 for earthquake loading. The miter gate must be evaluated in both the open and closed positions for earthquake loading. Earthquake loading is considered in both the upstream and downstream directions.

9.3.3 Load Combinations. General loads and loading combinations for gates are described in Chapter 4. Miter gates will be designed for the strength and fatigue limit states for the following load combinations. Principal load factors, γ_{pr}, and companion loads and load factors are defined in Chapter 4. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. The serviceability limit state is addressed in Chapter 4. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

9.3.3.1. Load Combination 1: Strength Limit State. Gate Closed.

9.3.3.1.1 Load Combination 1a. Upper gate subjected to maximum hydrostatic loading, Hs_{pr}, with applicable companion wave loads, Hw_c. The hydrostatic principal load factor is selected according to paragraph 4.3.3 based on the return period of the maximum hydrostatic loading. For locks that do not include an upstream dewatering system, the gate will be designed for the dewatered condition in the unusual load category.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + \gamma_{pr}\, Hs_{pr} + 1.0\, Hw_c \qquad \text{(Equation 9.1)}$$

9.3.3.1.2 Load Combination 1b. Gate subjected to maximum operational hydrostatic loading, Hs_{pr}, with companion hydrodynamic (temporal head), Hd_c, as defined in paragraph 9.3.2:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + \gamma_{pr}\, Hs_{pr} + 1.0\, Hd_c$ (Equation 9.2)

9.3.3.2. Load Combination 2: Strength Limit State. Gate Open.

9.3.3.2.1 Load Combination 2a. Dead load only.

$1.4\, D$ (Equation 9.3)

9.3.3.2.2 Load Combination 2b. Dead load plus mud and ice plus self-straining (prying).

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + T\, (1.2 \text{ or } 0)$ (Equation 9.4)

9.3.3.2.3 Load Combination 2c. Gate Operating on an Obstruction. Either gate subjected to dead, gravity, and maximum machinery load, Q_{pr}:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + \gamma_{pr}\, Q_{pr}$ (Equation 9.5)

The maximum machinery load should be considered an extreme load (Q_{pr}) with unknown return period ($\gamma_{pr} = 1.3$).

9.3.3.3. Load Combination 3: Strength Limit State. Gate Closed. Barge Impact. Loads consist of barge impact loads, BI_{pr}, and companion hydrostatic load, Hs_c:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.3\, BI_{pr} + 1.0\, Hs_c$ (Equation 9.6)

9.3.3.4. Load Combination 4: Strength Limit State. Gate Closed. Loads consist of extreme thermally expanding ice force, IX_X, (if applicable), and companion hydrostatic load, Hs_c:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + \gamma_{pr}\, IX_X + 1.0\, Hs_c$ (Equation 9.7)

Unless site information is available, the extreme ice load has unknown return period and $\gamma_{pr} = 1.3$.

9.3.3.5. Load Combination 5: Strength Limit State. Live Load. Gate Closed. Loads consist of live load, L, as the principal load, plus dead, gravity, and companion hydrostatic, Hs_c:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.6\, L + 1.0\, Hs_c$ (Equation 9.8)

9.3.3.6. Load Combination 6: Strength Limit State. Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load and gravity loads.

9.3.3.6.1 For standard and site-specific OBE ground motion analysis:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.5\, EQ + 1.0\, Hs_c$ (Equation 9.9)

9.3.3.6.2 For standard MDE ground motion analysis:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.25\, EQ + 1.0\, Hs_c$ (Equation 9.10)

9.3.3.6.3 For site specific MDE and MCE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ EQ + 1.0\ Hs_c \quad \text{(Equation 9.11)}$$

9.3.3.7. Load Combination 7: Fatigue Limit State. Miter gates will be designed for fatigue for the stress range described in Chapter 5. Design must satisfy requirements for either Infinite Life or Finite Life. See section 5.1 for more information on load factors for fatigue.

9.3.3.7.1 Load Combination 7a. Fatigue Limit State I. Infinite Life. Fatigue created by hydrostatic forces on a closed gate and by hydrodynamic forces from opening and closing the gate.

$$2.0\ Hs \text{ or } 2.0\ Hd \quad \text{(Equation 9.12)}$$

9.3.3.7.2 Load Combination 7b. Fatigue Limit State II. Finite Life. Fatigue created by hydrostatic forces on a closed gate and by hydrodynamic forces from opening and closing the gate.

$$1.0\ Hs \text{ or } 1.0\ Hd \quad \text{(Equation 9.13)}$$

9.3.3.7.3 Load Combination 7c. Fatigue Limit State I. Infinite Life. Fatigue in anchorage components.

$$2.0\ D + 2.0\ G + 2.0\ Q + 2.0\ T \quad \text{(Equation 9.14)}$$

9.3.3.7.4 Load Combination 7d. Fatigue Limit State II. Finite Life. Fatigue in anchorage components.

$$1.0\ D + 1.0\ G + 1.0\ Q + 1.0\ T \quad \text{(Equation 9.15)}$$

9.3.4 Design for Diagonal Members. See Appendix C.

9.3.5 Support Conditions. A miter gate in the closed position resists horizontal loads as a three-hinged arch if the gate is properly supported at the quoin and miter ends. The load distribution must be evaluated for this condition. Frequently, proper support is not provided because contact is lost between the gate and the quoin along some portion of the height of the gate.

9.3.5.1. For horizontally framed miter gates, this can lead to significant load shedding from horizontal girders with no contact to girders with contact. To account for this unforeseen support condition, the load distribution in the gate must be evaluated assuming a loss of quoin bearing over a certain distance. When determining the appropriate support gap length to use in design, consider site specific information, such as anticipated maintenance frequency and past performance of gates in the given region or site. As a minimum, assume this distance to be equal to the tributary height of the girder (i.e., girder spacing), and the resulting gate load distribution must be evaluated assuming that contact is not achieved at the horizontal girder location being investigated.

9.3.5.2. Actual gap lengths in quoin bearing can be several girder spacings tall. As a minimum, the top and bottom girder check will include tributary loading of at least the full spacing height between that girder and the adjacent girder due to loss of quoin bearing at the adjacent girder. This loss of quoin bearing must be included in the analysis of the miter gate to compute maximum possible forces in the miter gate members including the pintle and top anchorage.

9.3.6 Design Approach. Flow charts to guide the design of miter gate components for horizontally and vertically framed miter gates are provided in Plate 9.1 and Plate 9.2, respectively.

9.4 Miter Gate Types. There are three main framing types for miter gates: horizontally framed, arch type, and vertically framed, which are discussed in general in section 9.2. Most of the typical miter gate features are discussed with the horizontally framed miter gates in paragraph 9.4.1, while features specific to arch type and vertically framed are discussed in paragraphs 9.4.2 and 9.4.3, respectively.

9.4.1 Horizontally Framed Miter Gates.

9.4.1.1. Geometrical Layout. At the quoin support, the local geometry of the contact and rotation axis must be chosen with precision in order to ensure proper quoin block contact. Figure 9.3 shows two methods for determining the axis of rotation of a miter gate leaf (Daniel, 2011). Three-dimensional models are also useful aids in laying out geometry and validating the final geometry, including gate motion.

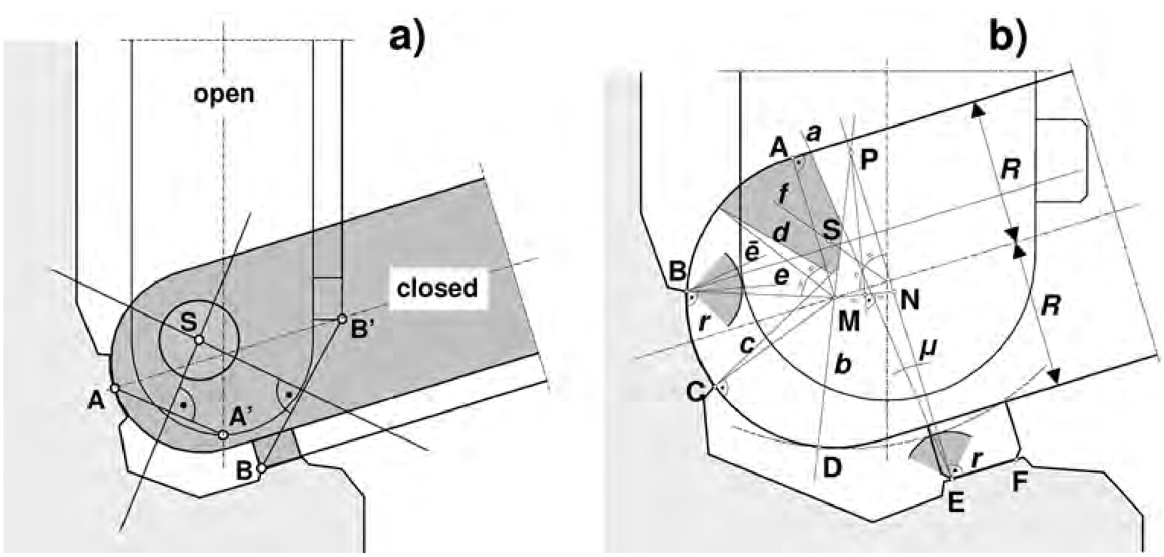


Figure 9.3. Geometrical Relations for Determining the Miter Gate Axis of Rotation (with a showing a simple method that gives one location, and b showing a more complex method that gives an area of possible locations)

9.4.1.1.1 Method (a), (Daniel, 2011):

• Draw the desired heel post geometry in both the open and closed position of the gate.

• Draw the line connecting the middle points of the heel support in both positions (A – A′).

• Draw the line connecting the outer edge of the heel support in both positions (B – B′).

• Draw perpendicular lines in the middle points of both line sections A – A′ and B – B′.

• The intersection of these perpendicular lines gives one sole point: the center of rotation.

9.4.1.1.2 Method (b), (Daniel, 2011):

• Assume the required gate clearance r during motion and in the open position. A practical condition that gives good water discharge and resistance to debris is:

$$r \geq \begin{cases} 0.2 \cdot R \\ 50\ mm \end{cases} \qquad \text{(Equation 9.16)}$$

• Draw lines perpendicular to contact surfaces in all characteristic points of heel post contact (B, C, and E). The gate will open if the center of rotation lies on the left sides.

• Set the R+r distance on the perpendicular line drawn from point E. Connect the received point N with the M center of heel post rounding. Draw a perpendicular line in the middle of MN and determine the intersection P with the EN line.

• Reach from P the farthest point D of the rounding and check if it remains outside r in E when the gate opens. If not, rearrange the position of EF contact.

• To avoid the highest point A of the rounding hitting B during gate opening, divide the angle AMB in half and find the intersection of its middle line with the rounding.

• The above steps enable gate opening but do not prevent rubbing against support surfaces. To prevent that, draw the lines a, c, d, and e at angle μ to the lines drawn before, on their left sides. Calculate an assumed value for μ by the following equation:

$$\tan(\mu) \geq 0.10 \qquad \text{(Equation 9.17)}$$

• This equation was developed for timber gates with lock crown (monolith) lining of natural stone. It can be modified slightly if other materials are used. However, it has proven to work well in many material combinations. Deviation is not recommended.

• The gate's center of rotation S must be inside the polygon drawn by the lines a, b, c, d, and e (the latter outside the polygon in Figure 9.3). Select the most convenient location of S and draw the gate in the open position accordingly.

9.4.1.1.3 Both methods have been presented globally with no consideration to some secondary issues, such as:

• Presence (or absence) and size of system clearances in hinges; see discussion on free, floating, and fixed hinged gate types further in this manual.

• Possible elastic and plastic deformations, if significant.

• Wear of hinge and post lining materials during service.

9.4.1.2. Girder Geometry. The primary structural elements of a single horizontally framed miter gate leaf consist of a series of horizontal girders (Plate 9.3 and Plate 9.5). The following symbols are used in these plates:

R = reaction of the girder at the wall quoin and miter blocks

N = component of R perpendicular to work line of leaf

P1 = component of R parallel to work line of leaf

P2 = the corresponding force on the end of each girder, determined from the horizontal loads acting on the surface extending from the contact point to the skin plate's upstream side

W = total horizontal force on each girder

9.4.1.2.1 Since the three-hinged arch formed by the two leaves is symmetrical about the center line of the lock, the miter end reaction R is perpendicular to this center line. Since the geometry of each leaf is also symmetrical about its own centerline, the angle that R forms with the leaf is identical at both ends.

9.4.1.2.2 The girder is subject to bending and compression. The downstream flange will be in tension near the center of the leaf but in compression near the ends under most load conditions. Greatest economy is usually achieved by keeping the work line as far downstream from the neutral axis as is practicable. (The work line is formed between the bearing contact points at the miter and quoin ends of the leaf.)

9.4.1.2.3 Efficient girder design can be achieved through trial and error. The computer program for the design or investigation of horizontally framed miter gates (CMITER) is a practical tool that may be used to determine girder spacing and sizing. Initial approximate dimensions may be taken as follows:

• Set the slope of the gate to 1:3, θ = arc tan 1/3.

• The corresponding length of the leaf for a gate angle can be 1:3 and is 0.527 times the distance between contact points of the gate at each wall.

• Estimate girder depth for gates of moderate height to be 0.07 times L.

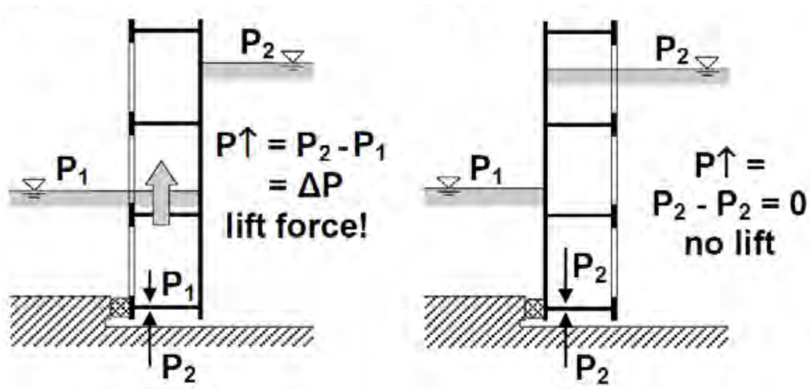
• Set the distance from the downstream girder flange face to the work line at 4 in.

9.4.1.3. Loads and Reactions. Required loads and load combinations are defined in Chapter 4 and section 9.3. Prestressing loads in the gate diagonals must be considered and are generally treated as an external applied load. Reactions to water and impact loads are assumed to be entirely at the pintle and quoin blocks (Plate 9.10). Reactions due to gravity and machinery loads are at the hinge and pintle.

9.4.1.4. Skin Plate. The skin plate is typically located on the upstream face of the girders, but the skin plate can also be on the downstream face of the girders. A disadvantage of the downstream face configuration is that debris is more likely to get caught in between girders from the upstream side. An advantage of a downstream skin plate is that the skin plate protects girders and diagonals from damage due to impact of barges in the recessed position. This may be a consideration at locks where barges must approach at an angle into the lock wall where the gate leaf is recessed.

9.4.1.4.1 Location of the skin plate and bottom seal affects the buoyant forces on the gate (Figure 9.4). The skin plate is designed for the water load. The edges of the skin plate panels are assumed fixed at the center line of intercostals and the edge of girder flanges. When the girder flanges are greater than 12-in. wide, the skin plate is assumed fixed at a point 6 in. from the center line of the web.

9.4.1.4.2 Plate-bending stresses can be obtained from textbooks such as Young, Budynas, and Sadegh (2012). The skin plate is also considered an effective part of the upstream girder flange. See AASHTO LRFD Bridge Design Specifications for skin plates with a higher yield strength than the girder. Skin plate stresses should be checked for the combined effects of plate bending plus stresses due to bending as part of the girder flange. The skin plate deflection will be limited to 0.4 times the plate thickness under hydrostatic service loading (Hs_{pr}).



a) Plate Upstream b) Plate Downstream

Figure 9.4. Skin Plate Location Relation to Uplift Force (Ryszard Daniel, 2019)

9.4.1.5. Intercostals. An intercostal is a plate used to stiffen the skin plate. A portion of the skin plate attached to the intercostal forms a composite section that transfers horizontal loads to the girders. This section (see Figure 9.6) is designed as a vertical fixed-end beam supported at the girder webs. The effective width of skin plate forming the section is determined by assuming the flange to be a noncompact member under uniform compression in all other stiffened elements. A uniform water pressure is used for design of the intercostal with the loading extending between the flanges of the girders as shown in Figure 9.5.

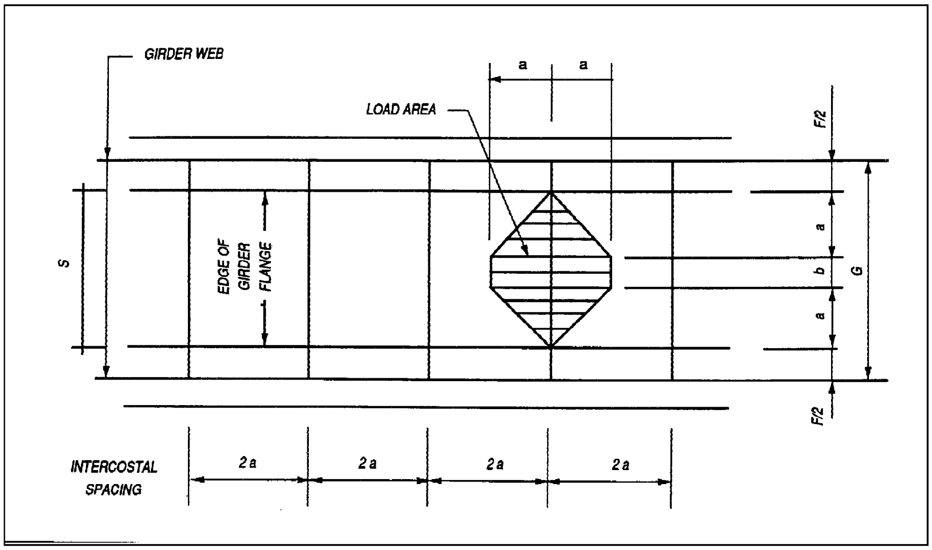


Figure 9.5. Nomenclature and Assumed Load Area for Intercostal Design (with 2a equal to the intercostal spacing, G the spacing between centerlines of the girder webs, and S the spacing between edges of girder flanges)

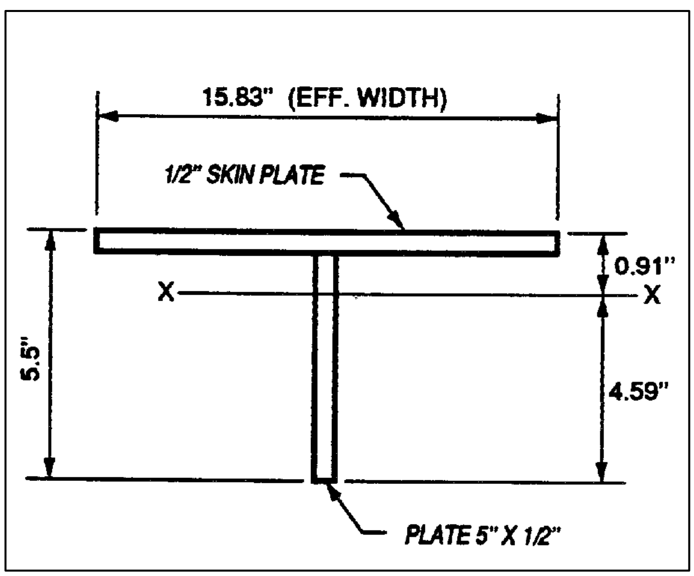


Figure 9.6. Sample Intercostal Section

9.4.1.6. Diaphragms. Diaphragms provide significant structural stiffness by distributing horizontal loads between girders. Diaphragms are vertically oriented full-depth members consisting of flanges and full-depth stiffener plates attached to girder webs.

9.4.1.6.1 The end diaphragms are located near the beginning of the girder taper and are stiffened at mid-depth with a horizontal stiffener (see Figure 9.8). End diaphragms that serve as a damming surface are designed as skin plate panels with the effective panel height being between the horizontal stiffener and the next lower horizontal girder. Design the end diaphragms for the hydrostatic head acting on the tributary area of the effective panel, forces from the thrust diaphragm, and the diagonals' force. Intermediate diaphragms should be spaced and sized as follows:

• To provide support for horizontal girders (weight and lateral buckling);

• For shear forces resulting from differential deflections between adjacent horizontal girders due to variation of hydrostatic and impact loads; and

• To resist operating machinery, jacking support, lifting loads, and diagonal tension-related loads.

9.4.1.6.2 Diaphragms are made as deep as the girder webs and usually have vertical stiffeners for stability. Critical buckling stresses in flat plates in edge compression and shear and can be found in textbooks such as Timoshenko (1936), Bleich (1952), and Priest (1954). The connection of diaphragm flanges to girder flanges should be detailed similar to Figure 9.7 to avoid fatigue cracking.

9.4.1.7. Horizontal Girders. Horizontal girders are subject to bending plus axial compression. They are spaced so that variations in the girder flange sizes and skin plate thicknesses are minimized. The spacing usually varies from a maximum of 6 ft at the top to a minimum of 4 ft at the bottom of the leaf. Girder spacing also influences the size of intercostals and diaphragms. Girder loading is primarily the differential water load on the gate. Barge Impact loads will govern the design of upper girders.

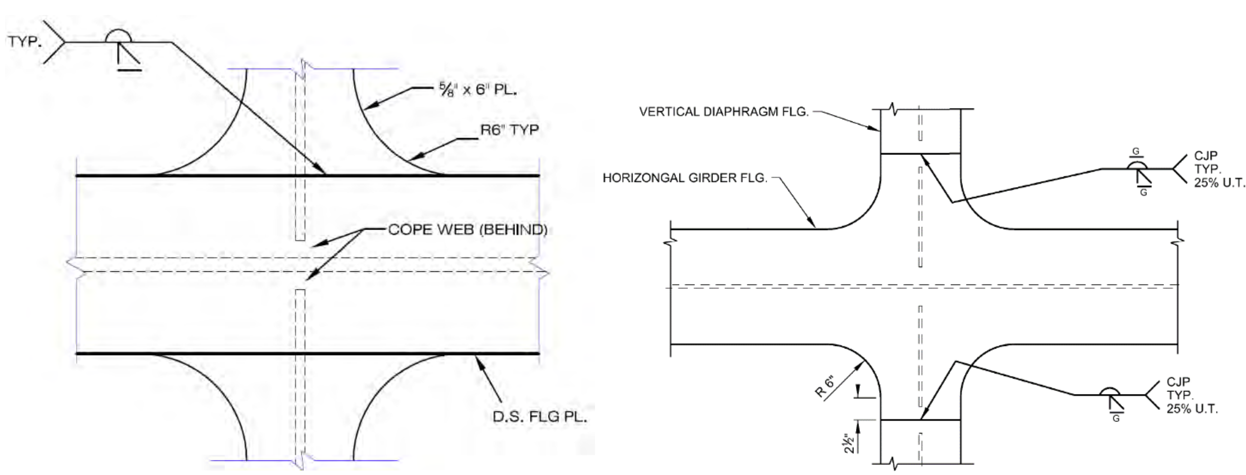


Figure 9.7. Preferred Detailing Options for Diaphragm Flange to Girder Flange Connection

9.4.1.7.1 The ratio of the depth of girder web to the length of leaf varies from 1:8 to 1:15 for most gates, the greater value for gates having higher heads. Deeper girders make the leaf torsionally stiffer, but may require web stiffeners. The web depth-to-thickness ratio should be such that no reduction in flange stress is necessary. Girder webs may require transverse horizontal stiffeners to meet the criteria for web buckling for axial loaded columns. Diaphragm spacing defines the effective column length. Minimum longitudinal horizontal stiffeners are generally used on girder webs even though not required by web buckling (Plate 9.5 and Plate 9.7).

9.4.1.7.2 Buckling of the girder about the major axis is not a concern since the skin plate provides lateral support to each girder. However, lateral stability of the compression flange should be checked where that flange is in compression near the end diaphragms.

9.4.1.7.3 Most girder flanges are symmetric about the web. However, the flanges of the bottom girder may be offset to provide adequate clearance between the flange and sill. Girder flange size may change along the length of the girder due to changing bending stresses. Flange width and thickness transitions should be detailed to provide sufficient fatigue resistance (Plate 9.7). See Chapter 5 for guidance on fatigue resistant details.

9.4.1.7.4 Drain holes must be provided in all girder webs except on a bottom girder where the web forms a damming surface. Draining of the bottom girder is not necessary as it is normally submerged and may need to be avoided if the girder serves as a damming surface. Four-inch diameter drain holes or larger are preferred. Smaller drain holes are more likely to get plugged. If the web of the top girder forms part of the damming surface during high water drain holes can be placed in the upstream flange.

9.4.1.7.5 The load in the diagonal is resisted by members connected to the gusset plate. The horizontal component of this load is distributed among several girders. The design of all girders attached to the gusset plate must include provisions for this additional eccentric axial load. A discussion of the distribution of this load among the girders may be found in Hartman, Gibson, and Nelson (1987).

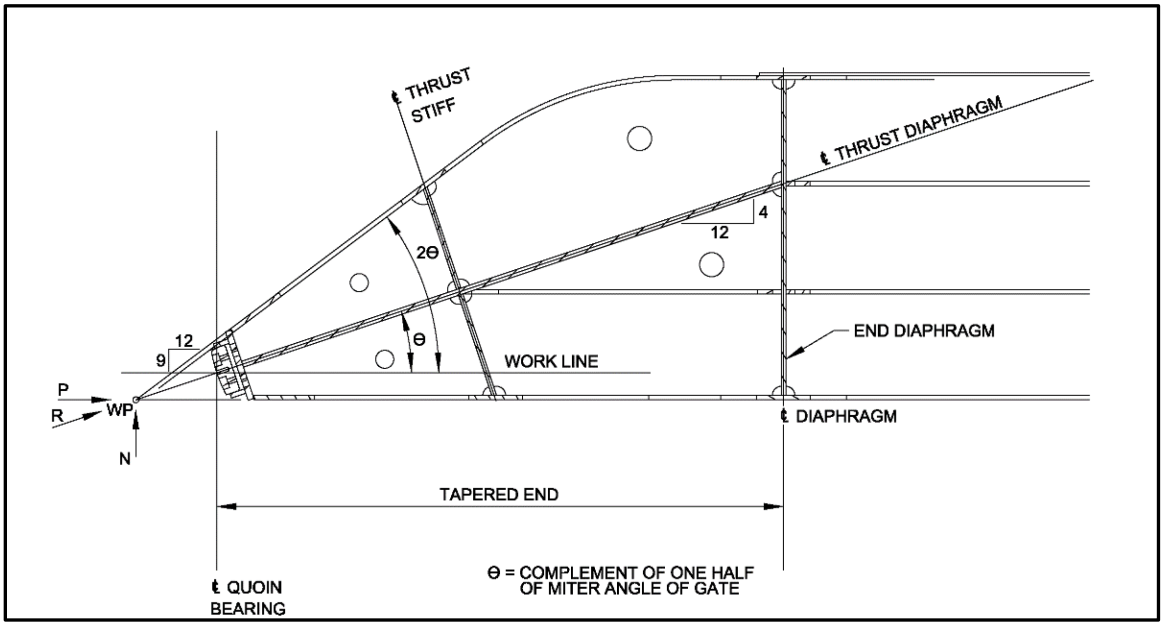


Figure 9.8. Girder Tapered End Section

9.4.1.8. Tapered End Section. The tapered end sections are used to transfer compression loads along the thrust line (see Figure 9.8 and Plate 9.6) of the girders. The load is primarily compression, but the stress distribution is difficult to determine due to complex geometries. Conservative assumptions and liberal use of stiffeners should be used to prevent local buckling of structural elements. Refined analyses can be used to provide a more thorough understanding of actual stress distribution. The minimum bottom girder web thickness over the pintle is usually at least ¾ in. and is machined to at least a 250-microinch finish or to match the machine finish to the top of the pintle socket. Also consider specifying a flatness tolerance between the web and socket. The top and bottom webs are wider at the quoin end to accommodate the gudgeon pin and pintle. Plate 9.5, Plate 9.6, and Plate 9.7 present additional information on girders.

9.4.1.9. Thrust Diaphragm. The thrust diaphragm forms a damming surface between the end plate and the end diaphragm and distributes girder reactions from the quoin block into the girder webs through shear at the thrust diaphragm to girder web connection. A portion of the thrust diaphragm also forms part of the quoin post and is subject to compression and bending stresses in this region. Behavior of the thrust diaphragm is complex due to multiple types and orientations of forces. Conservative assumptions should be used in the analysis and sufficient support provided to give adequate stability.

9.4.1.10. Quoin Post. The quoin post transfers all gravity loads into the pintle. Bending stresses are also present due to eccentricity of the pintle and gudgeon pin with respect to the centroid of the quoin post. The quoin post is a column formed by the end plate, a portion of the thrust diaphragm, the thrust diaphragm stiffeners, and portions of the girder flanges. See Plate 9.7 for geometry. The maximum combined stress may occur at the center of the lower edge of the thrust diaphragm panel or at any of the extreme corners of the quoin post cross-section (points A-E shown on Plate 9.7).

9.4.1.11. Gudgeon Pin Hood.

9.4.1.11.1 The gudgeon pin hood is an arrangement of plates forming the hinge connection at the top of the miter gate leaf (Plate 9.8). The top pin plate has sections sloping from the top height down to the girder web. When designing the gudgeon pin hood, clearances on the side of the hood need to be checked to avoid interference between the hood and the anchorage links during swinging of the gate or for maintenance removal of the gate or anchorage links. Steel rings varying in thickness from 1/16 in. to 1/4 in. are used to adjust the vertical clearance between the gudgeon pin barrel and the pin hood.

9.4.1.11.2 The top pin plate should be designed as a curved beam with a uniform load rather than assume the plate to be an eye bar. Plate 9.9 illustrates formulas from Seely and Smith (1952). This assembly is typically an FCM. See Chapters 6 and 8 for guidelines on detailing and fabricating FCM. The gudgeon pin is typically fixed and rotates with the gate. The pin generally has a minimum diameter of 12 in. to give an additional factor of safety and to standardize the barrel and hood arrangement. The pin is usually machined out of commonly available rod stock material (e.g., American Iron and Steel Institute 4140, 4340).

9.4.1.12. Gudgeon Anchorage.

9.4.1.12.1 The upper anchorage system supporting the miter gate leaves is typically comprised of the gudgeon pin barrel, anchorage links, and embedded anchorage. These components are typically FCM. The design loads are the calculated gate reaction forces, increased 10% for impact. The governing loads usually occur at the recessed (open) or mitered (closed) positions of the gate leaf. Fatigue design of the Primary anchorages (see Anchorage links for a definition of Primary) must double the total lockage cycles to account for the observed behavior of nearly two load cycles per lockage. When detailing gudgeon and anchorage pin plates, it is better to use thicker, single plates than to use welded doubler plates to avoid problems with welds subjected to tension and fatigue.

9.4.1.12.2 To develop maximum operating strut forces, the leaf is assumed obstructed near its miter end. Plate 9.13 and Plate 9.15 show the layout of typical anchorage systems. Consideration should be given to which component of the anchorage system should fail when an overload situation occurs. It is most likely advantageous for the anchorage link to be weaker link of the anchorage system since the gudgeon hood and embedded anchorage are harder to repair and spares are typically not available.

9.4.1.13. Gudgeon Pin Barrel. The gudgeon pin barrel is composed of welded or forged alloy steel plates and is designed as a continuous beam supported by vertical stiffeners while at the same time as a curved beam made up of a horizontal plate and an effective section of the plate cylinder that forms the pin barrel. The thickness of the barrel or horizontal plate should not be less than 1½ in. Plate 9.11 shows a typical barrel arrangement and formulas.

9.4.1.13.1 The alternate method of analysis (shown in Plate 9.12) may be used in lieu of the more precise method beginning in Plate 9.11. Since the barrel is a critical item, the strength capacity should be kept low, in the range of approximately 0.5 F_y and connections detailed to provide sufficient fatigue resistance.

9.4.1.13.2 The gudgeon pin bushing is typically fabricated from aluminum bronze and bearing pressure should be limited to a maximum of 1,500 psi. Consider using self-lubricated bushings for reduced maintenance. See UFGS 35 05 40.17 for approved self-lubricated bushing products and consult a mechanical engineer to ensure the correct product is selected. Some self-lubricated materials may not be appropriate for applications with high silt and debris loadings.

9.4.1.14. Anchorage Links. The pinned end links connect to the embedded anchorage with an adjustable section between the embedded anchorage and the gudgeon pin. Each link is designed as a tension or compression member individually, and the two links are checked as a unit (Plate 9.13 and Plate 9.15). The links are usually adjustable to align the top hinge axis since typically the bottom hinge is not easily adjustable. Threaded turnbuckles and wedge pins have been used for adjustment. Both have had maintenance issues with seizing and corrosion. Some strategies to avoid seizing of turnbuckles are to use split nut turnbuckles or apply corrosion protection to the threads (e.g., electrodeposited cadmium coating ASTM B766, type II).

9.4.1.14.1 Anchorage links are typically oriented parallel to the recessed and mitered direction of the gate leaf. Alternatively, one anchorage link may be slightly angled from the lock wall face and the other perpendicular or slightly angled from perpendicular to the lock wall face direction. In the latter case, the perpendicular anchorage link will be subjected to compressive loading when the gate is in the recessed position. The anchorage oriented in the miter direction or perpendicular to the lock wall is typically referred to as the Primary and the other as the Secondary.

9.4.1.14.2 Commonly used anchorage linkage systems include the 3-pin gudgeon barrel, 5-pin, and 3-pin lapped bars. The 3-pin gudgeon barrel system has a history of fatigue cracking due to poor fatigue detailing. The 5-pin system has more degrees of freedom than the 3-pin gudgeon barrel system that allows for some misalignment and avoids putting compression into the anchorage links and embedded anchorage.

9.4.1.14.3 Spherical bushings have also been used for anchorage link pins. The spherical bushings help to reduce unintended bending stresses that can occur due to misalignment, self-weight of the anchorage links, and vertical thermal expansion (associated with tall gates). Anchorage links can also have slotted rear pin holes on the secondary (recess-direction) bar to allow for the gate quoin block to move towards the wall quoin block.

9.4.1.14.4 The links acting as a unit are assumed to have a maximum misalignment of 2½ in. at the gudgeon pin. This introduces a bending stress in conjunction with the axial load. Design is usually governed by the fatigue design requirements. The threaded section of each link is constructed with forged steel with a minimum diameter of 6 in. Hexagonal sleeve nuts are used for adjustment.

9.4.1.14.5 Usually, one side of the link has 3/8 in. right-hand threads and the other side has 1/2 in. right-hand threads. This difference in pitch and the same thread direction allows for finer adjustment of the sleeve nut. Right- and right-hand threads are recommended (Plate 9.14). Some are left-handed on one bar and right-handed on the other bar with 1/2 in. threads, but this type is more susceptible to the sleeve nut loosening.

9.4.1.14.6 Threads with a root radius groove (e.g., Acme-4C) are recommended over threads square root grooves due to greater fatigue resistance. The first incomplete thread (fin) should be ground for additional fatigue resistance (Plate 9.14). The outside diameter of the section threaded for the sleeve nut should be the same as the largest dimension of the rectangular section. The rectangular section of the link is made from forged steel and should have a minimum size of 6 by 4 in.

9.4.1.14.7 The pin-connected ends of the rectangular sections are designed as eye bars. Pins should be designed for both bending and bearing. Attention should be paid to the tolerancing of the main sleeve nut threads and any lock or jam nut threads in the system to ensure loads are transferred as intended.

9.4.1.14.8 Anchorage links with wedge pin adjustment have a similar geometrical layout to the threaded anchorage links except that there are not threaded turnbuckles and the rear anchorage pin area uses two wedge pins (Plate 9.15). In and out adjustment of the gudgeon pin from the lock wall is made by adjusting the wedge pins up and down. Some advantages include avoiding fatigue-prone details and machining associated with threaded connections. Also, the wedge system eliminates all lash from anchorage system, avoiding undesired movement and thread wear that occurs in the turnbuckle system due to the tolerance in the threads. Do not weld anchorage links together for installing the gudgeon bushing.

9.4.1.14.9 The loadings on the anchorage system can vary greatly depending on the alignment of the gate, environmental factors, and type of operating machinery (Figure 9.9, Figure 9.10, and Figure 9.11). Quoin block misalignment and wear can cause conditions where either there is a gap or interference (prying) between the quoin blocks when the gate is in the mitered position and subjected to hydrostatic load. This can significantly change the load magnitudes on the anchorages.

9.4.1.14.10 The quoin gap set at time of installation is small; on the order of thousandths of an inch. Seasonal temperature changes can close this gap and impose a prying condition on the quoin blocks that creates additional load in the anchorage bar. Strength and fatigue design of anchorages should consider these load variations and account for them based on the assumed conditions over the anchorage life (e.g., % gapped, % prying, load fluctuation due to seasonal temperature changes).

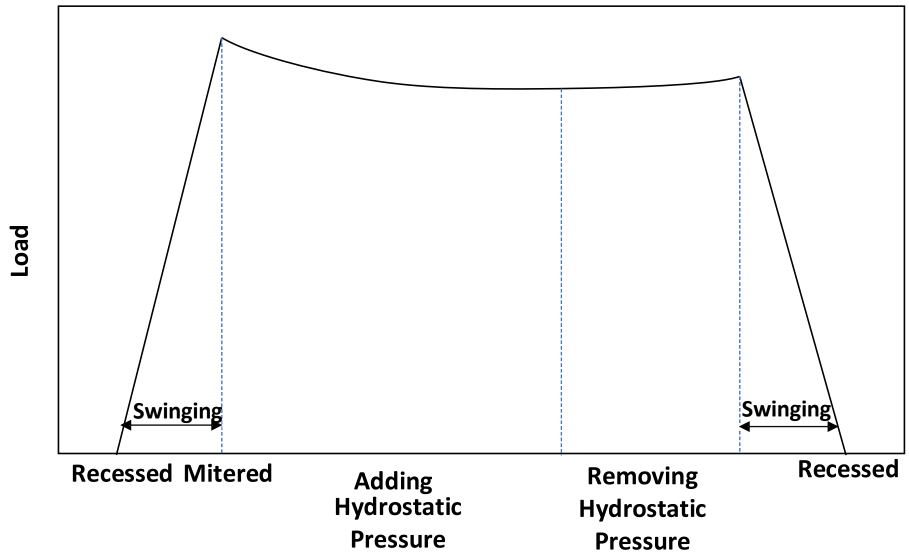


Figure 9.9. Generalized Primary Anchorage Load (Stress) Cycle for Normal Alignment of Quoin Blocks (Blocks in Contact with Hydrostatic Loading)

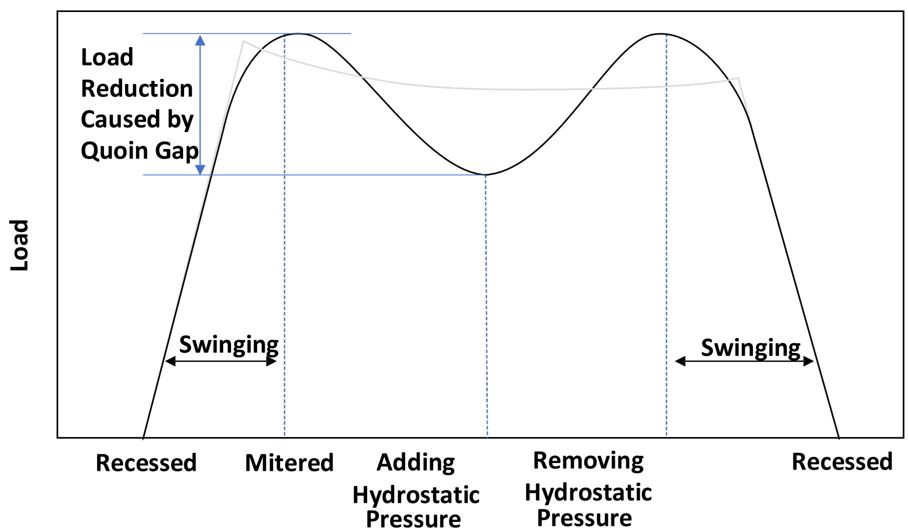


Figure 9.10. Generalized Primary Anchorage Load (Stress)
Cycle for Gap Misalignment of Quoin Blocks
(load reduction can be less, equal, or greater in magnitude to the swinging load depending on the magnitude of the gate weight and hydrostatic load)

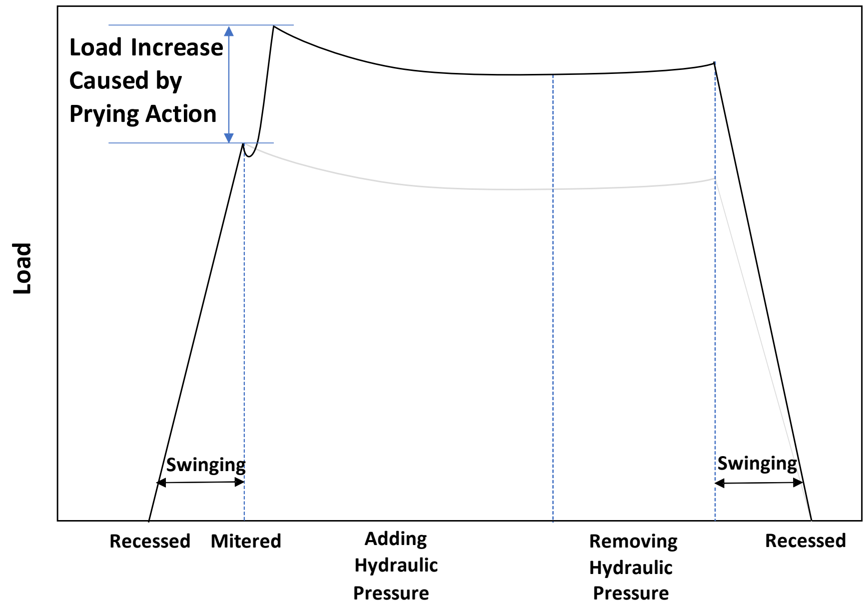


Figure 9.11. Generalized Primary Anchorage Load (Stress)
Cycle for Prying Action Misalignment of Quoin Blocks
(the magnitude of the load increase can be quite high (e.g., nearly double
the usual swinging load was seen at Lock 24))

9.4.1.15. Top Hinge Anchorage.

9.4.1.15.1 The top hinge anchorage distributes the anchorage link load into the lock wall. Previous top hinge anchorage design guidance recommended the use of an embedded triangular frame anchored at its base with post-tensioned anchor bolts into the lockwall concrete. The triangular frame was designed without including the benefit of concrete interaction or bearing.

9.4.1.15.2 There is a history of failures in miter gate and Tainter gate post-tensioned anchorages. Post-tensioned systems are more susceptible to difficulties in installation, quality control, and loss of structural system integrity due to degradation than non-post-tensioned systems. A structural system with its primary load path utilizing post-tensioned components is not as reliable or robust as a structural system utilizing concrete bearing or shear strength. For these reasons, new embedded anchorage systems must not include post-tensioned components in their primary load path.

9.4.1.15.3 The recommended top hinge anchorage is a passive structural system with a structural load path that transfers its load into the lockwall concrete through concrete bearing and shear strength. The anchorage can be exposed or embedded. Examples are shown on Plate 9.17 and Plate 9.18. Design considerations for new anchorages must include ease of inspection, ease of maintenance and repair, corrosion resistance, and fatigue. There are design consideration trade-offs between the exposed and embedded top hinge anchorage system types which as discussed below. A life-cycle cost analysis may prove helpful in selecting an anchorage system.

9.4.1.15.4 Exposed top hinge anchorage offers the advantages of easy inspection, maintenance, and replacement. Disadvantages include the anchorage being open to the weather and salt applied to the lockwall walking surfaces. The anchorage needs to be high enough on the lock wall to be above the normal water line and allow for drainage. In northern climates, the top several inches to feet of the concrete wall surface is subjected to freeze-thaw damage. Consider how to protect the condition of the concrete the anchorage bears on and if the concrete is damaged, how it will be replaced.

9.4.1.15.5 Embedded top hinge anchorage has the advantage that most of it is protected from corrosion except for the portion sticking out of the concrete. The disadvantages are that the embedded portion cannot be inspected; corrosion can occur at interface between concrete and the exposed portion; and it is harder, slower, and more expensive to do a complete replacement. However if the pin plates that extend through the concrete interface are bolted to the frame as shown in Plate 9.17, limited inexpensive concrete demolition can be done to replace the pin plates which are most susceptible to fatigue, corrosion, and damage. If an embedded anchorage is used, it is recommended to consider future replacement (e.g., bolted connection) of the exposed portion.

9.4.1.15.6 Retrofitted or replacement top hinge anchorages may have site conditions and schedules that do not allow for passive anchorage systems, therefore post-tensioned solutions are allowed. One of the main considerations when choosing a type of replacement embedded anchorage is to minimize the amount of concrete removal since it reduces navigation outages, cost, and schedule. For replacement of existing embedded anchorages, a top-mounted anchorage system similar to that used at the Poe Lock has advantages (Plate 9.18). The primary load path is through the horizontal beam to the shear key in the back. The vertical concrete anchors between the pin plates and the shear key help to resist overturning and strengthen the concrete's resistance to shear loading.

9.4.1.15.7 In general, there are 5 common types of top hinge anchorages used on USACE locks. Type 1 is an embedded frame with no post-tensioning (Plate 9.17). Type 2 is a top-mounted structure using a shear key for its primary load path (Plate 9.18). Type 3 is a dead man system with no post-tensioning. Type 4 is an embedded frame with post-tensioned anchors. Type 5 is a compact post-tensioned exposed anchorage (Figure 9.12). Types 1–3 must be used for new construction. Types 4 and 5 may be used for retrofits where existing geometrical or site conditions do not accommodate the usage of types 1–3.

9.4.1.15.8 For embedded anchorages, the understanding and behavior of the steel-to-concrete interaction has been greatly improved by the Engineer Research and Development Center (ERDC) finite element modeling and full-scale laboratory testing done in consultation with the INDC. The results show how loading stresses are distributed into the embedded anchorage and what portions of the anchorage are subjected to fatigue. See Levine, Eick, et. al. (2019), Eick, Levine, et. al. (2021), and Eick, Levine, et. al. (2022) for more details.

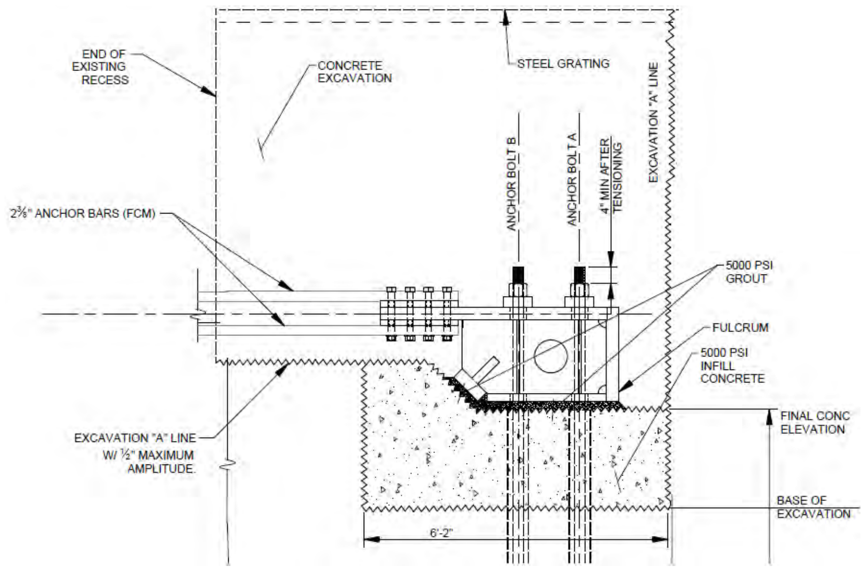


Figure 9.12. Type 5 Top Hinge Anchorage – Compact Post-Tensioned Exposed Anchorage (Elevation)

9.4.1.16. Pintle Assembly. The pintle and related components support the gravity loads of each leaf of the miter gate as well as incidental hydrostatic loading due to lack of support conditions as discussed in paragraph 9.3.5. Short gates can have a large horizontal component of the gravity load reaction, and therefore a larger pintle reaction than would be expected.

9.4.1.16.1 The unit is made up of five major components: pintle socket, pintle bushing, pintle ball, pintle shoe, and pintle base (Plate 9.19). The center line of the pintle (vertical axis of rotation) is located eccentrically upstream of the center of curvature of the bearing face on the quoin contact block as described in paragraph 9.4.1.1. Studies and experience show that these eccentricities will minimize interference and binding between the quoin blocks. EM 1110-2-2610 provides additional details on pintle design.

9.4.1.16.2 The pintle region has typically been the major source of problems with miter gates in service. The issue is widespread as identified in the Permanent International Association of Navigation Congresses International Commission Working Group (PIANC InCom WG) Report No. 154 effort. Typical pintle problems include:

- Cracking in the pintle region of the gate or pintle caused by the hydrostatic load path going through the pintle instead of the quoin block due to loss of quoin block contact (wear, degradation, or misalignment).

- Excessive bushing wear which can cause the pintle to shift toward the quoin and cause quoin block wear or interference. Bushing wear can be caused by lack of lubrication or higher than allowable bearing stresses.

- Pintle galling or seizing. Typically, due to lack of lubrication, the gate can start “jumping” or shuddering during swinging instead of a smooth steady rotation.

• Wear of the back (quoin side) of the pintle base. This has been common with the floating pintle type base where the pintle either starts moving along the circumference of the pintle base retainer or rotating against (i.e., pintle seized in bushing and gate rotating at pintle base). This wear also causes the gate to move toward the quoin causing quoin block wear or interference.

9.4.1.16.3 When a pintle problem occurs, the consequences are high since it affects the ability for the miter gate and therefore, the lock to function. Additionally, pintles are hard to maintain since they are under water. Repairing pintle problems typically requires lock dewatering or lifting the gate, both of which are costly and cause navigation outages.

9.4.1.16.4 The current USACE practice of using fixed pintles lends itself to pintles taking unintended hydrostatic load because, if there is any quoin block wear or misalignment, the gate cannot move toward the wall quoin block to make contact and proper load transfer. Research efforts are underway to investigate better anchorage and pintle hinge behavior that allows the gate to make more reliable quoin block contact.

9.4.1.16.5 Pintle Socket. The pintle socket is made of cast steel or forging and is connected to the bottom of the lower girder web with turned bolts (machined bolts to a machined-fitted hole size). The bolts are sized to carry the gate leaf reaction in shear, but as an added safety factor, a thrust plate should be connected to the underside of the bottom girder web with a milled contact surface between the plate and pintle socket. The socket encloses the bronze bushing, which fits over the pintle ball. Maximum bearing stresses are discussed in EM 1110-2-2610. Plate 9.20 presents additional information.

9.4.1.16.6 Pintle Bushing. The pintle bushing is important to proper pintle operation. Traditionally, the pintle bushings have been grease-lubricated aluminum bronze. Self-lubricated bearings have proven successful and are becoming more widely used. Most self-lubricated bearings are proprietary and there is a wide range of quality and performance available. Therefore, it is important to consult a mechanical engineer to ensure the right product with a good USACE test record is selected. See EM 1110-2-2610 for more information on pintle bushings and self-lubricated bearings. Some self-lubricated materials may not be appropriate for applications with high silt and debris loadings.

9.4.1.16.7 Pintle Ball. The pintle ball usually has a diameter of 10 to 22 in., with the top bearing surface in the shape of a half sphere and a cylindrical shaped bottom shaft. For salt or brackish water, pintles are typically cladded with corrosion-resisting steel. The pintle ball and bushing are typically finished to a 16-microinch (μin) finish where the two come in contact. Often the pintle ball is fitted to the bushing by scraping or lapping in the shop until uniform contact is attained over the entire bearing surface. For more information including materials used for pintles, see EM 1110-2-2610.

9.4.1.16.8 Pintle Shoe. There are three types of pintle assemblies for horizontally framed miter gates: fixed, floating, and free. These types of pintle assemblies will be briefly covered here with more thorough coverage available in PIANC WG Report No. 154.

9.4.1.16.9 Fixed Pintle. Plate 9.19 shows a typical fixed pintle. This pintle does not allow for movement of the gate relative to the gate's pintle base or lock floor. While this type of pintle has problems with not accommodating quoin block misalignment or wear, it does have a better performance record than the floating pintle and therefore is currently recommended for new construction and rehabilitation until a better performing pintle design has been researched and tested.

• The pintle fits into the pintle shoe, which is bolted to the embedded pintle base. The degree of fixity of the pintle depends on the shear capacity of the pintle shoe bolts. The pintle should be designed so that after relieving the load on the pintle by jacking, the pintle assembly is easily removable.

• The pintle base, made of cast steel, is embedded in concrete, with the shoe fitting into a curved section of the upper segment of the base. The curved section of the same radius as the pintle shoe is formed so that under normal operation the reaction between the shoe and base is always perpendicular to the curve of both shoe and base at the point of reaction.

9.4.1.16.10 Floating Pintle. Plate 9.20 shows a typical floating pintle. This type of pintle is not recommended for new construction. The pintle is fitted into a cast steel shoe with a shear key provided to prevent the pintle from turning. The shoe is not fastened to the base, thereby allowing the gate leaf to move outward in case of debris between the quoin and wall quoin, which prevents the leaf from seating properly. Damage to the pintle bearing has occurred frequently with this type of pintle due to the relative movement between the pintle shoe and base.

• The movement can consist of the shoe sliding on the base during leaf operation from either the mitered or recessed position until the leaf reaches approximately the mid-position, at which time the shoe slides back against the flange on the base. This type of movement is generally visually detectable and causes serious wear.

• An alternative to the floating circular shoe is to make the shoe three sided with one corner having the same radius as the circular shoe and attaching a steel keeper bar to the embedded base in front of the shoe. This would prevent the shoe from rotating on the embedded base and prevent the pintle from moving out of pocket. Again, the degree of fixity would depend on the shear capacity of the bolts in the keeper bar. This alternative will meet the requirements of the fixed pintle as well as the capacity to minimize damage in case of emergency.

• For floating pintles, a calculation must be performed to show that when the pintle has been dislodged due to debris that it can reseat itself as the gate swings (i.e., the horizontal reaction at the pintle base due to gate weight must be greater than the static frictional force between the pintle shoe and pintle base).

9.4.1.16.11 Free Pintle. The free pintle allows for movement between the pintle ball and the bushing or socket but no movement between the pintle, pintle base, or lock floor. This allows for movements due to wear or misalignment so the horizontal loads are transferred to the quoin blocks instead of the pintle. This type of pintle has been widely used on European waterways which typically have narrower locks; however, it has not been used in the USACE lock inventory. Research is ongoing within USACE to investigate future implementation.

9.4.1.16.12 Pintle Base. The pintle base is designed so that there will be a compressive force under all parts of the base. The overturning moment is caused by the horizontal force on the pintle and will be resisted by the reaction on the section being investigated. Due to the fine tolerances of the standard practice of using fixed pintles and tight-fitting quoin blocks with little room for error, it is important to consider constructability and installation of the pintle base so it can be precisely located in alignment with the quoin blocks and gudgeon anchorage.

9.4.1.17. Miter Blocks and Wall Quoins. Plate 9.24 shows typical quoin and miter block details. Miter blocks transfer the hydraulic load from gate leaf to gate leaf. Quoin blocks are reaction bearings that transfer the hydraulic load from the gates into the lock walls. For large gates, the hinges cannot be designed to withstand the large hydraulic load on the gate so it is imperative the hydraulic load be transferred through the quoin contact blocks to the lock wall.

9.4.1.17.1 It can be challenging to achieve the tight tolerances needed to achieve good contact bearing while still allowing the gate to freely rotate. The center of the quoin block is designed to be eccentric from the gate's hinge axis (Figure 9.3) so that the blocks only make contact when the gate is in the miter position. Free hinges that allow for movement also help the gate to move in and out of contact for loading and swinging.

9.4.1.17.2 Contact Block Surface Geometry. Different geometries have been used with balancing the trade-off of bearing surface area versus ease of disengagement to limit frictional loads during initial gate movement. These geometries include:

• Concave wall block with convex gate block: same radius into same radius or different radii;

• Flat wall block with flat gate block;

• Flat wall block with convex gate block;

• Flat gate block with flat gate block (miter); and

• Concave gate block with convex gate block (miter).

• For quoin blocks, the most common type is concave wall block with convex gate block. For miter blocks, the most common types are concave gate block with convex gate block (same radius) and flat gate block with flat gate block. The blocks are usually about 8 in. wide. The radius for the convex and concave blocks are usually 1.5 ft or greater. Sometimes the convex block has a slightly smaller radius than the concave block that it mates with.

9.4.1.17.3 Continuous Contact Blocks. Continuous contact blocks form a bearing and sealing surface between the gate and wall at the quoins and between the miter ends of the leaves. The gate blocks are typically made up of 10 to 20-ft-long sections with transverse joints occurring at the center lines of horizontal girder webs. This is currently the standard practice in USACE.

9.4.1.17.4 Discontinuous Contact Blocks.

• Discontinuous contact blocks are contact blocks located intermittently along the height of the gate at the horizontal girder locations. This type of contact block system has the advantage of being easier to adjust and readjust. Continuous contact blocks are difficult to adjust for continuous bearing along the full height of the gate and often in reality do not continuously bear. Also, the current systems of adjusting continuous blocks do not lend themselves to easy readjustment.

• Discontinuous contact blocks have been successfully implemented internationally and Eick, Smith, and Fillmore (2019) shows them to be feasible for USACE miter gates. Design and details of discontinuous contact blocks will be forthcoming.

9.4.1.17.5 Wall Quoin. The quoin block on the lock wall is essentially the same as the miter block with the wall quoin having the concave surface and the quoin block on each leaf having a convex surface. There are two recommended types of wall quoin systems:

• The first, an adjustable type, consists of a 10 x 3½-in. bar welded to a 1¼ x 1 ft, 5-in. base plate (Plate 9.24, Plans 1 and 2). The base plate is attached to a vertical beam with jacking and holding bolts to facilitate adjustment and replacement. The vertical beam is embedded in second-pour concrete and transmits the quoin reaction forces into the wall. The space between the base plate and the embedded beam is filled with epoxy filler after final adjustments have been made. Zinc has also been used as a backing material, but the high temperatures involved may damage the concrete.

• The second, a fixed type, consists of a 10 x 3½ in. bar welded to a vertical beam as described previously (Plate 9.24, Plan 3). Proper detailing of the first and second concrete placements are important for installation and successful alignment of the wall quoin. Concrete consolidation behind the wall quoin blocks during construction should also be considered.

9.4.1.17.6 Adjustment of Contact Blocks. Proper adjustment of the contact blocks is critical to gate performance both structurally and for water tightness. Typically, the rigid contact blocks are adjusted to the desired bearing or gap tolerance using push and pull bolts staggered along the height of the contact blocks. There is no consensus on what the correct quoin block gap setting should be. With the current gate hinge systems in use, the quoin gap tolerance is very tight and unforgiving to achieve proper quoin contact.

• Values of quoin gap settings in USACE currently vary from zero to 0.030 in. with most at approximately 0.010 in. but varies dependent on temperature during installation. Mechanically adjustable contact blocks can be used instead of push and pull bolts. Adjustments are made by moving wedge sections that allow the bearing surface to move in and out. However, adjustable contact blocks are very expensive, may see increased leakage through the block, and may not be adjustable over time with exposure to a submerged environment.

• After adjustment, a backing material is poured behind the contact blocks to permanently set them. When backing material is used, the contact block supports on the gate should be designed with plenty of holes along the height of the block to use as pouring ports. The hole should be at least 1 in. in diameter and threaded so it can be blocked with a bolt as the pour procedure moves up the gate. More details on contact block setting and installation are given in section 9.5.

9.4.1.17.7 Contact Block Materials.

• Contact blocks can be made out of:

– Carbon steel,
– Alloy steel,
– Forged steel,
– Corrosion-resisting clad steel,
– Solid corrosion-resisting steel, and
– Glass fiber reinforced polymer.

• The corrosive conditions of the lock site and bearing stresses must be considered in selecting the best performing material. Corrosion-resisting steel blocks can be very expensive. Hardened corrosion-resisting steel can also be difficult to machine due to warping. In addition, galvanic corrosion and isolation must be considered when using corrosion-resisting steels. When carbon steel contact blocks are used, the sides of the blocks should be painted.

9.4.1.17.8 Backing Material.

• After final adjustments have been made to the miter and quoin blocks, a gap of about 1/2 in. between the backing plate and the blocks is filled with zinc or epoxy. The filler layer assures a uniform transfer of the loads from the leaf into the blocks. Epoxy is typically easier and safer to install. Zinc can be dangerous to work with since it is a molten metal, exhausts fumes, and splashes when in contact with water or moisture. A disadvantage of epoxy is that it requires warm temperatures and surfaces for placement and takes longer to cure in cold temperatures.

• Precautions should be taken to prevent leakage of the filler and to prevent air entrapment. See Foltz, Trovillion, and Ryan (2015) for evaluation and techniques for sealing materials. A bond-breaking material should be applied to jacking bolts, holding bolts, and contact surfaces. Where zinc is used, a seal weld is needed at the end joints of the blocks after cooling. Welds should be ground smooth to prevent interference with bearing surfaces.

9.4.1.18. Diagonals. Each miter gate leaf is similar to a horizontal cantilever beam. Resistance to vertical loads is provided by the skin plate acting as a girder web. However, a cross-section of the leaf looks like a stacked series of channels. The vertical shear center of channel members is on the opposite side of the web from the flanges.

9.4.1.18.1 The offset between the leaf shear center and the center of gravity loads causes the leaf to twist so that the miter post is no longer vertical. Since channel sections are very flexible torsionally, diagonals are added to the back side of the leaf to increase the torsional stiffness as the gate moves through the water. The diagonals are also tensioned to adjust the plumbness of the miter post. Plate 9.22 and Plate 9.23 show examples of diagonal configuration.

9.4.1.18.2 The two main goals of diagonal tensioning are to plumb the gate and to provide a minimum pretension such that during operation the tension in the diagonal does not dip below 1,000 psi to avoid slackened, loosened, or buckled diagonals. Strain gages should be used for determining the stress in each diagonal—typically a minimum of three gages per diagonal. The maximum stress for temporary conditions must not exceed 0.75 F_y.

9.4.1.18.3 Information for the design of diagonals is provided in Appendix C. The method provided is not exact but experience has shown that the results obtained from this method have been very close to the values needed in the field for plumbness and minimum pretension. The stiffness of welded miter gates appears to be considerably greater than that assumed in the reference material. While this does not affect the overall pattern of diagonal design, it should be kept in mind when selecting the values for deflection of the leaf.

9.4.1.18.4 Diagonal tensioning is sensitive to gate out-of-planeness or warping due to fabrication and assembly. If the gate is slightly warped, the applied diagonal tension has P-Delta effect on magnifying the out-of-plane deflections. It is recommended that the gate be measured for planarity in the vertical position after assembly but prior to diagonal tensioning and acceptability tolerances being set.

9.4.1.18.5 Recently, several miter gates have had diagonal tensioning performed in the horizontal flat position at the shop prior to delivery. When the gates were installed vertically in the field, not much, if any, tensioning adjustment was needed. A large degree of this success is believed to be due to the planar trueness of the assembled gate leaf in fabrication. The horizontally framed gates tensioned in the shop ranged from approximately 25 to 80 ft in height. The vertically framed gates tensioned in the shop ranged from approximately 18 to 33 ft in height.

9.4.1.18.6 While it is still recommended to contractually have the contractor tension the gate leaves when installed at the lock, it may be acceptable to allow the fabricator to shop-tension the leaves provided that the strain gages are left attached and protected for retensioning in the field.

9.4.1.18.7 Commonly, there are diagonals spanning both directions on the back of the gate to form an "X," but in lower head or smaller gate applications sometimes only one diagonal is present. Some smaller gates rely on fixed fabricated plates to carry the shear and torsional forces without relying on an adjustable diagonal member. Gates of greater width to height ratios may have multiple sets of diagonals present (such as a "XX" or other arrangement, see Plate 9.3 and Plate 9.4).

9.4.1.18.8 Design of the diagonals for a vertically framed gate is similar to a horizontally framed gate. The number of panels of diagonals used on vertically framed gates depends on the spacing of vertical girders. The panel size with the height equal to 1.50 times the width is desirable but should not in itself be the only consideration for vertical girder spacing, which sets the panels for diagonals. Usually leaf dimensions are such that three sets of diagonals on a leaf face are used.

9.4.1.18.9 The diagonals may be pin connected or welded to the gusset plates. Gate diagonals are often adjusted via a threaded end or turnbuckle on smaller gates. Larger gates may require larger jacking device diagonals. While Appendix C.2 and Appendix C.4 predict the extent of adjustment and amount of post-tensioning force required to plumb the gate, actual adjustment and associated forces may vary in the field when conducting the work. Hence, the design should incorporate a reasonable factor of safety when sizing diagonal components.

9.4.1.18.10 The act of adjustment itself may require temporary supports and/or jacks to allow adjustment mechanisms (e.g., nuts or turnbuckles) to operate. During the adjustment procedure, vertical plumbness of the miter end of the gate is often verified using a plumb-bob. The plumb-bob is sometimes submerged in oil to counter the effect of wind. Often the gate is adjusted to plumb and the sill is later aligned to mate properly with the plumbed gate.

9.4.1.18.11 There are two general methods of prestressing diagonals. In one method, the leaf is twisted a precomputed amount and the slack in the diagonals is removed (twist-of-the-leaf method). In the other, the diagonal is tensioned to a precomputed amount (measured strain method). Measured strain method is more commonly used and typically recommended due to its ease of use and avoidance of gate damage. Caution should be taken when using the twist-of-the-leaf method where the leaf has top and bottom torque tubes.

9.4.1.18.12 Due to the increased leaf stiffness, there is a need for a higher jack capacity (150+ tons), and a possibility that damage could be caused to the leaf or other gate components. The high jacking loads could cause damages such as localized buckling of plates, excessive deflection in the quoin post, damage to the grease seals, pintle, and pintle socket, etc. The two diagonal adjustment methods are discussed below.

• Twist-of-the-Leaf Method. The twist-of-the-leaf method is an older method that is no longer used as much with the increased availability and performance of hydraulic jacking devices. In the twist-of-the-leaf-method, the quoin end of the leaf is made plumb and the miter end is anchored to prevent horizontal movement in either direction. This is done by either (1) tying the bottom of the miter end to the sill, or (2) tying the top miter end to the lock wall and using a hydraulic jack at the bottom. Then, with a power-operated cable attached to the top of the miter end, the leaf is twisted the computed distance "D" for one set of diagonals and the slack is removed from this set. During this operation, the other set of diagonals must maintain slack. The leaf is then twisted in the opposite direction the computed distance "D" for the other set of diagonals, and the slack is removed from them.

– It is important that all the slack be removed without introducing any significant tension in the diagonal. This can best be accomplished by lubricating the nut and manually turning it with a short wrench. Since the turning resistance increases abruptly with the removal of the slack, the point of removal can be felt.

– As a further precaution, a strain gage is recommended on the diagonal being tightened. The maintained deflection of the leaf should also be monitored since more than a slight change in the tension in the diagonal can cause a change in deflection of the leaf. On existing gates where the diagonals were not designed by this method, it may be necessary to overstress some diagonals during the prestress operation. A stress of 0.67 F_y for this one-time load is considered permissible, where F_y is the yield strength of the diagonal material.

• Measured Strain Method Using Tensioning Mechanisms. There are several different types of tensioning mechanisms used to tension individual diagonal bars:

– Turnbuckles,

– Single Nut,

– Multiple Nut,

– Multi-Bolt Jacking Device, and

– Jack and Shim.

• Turnbuckles. Historically, turnbuckles were the most common, however, they were hard to tension without twisting the diagonals and often seized over time due to corrosion. If re-tensioning is required later in the gate's life, the turnbuckles often had to be cut out of the diagonal and replaced with a new turnbuckle. As such, turnbuckles are no longer recommended.

9.4.1.18.13 The other types of tensioning mechanisms listed typically use hydraulic jacks with the exception being the multi-bolt jacking device, which uses bolts as the jacking mechanism. When possible, it is recommended to locate the tensioning region of the diagonal above the waterline to prevent corrosion and to allow for tensioning in service.

• Single Nut. A hollow, cylindrical hydraulic jack is threaded onto the threaded end of the diagonal as shown in Figure 9.13. The diagonal is stressed to the design value and the nut below the jack is tightened. This mechanism is popular due to its simple set up and procedure. After tensioning, the threads are protected with grease, plastic wrap, and a polyvinyl chloride (PVC) cover in an effort to prevent corrosion of the threads. This mechanism is the preferred mechanism on the Upper Mississippi River.

Figure 9.13. Diagonal with Threaded Ends and Large 4.5-in. Diameter Nut (Left) and with Hydraulic Tensioner Attached (Right)

• Multiple Nut. Similar to the single nut, the multiple nut has more than one threaded stud per diagonal. These smaller diameter threaded studs are tensioned at the same time using multiple small stud tensioners (Figure 9.14). This tensioning mechanism is common on the Lower Ohio River.

• Multi-Bolt Jacking Mechanism. The multi-bolt jacking mechanism is similar to the nut tensioning mechanism except that instead of using a hydraulic jack to pull on the threaded end, several jack bolts are turned to jack the single nut (Figure 9.15). The advantage of this mechanism is that no special equipment is needed such as a hydraulic tensioner or jack. However, with more threaded pieces, it may be more susceptible to corrosion. This mechanism is the preferred tensioning mechanism on the Upper Ohio River.

Figure 9.14. Multiple Nut Tensioning Mechanism Using Multiple Stud Tensioners

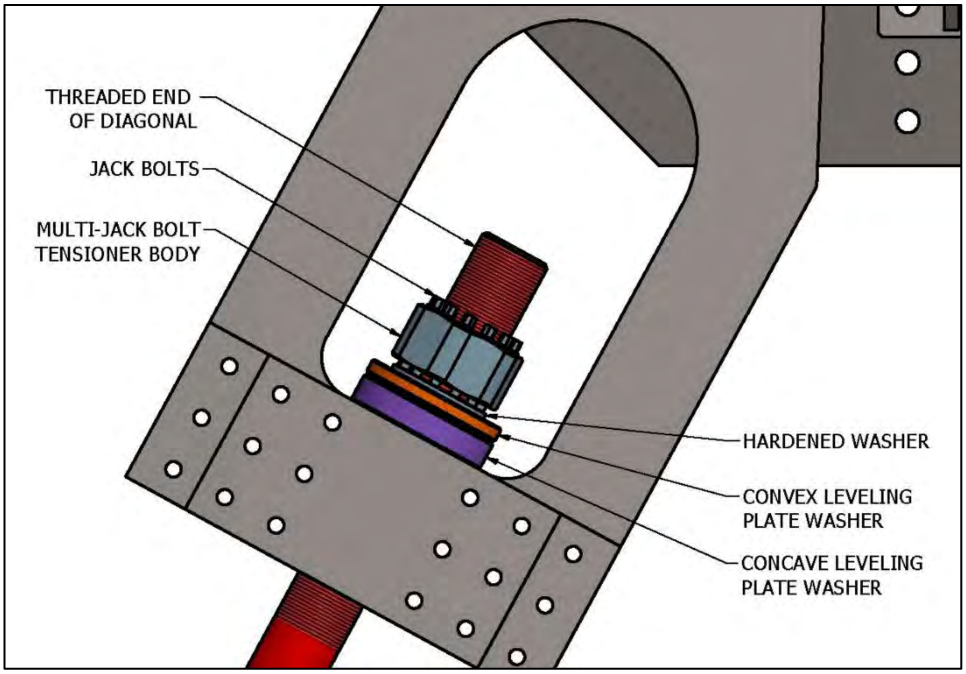


Figure 9.15. Multi-Bolt Jacking Mechanism (USACE, Inland Navigation Design Center)

• Jack and Shim Mechanism. A new tensioning system using shims has been engineered and used by The St. Lawrence Seaway Management Corporation (SLSMC) (Roby, P., 2016). The design objectives were to remove threaded parts that can corrode over time and avoid torque reaction when pretensioning by using a linear force. The system does not rely on threads. This was achieved using hydraulic force and plate work, rendering it low maintenance and easy to fabricate. No costly or longer lead time parts, such as forgings, are needed.

– As shown in Figure 9.16, a removable hydraulic jack is inserted in the “eye” of the diagonal and jacks the diagonal tight against a reaction gusset plate. When the proper tension is reached, shims are inserted in the gaps between the diagonal and reaction gusset plate.

– For maintenance purposes, the SLSMC (Roby, P., 2016) built a chart correlating the tension in the diagonal to the hydraulic pressure allowing mechanics to adjust it without using strain gauges. As an inspection tool, a second chart correlating the natural frequency with the tension increase in the diagonal allows for a quick check with few impulses.

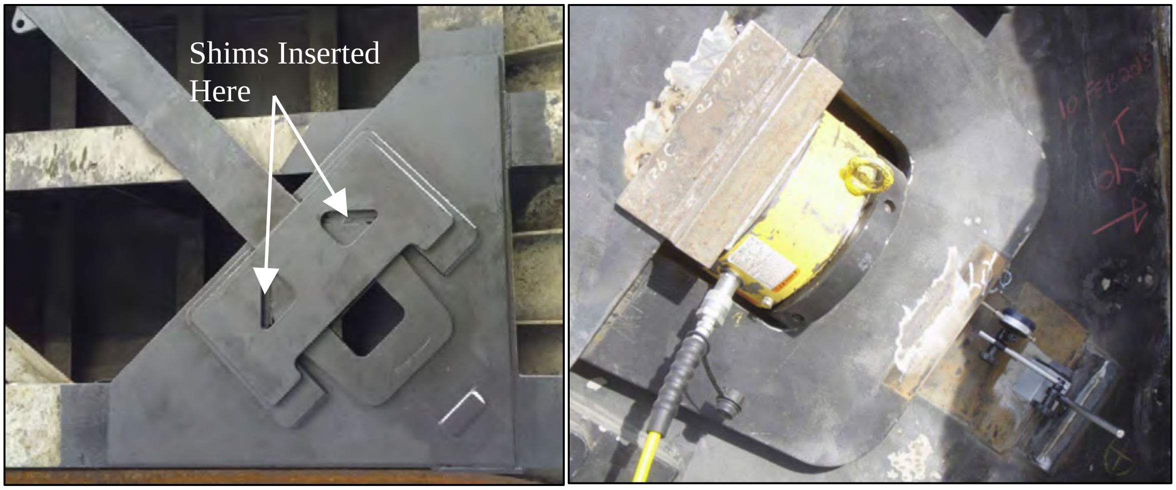


Figure 9.16. Jack and Shim Mechanism
(The St. Lawrence Seaway Management Corporation) (Roby, P., 2016)

9.4.1.19. Seals. Seals serve an important role by enabling the gate to successfully dam water by sealing the perimeter of the gate against the lock walls and sills. The bottom seal between the gate and the lock sill prevents excessive leaking and damaging vibrations that are caused by high water pressures. The side seals between the gate and the lock walls are primarily for water retention. The typical sealing arrangement for a horizontally frame miter gate is shown in Figure 9.17.

9.4.1.19.1 On small watersheds, water conservation is very important, so the ability of the gate seal to successfully perform is essential. On larger watersheds and rivers, side seal leakage may be aesthetically undesirable but is tolerable since water is plentiful. Water sealing can be achieved through rubbing, water pressure deflecting a seal against a surface, seal self-weight, or external mechanisms. The three primary material sealing configurations are:

- Elastic polymers (rubber) bearing on metal or concrete contact. The stiffness and pliability of the rubber depends on the situation. This is the most common and recommended type.

- Metal bearing on metal contact. Special attention must be given to ensure proper sealing is made. There is no forgiveness for misalignment due to installation or damage.

- Wood bearing on metal contact or concrete lock wall. Wood is not as durable as elastic materials for comparing wearing, rubbing, or rotting.

9.4.1.19.2 Galvanized bolts painted with an approved primer for galvanized steel and top-coated with a vinyl paint system are recommended for attaching seals to the gate. Painted galvanized bolts are recommended over plain steel bolts since seal bolts are typically installed after everything else has been painted and they do not require blasting, which could damage the surrounding paint.

9.4.1.19.3 Stainless steel bolts cause galvanic corrosion and typically seize up over time as much as steel bolts. Typical removal and replacement of seal bolts if they cannot be unbolted is cutting them and installing new bolts. More in-depth coverage of miter gate seals is provided in PIANC InCom WG Report No. 154.

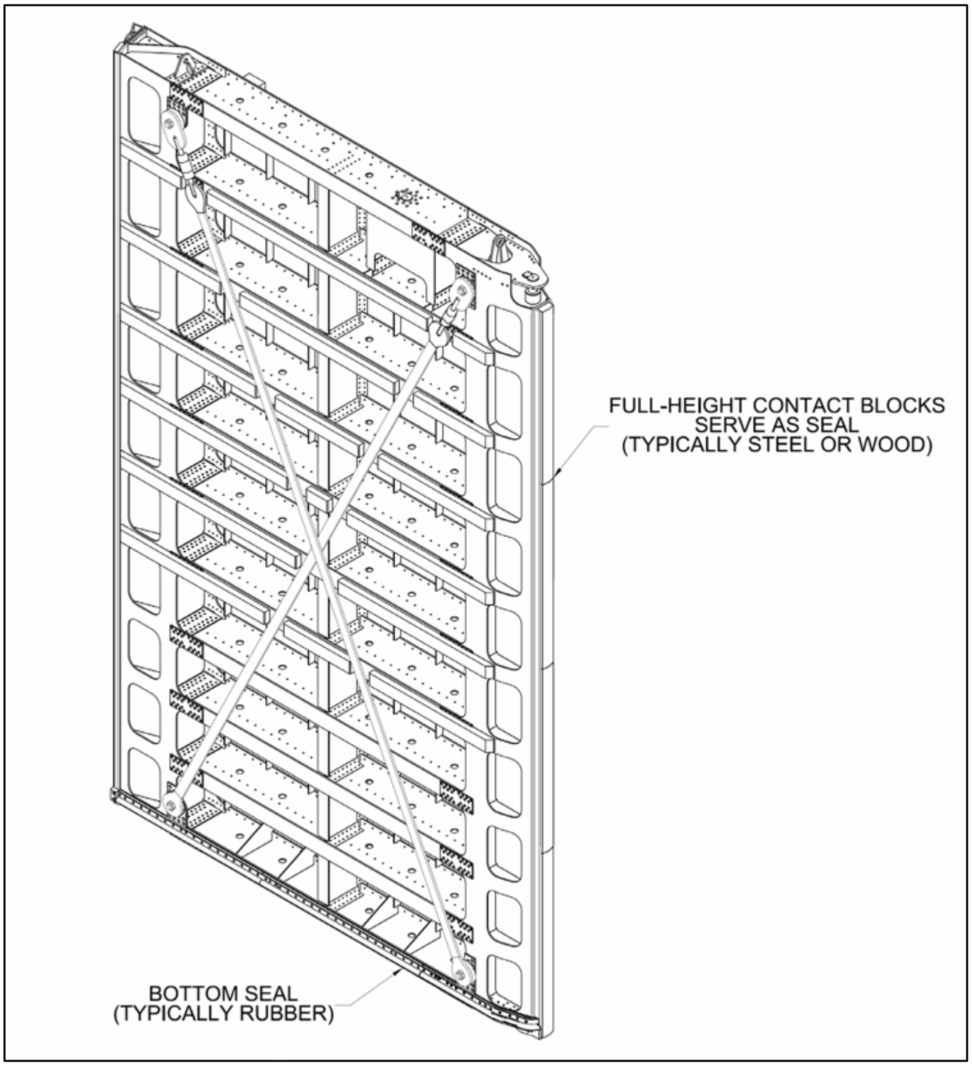


Figure 9.17. Seals on Horizontally Framed Miter Gates

9.4.1.19.4 Rubber seals should be installed on the bottom of each leaf to seal the gate to the miter sill, as shown in Plate 9.26. These details have been successful in reducing vibrations and in accommodating large temperature variations. It is also less susceptible to damage from debris than other seal arrangements.

9.4.1.19.5 Seals might also be required above ends of the quoin and miter blocks if a water-tight surface is required above that point. However, a major disadvantage of supplementary seals at the quoin and miter blocks is that they can mask major contact problems between the contact blocks.

9.4.1.19.6 Bottom Seals. Bottom seals are located at the base of the miter gate leaves and seal the base of the gate to the sill. Bottom seals are designed to prevent chamber water from leaking under the gates when the gate is mitered. Seals can be attached to the sill or to the gate. If the seal is attached to the sill, the seal has to be adjusted to properly fit-up with the gate during installation. After adjustment, a second pour of concrete is placed to secure and support the seal.

9.4.1.19.7 The sections below detail various common types of gate bottom seals. It is recommended to attach the bottom seal to the gate rather than the sill for easier repair and adjustment. The recommended bottom seal type is the inclined J-bulb seal. It is also recommended to apply a fluorocarbon coating to the seal bearing surface for improved performance and longevity.

- Inclined J-Bulb Seal. The round seal, J-bulb seal, and the "porkchop" seal are subject to damage due to debris accumulation at the sill. If a foreign object, such as a log, becomes stuck against the sill, mitering the gate may tear or bend the bottom seal, thus reducing its effectiveness.

- Damaged bottom seals are a common source of gate vibration problems, which is especially noticeable during the lock filling process. As the hydraulic pressure head increases, a damaged or improperly seated bottom seal will allow water to rush through the opening. This high-velocity flow can induce resonant physical and audible vibrations in the gate leaf. Therefore, the round seal, J-bulb seal, and the "porkchop" seal are not recommended for new miter gate construction.

9.4.1.19.8 To mitigate the potential for seal damage due to foreign debris, an inclined bottom sill seal is preferred for new gate construction as shown in the Figure 9.18. The rubber seal is located on the top of the sill, rather than the side of the sill, as found with the round, J-bulb and "porkchop" seals. This geometry reduces the susceptibility of debris accumulation at the sealing surface. It is also important for the bottom seal to allow horizontal deflection girder while maintaining a seal, which the inclined seal allows. This seal accommodates unbalanced head conditions while still providing a positive seal.

9.4.1.19.9 This inclined seal design shows a horizontally mounted, flexible seal forced against the sill with differential water pressure. The seal may be set to touch the sill. It accommodates up to 1 in. of horizontal misalignment, and it does have some sensitivity to vertical misalignment. It could deflect to seal over a relatively large range, up to 0.75 in. If the seal is still in firm contact with the sill after hydraulic head pressure is removed, the seal may bind and be bent over when the gate is opened.

9.4.1.19.10 Seal design should incorporate slotted holes to allow for adjustment in both the vertical and horizontal directions. Vertical adjustment becomes important when accounting for pintle wear and movement or during pintle replacement where the height of the gate may change due to the new pintle and bushing combination. Allow enough sill plate surface to accommodate for the change in miter location (upstream-to-downstream direction) as the gate length changes due to temperature fluctuations. Usually with increase in temperature, the gate lengthens and the miter contact point moves further upstream relative to when temperatures are lower.

9.4.1.19.11 Installers should set the miter point according to the ambient temperature conditions to allow the sealing in all operating ranges. Provide gate stops as a secondary means to prevent gate from overtraveling the sill on horizontal style seals.

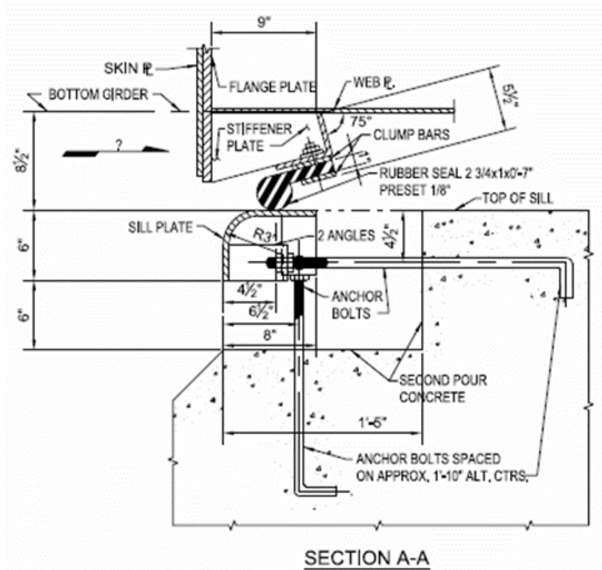


Figure 9.18. Inclined J-Bulb Seal

• Bottom Block Seal in the Pintle Region. In the pintle region where the pintle socket rotates against the sill, a rubber block seal has been used successfully as shown in Figure 9.19. The block seal is a rectangular piece of rubber that is anchored to the concrete surrounding the miter gate pintle as shown below. The pintle socket/casting bears against the block seal when the miter gate is in the mitered position. The rubber hardness is typically a Shore Type A durometer value of 60. This block seal is robust and is anticipated to have a long service life. One disadvantage is that it is attached to the concrete instead of the gate where it would be easier to replace.

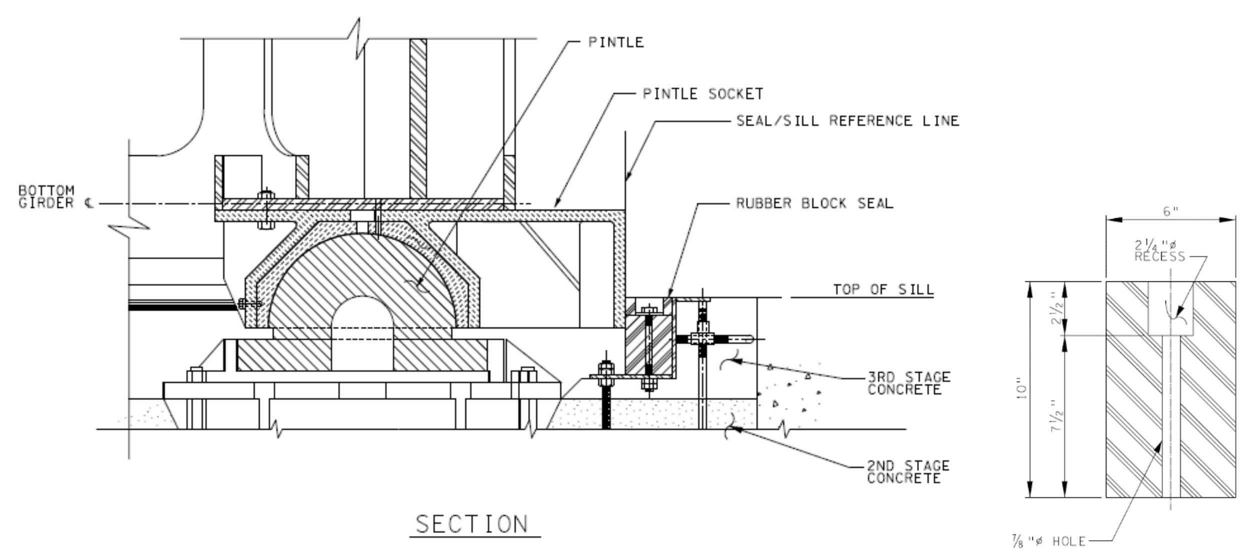


Figure 9.19. Block Seal (Kentucky Lock)

• Bottom Seal Transition from Round Rubber Seal to Block Seal Near the Pintle. A transition joint is required where the round rubber seal, which runs a majority of the gate length, intersects the block seal in the pintle region. This transition involves a sliding lap joint between the round seal and the block seal as shown in Figures 9.20 and 9.21. The round rubber bottom seal moves with the gate while the block seal in the pintle region is stationery. As the gate and round rubber seal rotate into the mitered position, the round rubber seal extension slides into the lap joint cut-out in the stationery block seal. The length of this lap joint is typically 3 in. long. In the case of the 4 in. round rubber seal, the cut-out into the block seal has a radius that is 1/8 in. larger than the radius of the round seal.

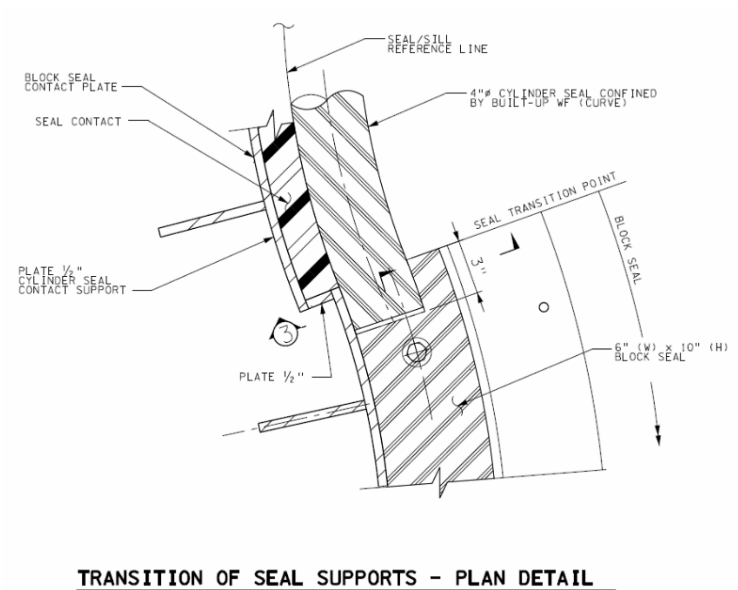


Figure 9.20. Plan View of Transition Between Round Seal and Block Seal (Kentucky Lock)

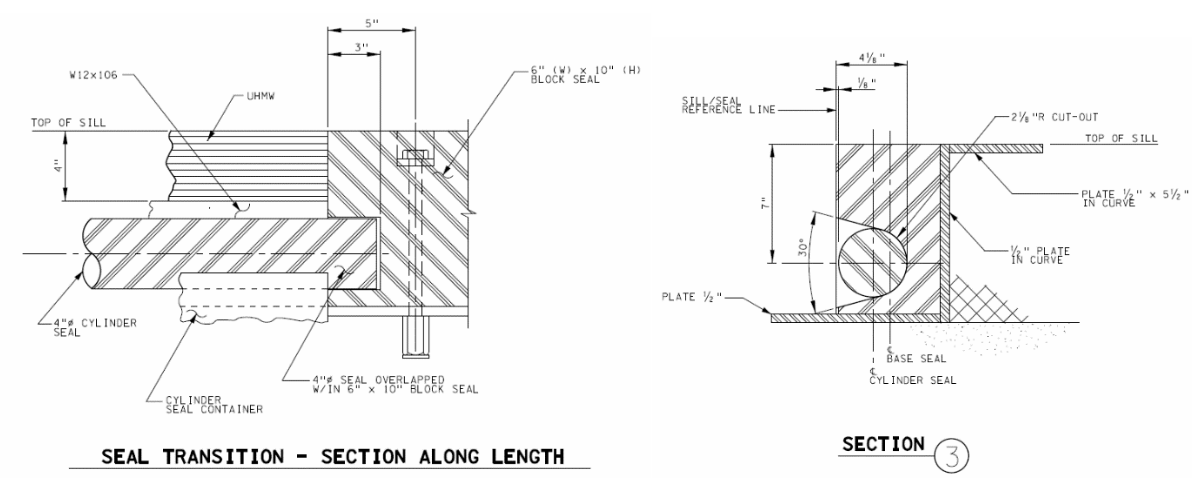


Figure 9.21. Sections of Transition Between Round Seal and Block Seal (Kentucky Lock)

• Bottom J-Bulb Seal. Near the pintle, as an alternative to the block seal, a rubber J-bulb seal can be used to seal the base of the gate leaf against the bottom sill as shown in Figure 9.22. Round rubber seals do not perform satisfactorily around sharp bends due to their large diameter. Locations such as where the bottom sill bends around the base of the pintle are ideal for the use of a J-bulb seal. Although care should be taken where the J-bulb is bent around a corner or radius. On the example shown in Figure 9.23, the J-bulb in the curved portion is pulled in tighter to the gate than the straight portion. In other words, the offset from the front face of the J-bulb seal to the gate changed from the straight portion to the curved portion and required shimming of the seal flange behind the clamping bar in the curved portion to maintain a consistent offset to match the sill.

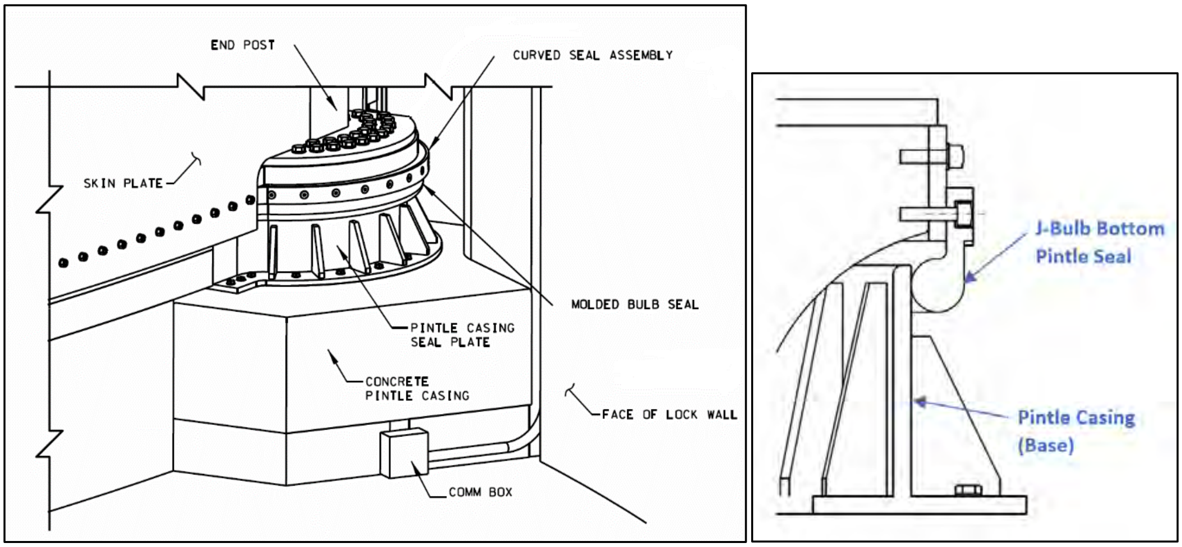


Figure 9.22. J-Bulb Bottom Seal at Pintle Where Seal Is Below Bottom Girder Height (The Dalles Lock)

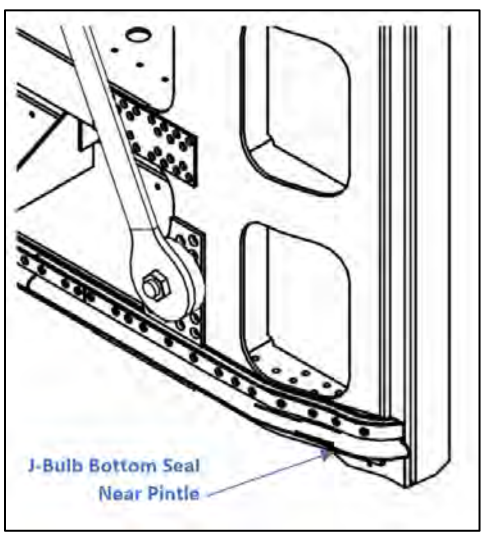


Figure 9.23. J-Bulb Bottom Seal Near Pintle Where Seal is at Bottom Girder Height (Troy Lock)

9.4.1.19.12 How a J-bulb seal is mounted can greatly affect its performance. J-bulb bottom seals may be mounted vertically or on an incline. Overall, a J-bulb gate-mounted seal at an angle riding up over the sill is a better detail than the typical vertically mounted J-bulb seal. Problems with J-bulb seals arise due to the seal's thinness and flexibility. J-bulb seals tend to vibrate the gate as the gate gets close to the miter position. Debris can also get stuck behind the J-bulb seal if mounted on the gate, preventing it from deflecting or sealing against the sill. Also, for vertical J-bulb seals, the fit-up with the sill varies depending on the seasonal temperatures.

9.4.1.19.13 The vertically mounted flexible seal mounted to the embedded sill as shown in Figure 9.24 can only be serviced when the lock is dewatered. Vertically mounted J-bulb seals used as a bottom seal along the length of the gate attached to the bottom girder will not be able to maintain proper sealing as the bottom girder horizontally deflects (see inclined seal for improved performance). Typically, the preference is to have the seals mounted to the gate when possible because it is easier to remove the gate than to dewater the lock for maintenance.

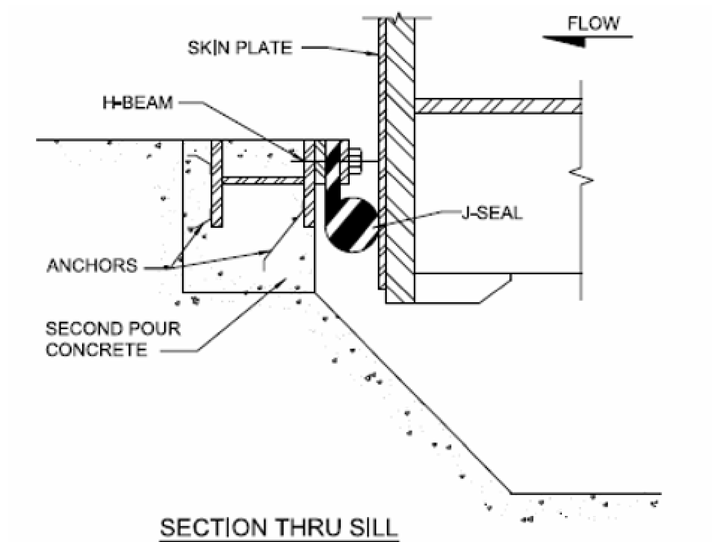


Figure 9.24. J-Bulb Seal

9.4.1.20. Operating Strut Connection.

9.4.1.20.1 Hood. Plate 9.33 shows typical details for the hood-type connection. This connection is typically attached above the top girder, but can also be below the top girder. The operating strut is connected to the hood by two pins: one larger vertical pin and a smaller horizontal pin through the vertical pin, forming a universal joint to minimize moment in the strut. The hood supports the pins through a set of plates and tees. Moment caused by the offset between the strut and the girder web is resisted by a series of diaphragms that span between the top two girders.

9.4.1.20.2 Vertical-Shaft. The vertical-shaft-type connection is somewhat similar to the vertical pin of the hood-type connection. However, in this case, the vertical pin extends between the top two girders to resist the moment caused by the offset between the strut and the top girder web, and the pin extends as a cantilever above the top girder. This system is simpler than the hood type. However, the cantilevered length of the pin above the top girder can make design difficult.

9.4.1.20.3 Direct-Acting. This connection has normally been used only for direct-acting-cylinder machinery. It is bolted directly to a section of the upstream flange of the top girder. The flange thickness is increased near the connection and stiffeners help distribute the load into the girder. The operating strut is connected by the same universal-type vertical and horizontal pins as the hood connection (Plate 9.33) for typical arrangement of this type of connection. The direct-acting connection is the simplest of the three types, but it might require a wider wall recess if used with machinery other than the direct-acting cylinder. This is due to having to move the machinery back from the face of the lock wall. If the machinery is kept in the same position as the hood or vertical-shaft connections, the strut would have to be reduced in length, thereby creating potential interference between parts of the strut.

9.4.1.21. Miter Guide. Proper miter between the gate leaves is critical to miter gate performance. The miter guide is used to bring both leaves of the gate into the mitered position simultaneously, facilitating seating of the miter blocks. The guide assembly may be located on the upstream side of the top girders or on top of the top girder web of each leaf. The miter guide is made up of two major components: the roller, mounted on an adjustable bracket, and the two-piece, adjustable, v-shaped contact block with its support. The roller bracket and the contact block are connected to their supports with a series of bolts to permit field adjustment. Steel shims or epoxy filler may be used to secure the miter guide components in their final positions. Plate 9.25 shows typical details.

9.4.1.22. Miter and Recess Proximity Sensors. Miter and recess proximity sensors should be installed per EM 1110-2-2610 for proof of miter and recess. Closed-circuit television cameras at miter may also be helpful to verify proper miter. USACE Divisions may have specific interlock guidance that may need to be met as well.

9.4.1.23. Walkway. Miter gates should be equipped with a walkway across the top of the gate. Walkways and handrails must be designed according to EM 385-1-1. A minimum width of 4 ft is recommended, with the top of the walkway flush with the top of the lock wall. Walkway widths vary with the intended purpose of use. Depending on the relative elevations of the lock wall and the top girder, the walkway can rest directly on the top girder or be elevated and supported off the top girder as it is in most cases. The walkway beams are usually constructed of structural angles which act as a toe board to prevent objects from falling off.

9.4.1.23.1 Walkways are typically made out of weather-resistant materials such as galvanized steel, coated steel, aluminum, or stainless steel with consideration for galvanic corrosion where dissimilar metals are in contact. The slip-resistant walking surface is typically open grating—either galvanized/coated steel or fiber reinforced polymer. Grating panels are sized so they can be easily removed for maintenance and inspection. Usually, walkways are detachable (bolted) in order to be removed for maintenance or replaced after a vessel impact. Framing members above lifting points and operating strut arm connections should be made detachable for maintenance.

9.4.1.23.2 The end of the walkway adjacent to the lock wall is made on a radius, typically from the center line of the gudgeon pin to the outside edge. It is recommended to hinge this walkway section on the gate end and rest the other end on the lock wall. This ensures the walkway is flush with the top of lock wall despite minor discrepancies in walkway height during fabrication.

9.4.1.23.3 Instead of a walkway, a maintenance bridge-way may be provided over (and supported by) the lower miter gates to accommodate vehicular access or a mobile crane, thereby eliminating the frequent need for a floating plant for many maintenance and repair operations.

9.4.1.24. Fenders and Bumpers. Miter gates should be equipped with fenders to protect the gate from impact and to prevent damage by passing tows when the gate is in the recess. Fenders can be wood, rubber, metal, or other materials installed on the downstream flanges of all horizontal girders subject to impact. Generally, this extends from a point at or slightly below minimum pool up to a point approximately 6 ft above the maximum pool during lock operation. Consideration should be given to placing fenders 2 ft on center vertically in areas where heavier tows are likely to cause considerable damage to gates. Vertical beams spanning between horizontal girders should be used to support the extra fenders. See Lampo and Foltz (2017) and PIANC InCom WG Report No. 154 for more in-depth coverage of fenders and considerations.

9.4.1.24.1 Bumpers are fastened to the wall of the recess, or to the gate, to cushion any impact between the gate leaf and the wall as the gate is opened. There should be at least two bumpers, possibly more on tall gates. Where ice buildup on the recess occurs, it is recommended to attach the bumper to the gate. The location of the bumper(s) along the length of the gate is more toward the miter end—sometimes at the last vertical framing member. Along the height of the gate, the bumper(s) are typically placed toward the top of the gate at the horizontal girders. For tall gates, bumper(s) may be needed at the bottom of the gate as well.

9.4.1.24.2 Rubber D-shaped seals mounted on top and bottom steel angles with slotted holes for adjustment have worked well.

9.4.1.25. Gate Latches.

9.4.1.25.1 Latches should be provided to hold each leaf in the recess against temporal hydraulic loads and for emergencies. A single latch at the top of the leaf is normally sufficient. Taller leaves typically have two latches: one on top and one lower on the leaf. Where the lock is used as a floodway during high flows, additional latches may be required. Latches should be constructed so the leaf is held snug against the bumpers and vibration is kept to a minimum. If a lower latch is used, the latch structure attached to the leaf should be designed so it can be removed by divers in case the latch fails with the leaf tied to the wall.

9.4.1.25.2 A latch or tie should also be provided to tie the leaves in the miter position, to pull the leaves together to reduce vibration. Plate 9.27, Plate 9.28, and Plate 9.29 show a variety of latching devices.

9.4.1.26. Corrosion Protection.

9.4.1.26.1 The recommended paint system for miter gates in fresh water is the USACE aluminum vinyl 3-A-Z system. This system has more ultraviolet resistance and less permeability than the USACE vinyl 3-E-Z system. However, if abrasion is of utmost importance the 3-E-Z system may be preferable since it is more abrasion resistant than the 3-A-Z system. The recommended paint system for miter gates in salt or brackish water is the USACE coal tar epoxy 6-A-Z system. Polyurethane coating systems are more brittle than the vinyl paint systems and do not perform as well.

9.4.1.26.2 For more coating information, see UFGS 09 97 02 and ERDC/CERL Paint Technology Laboratory. Depending on the water conductivity and corrosiveness, cathodic protection may be needed. For guidance on cathodic protection, please consult the USACE Corrosion Control and Cathodic Protection Technical Center of Expertise (TCX) at Mobile District.

9.4.1.27. Structural Health Monitoring and Instrumentation. The goal of structural health monitoring and instrumentation is to provide engineers and operations personnel with an effective tool for lock gates that will provide advance notice of deteriorating conditions before they hamper operations or safety.

9.4.1.27.1 The USACE Structural Monitoring and Analysis in Real Time (SMART) Gate program has instrumented several miter gates to monitor its performance over the course of the gate's life. The first gate was instrumented in 2003 at Greenup Lock on the Ohio River. The types of instrumentation used include full-bridge strain gages, bi-axial tilt-meters, tri-axial accelerometers, temperature sensors and water level sensors.

9.4.1.27.2 The data is fed into a data logger at the lock site and transmitted to a centralized data repository. The data can be accessed through a web application in real time. The web application is capable of sending email alerts and generating reports. Since the first generation of SMART gate, efforts have been made to target components or behaviors that indicate or predict operational problems.

9.4.1.27.3 A recommended best practice for miter gate instrumentation is using load pins for the rear anchorage pins (Figure 9.25). This is a good way to be able to monitor anchorage forces and behavior. If continuously monitored, it would also provide a fatigue cycle history. The cost of load pins is relatively small. The advantage of load pins over strain gages is that load pins measure absolute force as opposed to relative stresses based on the in situ stress of when a strain gage is installed.

9.4.1.27.4 The cost of data acquisition and monitoring depends on the level of data collected. If data is collected continuously, it can be more complicated with permanently connected wires and data transmission systems. If data is collected manually and intermittently, it is easier and less costly. For more information on structural health monitoring and load pins, contact the USACE INDC.

Figure 9.25. Load Pin

9.4.2 Arch Type and High Lift Miter Gates. A horizontally framed gate can use arches instead of girders. This style of miter gate has been used for very tall, high-head gates on the Tennessee, Columbia, and Snake rivers. The arch is more efficient at minimizing bending moments. Other than the arches, the basic components are the same as for a gate with girders. Plate 9.30 shows a typical horizontal arch layout.

9.4.2.1. High lift miter gates have many unique design considerations all to themselves. For miter gates with high lifts, the sensitivity of gate dimensional tolerances and proper alignment become more important due to higher hydraulic loading. High water pressure requires the gate to be designed against leakage since it could cause vibration problems if the leakage is at the bottom of the gate or eroding the bearing blocks. Due to the large gate dimensions, adjustment of the gate diagonals is sensitive to air temperature and solar loading (thermal expansion and contraction).

9.4.2.2. The larger and heavier gates will require a larger and heavier crane. Hence, crane access and surcharge loading at the site need to be considered. Higher head gates are tall and bending of the gate as a simply supported structure should be considered during the erection process to ensure the gate and pick points are not overstressed.

9.4.2.3. Thermal effects also need to be accounted for, given the height of the gate can vary with both seasonal and daily conditions. For example, the gate may be shorter during cooler temperatures at night and expand the next day under exposure to the sun. This may impact detailing practices at the gudgeon pin (anchorage) and also along the gate seals.

9.4.3 Vertically Framed Miter Gates. A vertically framed gate resists the water pressure by a series of vertical girders. The girders span from the sill at the bottom to a horizontal girder at the top. The top girder spans between walls similar to a horizontally framed gate. The lower ends of all vertical members are supported directly by the sill, with a bottom girder acting to transfer the concentrated loads into a more uniform reaction on the sill. Vertically framed gates are recommended for wide, shallow gates, usually when the height-to-width ratio of a leaf is less than about 1H:2W. Due to the simpler boundary conditions of only having one wall contact block to adjust at the top girder and above the waterline, vertically framed gates are easier to maintain and can be replaced in-the-wet without lock dewatering. Plate 9.4 shows an example of a vertically framed miter gate.

9.4.3.1. Skin Plate and Intercostals. The analysis of skin plate and intercostals is the same as for horizontally framed gates, except the intercostals span horizontally between girders. The skin plate is generally on the downstream side of the vertical beams, to minimize uplift forces on the gate and to protect the vertical beams and diagonals from barge impact when the gate is in the recessed position. However, that position also maximizes opportunity for silt to accumulate above the bottom girder. A way to reduce silt buildup on the bottom girder is to hold the upstream flange below the girder web as show in Figure 9.26.

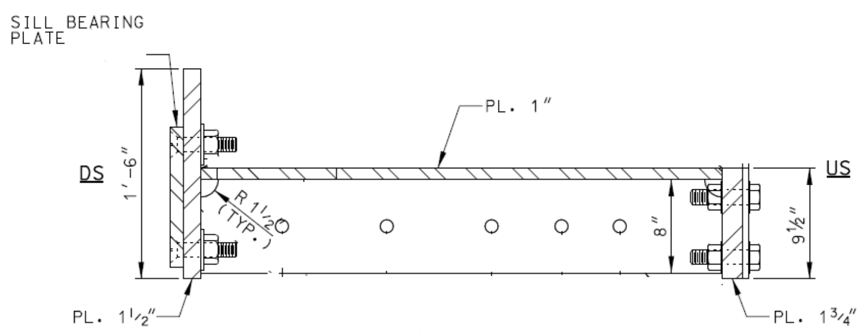


Figure 9.26. Bottom Girder with Upstream Flange Held Below Girder Web

9.4.3.2. Vertical Girders. Vertical girders function as vertical beams and serve as support members for the top and bottom girders. They are located so that practically all vertical forces caused by the diagonals are carried by the vertical girders. The vertical girders and the bottom girder are normally the same depth so as to simplify framing and to make the bottom girder flanges more directly effective in taking the components of the diagonals.

9.4.3.3. Vertical Beams. Vertical beams also span between the top and bottom girders and are located between the vertical girders. Spacing of the beams is determined largely by support requirements for the skin plate system, with a normal spacing being at the quarter points between vertical girders. The beams are assumed to be simply supported top and bottom, with simple moment and shear dictating beam size.

9.4.3.4. Top Horizontal Girder. The top horizontal girder is designed as a fracture critical member to withstand a simultaneous load of water force and boat impact. The top girder design is essentially the same as that for girders in a horizontally framed gate. The reaction of the top girder is transmitted through steel bearing blocks at each end of the girder. These blocks are similar to the bearing arrangement for horizontally framed gates, having the same convex and concave faces and the same adjustment.

9.4.3.5. Bottom Horizontal Girder.

9.4.3.5.1 Under normal conditions, the bottom girder does not function as a girder, but rather as a member to transfer the concentrated vertical beam and girder loads into a uniformly distributed horizontal force on the sill. For most gates, the bottom girder center line is located approximately 4 in. below the top of the sill to provide sufficient bearing surface between the girder and the embedded metal.

9.4.3.5.2 The girder is also checked for sufficient capacity to carry the reaction from any vertical beam or girder to adjacent beam or girder points if irregularities or obstructions between the sill and bottom girder prevent bearing at a vertical beam location. The minimum effective length for this should be twice the vertical beam spacing. The bottom girder is also in compression, due to cantilever action of the gate leaf and prestress in the diagonals. The girder is also subject to vertical bending due to uplift pressures or silt load, depending on location of the skin plate.

9.4.3.6. Diagonals. Design of the diagonals for a vertically framed gate is essentially the same as that for a horizontally framed gate. If the skin plate is on the downstream face of the leaf, diagonals are positioned on the upstream face. The number of panels of diagonals depends on the spacing of vertical girders. Usually leaf dimensions are such that three sets of diagonals are used. Due to flexibility of a vertically framed gate, top-mounted tensioning mechanisms are recommended on all diagonals to allow for easier adjustments, preferably above the waterline.

9.4.3.7. Miter and Wall Quoins.

9.4.3.7.1 The bearing block arrangements are similar to a horizontally framed gate, except for the vertical height of the bearing area. The wall quoin of a vertically framed gate is about 24 in. high and 20 in. wide. It should be of sufficient size to maintain bearing on the concrete to approximately 600 psi or less, so that cracks in the concrete around the corner of the gate recess will be kept to a minimum.

9.4.3.7.2 The beam is generally placed horizontally in first-pour concrete with the bearing being detachable, with provisions for adjustment in and out and laterally side to side. Total in and out adjustment of the wall quoin is typically about 5/8 in. Total lateral side to side adjustment of the wall quoin is approximately 1/2 in. in each direction. The gate contact blocks can be adjusted by the use of finger shims inserted from the downstream side.

9.4.3.8. Seals. Seals used on a vertically framed miter gate are shown in Figure 9.27. Since there is not a continuous full-height quoin block to serve as a seal between the leaf and the wall, a quoin seal is attached to the downstream face of the gate which mates up with the downstream side of the gate recess in the mitered position. Similarly, at the miter end, a full-height miter seal attached to one leaf mates up against a seal bearing plate on the opposite leaf.

9.4.3.8.1 The omega style seal is the recommended seal for both the quoin and miter seal. It is much more durable and can withstand more abrasion than the J-Bulb seal. For improved durability, 3/8” thick plate is embedded in the stem with 3-ply nylon above and below it to prevent the seal from being ripped off the connection bolts during abrasion (Figure 9.28).

9.4.3.8.2 The bottom seal is formed by the contact between the bottom girder and the embedded metal of the gate sill. A metal bearing plate is attached to the downstream flange of the bottom girder and this also acts as a seal plate. At the end of the leaf adjacent to the pintle, a steel projection that is part of the pintle base extends upward to meet up with the quoin seal to seal between the leaf and pintle base.

9.4.3.9. Pintle Base. The recommended pintle base for vertically framed miter gates is a floating style pintle that has successfully performed on the Upper Mississippi River (Figure 9.29). The embedded pintle base fits the radius of the pintle shoe on the quoin side, but then has a 2 in. gap between the pintle shoe and the stop on the upstream side of the embedded pintle base. This allows for the gate and pintle to shift upstream if the gate closes on a sill obstruction.

9.4.3.9.1 The typical problems with floating pintles on horizontal gates causing wear and quoin block wear do not apply here because there are no quoin blocks at the bottom of the gate. Similar to floating pintles, a calculation must be performed to show that when the pintle has been dislodged due to obstruction, it can reseat itself as the gate swings (i.e., the horizontal reaction at the pintle base due to gate weight must be greater than the static frictional force between the pintle shoe and pintle base).

9.4.3.9.2 A terminology note: sometimes the terms pintle shoe and pintle base are reversed as to which one is the embedded piece. For consistency in this guidance, pintle shoe is the bottom piece connected to the pintle ball and pintle base is the piece embedded in concrete.

9.4.3.9.3 Other Components. Other components such as pintles, strut connections, walkways, fenders, and latches are similar to those used for horizontally framed gates.

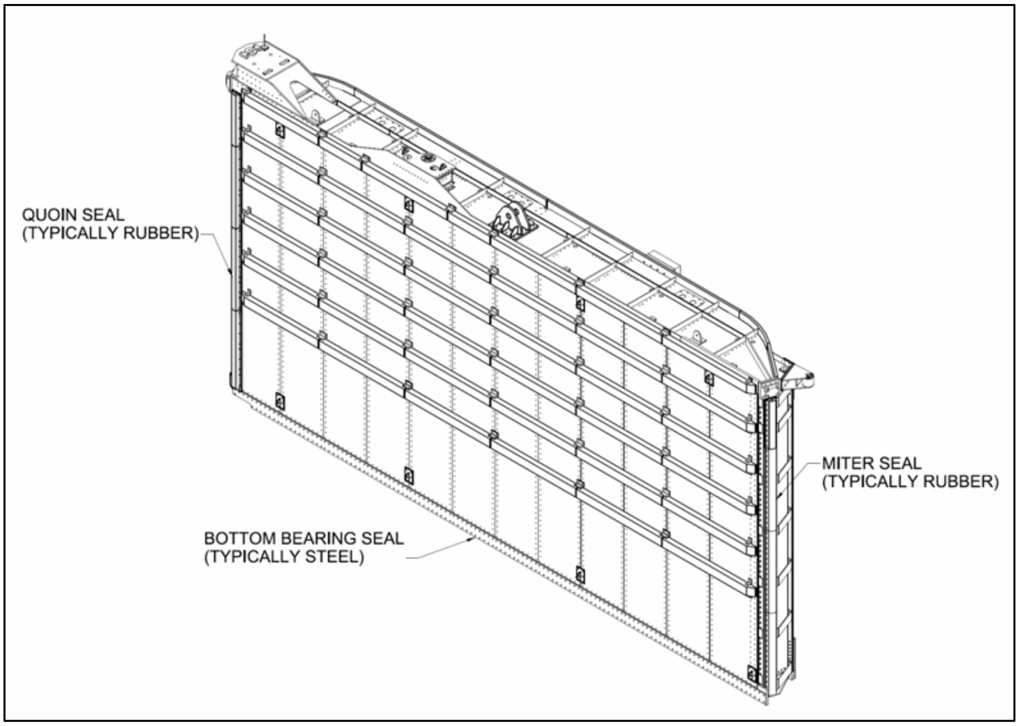


Figure 9.27. Seals on Vertically Framed Miter Gates (Upper Mississippi Locks)

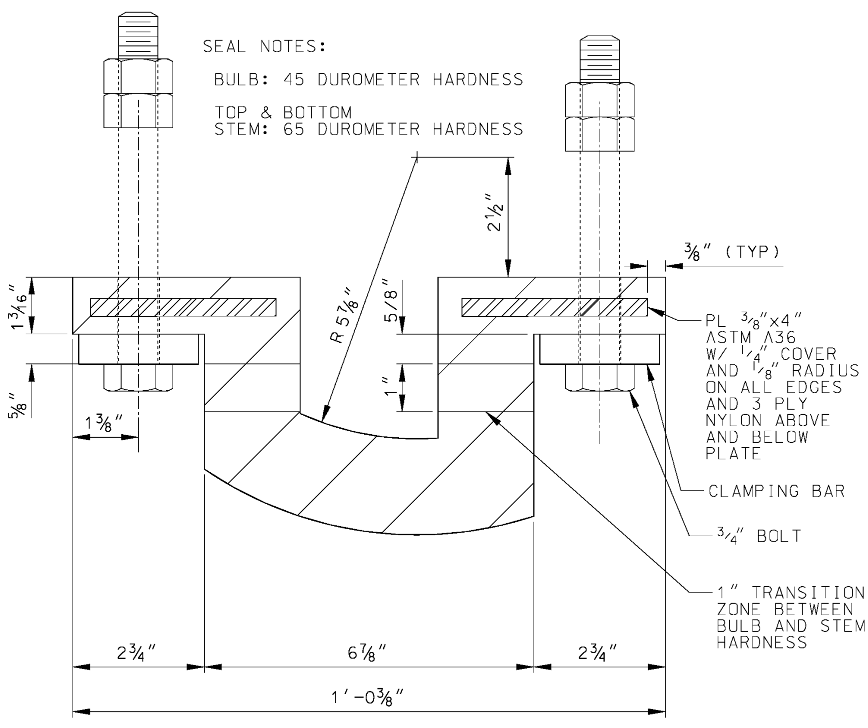


Figure 9.28. Omega Seal Detail

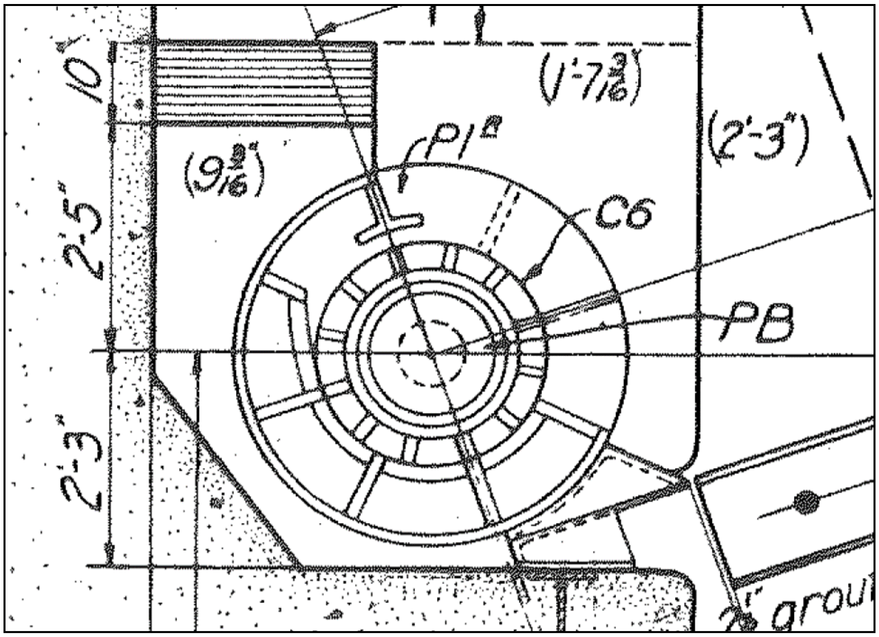


Figure 9.29. Plan View of Pintle Sitting in the Embedded Pintle Base
with Upstream Toward Top and Miter Sill to the Right

9.5 Erection and Testing. Proper fabrication, installation, adjustment, and commissioning of miter gates are of as equal importance as design to ensure successful performance. Miter gates should be completely shop assembled, if size permits, with adjoining pieces fitted together to ensure satisfactory field connections. The tolerances should follow the guidelines of UFGS 055920 or project requirements. Rubber seals should be fitted and assembled to the gate leaf in the shop, with holes drilled to match the seal supports on the gate leaf and then removed for shipment. Before disassembly of the leaf each piece should be match-marked to facilitate erection in the field.

9.5.1 Before gate installation, pull contact blocks inward so they do not make premature contact, or cause interference or prying during gate installation.

9.5.2 The bottom pintle casting must be adjusted to proper elevation and position and then properly concreted in place before erection of the leaf. The bearing surface of the pintle and bushing should be thoroughly cleaned and lubricated before setting in place. After placement of the leaf, the top anchorage links should be installed and adjusted so that the center of the gudgeon pin is in vertical alignment with the center of the pintle. Check alignment in the mitered and recessed positions. There are competing fit-up requirements such as trying to make the miter and quoin block contact, as well as the bottom seal to meet the sill.

9.5.3 Procedure for Prestressing Diagonals. See Appendix C, sections C.2 and C.4, for establishing pretension values for prestressing diagonals. There are different procedures for stressing diagonals, this being just one. Use Figure 9.30 with this procedure:

9.5.3.1. With all diagonals slack, adjust anchorage bars so quoin end is plumb and bottom girder is horizontal. Pintle shoe must be fully seated against the back of the pintle base.

9.5.3.2. Lubricate the nuts on the diagonals so they can turn easily.

9.5.3.3. Place rosettes for strain gages on all diagonals a minimum of 20 hours before prestressing unless approved quick-setting cement is used.

9.5.3.4. Without the restraint of any guides or jacks, the leaf will deflect in a negative direction under its own dead load weight. Measure this deflection.

9.5.3.5. Tighten the diagonals to the design values.

9.5.3.6. During the prestressing operation use a strain gage to determine the stress in the diagonals. Establish acceptable final minimum and maximum stresses for all diagonals. An upper limit for maximum stresses may be approximately 0.55 F_y. The maximum allowable stress is 0.75 F_y.

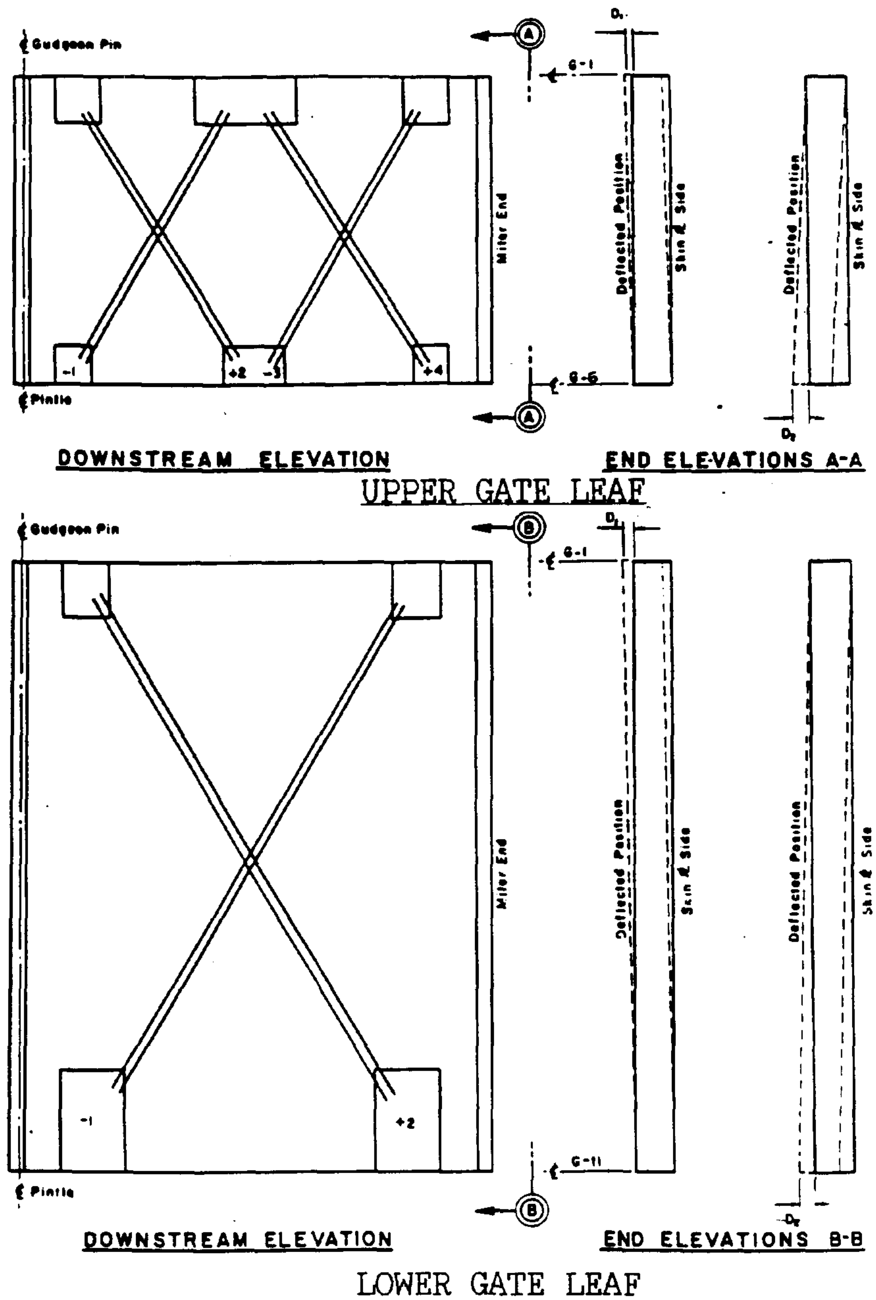
9.5.3.7. During the prestressing operation use a strain gage to determine the stress in the diagonals. The maximum allowable stress is 0.75 F_y.

9.5.3.8. After the final adjustments of the diagonals, the leaf should be plumb. A deflection ±1/4 in. will be permitted in the lower leaf and ±1/8 in. on the upper leaf. A larger tolerance is allowed for the lower leaf because it is much taller than the upper leaf. If plumb is not achieved, adjust the tension in the diagonals to achieve plumb.

9.5.3.9. After diagonals have been prestressed and final adjustments have been made to the anchorage, the fit-up between the gate leaf and sill should be checked to ensure good sealing. If installing a new sill, adjust the sill to the gate but do not place second concrete pour until after contact blocks have been set.

9.5.3.10. If the gate seal does not clear the sill or rubs too hard on the sill, the primary anchorage link needs to be adjusted to bring the gate leaf up or down. Level should be checked with the gate in recess. This should be level within 1/8” but depending on circumstances, 1/4” is acceptable.

9.5.3.11. Adjustment of the secondary anchorage link primarily affects level in the recessed position. If the quoin is not aligned well when the gate is level in the recess, a compromise will have to be met with quoin alignment and the level of the gate. For successful operation, positioning and plumbing of the vertical axis of rotation (gudgeon pin centered on pintle) is more critical than gate levelness. Once gate anchorages have been adjusted, it is recommended to fan the gate several times to verify the pintle has fully moved into the back of the pintle socket casting.



NOTE:

A deflection of the leaf is defined as a twisting of the leaf such that the miter end is out of plumb. A positive deflection of the leaf is one in which the top of the miter end is moved upstream relative to the bottom. The magnitude of the deflection is the amount by which the top of the miter end is out of plumb, as shown in the figure.

When any diagonals are tightened, they shall be taken up just to the point where all of the slack is removed and a very slight tension exists. Care shall be exercised that the amount of this initial tension is as small as possible. The slack shall be considered to be removed when the diagonal does not bow in or out from the leaf. No attempt shall be made to remove the slight vertical sag which will always exist in the diagonal because of its dead weight.

Figure 9.30. Methods for Prestressing Diagonal

9.5.3.12. Next, start the adjustment of the miter blocks.

9.5.3.12.1 The contact block retaining channels should be set to be aligned, and it should be verified that the bottom rubber J-seal is adequately on the sill. Set the rubber J-seal compensating for the change in miter point due to the linear temperature expansion of the gate leaves.

9.5.3.12.2 For example, on a 110 ft lock, it is possible that the leaves will miter approximately 7/8" further downstream in the winter if the seals are set in the summer, and the reverse if the installation happens in the winter. Adjustments should be made to the mitering device brackets so that either leaf may be mitered or opened without disturbing the other leaf.

9.5.3.13. After the miter guide is set, measure the distances between the miter block retainer channels at the top and bottom of the gate. Record these measurements and mark them on the gate. Record the measurement to the sill reference angle.

9.5.3.14. To setup the piano wire for setting the land wall leaf, divide the top channel measurement by 2, and then add the measurement for the keystock used to measure to the wire. This is typically 1/4 in. The bottom measurement should be found by measuring the channel gap, subtracting 3/8 in., dividing by 2, and then adding the measurement for the keystock. (Example: Measurement = 2 in. – 3/8 in. = 1 5/8 in./ 2 = 13/16 in. + 1/4 in. = 1 1/16 in.). After establishing this measurement, attach the steamboat ratchets to the bottom of both leaves.

9.5.3.15. Pull both leaves downstream equally until the bottom channel measurement is reduced by 3/8 in. (this 3/8 in. referred here and above is arbitrary and gate dependent, see reasoning for 3/8 in. below). This is to simulate a loaded head condition. If the blocks are not gapped at the bottom, the gates will have a tendency to spread open at the top when a head is applied. Experience has shown 3/8 in. to be suitable for the Louisville District high lift locks with gate heights ranging from 53 to 67 ft tall.

9.5.3.16. Record the measurement at the sill angle. This is typically 3/8 in. to 1/2 in. less than the measurement taken at the mitered without simulated head condition. With these measurements recorded, the gates can be released, piano wire installed on the land wall leaf, and blocks adjusted to the piano wire. Once blocks are satisfactorily adjusted to the piano wire, the gates are again mitered.

9.5.3.17. Steamboat ratchets are installed and gate leaves are pulled together to the bottom channel measurement of 3/8 in. The top and bottom of the gate are positively (mechanically) connected to keep the gates from spreading while pushing the blocks together. After completing miter adjustments, disconnect the leaves and miter several times to check that the mitering device repeatedly miters the gates. Check that the gap at the bottom between the blocks is approximately 3/8 in. Check that the gate stops are 1 in. to 1½ in. off of the sill.

9.5.3.18. Quoin blocks are set with the gates mitered but not preloaded. As discussed in paragraph 9.4.1.17, there is not currently consensus in USACE on the proper quoin block tolerance setting or methodology. Since the quoin gap setting is usually in thousandths of an inch, great care must be taken in setting and measuring quoin block adjustment. Often, quoin blocks are set in the early morning before the sun rises to avoid thermal expansion issues. It is important for the quoin gap to be uniform over the full height of the gate for uniform bearing to be achieved.

9.5.3.19. Tools used for setting the blocks can include long blade (12 in.) feeler gages. Metal foil or polycarbonate sheets of a specific thickness can be placed between the wall quoin block and the gate quoin block and the blocks adjusted tight to the material.

9.5.3.20. After adjustment of the blocks, the leaves should be swung out and zinc or epoxy filler poured between the seal blocks and the end plates of the leaves. If zinc is used, blocks and plates adjacent to the zinc must be preheated to a temperature between 200 and 250 °F, immediately preceding the pouring to prevent the zinc from cooling before it can fill the area behind the blocks. The pouring temperature of the zinc must be maintained between 810 and 900 °F to avoid volatilizing or oxidizing the metal and to ensure that it will fill the area behind the blocks. Pouring holes should be located 2 to 3 ft apart.

9.5.3.21. After completion of the gate, including prestressing of the diagonals, installation of all seals, and all adjustments the gate leaves should be swung through a sufficient number of opening and closing operations to assure the leaves are in true alignment and that necessary clearances have been provided. After this trial operation the leaves should be swung out and the second-pour concrete placed in the sill and wall quoins.

9.5.3.22. Check the operation of the operating strut machinery to ensure it has the right travel distance and pressure on the gate in the mitered position. Have an electrician experienced in setting gate limits check the operating equipment limit switches and install or adjust the miter proximity sensors. Place the second pour concrete on the sill if a new sill is being installed.

9.5.3.23. The final test should consist of operating the gate under power, by means of the permanent operating machinery, first during the unwatered condition and then after water is allowed in the lock chamber. The leaves should be operated through their entire travel a sufficient number of times to indicate that all parts and equipment are in proper operating condition. Check if diagonals loosen and retension if required.

9.6 Operating Machinery. Operating machinery for miter gates is described in EM 1110-2-2610. It generally consists of electric motors, bull gears, and strut and sector arms, or it might be a directly connected hydraulic cylinder. The primary factor affecting structural design of the gate is the force the machinery can exert on the gate.

9.7 Gate Recess Bubbler Systems. Miter gate operation can be disrupted by accumulation of ice or debris in the recess behind the gate. This prevents the gate from fully opening to pass vessels. High flow air bubblers placed in the gate recess can effectively clear the ice and debris. Standard pipe is used for the supply and distribution lines. The supply feed is from the quoin end. Proper spacing and nozzle size will ensure maximum nozzle flow for a given air supply.

9.7.1 Output pressure must be high enough to overcome hydrostatic pressure at the submergence depth, frictional losses in the supply and distribution lines, and still provide a pressure differential at the last orifice to drive the air out at the desired rate. Supply and distribution line diameters should be large enough so that frictional pressure losses along the line are small. A small increase in line diameters often results in significant reduction in frictional losses and results in more uniform discharge rates along the line.

9.7.2 Orifice diameter and spacing should be selected to maximize rates. Too large an orifice diameter can result in all the air being discharged at one end. Submergence depth will be dictated by operational limitations but should be lower than the expected depth of trash pile-up. Typical installation depths are 10 to 15 ft.

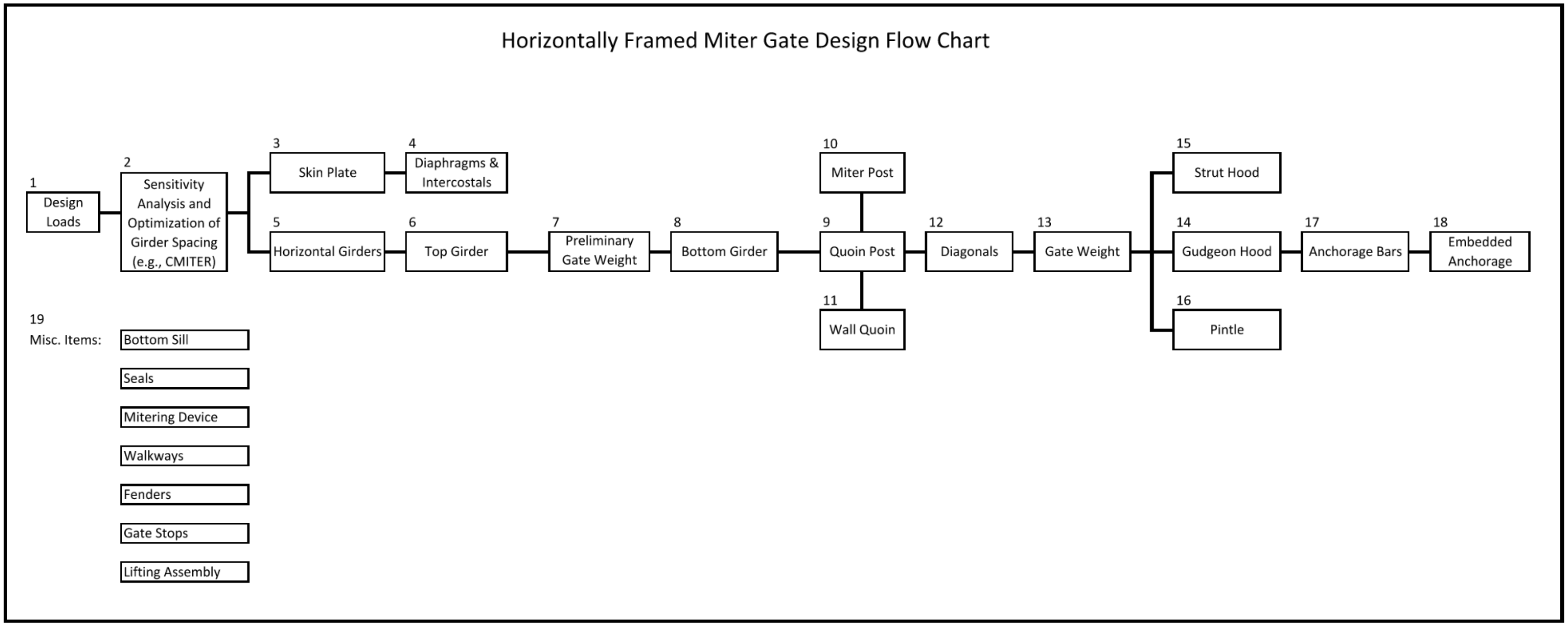


Plate 9.1. Horizontally Framed Miter Gate Design Flow Chart

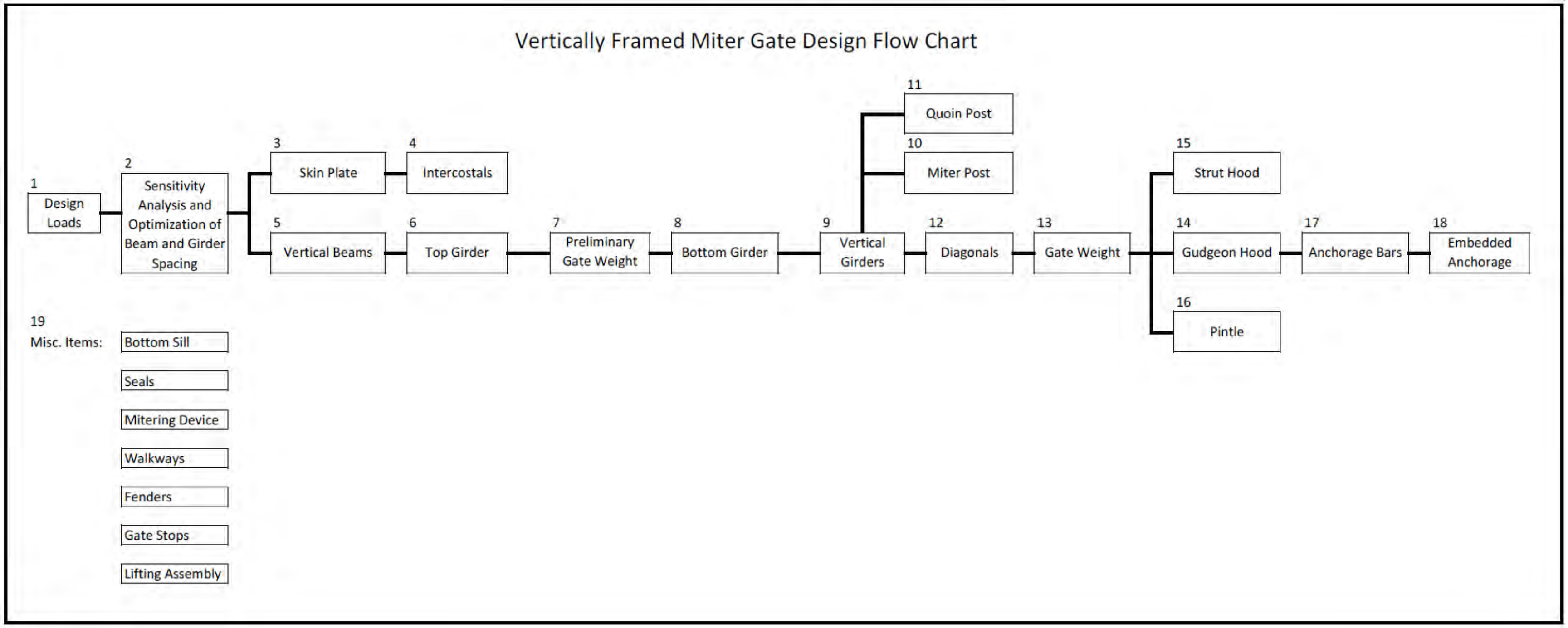


Plate 9.2. Vertically Framed Miter Gate Design Flow Chart

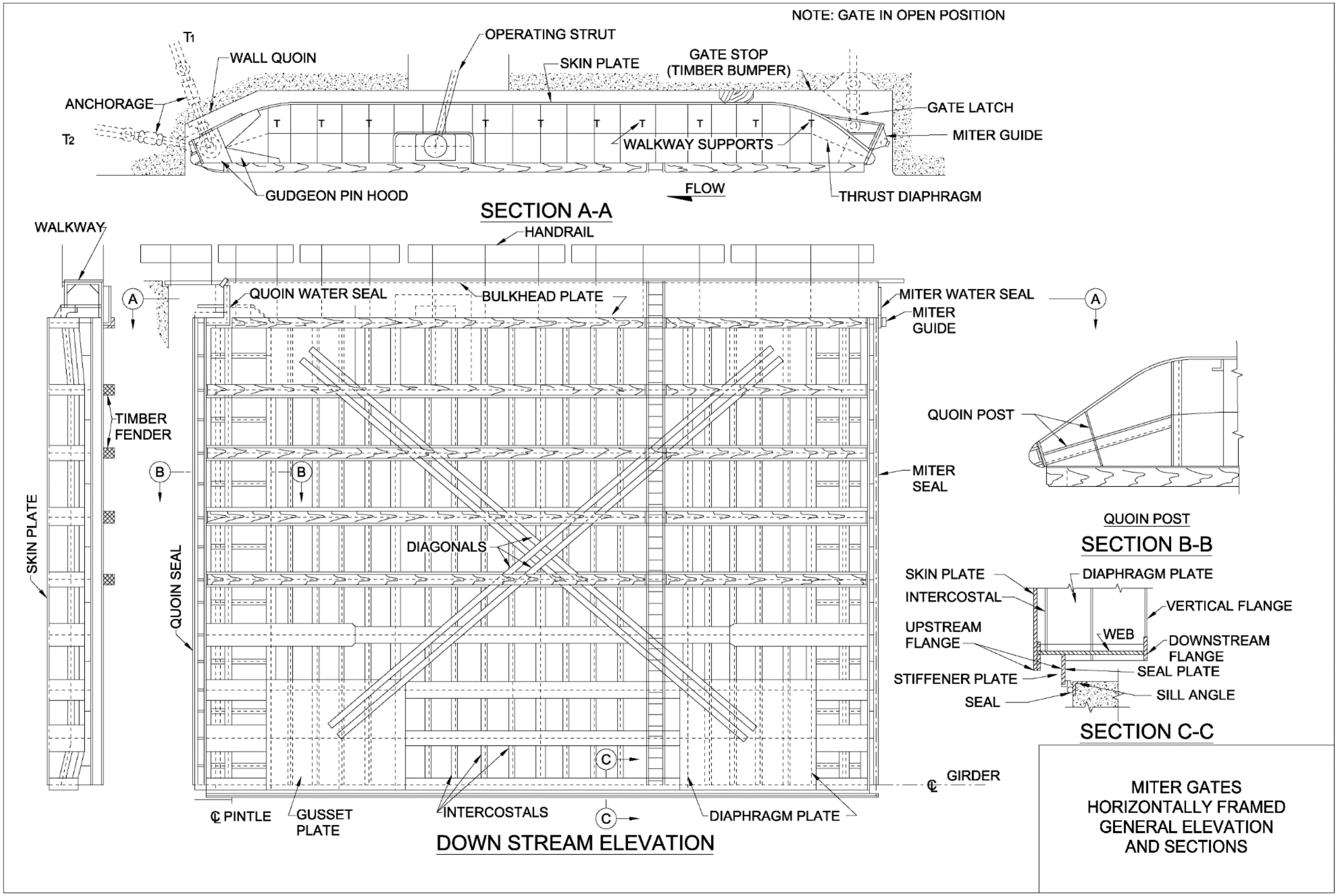


Plate 9.3. General Elevation and Sections, Horizontally Framed Miter Gates

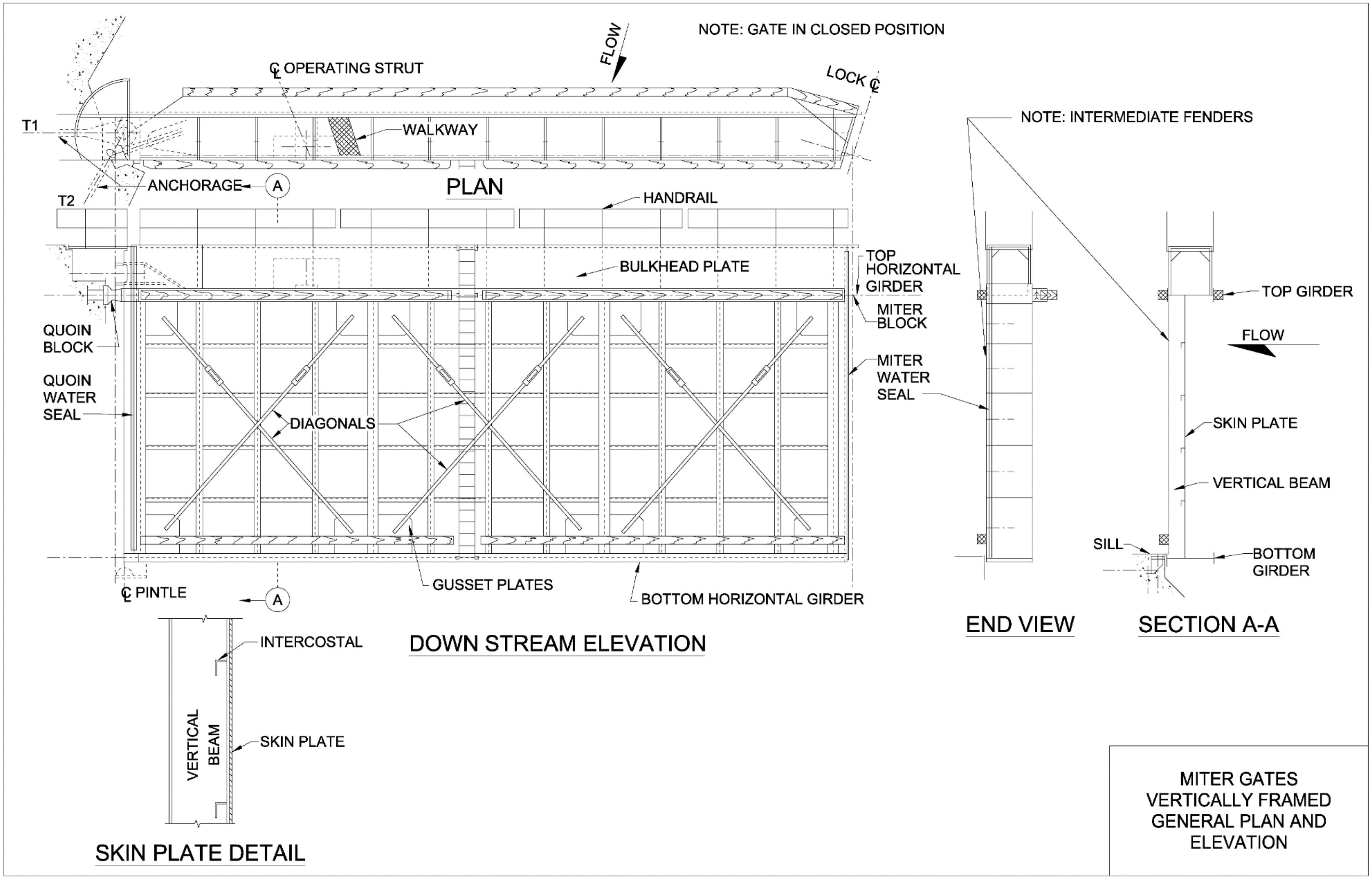


Plate 9.4. General Plan and Elevation, Vertically Framed Miter Gates

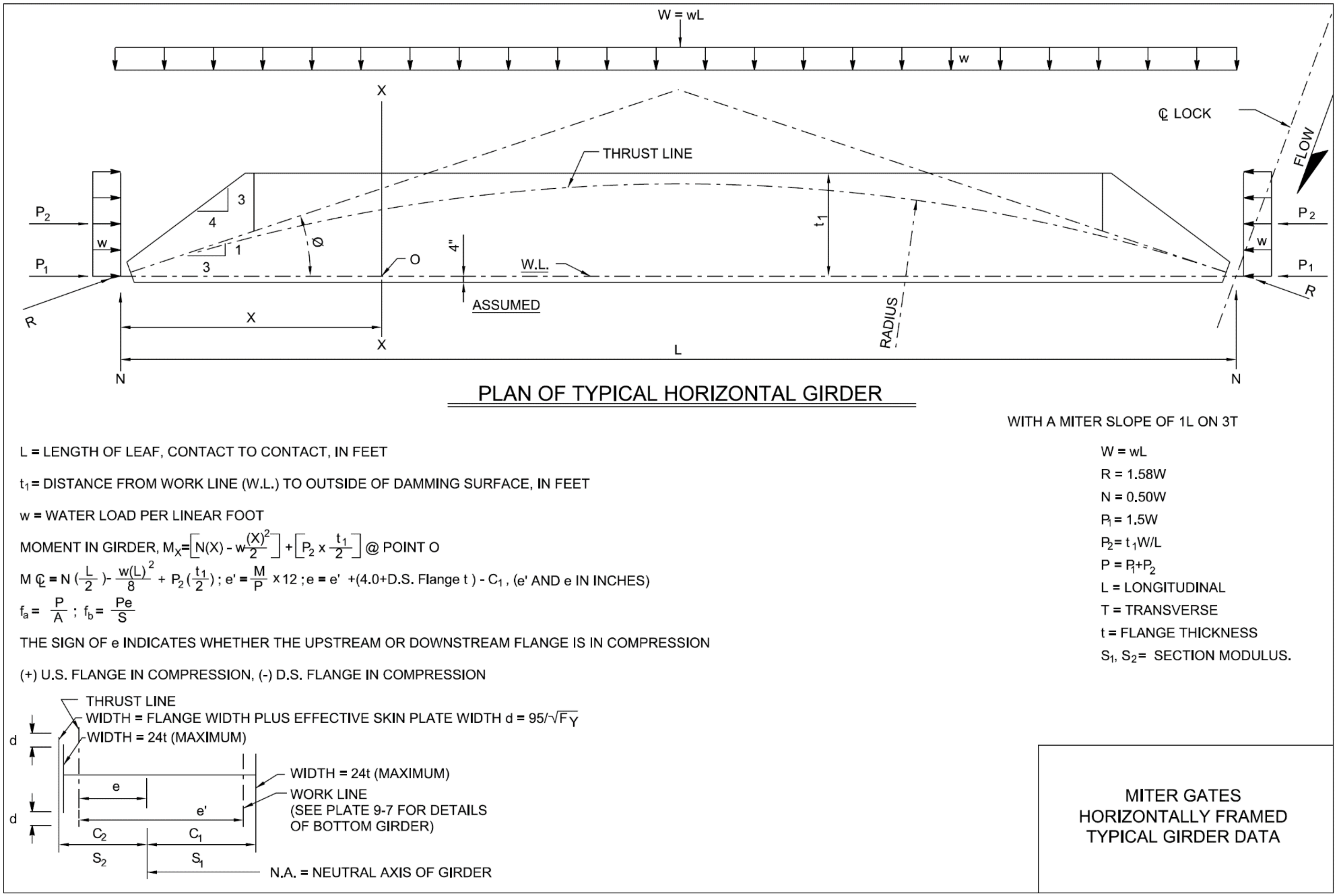


Plate 9.5. Typical Girder Data, Horizontally Framed Miter Gates

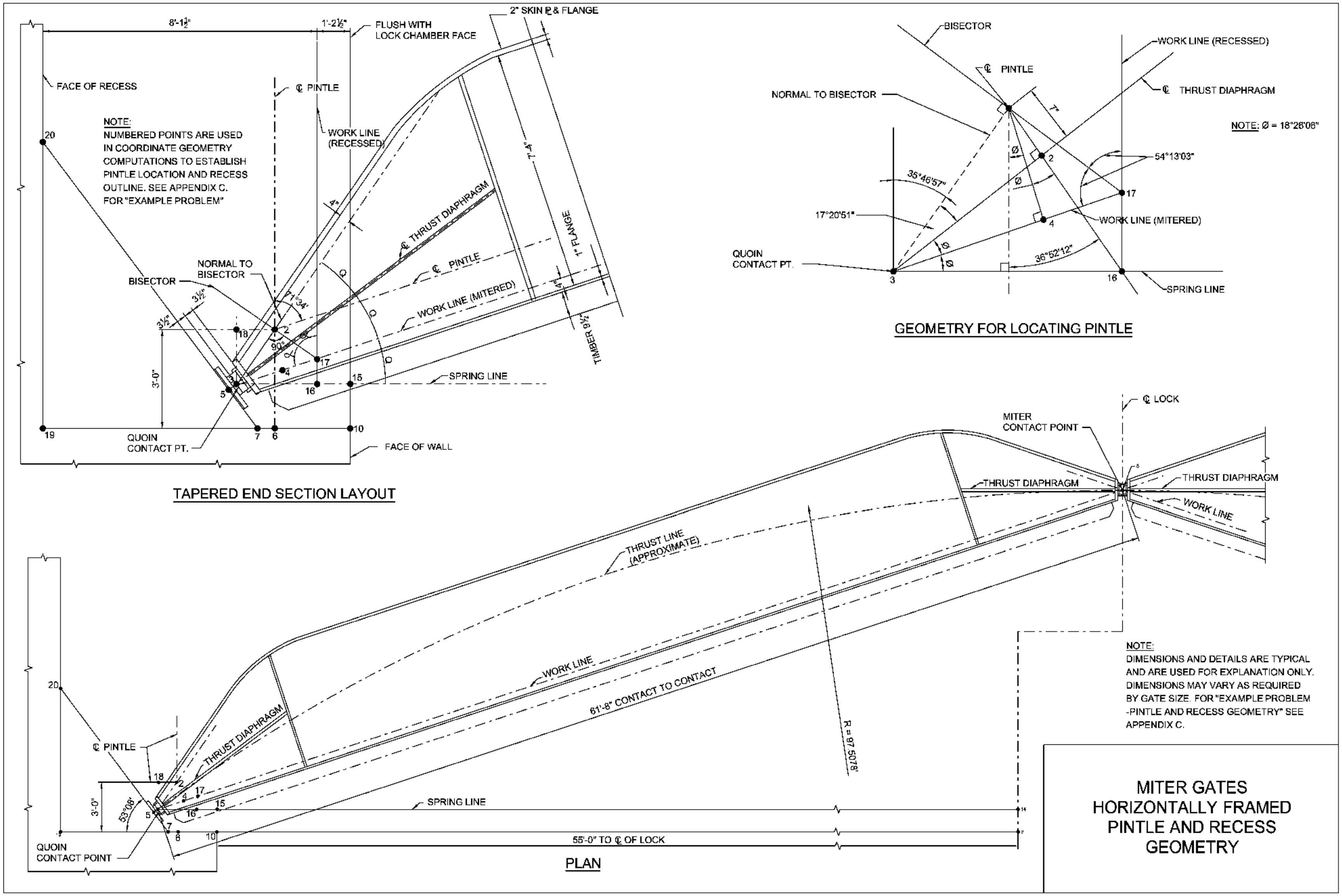


Plate 9.6. Pintle and Recess Geometry, Horizontally Framed Miter Gates

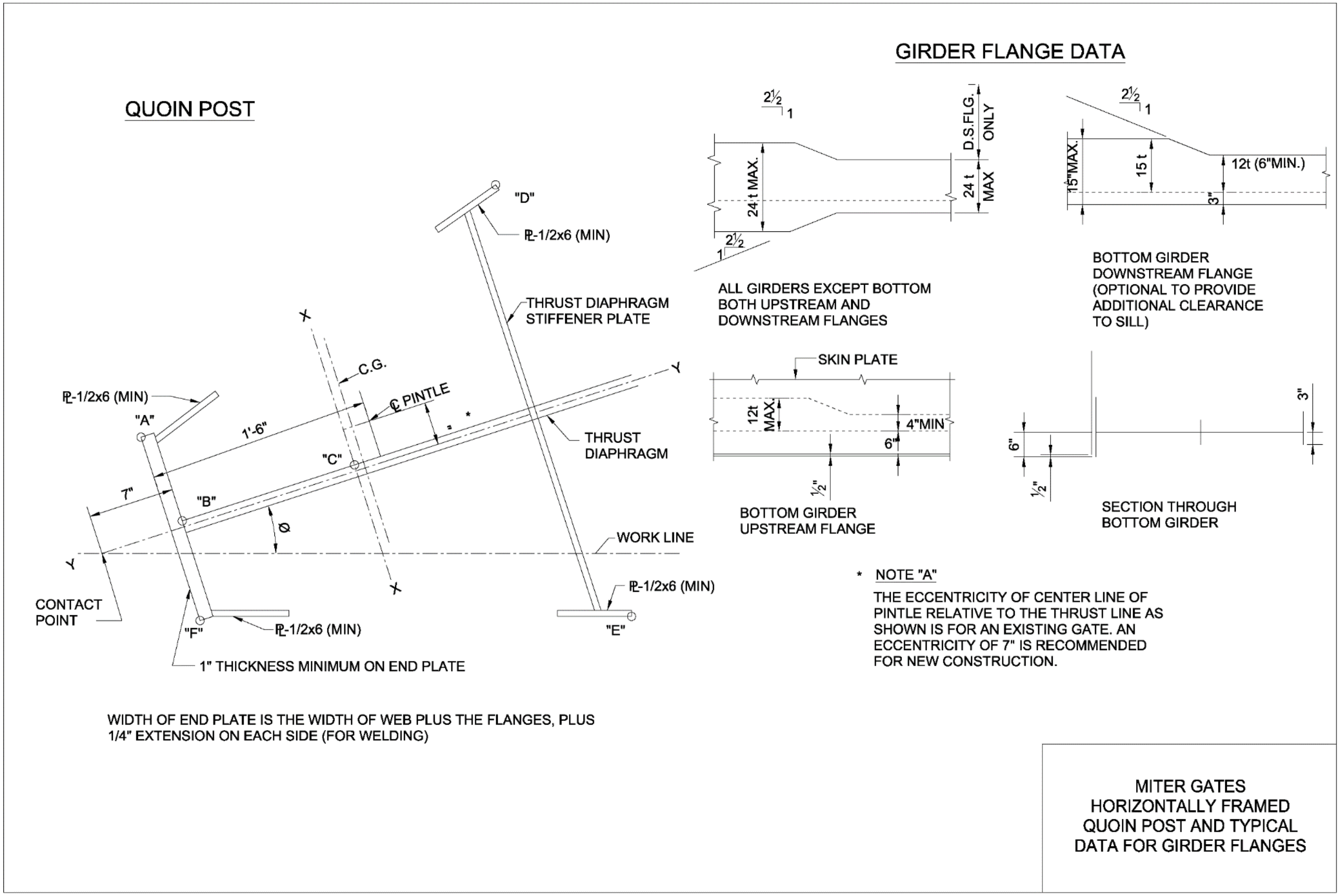


Plate 9.7. Quoin Post, Horizontally Framed Miter Gates

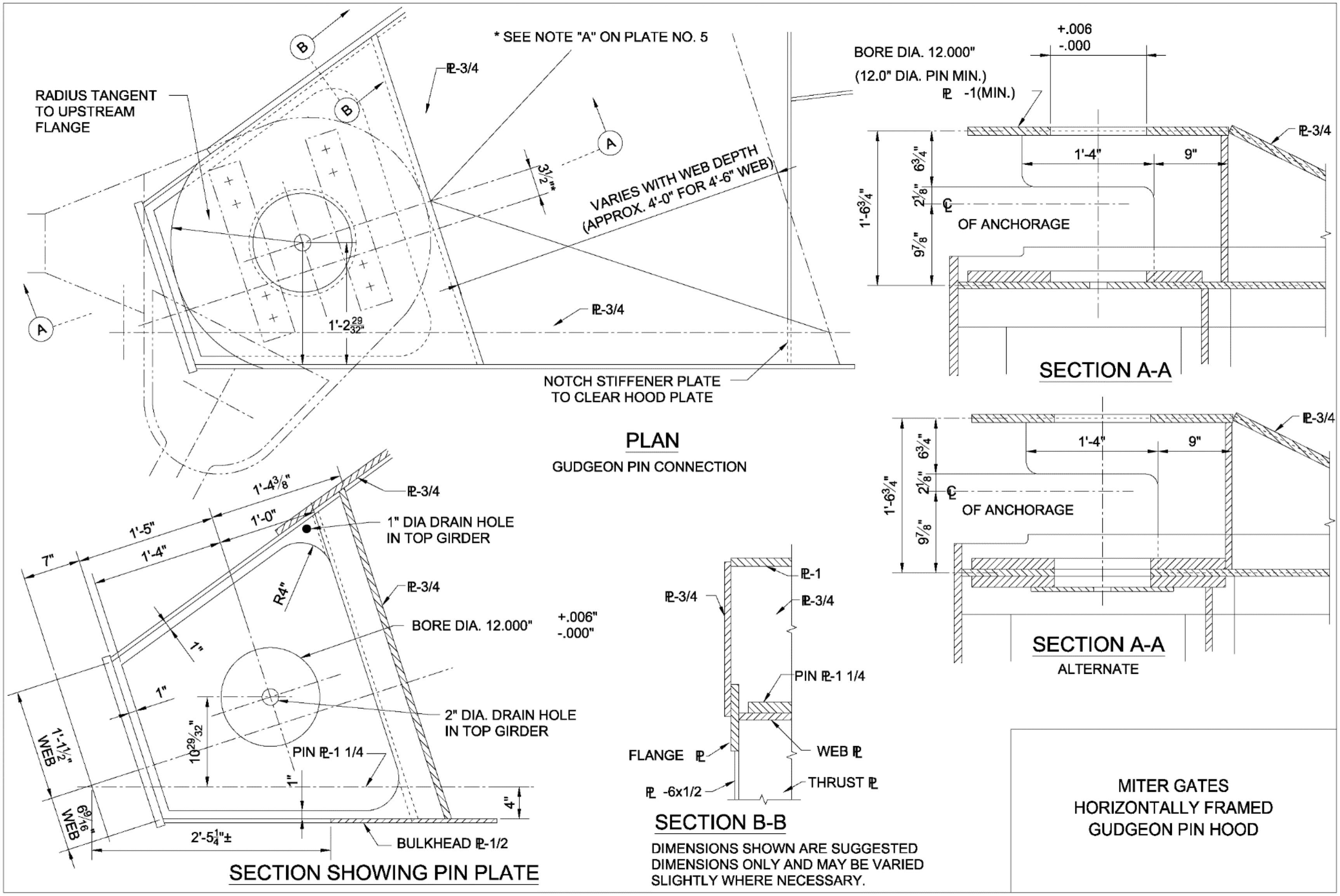
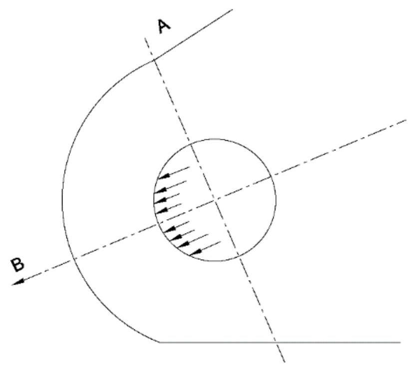


Plate 9.8. Gudgeon Pin Hood, Horizontally Framed Miter Gates

FROM: "ADVANCED MECHANICS OF MATERIALS" BY SEELY AND SMITH (1952) 2ND EDITION, PAGE 183

a = AREA OF SECTION
d = MEAN DIAMETER OF RING
W = TOTAL VERTICAL LOAD
Y = DISTANCE FROM CENTROIDAL AXIS, (Y) IS NEGATIVE TOWARD THE INNER FACE AND POSITIVE TOWARD THE OUTER FACE

w = UNIT LOAD
1/2P = WR



PLAN OF GUDGEON PIN HOOD

$R = \frac{d}{2}$

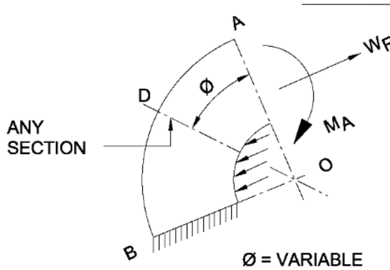
MOMENT

$M_A = \frac{1}{16}\, wd \qquad M_B = \frac{Wd}{16}$

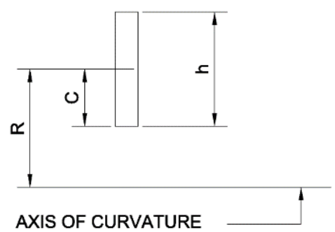
STRESS

$$S = \frac{1/2\, P \cos Ø}{a} + \frac{M}{aR}\left(1 + 1\,\frac{1}{Z}\,\frac{y}{R+y}\right)$$

$$\text{FACTOR OF } Z = (-)\,1 + \frac{R}{h}\left[\log_e\left(\frac{R+C}{R-C}\right)\right]$$



UNIFORM LOAD



MITER GATES
HORIZONTALLY AND
VERTICALLY FRAMED
GUDGEON PIN HOOD
DESIGN DATA

Plate 9.9. Gudgeon Pin Hood Design Data, Horizontally and Vertically Framed Miter Gates

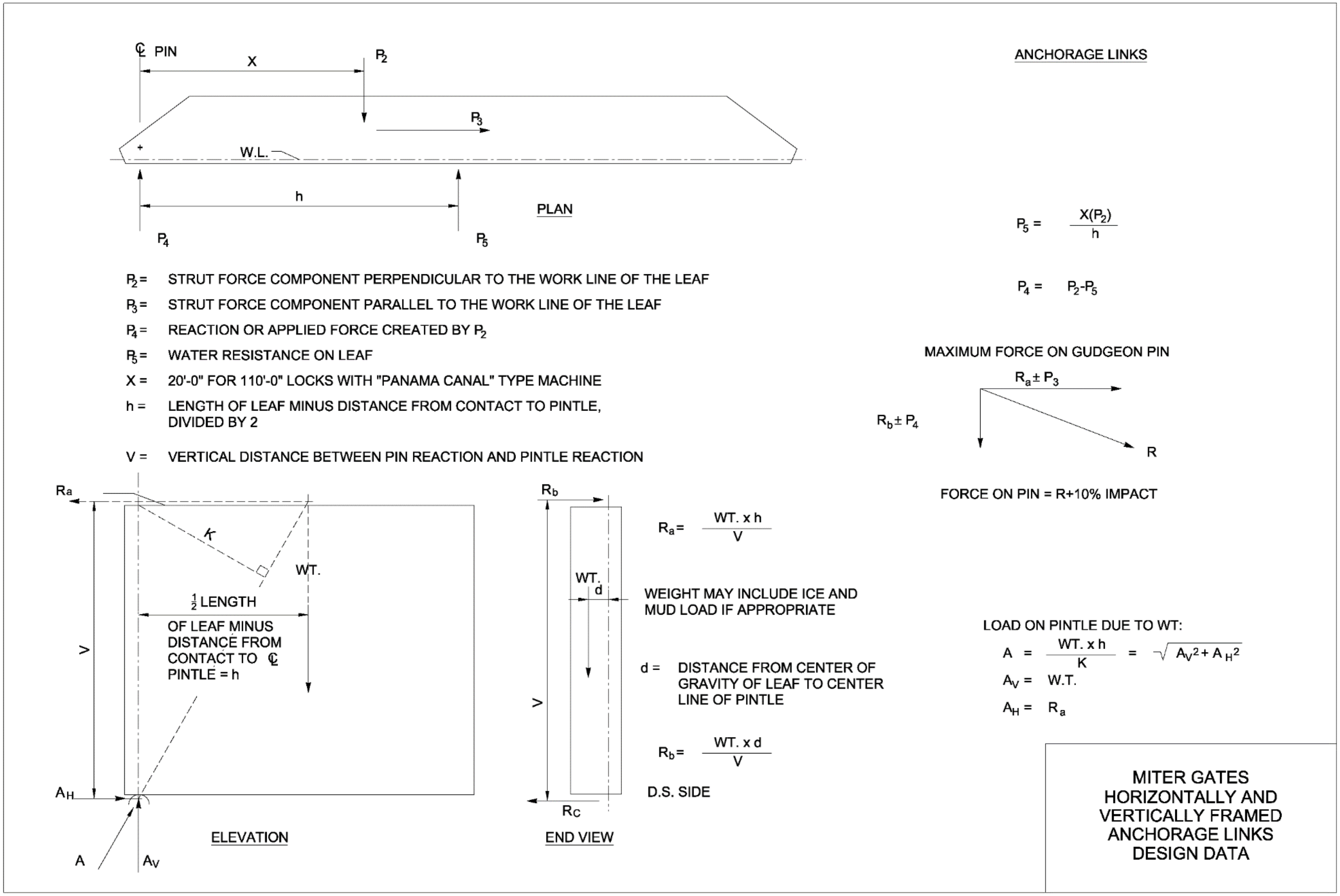


Plate 9.10. Anchorage Links Design Data, Horizontally and Vertically Framed Miter Gates

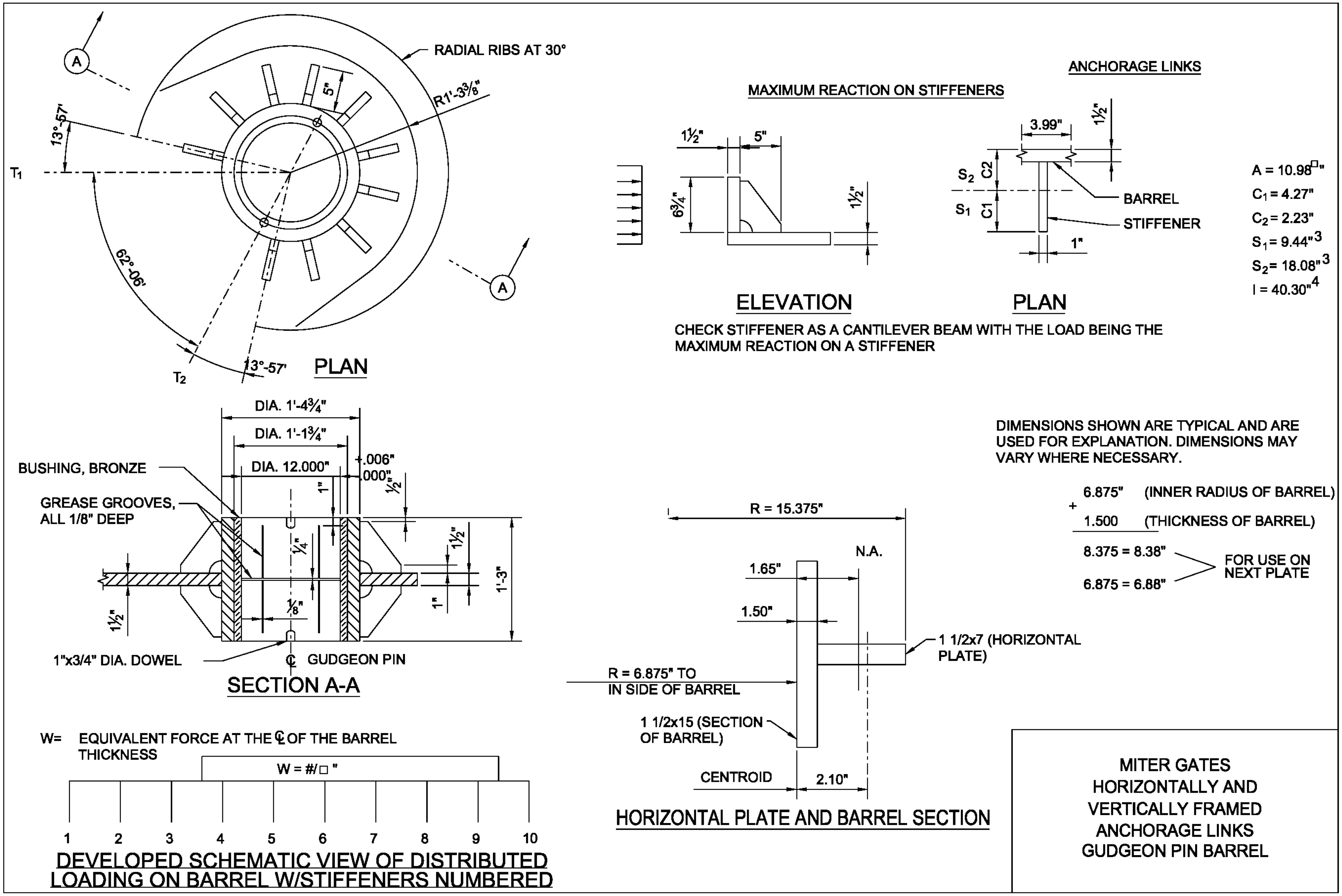


Plate 9.11. Anchorage Links Gudgeon Pin Barrel, Horizontally and Vertically Framed Miter Gates

ANCHORAGE LINKS

ALTERNATE METHOD OF PIN BARREL ANALYSIS

ASSUME THE COMPOSITE SECTION OF THE BARREL AND HORIZONTAL PLATE ACTS AS A CURVED BEAM WITH A UNIFORM LOADING ACROSS THE DIAMETER.(WHERE R IS TO THE CENTROID, AND D = 2R)

r_0 = RADIUS TO NEUTRAL AXIS

$$r_0 = \frac{A}{\int \frac{dA}{r}} = \frac{(7x1\ 1/2)+(15x1\ 1/2)}{15\int_{6.88}^{8.38}\frac{dr}{r} + 1.5\int_{8.38}^{15.38}\frac{dr}{r}}$$

$$r_0 = \frac{33.0}{(15xLOG_e\ \frac{8.38}{6.88})+(1.5\ LOG_e\ \frac{15.38}{8.38})} = 8.53"$$

Y = DISTANCE FROM NEUTRAL AXIS TO THE POINT OF STRESS

$$M = \frac{SR}{Y}\int (r-r_0)dr \qquad S = \frac{MY}{R\int (r-r_0)dr}$$

$$S = \frac{M_Y}{R[\underbrace{15}_{\text{WIDTH OF BARREL}}\underbrace{\int_{6.88}^{8.38} rdr}_{(171.68)} - \underbrace{127.95}_{\text{WIDTHx}r_0}\underbrace{\int_{6.88}^{8.38} dr}_{(-191.93)} + \underbrace{1.5}_{\text{WIDTH OF HORIZONTAL PL}}\underbrace{\int_{8.38}^{15.38} rdr}_{(124.74)} - \underbrace{12.80}_{\text{WIDTHx}r_0}\underbrace{\int_{8.38}^{15.38} dr}_{(-89.60)}]}$$

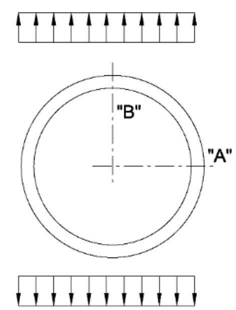
$$\int r\,dr = r^2/2$$

$$15\int_{6.88}^{8.38} r\,dr = 15\left[\frac{(8.38)^2}{2} - \frac{(6.88)^2}{2}\right] = 171.68$$

$$127.95\int_{6.88}^{8.38} dr = 127.95\left[(8.38-6.88)\right] = 191.93$$

$$1.5\int_{8.38}^{15.38} r\,dr = 1.5\left[\frac{(15.38)^2}{2} - \frac{(8.38)^2}{2}\right] = 124.74$$

$$12.80\int_{8.38}^{15.38} dr = 12.80\left[(15.38-8.38)\right] = 89.60$$



SEE "ADVANCED MECHANICS OF MATERIALS", SEELY & SMITH, (1952) 2ND. ED.

MA AND MB ARE MAXIMUM MOMENTS MA = -MB

$$MA = \frac{WR^2}{4} = \frac{Wd}{16}$$

MAX. STRESS = (DIRECT TENSION)+(CURVED BEAM STRESS) (POINT A, INSIDE FACE)+(LOCAL BENDING IN BARREL)

$$S = \frac{MxY}{Rx14.89}$$ (FOR THE EXAMPLE SHOWN)

(R IS THE RADIUS TO THE POINT UNDER CONSIDERATION)

(Y IS THE DISTANCE FROM THE NEUTRAL AXIS)

MITER GATES
HORIZONTALLY AND
VERTICALLY FRAMED
DESIGN DATA
ANCHORAGE LINKS
GUDGEON PIN BARREL

Plate 9.12. Design Data on Gudgeon Pin Barrel, Horizontally and Vertically Framed Miter Gates

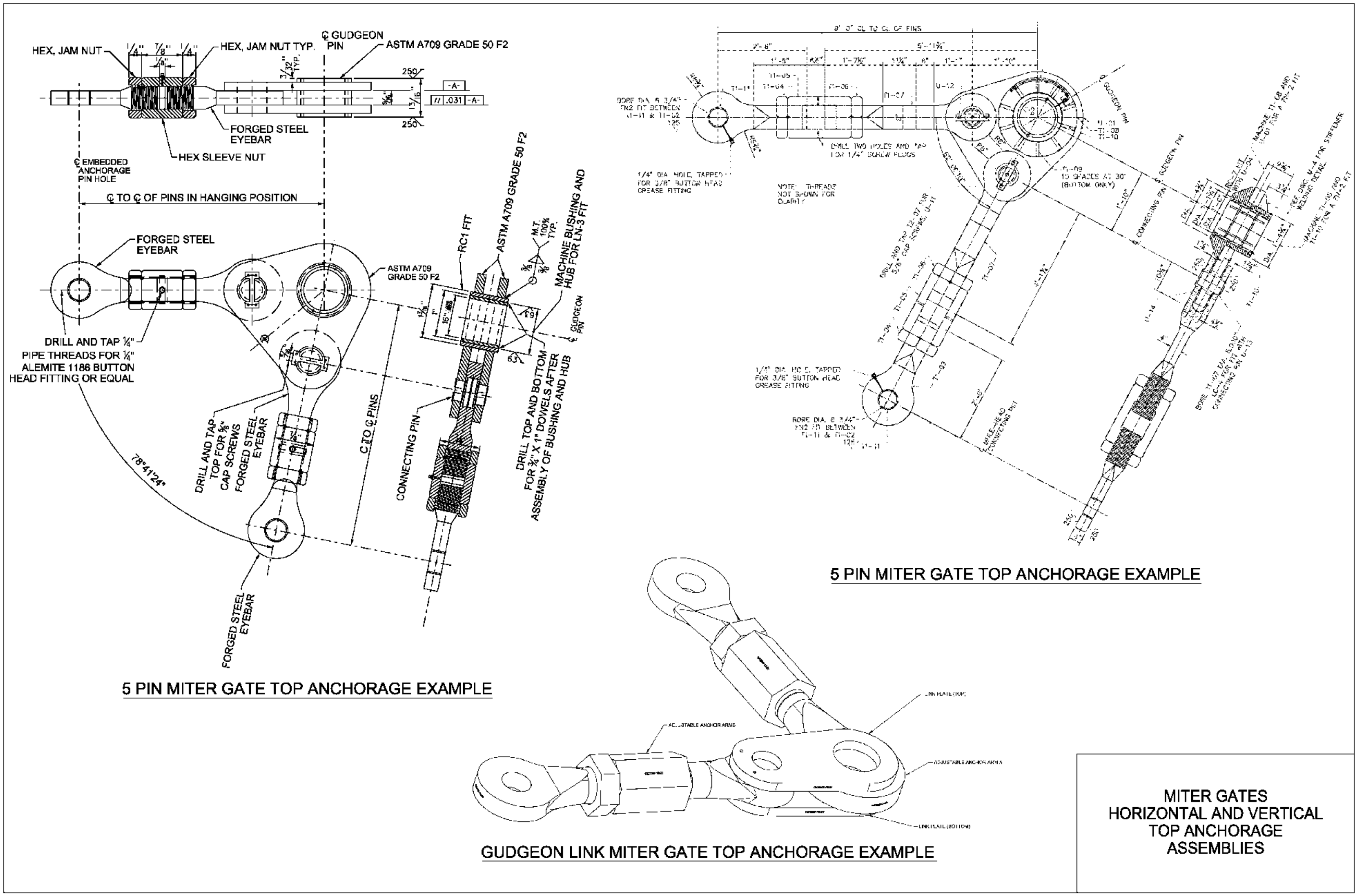


Plate 9.13. Miter Gates Top Anchorage Assemblies, Horizontally and Vertically Framed Miter Gates

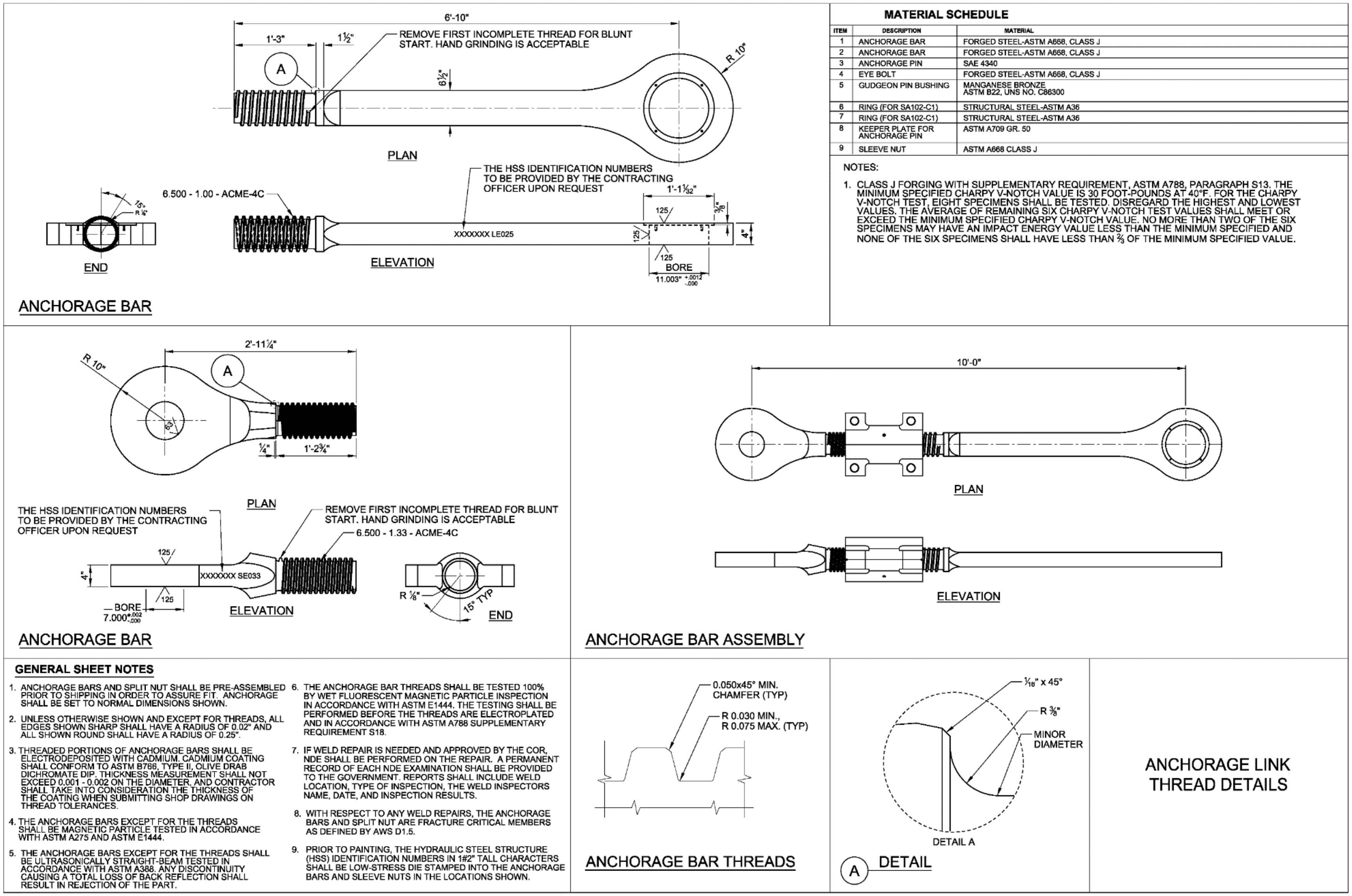


Plate 9.14. Anchorage Link Thread Details

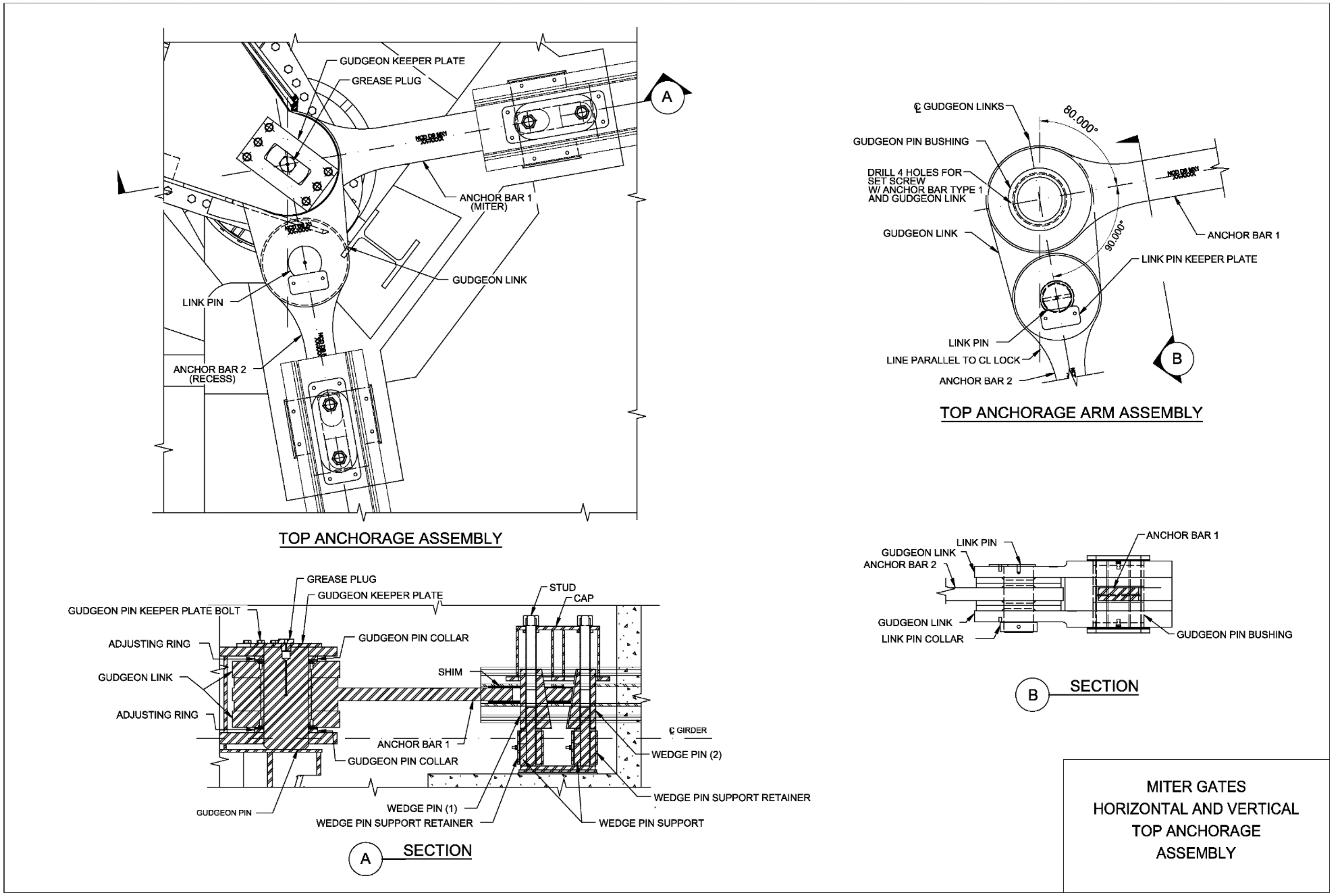


Plate 9.15. Wedge Pin Top Anchorage Assembly, Horizontally Framed Miter Gates

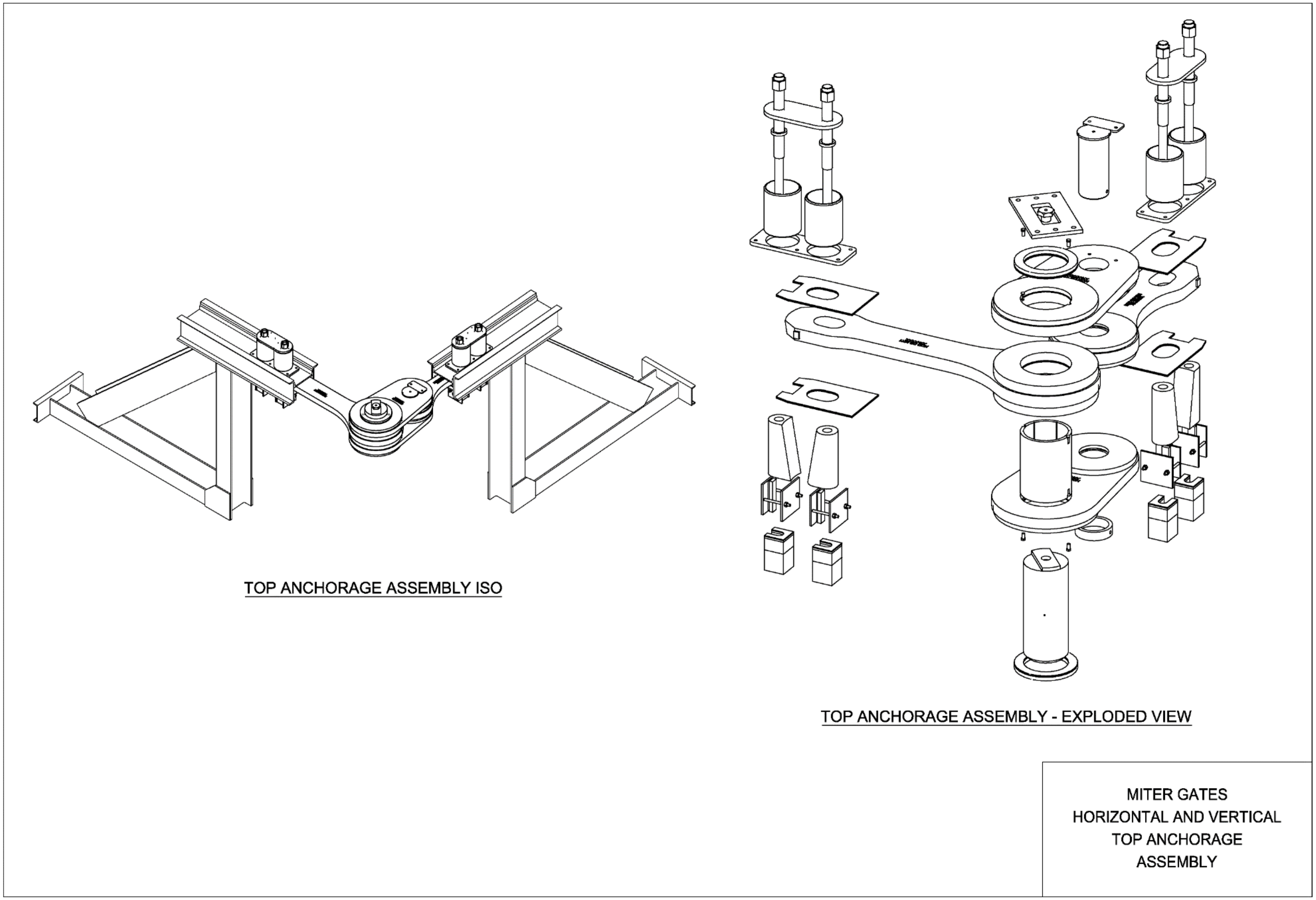


Plate 9.16. Wedge Pin Top Anchorage Assembly, Horizontally Framed Miter Gates

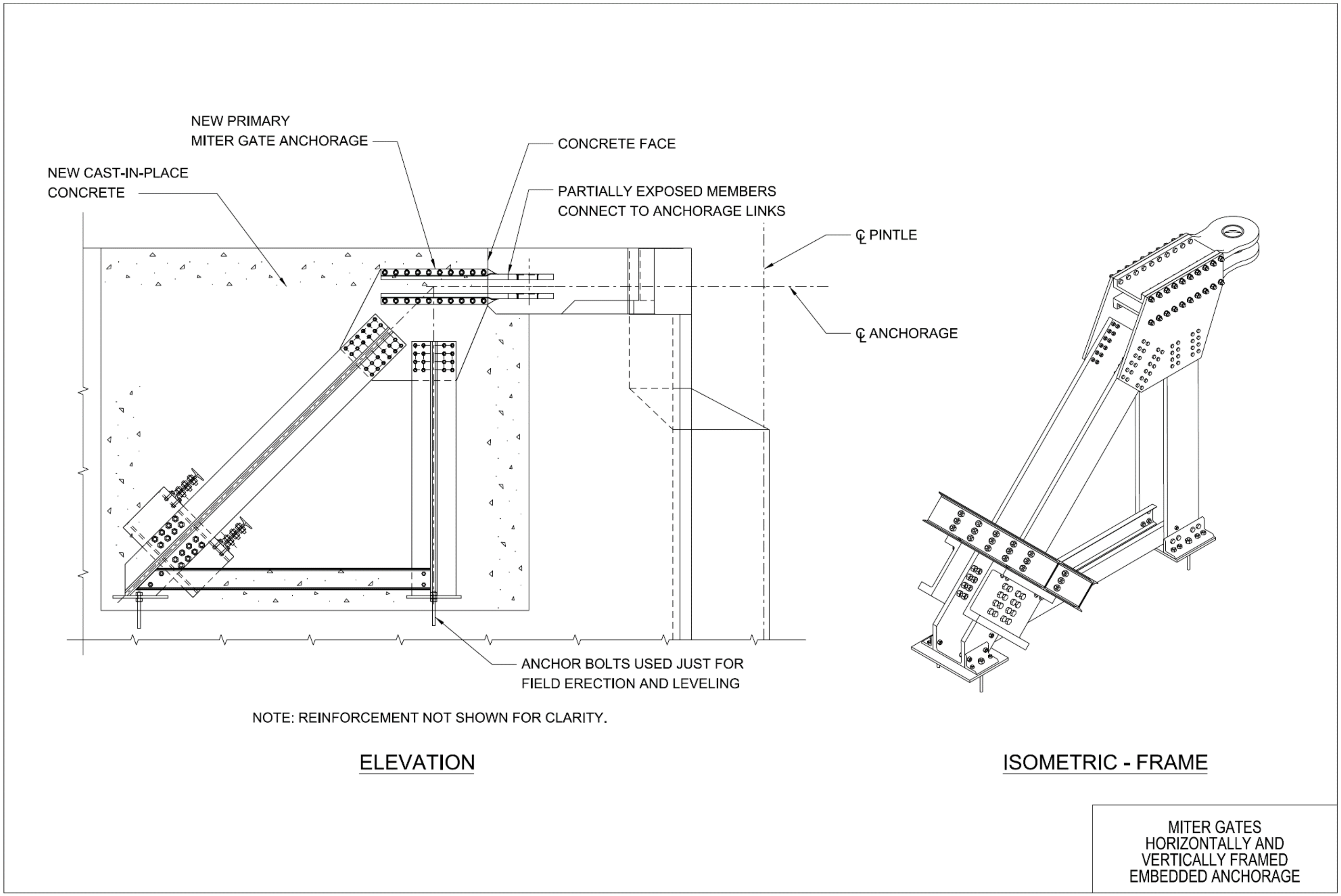


Plate 9.17. Embedded Anchorage, Horizontally and Vertically Framed Miter Gates

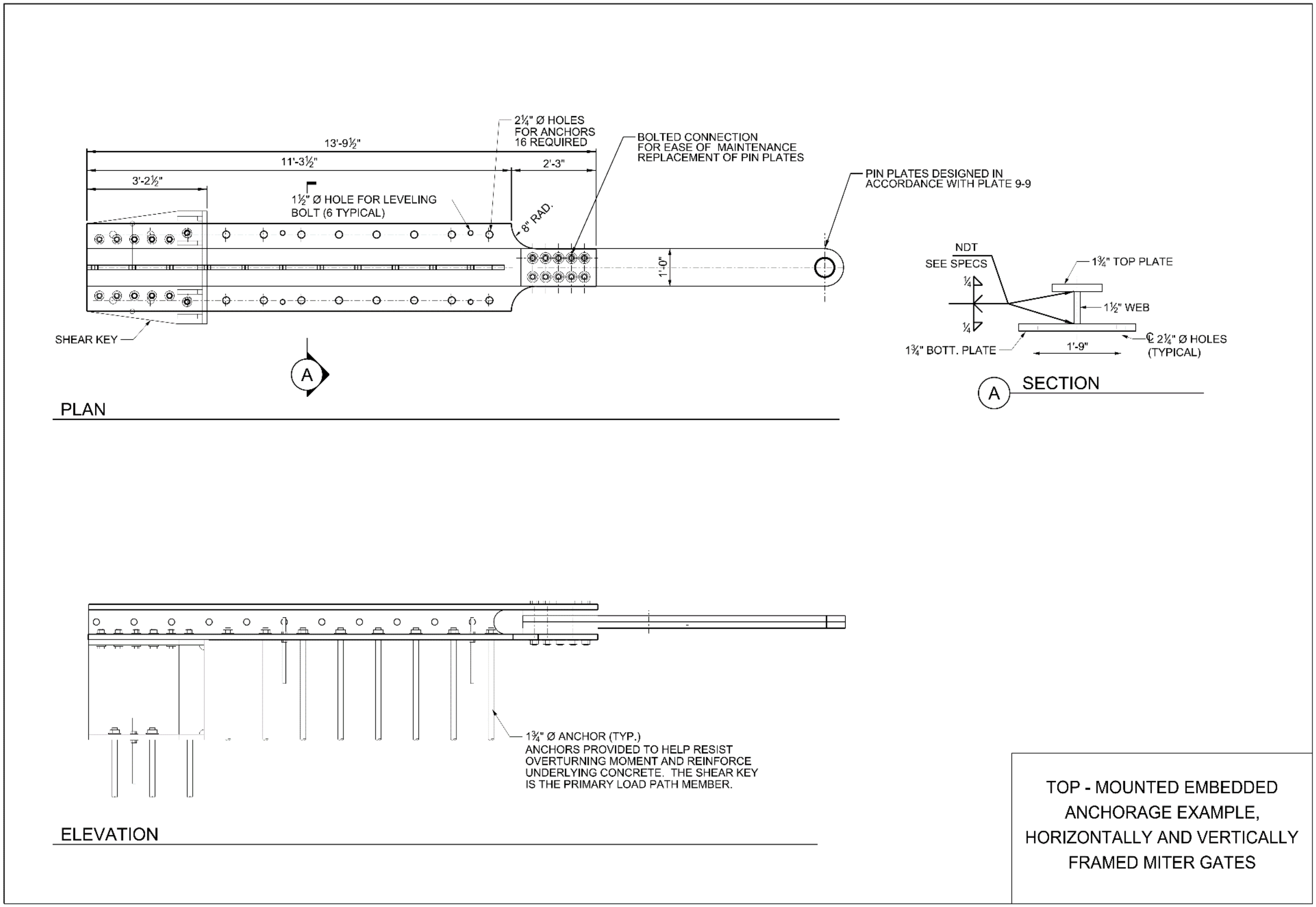


Plate 9.18. Top-Mounted Embedded Anchorage Example, Horizontally and Vertically Frame Miter Gates

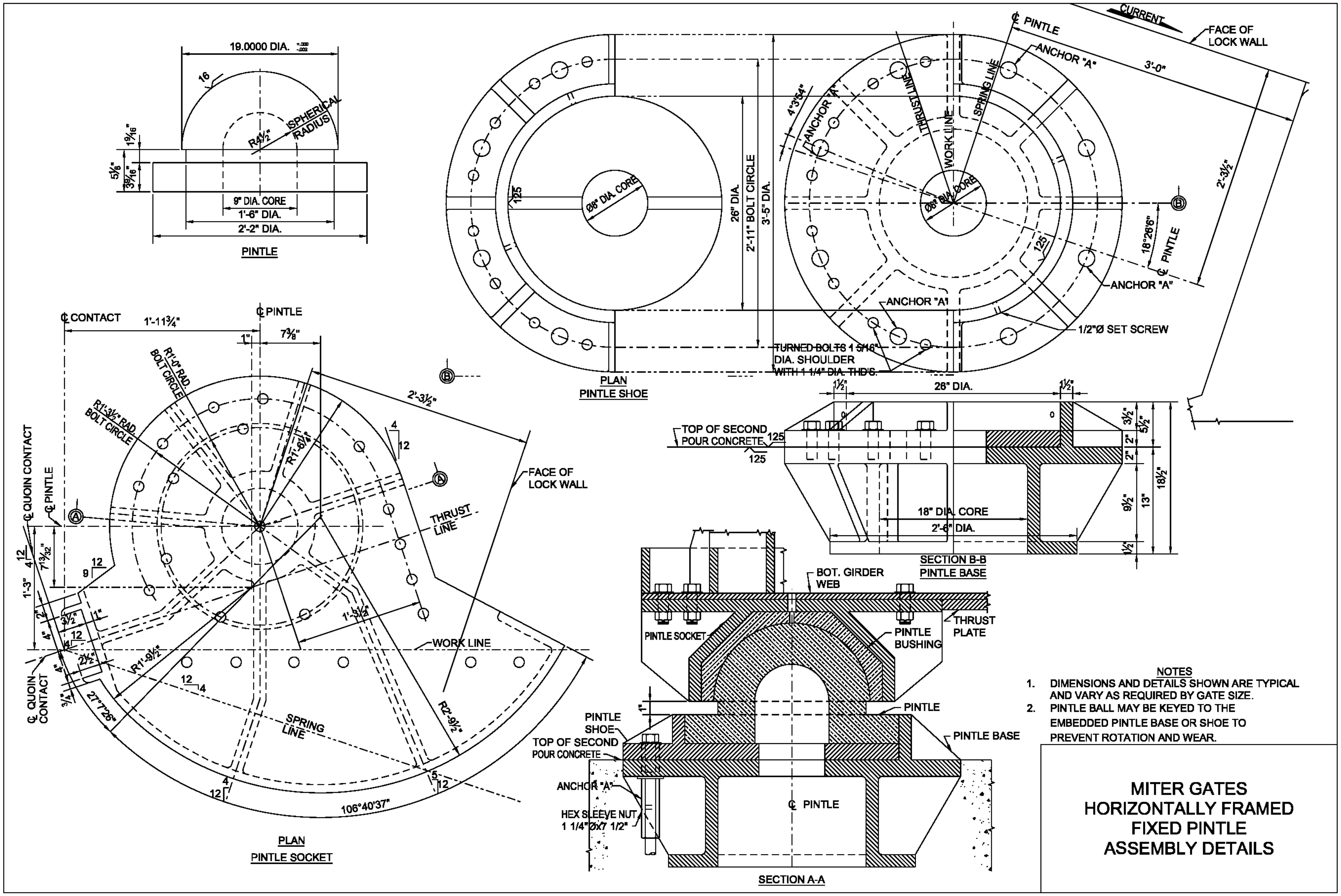


Plate 9.19. Fixed Pintle Assembly, Horizontally Framed Miter Gates

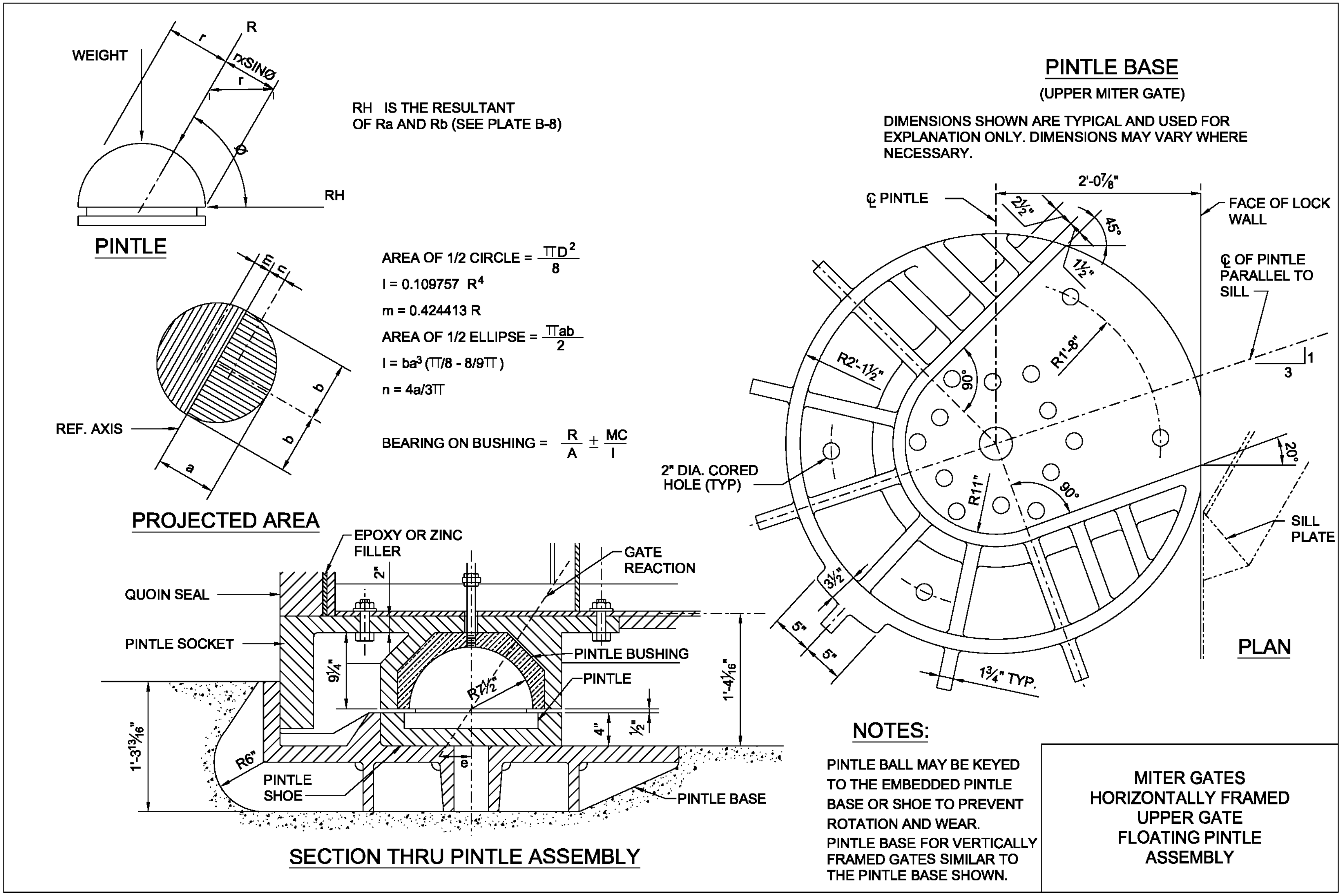


Plate 9.20. Upper Gate Pintle Assembly, Horizontally Framed Miter Gates

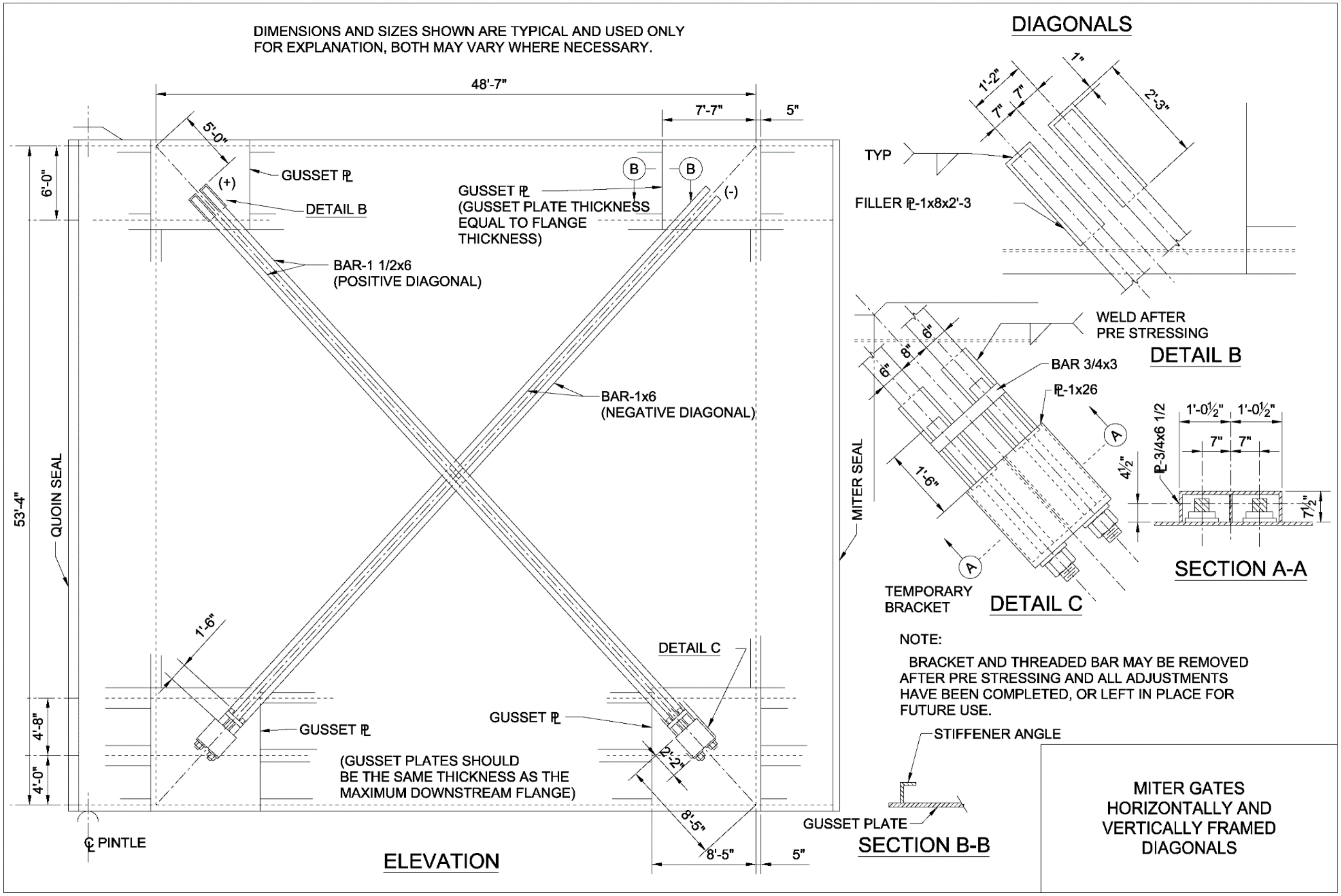


Plate 9.21. Upper Gate Pintle Assembly, Horizontally Framed Miter Gates

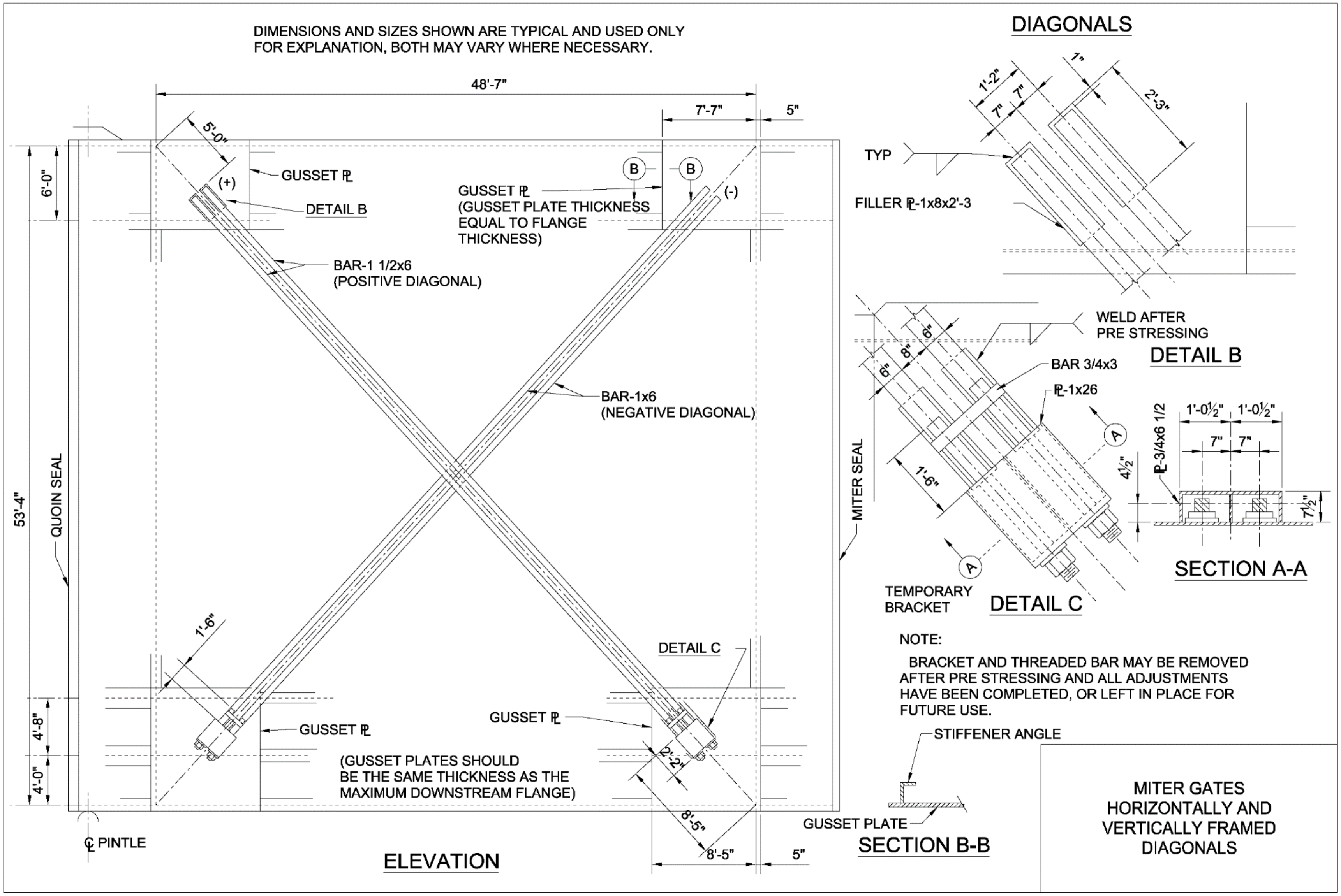


Plate 9.22. Diagonals, Horizontally and Vertically Framed Miter Gates (Sheet 1 of 2)

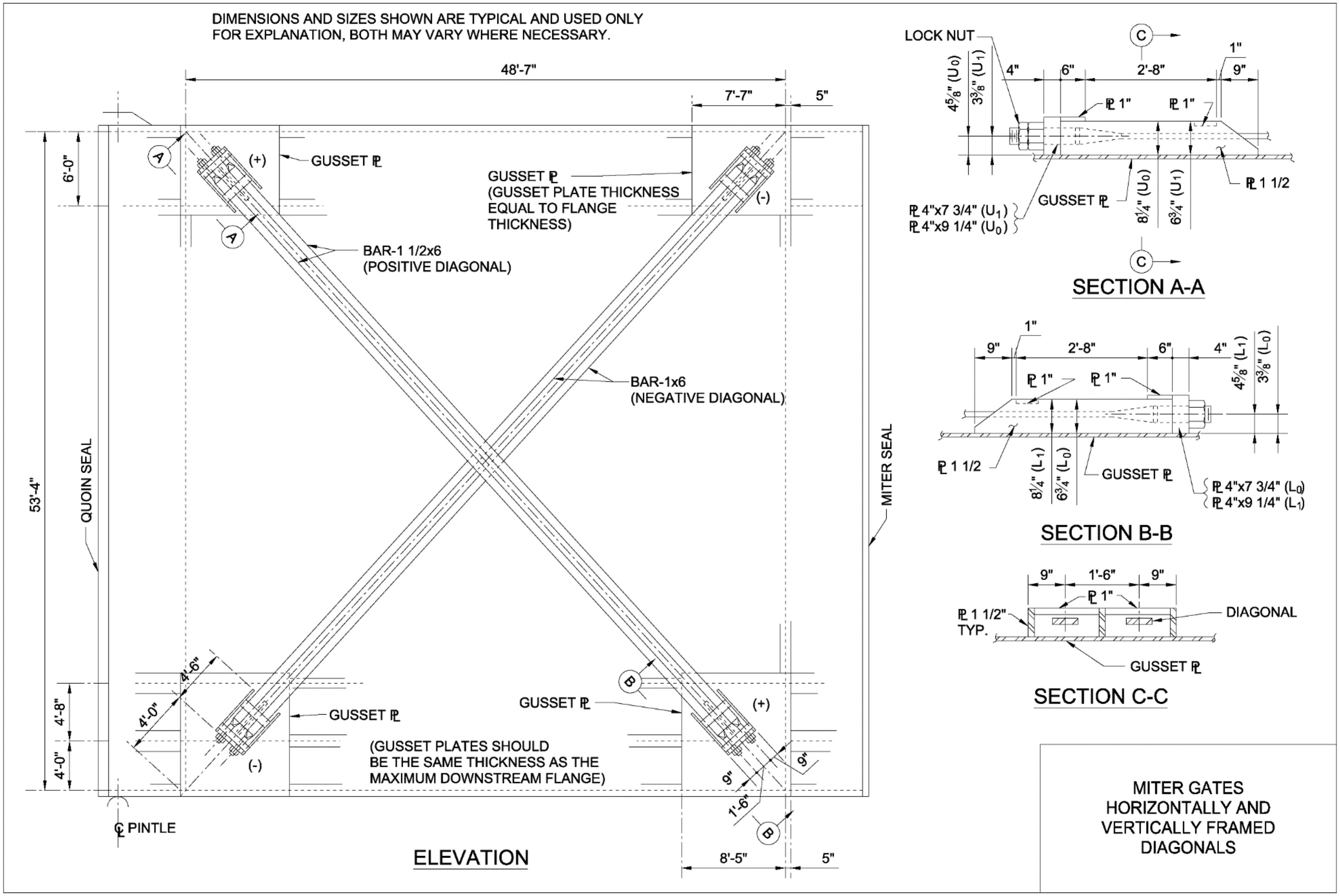


Plate 9.23. Diagonals, Horizontally and Vertically Framed Miter Gates (Sheet 2 of 2)

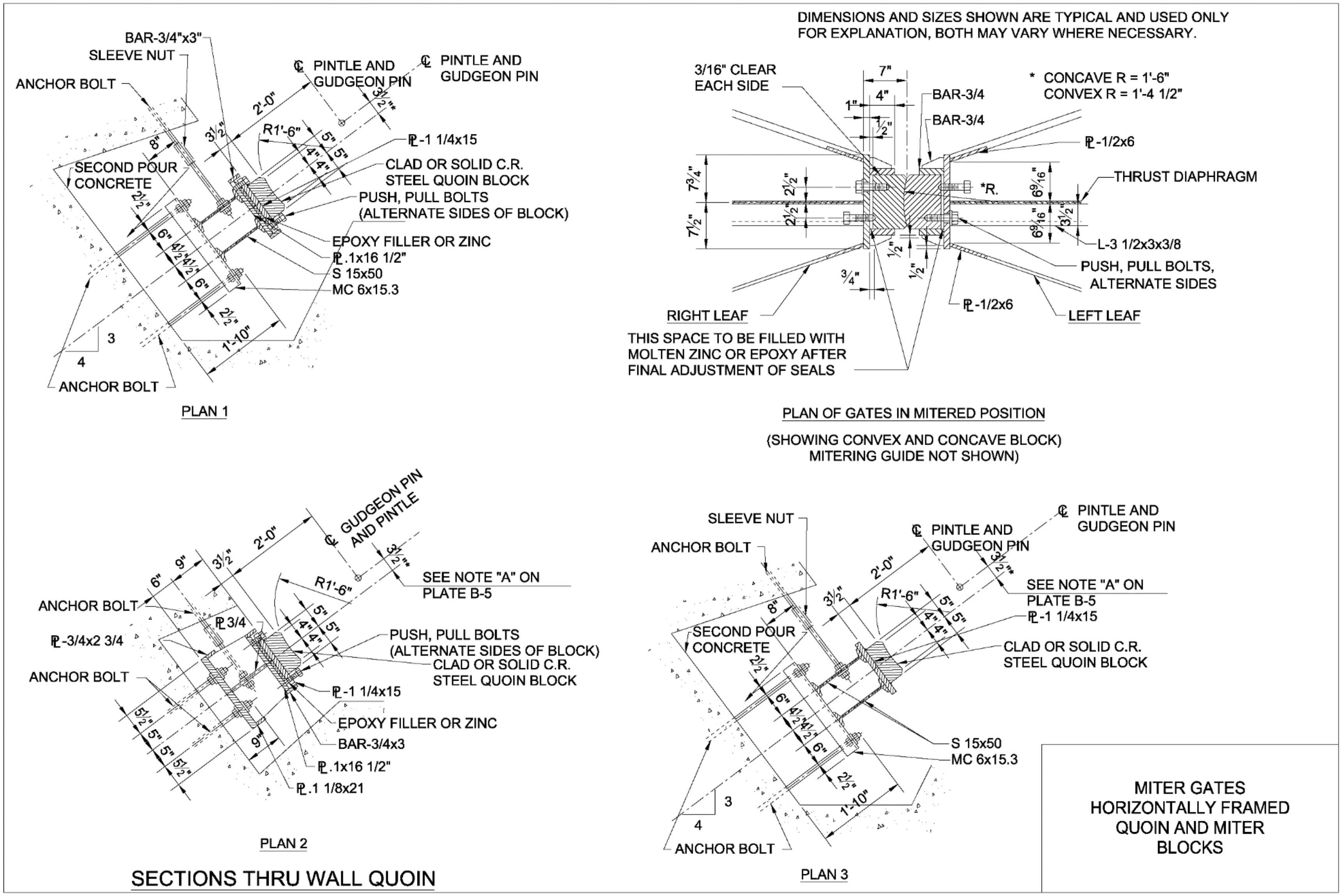


Plate 9.24. Quoin and Miter Blocks, Horizontally Framed Miter Gates

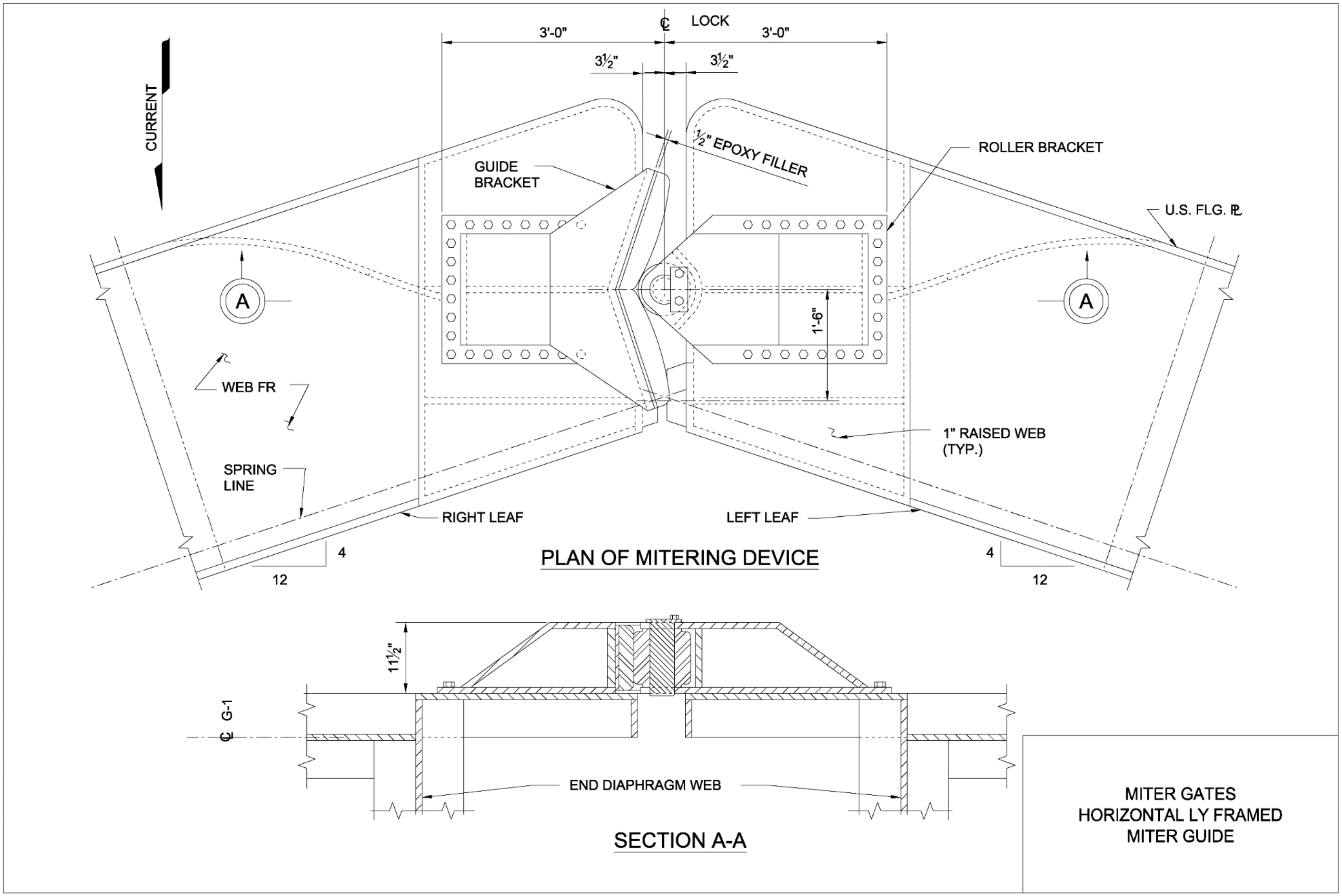


Plate 9.25. Miter Guide, Horizontally Framed Miter Gates

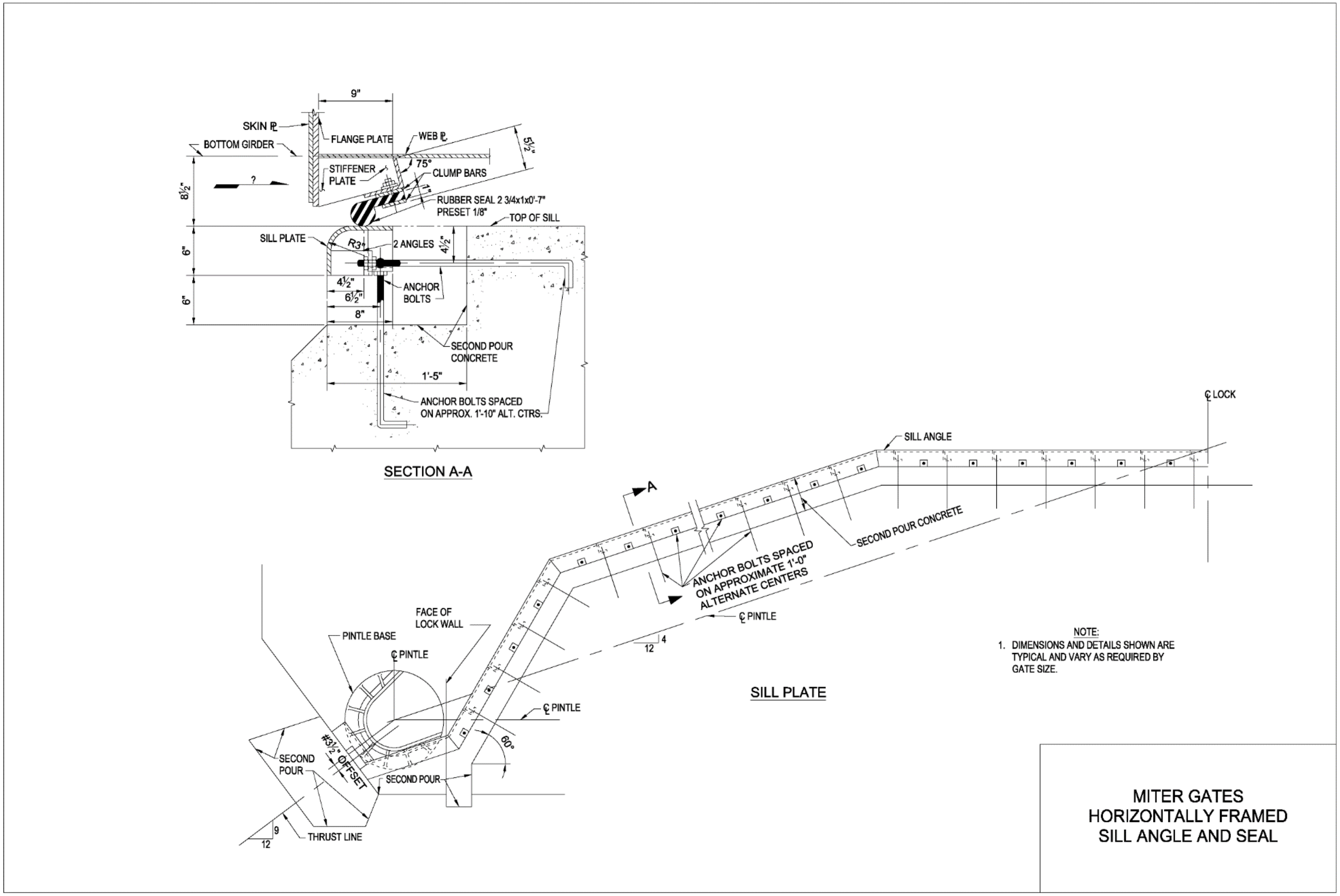


Plate 9.26. Sill Angle and Seal, Horizontally Framed Miter Gates

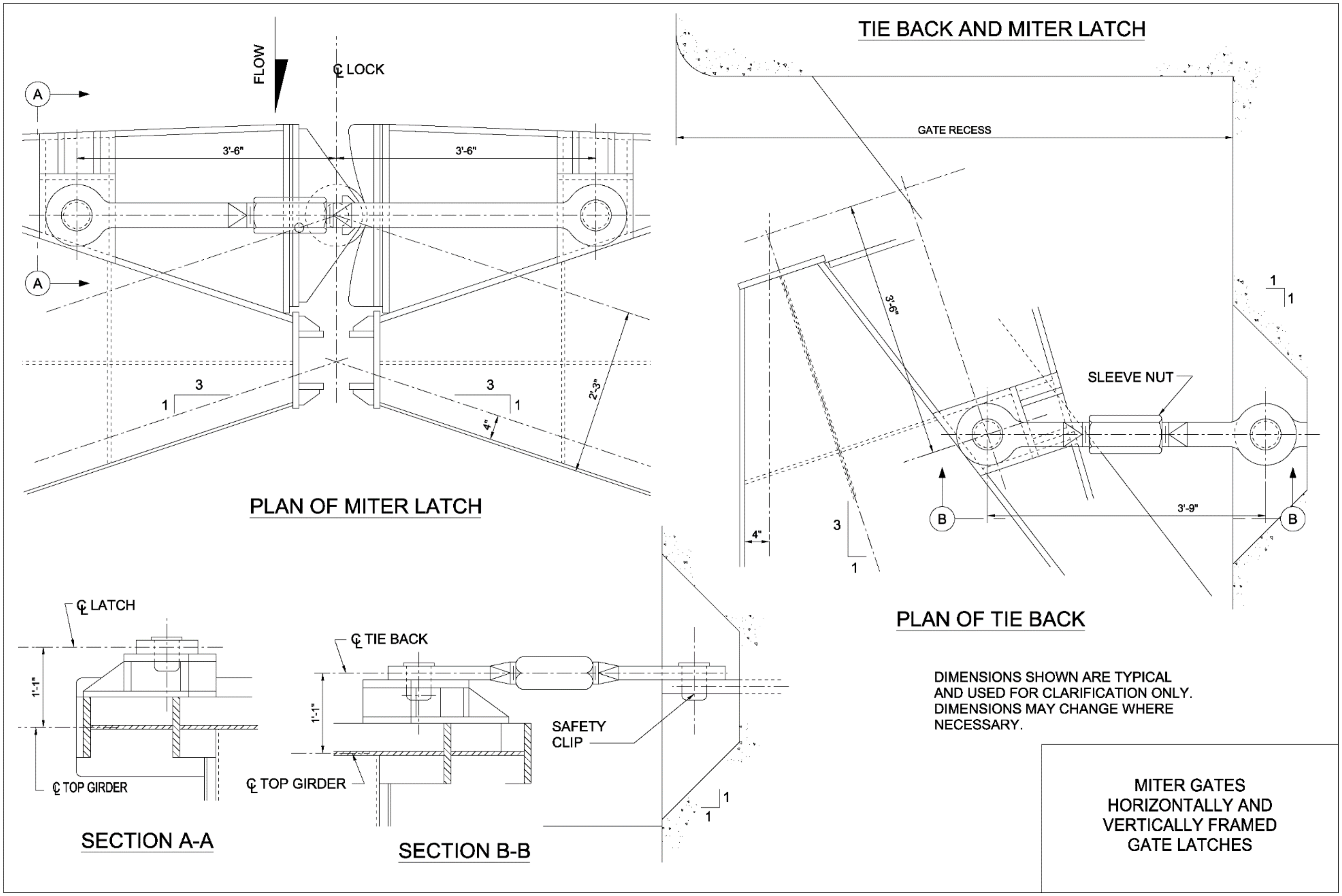


Plate 9.27. Gate Latches, Horizontally and Vertically Framed Miter Gates

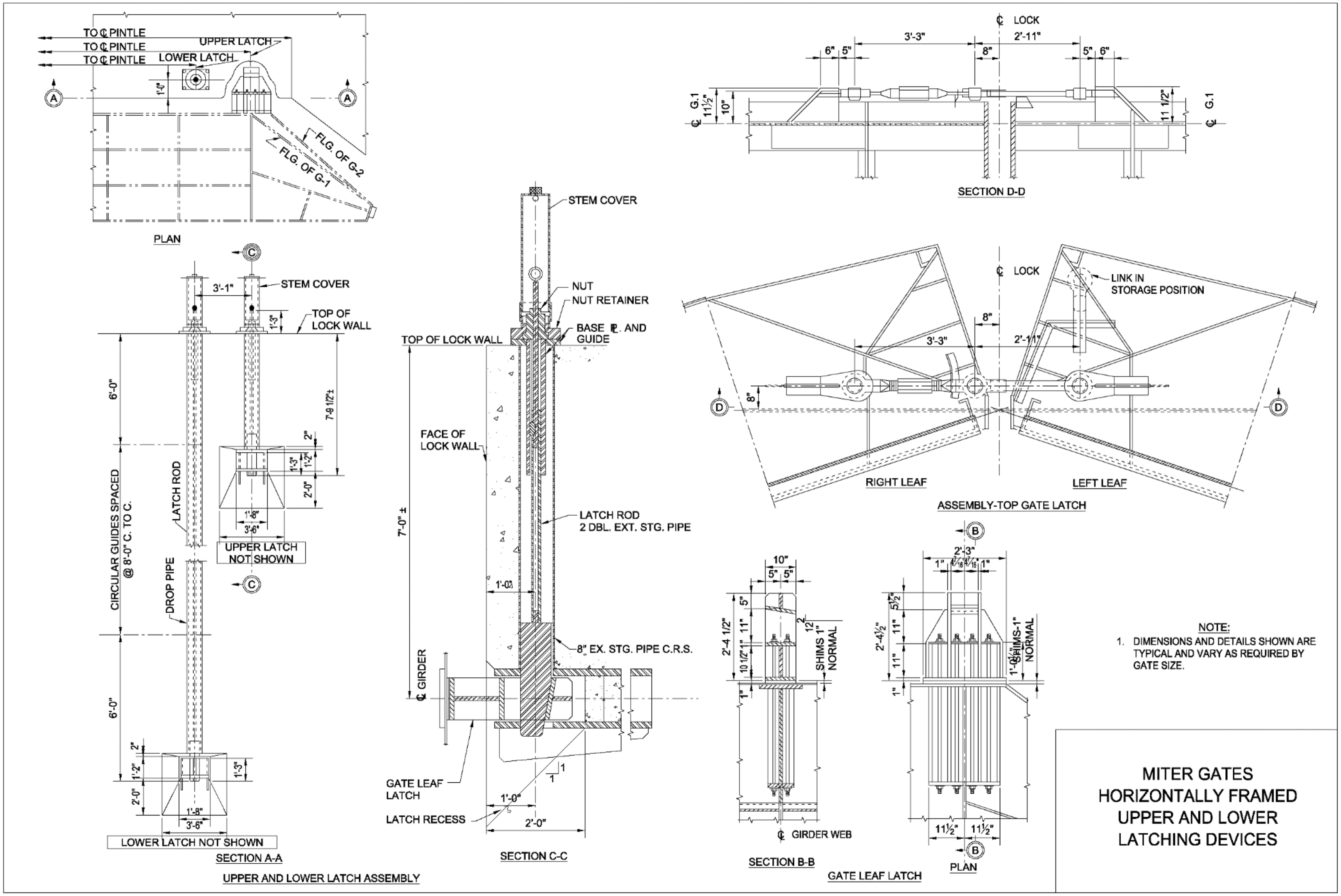


Plate 9.28. Upper and Lower Latching Devices, Horizontally Framed Miter Gates

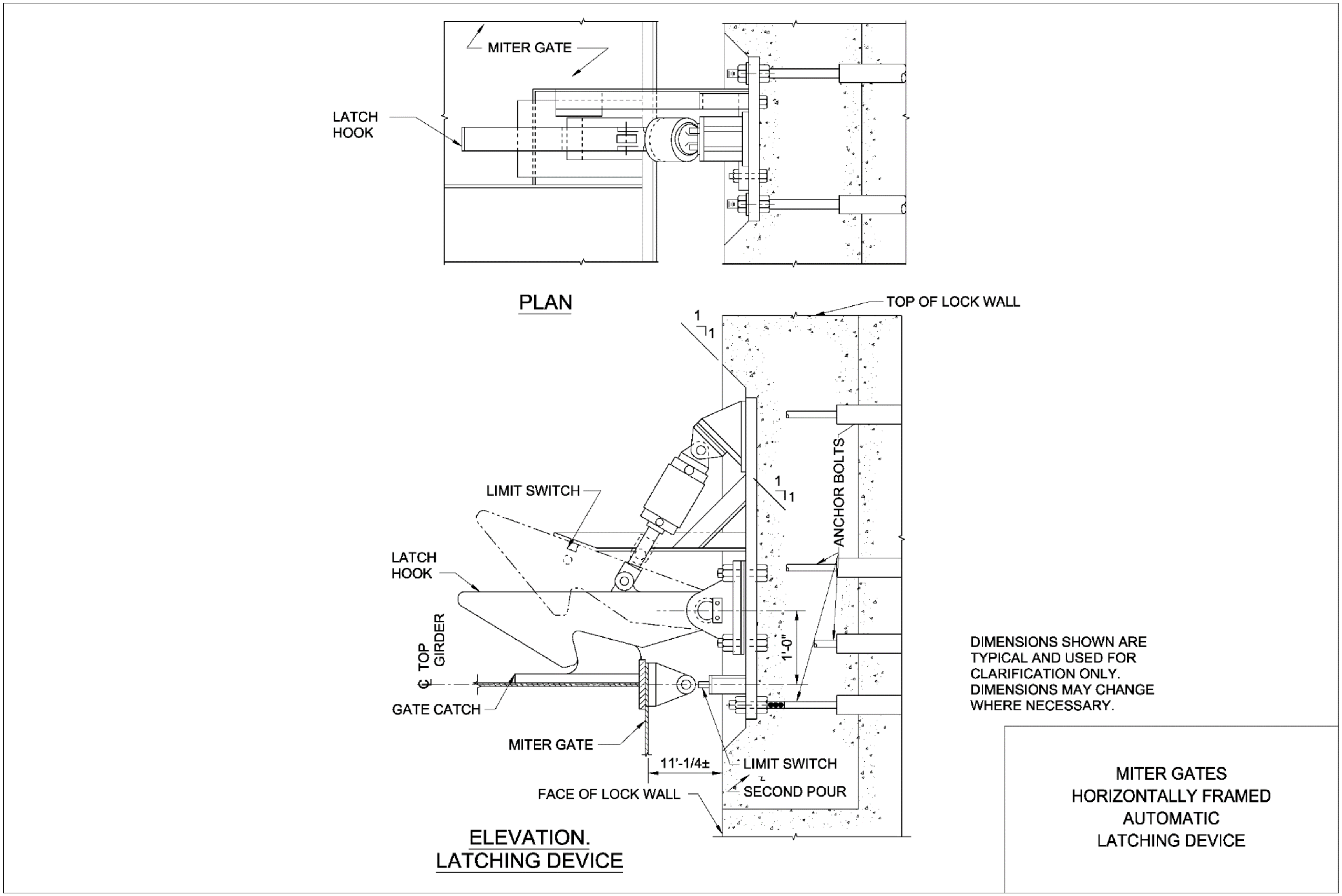


Plate 9.29. Automatic Latching Device, Horizontally Framed Miter Gates

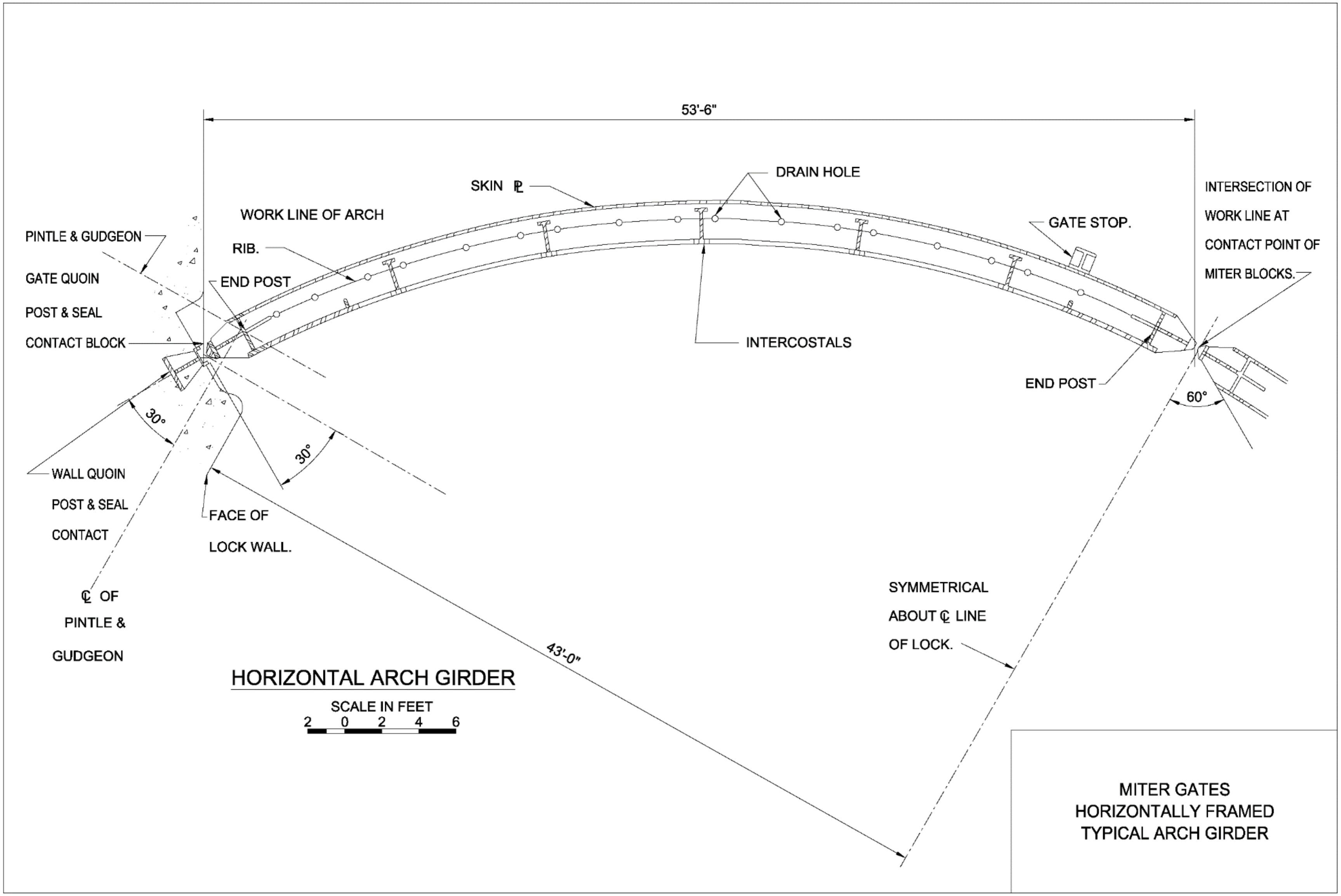


Plate 9.30. Typical Arch Girder, Horizontally Framed Miter Gates

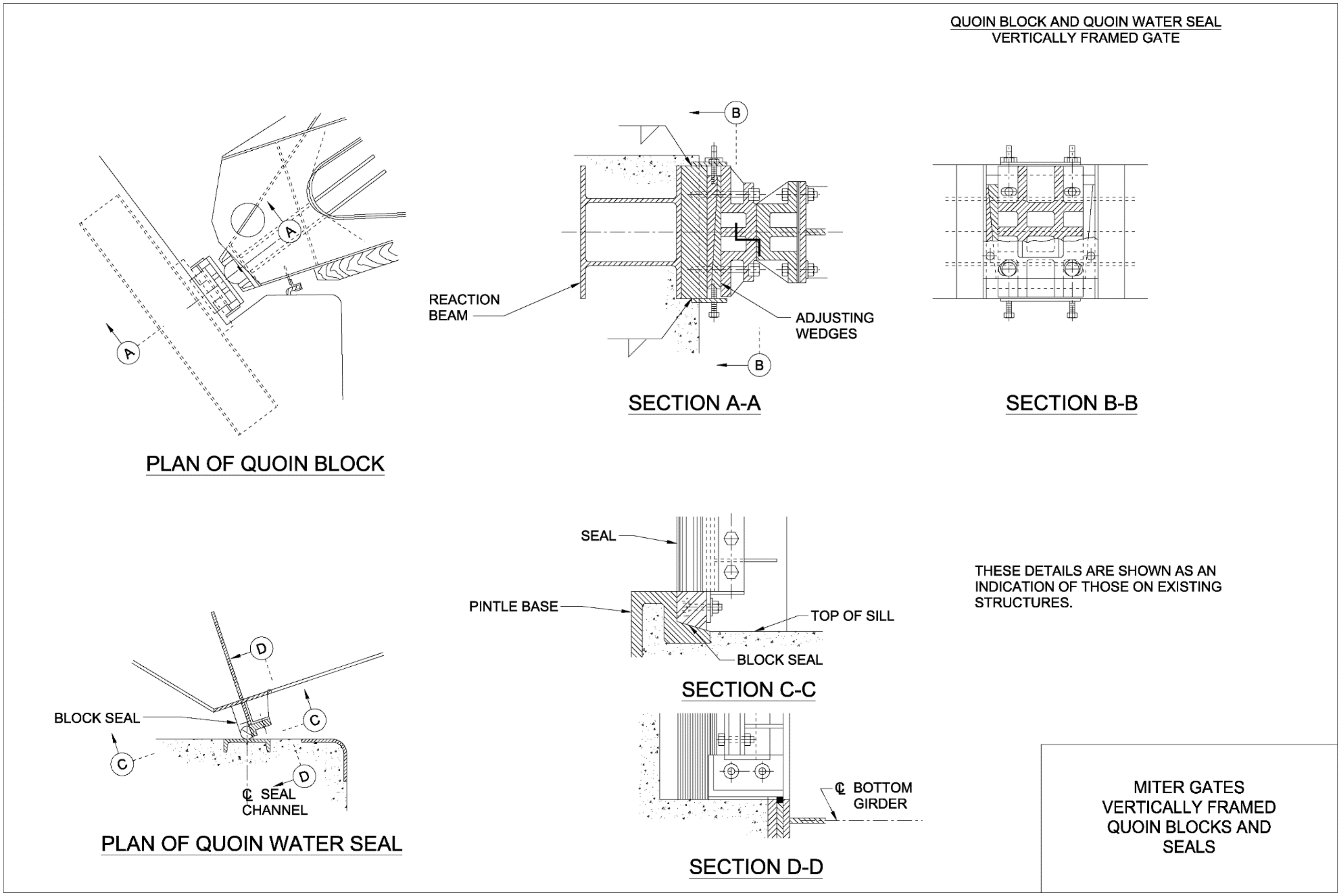


Plate 9.31. Quoin Block and Seals, Vertically Framed Miter Gates

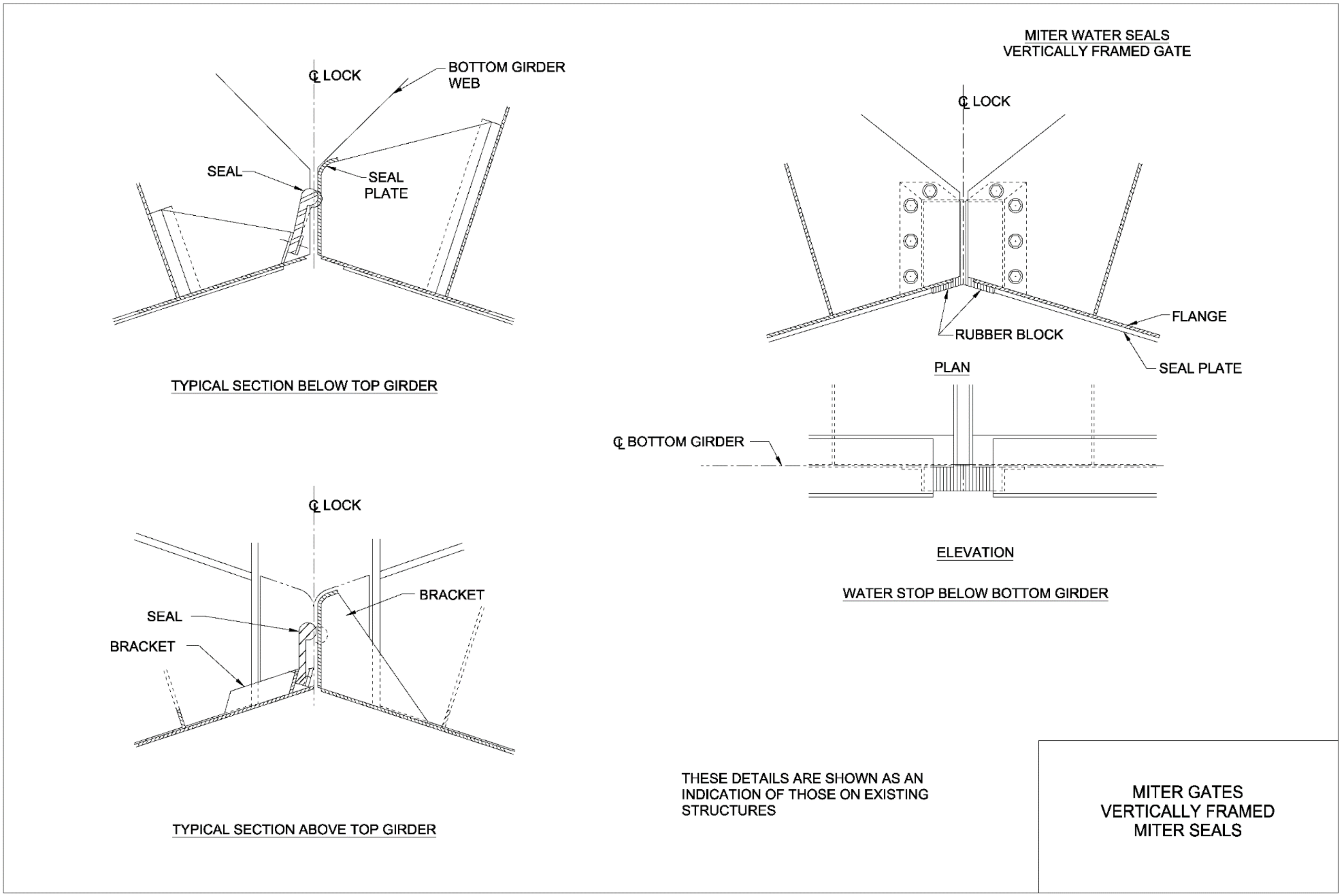


Plate 9.32. Miter Seals, Vertically Framed Miter Gates

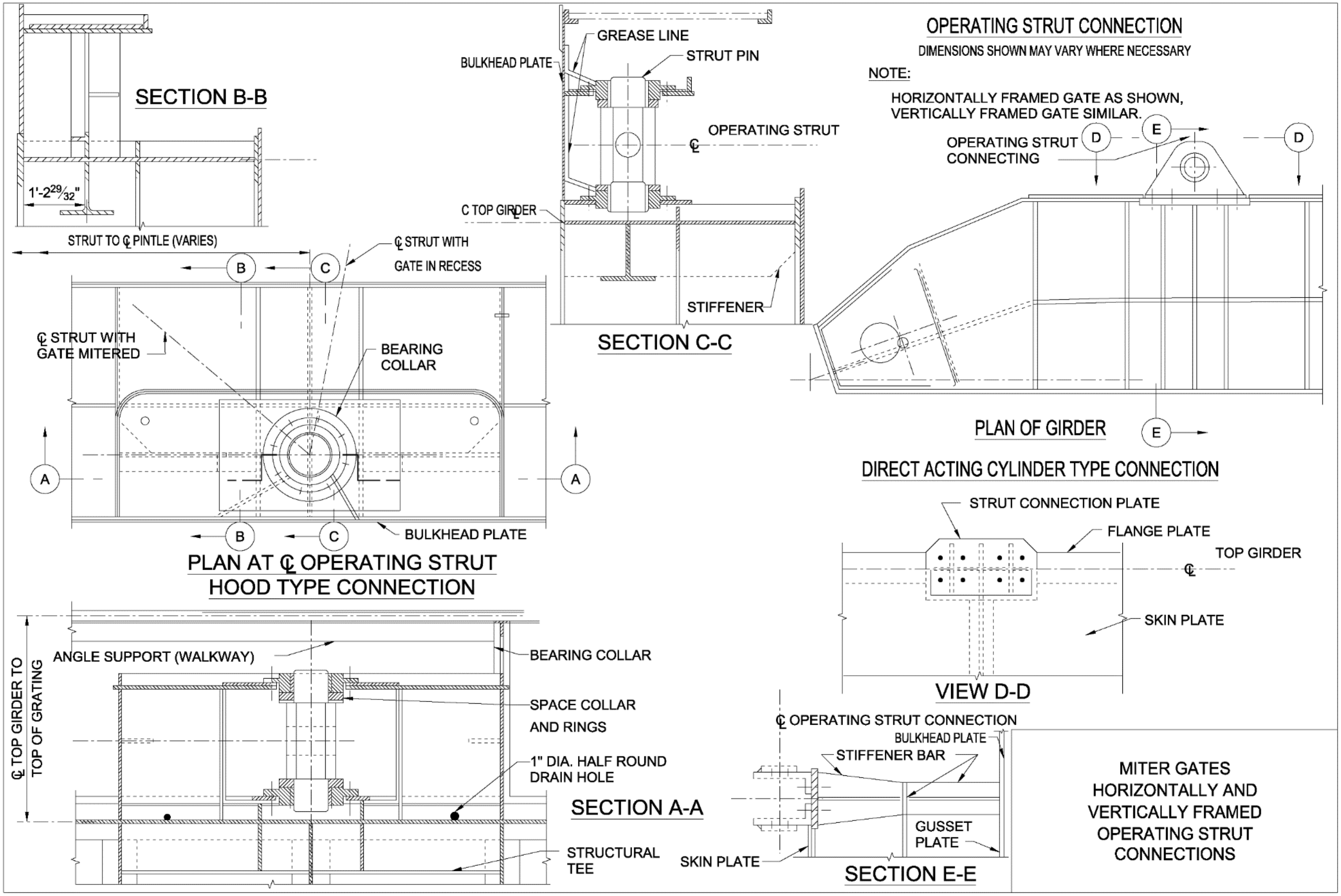


Plate 9.33. Operating Strut Connections, Horizontally and Vertically Framed Miter Gates

Chapter 10
Spillway Tainter Gates

10.1 General. This chapter provides guidance for the design, fabrication, and inspection of spillway Tainter gates, and associated trunnion girders and trunnion-girder anchorages for navigation, hydropower, and flood risk management projects. Design requirements using LRFD are provided in Chapters 3 and 4. This section provides information specific to design of spillway Tainter Gates for strength and serviceability.

10.1.1 Controlled Spillways. Controlled spillways use crest gates to serve as a movable damming surface allowing the spillway crest to be located below the normal operating level of a reservoir or channel. Information on the use of various crest gates and related spillway design considerations is provided in EM 1110-2-1603, EM 1110-2-1605, and EM 1110-2-2607. Tainter gates are considered the most economical and usually the most suitable type of gate for controlled spillways due to their simplicity, light weight, and low hoist-capacity requirements.

10.1.2 Configuration. This manual describes a conventional Tainter gate configuration. However, there are numerous unique variations of Tainter gate types. Figure 10.1 shows a navigation dam with Tainter gates. Figure 10.2 gives a downstream view of a typical Tainter gate. Gates are composed primarily of structural steel and are generally of welded fabrication. Structural members are typically rolled sections. However, welded built-up girders may be required for large gates.

10.1.3 Trunnion Assemblies and Trunnion Girders. Various components of the trunnion assembly and operating equipment may be of forged or cast steel, copper alloys, or stainless steel. Based on project requirements, trunnion girders are either post-tensioned concrete girders or steel girders.

10.1.4 Submergible Tainter Gate. The configuration and design of a submergible Tainter gate is similar to that of a spillway Tainter gate (see Figure 10.3 and Figure 10.4).

10.1.5 Advantages and Disadvantages of Tainter Gates.

10.1.5.1. Advantages of Tainter gates include:

10.1.5.1.1 The radial shape provides efficient transfer of hydrostatic loads through the trunnion.

10.1.5.1.2 A lower hoist capacity is required.

10.1.5.1.3 Tainter gates operate relatively quickly and efficiently.

10.1.5.1.4 Side seals are used so gate slots are not required. This reduces problems associated with cavitation, debris collection, and buildup of ice.

10.1.5.1.5 Tainter gate geometry provides favorable hydraulic discharge characteristics.

10.1.5.2. Disadvantages of Tainter Gates.

10.1.5.2.1 To accommodate location of the trunnion, the pier and foundation will likely be longer in the downstream direction than would be necessary for vertical gates. The hoist arrangement may result in taller piers, especially when a wire rope hoist system is used. (Gates with hydraulic cylinder hoists generally require shorter piers than gates with wire rope hoists.) Larger piers increase cost because of the increased amount of concrete required.

10.1.5.2.2 The larger piers associated with Tainter gates result in less favorable seismic resistance due to their greater height and mass.

10.1.5.2.3 End frame members may encroach on water passage. This is more critical with inclined end frames.

10.1.5.2.4 Long strut arms are often necessary where flood levels are high to allow the open gate to clear the water surface profile.

10.1.6 Use on USACE Projects. Spillway Tainter gates are used on projects for flood risk management, navigation, and hydropower. Although navigation and flood risk management Tainter gates are structurally similar and generally have the same maximum design loads, the normal loading and function may be very different. Generally, gates on navigation projects are subject to significant loading and discharge conditions most of the time, whereas gates on flood risk management projects may be loaded significantly only during flood events. These differences may influence selection of the lifting hoist system, detailing for resistance to possible vibration loading, and selection of corrosion protection systems. Figure 10.5 shows views of a typical Tainter gate.

10.1.6.1. Navigation Projects.

10.1.6.1.1 General Considerations. Navigation projects are normally built in conjunction with a lock. Navigation gates are designed to maintain a consistent pool necessary for navigation purposes, while offering minimum resistance to flood flows. Gate sills are generally placed near the channel bottom, and during normal flows, damming to the required upper navigation pool elevation is provided by Tainter gates. Under normal conditions, most gates on a navigation dam are closed, while several other gates are partially open to provide discharge necessary to maintain a consistent upper lock pool.

10.1.6.1.2 Normal Operating Conditions. Under normal conditions, navigation gates are often partially submerged and significantly loaded with the upstream-downstream hydrostatic head.

10.1.6.1.3 Flood Operating Conditions. During flood events, gates are open and flood flow is not regulated. The upper pool elevation often rises significantly during flood events and the open gate must clear the water surface profile to allow debris to pass. As a result, the trunnion elevation is often relatively high and the gate radius is often long.

10.1.6.2. Flood Storage and Hydropower Projects.

10.1.6.2.1 Flood Storage Projects. Flood storage projects provide temporary storage of flood flows. Many projects include gated spillways to regulate outflow. On flood storage projects with gated spillways, gate sills are generally located such that the gates are dry or only partially wet under normal conditions. In general, gates are exposed to the atmosphere and are subject to slight loads, if any. Only infrequently (during floods) are gates loaded significantly due to increases in pool.

10.1.6.2.2 Multipurpose Projects. Some unique multipurpose projects (projects that provide flood risk management and reservoir storage) and most hydropower projects include aspects of flood storage and navigation gates. Gates on these projects are normally subject to significant hydrostatic loading on the upstream side and may be used to regulate flow on a regular basis.

10.1.6.2.3 Trunnion Locations. Trunnions are typically located at an elevation approximately one-third the height of the gate above the sill.

10.2 Loads.

10.2.1 General. General loads and loading combinations are described in Chapter 4. Loads that are applicable to Tainter gate design include gravity loads, hydrostatic loads, operating loads, ice and debris loads, and earthquake loads. Reactions are not listed below or in the load cases. Reaction forces are not factored since they are determined from equilibrium with factored loads applied. As a result, reaction forces include the load factors of the applied loads.

10.2.2 D, Dead Load. Dead load is defined in Chapter 4. A load factor of 1.2 is used when dead load adds to load effects and 0.9 when it reduces load effects.

10.2.3 G, Gravity Loads. Gravity loads are defined in Chapter 4. Gravity load is applied with a load factor of 1.6 when it adds to load effects. When it reduces load effects it is not applied.

10.2.4 Hs, Hydrostatic Loads. Hydrostatic loads consist of hydrostatic pressure on the gate considering both upper and lower pools. Hydrostatic loads are described in Chapter 4.

10.2.4.1. Strength Design. For hydrostatic pressure as the principal load, Hs_{pr} is the maximum hydrostatic loading from differential head. Depending on the return period of the maximum loading, it may be extreme, unusual, or usual.

10.2.4.2. Companion Loads. For companion hydrostatic loads, Hs_c is the normal operating condition, with a return period of 10 years as defined in paragraph 3.3.3.3.

10.2.5 Q, Gate-Lifting System (operating machinery) Loads. See Chapter 3 for further discussion on operational loads. Operating machinery is provided to lift or lower gates. Under normal operating conditions the forces in the lifting equipment are reaction forces. Loads, Q, are operating machinery loads for conditions where the machinery applies forces to a closed or jammed gate. There are three types of design loads applied by the operating machinery to the gate.

10.2.5.1. Gates Operated on Hydraulic Cylinders-Downward Loads. The hydraulic cylinder maximum downward load, QD_{pr}, is the maximum compressive downward load that a hydraulic hoist system can exert if the gate jams while closing, or if the gate comes to rest on the sill and the machinery continues to operate. The load category of this principal load is dependent on operating conditions, machinery characteristics and other factors but this should be an unusual or extreme load.

10.2.5.2. Gates Operated on Hydraulic Cylinders-At-Rest Loads. The hydraulic cylinder at-rest load QDc is the usual downward load that a hydraulic cylinder exerts while the gate is at rest on the sill (due to cylinder pressure and the weight of the piston and rod) that is combined with other principal loads. Loads QD do not exist for wire rope hoist systems.

10.2.5.3. All Lifting Systems. The maximum upward operating machinery load QU_{pr} is the maximum upward load that can be applied by the wire rope or hydraulic hoist systems when a gate is jammed or fully opened. This force is determined by the limits of the machinery and lifting system. The design intention is that the machinery is not strong enough to damage the gate in a jammed situation. Coordination should be performed with the machinery designers to determine a reasonable upper limit of lifting loads.

10.2.5.4. Operating Load Considerations. The gate operating systems exert forces on specific gate members whether the forces are reactions or applied loads.

10.2.5.4.1 The connection of the wire rope or cylinder to the gate should not fail due to application of any possible machinery loads. This may result in an inoperable gate and may be very difficult to repair.

10.2.5.4.2 Where the wire rope bears on the skin plate, the rope exerts a contact pressure (line load) on the skin plate. The contact pressure force is equal to the rope tension force divided by the gate radius.

10.2.5.4.3 If the wire rope is not tangent to the skin plate, the rope will exert an additional concentrated load on the gate (Figure 10.8). Concentrated forces that typically vary with gate position in magnitude and direction are present at the attachment points for both gate-lifting systems.

10.2.5.4.4 Operating machinery loads must be quantified in consultation with the mechanical engineer who designs the machinery. The probability of the maximum operating machinery loads occurring is unknown and therefore the principal load condition 3 (extreme) load factor described in Chapter 4 applies.

10.2.6 IM, Debris Impact Load. This load accounts for the impact of debris (timber, floating ice, and other foreign objects). For sites with navigation or where floating ice is present, IM is specified as a uniform distributed load of 5,000 lbs/ft. It acts in the down-stream direction and is applied along the width of the gate at the upper pool elevation. Sites without floating ice may be designed for lesser values but design values should represent the upper bound of expected loads. IM must be placed to produce maximum effects. The probability of loading is unknown for IM and therefore the principal load condition 3 (extreme) load factor applies.

10.2.7 BI, Barge Impact Load. For navigable waterways, barge impact load for Tainter gates is specified as a point load to main framing members exposed to potential barge impact at locations that produce the maximum effects in the primary members of the gate. For gates in navigable waterways, the minimum design barge impact load is equal to 5 kips/ft multiplied by the width of the gate opening. Gates at locations in which failure may result in loss of life from uncontrolled release of water or high economic or environmental consequences may require higher design loads. See section 4.2.6.3 for additional guidance on selection of barge impact loads. For non-navigable waterways, this load is not applicable.

10.2.8 IX, Thermally Expanding Ice load. The load from thermal expansion of ice sheets is applied for sites where this load is possible. Thermally expanding ice is a temporary load that is 5,000 lbs/ft across HSS members exposed to ice. The thermally expanding ice load will be applied at the possible pool surface elevations to produce maximum effects in each member. The probability of loading is unknown for IX and therefore the principal load condition 3 (extreme) load factor applies.

10.2.9 F, Friction from various components as follows.

10.2.9.1. Fs, Side-Seal Friction. Loads exist along the radius of the skin plate because of friction between the side seals and the side-seal plate when the gate is opening or closing. The friction force is equal to the product of the coefficient of friction and normal force between the seal plates and the side seals.

10.2.9.1.1 Friction Coefficient. For rubber seals, a coefficient of friction of 0.5 is recommended. Seals that have Teflon rubbing surfaces provide a lower coefficient of friction and are recommended for serviceability. However, wear of the Teflon may increase friction, and applying a lower coefficient of friction for design purposes is not recommended.

10.2.9.1.2 Seal Forces. The normal force on the side seal is a function of the preset force in the seal and the hydrostatic pressure on the surface of the seal. For normal Tainter gate configurations, side-seal friction can be approximated by Equation 10.1. See Figure 10.15.

$$Fs = \mu_s Sl + \mu_s \gamma_w \, {}^{d}\!/_{2} \left(l_1 \, {}^{h}\!/_{2} + h l_2 \right) \qquad \text{(Equation 10.1)}$$

Where:

μ_s = coefficient of side-seal friction

l = total length of the side seal

l_1 = length of the side seal from the headwater to the tailwater elevations or bottom of the seal if there is no tailwater on the gate

l_2 = length of the side seal from the tailwater elevation to the bottom of the seal (equals zero if there is no tailwater on the gate)

S = force per unit length induced by presetting the seal and can be approximated as $S = 3\delta EI/d^3$, where δ is the seal preset distance

γ_w = unit weight of water

d = width of the J-seal exposed to upper pool hydrostatic pressure

h = vertical distance taken from the headwater surface to the tail water surface or the bottom of the seal if there is no tailwater on the gate

10.2.9.2. Fb, Side Sway Friction Load. If the gate is opened with uneven lifting forces, it may translate laterally and rub on the pier as it is lifted. The contact may be through the side seals, portions of the gate, or through bumpers often provided at the end of horizontal girders. The force normal to the pier from side sway is computed from analysis of the gate girder strut frame. This may be either a simplified 2-d analysis or a finite element analysis of the entire gate. The sidesway friction force should be included in design with load cases where the gate is lifted by one cable.

10.2.9.2.1 Friction Coefficient. The coefficient of friction used for design is the expected coefficient for the material used in the contact point on the gate and at the contact point on the pier.

10.2.9.2.2 Load Factor. Using an expected value, a load factor of 1.4 is applied in load combinations to account for uncertainty in the friction coefficient.

10.2.9.3. Ft, Trunnion Pin Friction. During opening or closing of gates, friction loads exist around the surface of the trunnion pin between the bushing and the pin and at the end of the hub between the hub bushing and side plate (yoke plate for yoke mounted pins). These friction loads result in a trunnion friction moment Ft about the pin that must be considered in design. The friction moment is a function of a coefficient of friction, the trunnion reaction force component R that acts normal to the surface of the pin (parallel to the pier face), and the radius of the pin. Reaction force, R, is determined using factored loads.

10.2.9.3.1 Friction Forces. The friction moment at the end of the hub is a function of a coefficient of friction, the trunnion reaction force component R_z that acts normal to the end of the pin (normal to the pier face), and the average radius of the hub.

10.2.9.3.2 Friction Coefficient. A coefficient of friction of 0.3 will be used. This value accounts for any bushing material (bronze or ultra-high molecular weight polymer) that may be slightly worn or improperly maintained and includes effects of thrust washer friction and bearing misalignment. A load factor of 1.4 is applied to the trunnion friction coefficient to account for the possibility of higher friction coefficients to be present.

10.2.10 EQ, Earthquake Design Loads. See Chapter 4 for earthquake loading.

10.2.11 Hw, Wave Loads. The primary hydrodynamic load considered in design of spillway Tainter gates is the wave load. See Chapter 4 for determining wave loads. Wave loads applied as principal load are extreme loads, Hw_X. Wave loads applied as companion loads, Hwc, are usual loads computed according to Chapter 3. Flow induced vibration is limited as discussed in Chapter 4 and this chapter.

10.2.12 W, Wind. The main environmental load considered for Tainter gate design is wind. See Chapter 4. Wind loads are small when compared to hydrostatic loads and only affect gate reactions when the gate is out of the water.

10.2.13 T, Self-Straining. Self-straining loads are not normally considered for Tainter gates because they are largely unrestrained and water and operation loads predominate. Self-straining loads should be considered if the geometry of the gate may provide restraint for strains from temperature change or variable support conditions.

10.3 Load Combinations. Tainter gates will be designed for the strength limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in Chapter 4. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. The serviceability limit state is addressed in paragraph 10.15.4.2.3. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

10.3.1 Load Combinations 1: Gate Resting on Sill.

10.3.1.1. Load Combination 1a. Maximum Hydrostatic. Loads consist of maximum hydrostatic loading, Hs_{pr}, with gate subjected to dead load, gravity loads, hydraulic cylinder residual pressure QD_c and wave or impact as a companion load. The hydrostatic principal load factor is selected according to paragraph 4.3.3 based on the return period of the maximum hydrostatic loading.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\, Hs_{pr} + (1.0 \text{ or } 0)\ QD_c + 1.0\ (Hw_c \text{ or } IM_c) \qquad \text{(Equation 10.2)}$$

10.3.1.2. Load Combination 1b. Maximum Ice, Impact or Wave.

10.3.1.2.1 Loads consist of extreme wave, impact, or thermally expanding ice, as applicable plus companion hydrostatic loading, Hs_c, with gate subjected to dead load, gravity loads, hydraulic cylinder residual pressure and weight, QD_c:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ (IX_X \text{ or } IM_X \text{ or } BI_X \text{ or } Hw_X) + 1.0\ Hs_c + (1.0 \text{ or } 0)\ QD_c \qquad \text{(Equation 10.3)}$$

10.3.1.2.2 Unless site specific data is available, the average annual return periods of the extreme thermal expansion ice load and debris and ice impact load are not known and $\gamma_{pr} = 1.3$. For barge impact, $\gamma_{pr} = 1.3$. Wave loads for return periods meeting the requirements of paragraph 4.3.4.1 can be estimated from wind data and $\gamma_{pr} = 1.2$ for waves.

10.3.1.3. Load Combination 1c. Maximum Hydraulic Cylinder Load (where applicable).

10.3.1.3.1 Loads consist of maximum hydraulic cylinder pressure, QD_{pr} with n loading, Hs_c, dead load, and gravity loads:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ QD_{pr} + 1.0\ Hs_c \qquad \text{(Equation 10.4)}$$

10.3.1.3.2 Unless there is evidence that the maximum hydraulic cylinder load occurs more frequently than an extreme load, it should be considered an extreme load (QD_X) with an unknown return period ($\gamma_{pr} = 1.3$).

10.3.2 Load Combinations 2: Gate Supported by Two Hoists.

10.3.2.1. Load Combination 2a. Maximum Hydrostatic. Loads consist of maximum hydrostatic loading, Hspr, with gate subjected to dead load, gravity loads, plus companion wave or impact. Operating machinery forces are a reaction:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs_{pr} + 1.0\ (Hw_c \text{ or } IM_c) \qquad \text{(Equation 10.5)}$$

10.3.2.2. Load Combination 2b. Maximum Impact. Loads consist of extreme wave, impact, or thermal ice expansion, as applicable plus companion hydrostatic loading with gate subjected to dead load and gravity loads. Operating machinery forces are a reaction. Values for γ_{pr} are described in Load Combination 1b.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ (IX_X \text{ or } IM_X \text{ or } BI_x \text{ or } Hw_X) + 1.0\ Hs_c \qquad \text{(Equation 10.6)}$$

10.3.3 Load Combination 3: Gate Operated by Two Hoists. Loads consist of maximum hydrostatic loading, Hs_{pr}, with gate subjected to dead load, gravity loads, and side seal and trunnion friction. Operating machinery forces are a reaction:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs_{pr} + 1.4\ Fs + 1.4\ Ft \qquad \text{(Equation 10.7)}$$

10.3.4 Load Combination 4: Gate Operating on One Hoist. Loads consist of unusual hydrostatic loading, Hs_N, (the principal load) with gate subjected to dead load, gravity loads, side seal and trunnion friction, and side sway friction load (if present):

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.4\ Hs_N + 1.4\ Fs + 1.4\ Fb + 1.4\ Ft \qquad \text{(Equation 10.8)}$$

10.3.5 Load Combination 5: Gate Jammed

10.3.5.1. Loads consist of maximum operating equipment forces, QU_{pr} plus companion hydrostatic loading, Hs_c, dead load and gravity loads.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ QU_{pr} + 1.0\ Hs_c \qquad \text{(Equation 10.9)}$$

10.3.5.2. The maximum operating load should be considered an extreme load (QU_X) with unknown return period ($\gamma_{pr} = 1.3$) unless site specific information is available.

10.3.6 Load Combination 6: Gate Fully Opened Supported or operating on two hoists.

10.3.6.1. Load Combination 6a. Loads consist of dead load and gravity loads plus wind where wind (W) is the principal load:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.0\ W \qquad \text{(Equation 10.10)}$$

10.3.6.2. Load Combination 6b.

10.3.6.2.1 Loads consist of dead load and gravity loads plus maximum operating equipment forces:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ QU_{pr} \qquad \text{(Equation 10.11)}$$

10.3.6.2.2 The maximum operating load should be considered an extreme load (QU_X) with unknown return period ($\gamma_{pr} = 1.3$) unless site specific information is available.

10.3.7 Load Combination 7: Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load, and gravity loads. The gate may be closed or open.

10.3.7.1. For standard and site-specific OBE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c \qquad \text{(Equation 10.12)}$$

10.3.7.2. For standard MDE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.25\ EQ + 1.0\ Hs_c \qquad \text{(Equation 10.13)}$$

10.3.7.3. For site specific MDE and MCE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ EQ + 1.0\ Hs_c \qquad \text{(Equation 10.14)}$$

10.4 Fatigue and Fracture Control.

10.4.1 Requirements Related to Fatigue and Fracture Control. Chapter 5 provides gate design requirements related to fatigue and facture control. Spillway Tainter gates usually do not experience sufficient load cycles for fatigue to be a concern. However, vibrations due to flows can create localized fatigue cracking.

10.4.2 Controlling Cracking. Trunnion yoke plates, trunnion bushing assembly, cable attachment brackets, steel trunnion girders, and built-up members may include weldments with thick plates and high constraint that can result in cracking. Appropriate fabrication requirements, including weld sequence and inspection, should be specified for thick plate weldments or highly constrained weldments. See Chapters 5, 6, and 8 for guidance on controlling cracking in these members.

10.4.3 Trunnion Location. Locate the trunnion above the maximum water surface profile where possible to avoid contact with floating ice and debris and to avoid submergence of the operating parts. However, it is sometimes necessary to allow submergence for flood events, especially on navigation dams. Designs allowing submergence 5% to 10% of the time are common. Gates incorporating a trunnion tie (a structural member along the gate's axis) should not be subject to trunnion submergence.

10.5 Gate Hoists. The type and position of the gate-lifting equipment can have a significant effect on gate forces as the gate is moved through its range of motion. The wire rope system incorporates wire ropes that wrap around the upstream side of the skin plate assembly (Figure 10.6) and attach near the bottom of the skin plate. The hydraulic hoist system incorporates hydraulic cylinders that attach to the downstream gate framing, usually at the end frames (Figure 10.7).

10.5.1 Wire Rope System. Figure 10.8 shows the three possible variations in cable layout in a wire rope hoist system: cable more than tangent to the skin plate; cable tangent to the skin plate; and cable less than tangent to the skin plate.

10.5.1.1. Cable Tangent. The ideal configuration is when the rope is pulled vertically and is tangent to the arc of the skin plate. For this condition, horizontal forces exerted on the hoist equipment are insignificant and rope contact forces act radially on the gate.

10.5.1.2. Cable More Than Tangent. With a rope in the more-than-tangent condition, an edge reaction force exists at the top of the skin plate due to an abrupt change in rope curvature. This force affects the rope tension, trunnion reaction, and rib design forces.

10.5.1.3. Cable Less Than Tangent. If the rope is in the less-than-tangent configuration, the rope force required to lift the gate increases exponentially as the direction of rope becomes further from tangent. The large lifting force affects the hoist and gate. Due to various constraints, some compromise on location of the hoist is usually required.

10.5.2 Hydraulic Cylinder Hoist System. Hydraulic cylinder hoist systems have some disadvantages and are not suited for all applications. A hydraulic cylinder hoist system generally comprises two cylinders, one located at each side of the gate. Each cylinder pivots on a trunnion mounted on the adjacent pier, and the piston rod is attached to the gate. The magnitude and orientation of cylinder force will change continually throughout the range of motion.

10.5.2.1. Cylinder Position. In determining the optimum cylinder position, the location of the cylinder trunnion and piston rod connection to the gate are interdependent. Generally, the piston rod connection position is selected first and then the cylinder trunnion position is determined to minimize effects of lifting forces. For preliminary design layouts, it is often assumed that the cylinder will be at a 45-degree angle from horizontal when the gate is closed, although optimization studies may result in a slightly different orientation. Generally, the most suitable location for the piston rod connection is on the gate end frame at or near the intersection of a bracing member and strut. It is preferable to have the piston rod connection above tailwater elevations. However, partial submergence may be acceptable for navigation projects.

10.5.2.2. Connection Location. The connection location influences the reaction forces of the gate trunnion. When the connection is located upstream of the gate center of gravity, the dead load reaction at the gate trunnion will be downward while the gate is lifted off the sill. However, if the connection is downstream of the center of gravity, the reaction at the gate trunnion will act upward while the gate is lifted off the sill.

10.5.3 Other Geometry Considerations. The face of the gate and the stop log slots should be located far enough apart to permit the installation of maintenance scaffolding. Spillway bridge clearance and any crane operations are factors in determining the gate radius and trunnion location. Operating clearances from the bridge and the location of the hoist will usually require that the sill be placed downstream from the crest, but this distance should be as small as possible to economize on height of gate and size of pier.

10.6 Tainter Gate Components. The principal elements of a conventional Tainter gate are the skin plate assembly, horizontal girders, end frames, trunnions (Figure 10.9), and bracing.

10.6.1 Skin Plate Assembly. The skin plate assembly consists of a skin plate stiffened and supported by curved vertical ribs. The skin plate acts compositely with the ribs (usually structural tee sections) to form the skin plate assembly. The skin plate assembly is supported by the horizontal girders that span the gate width. The downstream edge of each rib is attached to the upstream flange of the horizontal girders.

10.6.2 Horizontal Girder. Girders provide support for the skin plate assembly and transfer all loads from the skin plate assembly to the end frames. Horizontal girders typically consist of rolled wide flange beams.

10.6.3 End Frames. The horizontal girders are supported by the end frames. End frames include the struts and associated bracing, and the strut-to-trunnion hub transition. The strut-to-trunnion hub transition consists of a built-up section of flange and web plates that connect the struts to the trunnion hub. The end frames may be parallel to the face of the pier (supporting the horizontal girders at the ends) or inclined to the face of the pier (supporting the horizontal girders at some distance away from the end). The trunnion is the hinge on which the gate rotates. The trunnion is supported by the trunnion girder.

10.6.3.1. Parallel End Frames. End frames that are parallel to the pier and perpendicular to the horizontal girders will minimize debris accumulation and interference to flow. This will simplify geometry at the trunnion hub flange connections. However, this will greatly increase flexural loads in the struts and girders and will limit clearance for maintenance painting between the pier and struts.

10.6.3.2. Inclined End Frames. By inclining the end frames from the pier face, girder and strut flexural forces are reduced. The component of the end frame loads perpendicular to the pier is transmitted either directly to the pier or is resisted by a trunnion tie. While inclined end frames are usually desirable for flood storage projects, they are often not feasible for navigation dam projects where floating debris is a concern. Struts are typically oriented such that the horizontal girder end of each strut is an equal horizontal distance from the pier face. This results in the struts projecting on a conical surface with the apex at the trunnion.

10.6.4 Horizontal Girder Lateral Bracing. This bracing is generally only necessary for gates with large horizontal girders and is placed between adjacent girders in a plane perpendicular to the girder axes, sometimes at several locations along the length of the girders. Lateral bracing that is located in the same plane with the end frames is generally made up of significant structural members.

10.6.5 Lateral Bracing. Lateral bracing carries significant vertical forces from the skin plate assembly to the end frame. It is often considered a part of the end frame (Figure 10.10).

10.6.6 Intermediate Bracing. Intermediate bracing is located away from the end frames, supports girder dead loads, and helps transfer lifting loads from the skin plate through the girders (otherwise taken through torsion) into the end frames. It can be considered as one of the secondary members.

10.6.7 Downstream Vertical Truss. The downstream vertical truss provides lateral bracing for the horizontal girders and adds torsional rigidity under single sided loading. It also provides gate rigidity for resisting gravity loads with symmetric hoist support conditions, and structural rigidity during field erection. For gates that have a low height-to-width ratio, additional bracing may be required to achieve adequate stiffness to prevent unacceptable torsion and lateral displacement of the gate. Various configurations have been used depending on the gate size and configuration (Figure 10.11).

10.6.8 End Frame Bracing. Figure 10.12 and Figure 10.13 show bracing provided for the end frame struts. The end frame bracing members are ordinarily designed to brace the struts about the weak axis to achieve adequate slenderness ratios. As such, these members are considered secondary members. However, depending on their configuration and connection details, these bracing members may carry significant forces and act as primary members.

10.6.9 Trunnion Tie. A trunnion tie is a tension member provided on some gates with inclined strut arms that is designed to resist lateral end frame reaction loads (loads that are parallel to trunnion pin axis). Trunnion ties are not generally provided on gates with parallel strut arms, since the lateral reaction loads are small. The trunnion tie extends across the gate bay from one end frame to the other. It is attached to each end frame near the trunnion (Figure 10.14).

10.7 Alternative Gate Types. Many alternatives to the standard framing system have been designed and constructed. A brief description of some configurations is provided for information. The design guidelines presented herein are not necessarily applicable to these gates.

10.7.1 Vertical Girders. For the standard gate configuration, geometry at the trunnion normally limits the number of end frame strut arms to a maximum of four on each side. This would limit the number of horizontal girders to four. For tall gates, vertical girders have been used to transfer loads from more girders to fewer strut arms, to simplify the end frame and trunnion configuration.

10.7.2 Vertically Framed Gates. In vertically framed gates, vertical girders support ribs that are placed horizontally on the skin plate. The ribs replace the horizontal girders. The vertical girders are supported by two or more struts. This system has been used on small gates and gates with low hydrostatic head.

10.7.3 Orthotropic Gates. An alternative design approach is to design the gate as an orthotropic system. With the orthotropic approach, the skin plate, ribs, and horizontal girders are assumed to act as a stiffened shell. Typically, the ribs are framed into the horizontal girder webs. This approach can save material and gate weight and has improved detailing, but fabrication and maintenance costs may be higher.

10.7.4 Stressed Skin Gates. Stressed skin gates are a type of orthotropic gate in which the skin plate assembly is considered a shell or tubular structure spanning between trunnion arms. The skin plate is stiffened with horizontal and vertical diaphragms and intermediate stiffening members (usually horizontal tee sections parallel to the intermediate or midlevel horizontal diaphragm). As with other orthotropic gates, this type of gate can save material and gate weight, but fabrication and maintenance costs may be higher.

10.7.5 Truss-Type or Space Frame Gates. Three-dimensional truss or space frame gates were sometimes used in early Tainter gate designs in the 1930s and 1940s. These gates were designed as a series of two-dimensional (2-D) trusses and were referred to as truss-type gates. They were typically as heavy as (or heavier than) girder designs, so fabrication and maintenance costs were high. For this reason, they were not adopted as a standard design. More recently, the use of computer designed three-dimensional space frame gates constructed with tubular sections has been investigated and may be practicable in some situations. However, tubular sections are discouraged in HSS because the interior surfaces are difficult to inspect.

10.7.6 Overflow or Submersible Gates. These gates are generally of a standard configuration, but are designed to allow water to pass over the top the gate. Deflector plates are often provided on the downstream side of the gate to allow water and debris to pass over the framing with minimal impact. Other gates have been designed to include a downstream skin plate, so the gate is completely enclosed. Vibration problems have been prevalent with this type of gate. See Chapter 11 for design guidance on these types of gates.

10.8 Serviceability Requirements. Serviceability requirements for gates are provided in Chapter 4. Additional guidance follows.

10.8.1 Girder Deflections. Girder deflections must be limited to avoid unwanted vibrations and leakage at the sill, and plates must be sized to avoid plate membrane behavior. The maximum girder deflection between end frames must be limited to 1/800 times the span, and the maximum girder deflection for the cantilever portion between the end frame and pier face must be limited to 1/300 times the cantilever length. The skin plate deflection must be limited to 0.4 times the plate thickness. Consider whether bulkhead slots are needed to permit gate maintenance.

10.8.2 Sidesway and Binding. Gates may include various side bumpers or rollers to limit side sway deflection and binding, such that gate operation is not impeded. For conditions where the gate is supported on only one side and has sufficient flexibility, the gate will rotate so the bumpers bear on the side-seal plates. If this occurs, the normal force between the bumper and plate influences the potential for gate binding between the piers due to frictional forces that occur with gate movement. Bumpers are placed in line with top and bottom horizontal girders so forces are transferred directly into the girders.

10.8.2.1. Gate Twisting. The gate twisting forces occurring from the one side lifted case are resisted by the vertical truss and horizontal girder assembly with girders acting as chords and truss members acting as bracing. The entire assembly can be idealized as a truss or frame depending on connection stiffness and eccentricities. Two-dimensional models are adequate for computing reaction forces applied to the bumpers and internal forces in girder and brace members.

10.8.2.2. Torsion Forces. Torsion in other members, like strut arms, resulting from gate twist can be evaluated based on the geometry of the gate and on the maximum twist that can occur due to clearance between bumpers and piers. More refined analyses can be conducted if impacts to other members is needed.

10.8.2.3. Roller Failures. If operational requirements include lifting or closing the gate when it is supported on one side only, the designer should consider the possibility of roller failure, where rollers are provided, or degraded embedded metal surface conditions (due to corrosion or the presence of foreign materials or lichen or bryophyte growth) on the effective roller drag or frictional resistance.

10.8.3 Ice Control. Where ice may accumulate and inhibit gate operation, heaters must be considered in the design.

10.8.4 Vibration. Vibration due to flow under the gate is a consideration when designing Tainter gate details. To limit vibration, the bottom lip of the Tainter gate and sill should be detailed as described in the design details section of this chapter.

10.8.5 Debris. Consideration should be given to debris buildup in cases where there will be downstream submergence. In extreme cases, floating debris swirling behind the gate has damaged members. To avoid damage, some gates have been fitted with downstream deflector plates to protect the framing from impact due to debris. Debris protection should be provided as needed on the end frames and on the downstream flanges of girders to avoid impact damage and binding of lodged debris.

10.9 Material Selection. Material selection guidelines for gates are provided in Chapter 3.

10.10 Analysis and Design Considerations.

10.10.1 Two-Dimensional Analysis. Traditionally, 3-D behavior of Tainter gates has been simulated using independent 2-D models. The overall behavior is simulated by modeling separately the behavior of the skin plate assembly (composed of the skin plate and supporting ribs), girder frames (composed of a horizontal girder and associated struts), vertical end frames (composed of struts and braces), and the vertical downstream truss. Analysis of the 2-D models is interdependent. Loads on one model create reactions that are applied to other members and the reactions are used in other models. For example, the reactions of the skin plate assembly model are the loads applied to the horizontal girders and used in the girder frame model. An alternative for each 2-D model is described in the following paragraphs. Loads applied to the models as reactions are not factored since they are the result of the applied factored loads.

10.10.2 Skin Plate Assembly. The skin plate thickness is dependent on the rib spacing (skin plate span), and the rib size is dependent on the skin plate thickness since an effective portion of skin plate acts as a rib flange.

10.10.2.1. Skin Plate Design. The skin plate design stress should be based on the negative moment at the supports for equally spaced interior ribs (fixed-end moment). The spacing between the exterior ribs at the ends of the gate should be adjusted such that the moment does not exceed the fixed-end moment of the interior spans. For gates with a wire rope hoist, thicker plate and/or closer rib spacing is normally required under the wire rope due to the rope pressure exerted on the plate. Because of the varying loading on the skin plate, it may be economical to vary the thickness of the plate over the height of the gate. It is recommended to maintain a minimum thickness of 3/8 in., while a thickness greater than 3/4 in. should be avoided.

10.10.2.2. Rib Design. An effective width of skin plate acts as the upstream flange of the rib. The effective skin plate width of the rib, $b_{eff,}$ shown in Figure 10.17, may be taken as a non-compact flange using the case of "uniform compression in all other stiffened elements" as defined in AISC. Girder spacing is adjusted to equalize maximum rib bending moments at various locations along the rib with consideration for impacts of rib reactions on horizontal girders (see discussion in Horizontal Girder). Rib spacing is adjusted at wire rope locations to accommodate localized loads caused by the wire ropes.

10.10.2.3. Model Assumptions. For 2-D analysis, the skin plate and ribs are assumed to have zero curvature. The skin plate deflects in two directions: one in a direction transverse to the ribs, and the other parallel to the ribs. The resulting stresses can be determined using the following two simplified models.

10.10.2.3.1 Loading Transverse to Ribs. Each unit width of skin plate is assumed to act as a continuous beam spanning between ribs (Figure 10.16). Boundary conditions consist of simple supports located at each rib. The skin plate does not need to be designed for IM, IX, or BI. Reaction loads (sill, hydraulic cylinder, and wire rope reactions) are the result of factored loads and an additional load factor is not applied.

10.10.2.3.2 Loading Parallel to Ribs. The composite rib-skin plate acts as a continuous beam supported by the horizontal girders (Figure 10.17). Boundary conditions consist of simple supports at each girder for the rib members.

10.10.3 Girder Frame. The girder frame consists of a horizontal girder and the two struts attached to the girder. The girders act as rib supports and are located to achieve an economical design for the ribs. However, the location of the girders also affects the load on each girder since the rib reactions are the girder loads. The overall economy of rib and girder design should be considered. The analytical model is a simple frame (Figure 10.18) with the strong axes of the struts oriented to resist flexural forces in the plane of the frame. Under unsymmetrical loading, and since the frame is not fully restrained laterally, the frame must be subjected to a calculated sidesway. The amount of sidesway is limited by piers.

10.10.3.1. Strut Orientation. The distribution of flexure along the length of the girder is significantly influenced by the orientation of the end frames. Maximum girder moments will result with struts that are parallel to the pier face. The maximum girder moment is reduced if the struts are inclined and will be minimized if the struts intersect the girder at approximately one-fifth the gate width from the pier face.

10.10.3.2. Inclined Struts. With inclined struts, the limit state of lateral torsional buckling of the girder must be checked since a significant length of the downstream flange of the girder will be in compression. The unsupported length of the compression flange between the struts is the distance between downstream vertical truss connection, when the connection is in the compression region, and the strut. When the vertical truss connection is not in the compression region, use the point of contraflexure.

10.10.3.3. Modeling Assumptions. Except for the gate jammed load Combination, where lateral support is provided due to binding, the frame should be assumed to be unbraced for stability analysis, unless it can be shown otherwise that the frame is braced. Where cylindrical pins are used at the trunnion, a fixed support should be assumed. Where spherical bearings are used at the trunnion, a pinned support should be assumed.

10.10.3.4. Loading. Girder loading consists of rib reaction forces from the rib analysis described previously plus operating machinery loads. It is assumed that girder lateral bracing resists girder torsional forces that are caused by gravity loads, or that torsional forces are inconsequential where girder lateral bracing is not necessary, and thus torsional effects are not considered. Since the rib reaction forces are the result of factored loads, no additional load factors are required. All reactions from this analysis should be considered when calculating effects of friction at the trunnion.

10.10.4 End Frame. The analytical model for the end frame consists of struts, strut bracing, girder webs, girder lateral bracing, and the skin plate assembly (Figure 10.19 and Figure 10.20). The girder webs and skin plate assembly are included only to transfer loads and maintain the correct geometry, not for design of these elements, and thus should be relatively stiff compared to the other elements.

10.10.4.1. Bracing Configuration. End frame bracing should be spaced to achieve adequate weak axis slenderness ratios for the struts and must be designed to resist calculated forces. Bracing members may include significant flexural forces depending on member sizes, connection rigidity, and trunnion friction.

10.10.4.2. Modeling Assumptions. Element types (truss, beam, or frame) should be consistent with connection detailing except that girder webs should be pinned so that girder lateral bracing elements resist all forces transverse to the girder. This will ensure that bracing is proportioned so that girder torsion is limited. The end frame model is used to determine sill reactions, operating machinery reactions, trunnion reactions, and end frame member (weak axis strut and bracing) forces. The end frame analysis results are combined with the girder frame analysis results to analyze the struts for biaxial bending.

10.10.4.3. Loading. End frame loads consist of trunnion reactions from the girder frame analysis, gravity loads, and operating equipment forces. For gate operating load cases, the initial forces obtained from the end frame analysis are used to calculate trunnion reaction and the trunnion reaction is used to calculate the initial trunnion friction moment. Addition of this moment will increase the operating equipment reactions which in turn increases the trunnion reactions. The recalculation of reaction forces is iterated until the trunnion friction forces converge.

10.10.5 Downstream Vertical Truss Model. Bracing members that make up the downstream vertical truss are proportioned for forces that occur when the gate is supported at one side. To determine these forces accurately, a 3-D analysis is required because of the complex interaction of the skin plate assembly, end frames, and bracing members. However, a 2-D model can be used to conservatively approximate the forces. For gates that have a low height-to-width ratio, additional bracing may be required to achieve adequate stiffness to prevent unacceptable torsion and lateral displacement of the gate. Side bumpers should also be added to limit lateral movement. Single angles, double angles, or WT sections are commonly used for the vertical truss bracing members.

10.10.5.1. Modeling Assumptions. Continuous frame elements simulate girder flanges, and the bracing members are represented by truss elements. The model can be assumed pinned at one lower corner, and supported horizontally with the ability to translate in the vertical direction at an upper corner (see Figure 10.21).

10.10.5.2. Loading. Applied loads will include only dead and gravity loads for all load cases except the gate jammed case where the machinery load will be applied. This should result in conservative member forces as the skin plate assembly will not contribute to the resistance.

10.11 Fabrication and Maintenance.

10.11.1 Skin Plate Assembly. Skin plate splices are complete joint penetration welds. Where wear and deterioration of the skin plate thickness along the bottom of the gate is expected, additional plate thickness, beyond what is needed for strength and deflection limits, should be provided. Wear plates are typically provided under wire rope locations to protect against abrasion of the wire rope against the skin plate. Ribs must be proportioned to provide adequate clearances for welding and painting. The minimum rib depth is typically 8 in., although shorter clearances can be used if it can be shown that the required weld quality can be achieved.

10.11.2 Horizontal Girder. Use of a minimum number of girders will simplify fabrication and erection and facilitate maintenance. Drain holes with smooth edges should be provided in the girder webs at locations that ensure complete drainage of the web. The seal weld connecting the rib to the girder flange is prone to fatigue cracking. This may not be an issue for spillway Tainter gates unless significant vibration is allowed. This can be an issue for navigation Tainter gates due to stress cycles incurred under gate opening and closing operations. In cases where fatigue is a possible limit state, alternate gate geometry should be considered.

10.11.3 End Frames.

10.11.3.1. Bracing. Strut bracing are usually wide flange sections, with the same depth as the struts, to simplify connections.

10.11.3.2. Connection to Trunnion Hub. Struts are welded to trunnion hub flanges, with clearance provided between the ends of intersecting struts (Figure 10.22). These connections generally involve complete joint penetration splices involving thick plates, thus complicating fabrication requirements. The termination of the strut-to-trunnion hub transition webs creates congestion at the trunnion hub resulting in difficult welding conditions. The webs can be terminated short of the hub if strut-to-trunnion hub transition flanges alone satisfy all limit states. This results in less congestion and facilitates the welding to the trunnion hub.

10.11.3.3. Strut Geometry. Since the struts do not lie in the same plane (i.e., they are rotated with respect to one another), the trunnion hub must be fabricated to accommodate this rotation. This can be done by forming the strut-to-trunnion hub transition flanges with plates welded together at the rotation angle or by bending the plate to match the rotation angle. The struts can also be placed in a single plane (not rotated with respect to each other). For gates with more than two girders, this results in differing support locations for the horizontal girders. However, fabrication of the strut-to-trunnion hub flange connection is simplified since all struts fall in a single plane.

10.11.4 Downstream Vertical Truss. The downstream vertical truss provides lateral bracing for the horizontal girders and adds torsional rigidity when the gate is supported only at one end. It also provides gate rigidity for resisting gravity loads with symmetric hoist support conditions, and structural rigidity during field erection. For gates that have a low height-to-width ratio, additional bracing may be required to achieve adequate stiffness to prevent unacceptable torsion and lateral displacement of the gate. Side bumpers should also be added to limit lateral movement. Single angles, double angles, or WT sections are commonly used for the vertical truss bracing members.

10.11.5 FCM. Failure critical bracing members subject to flexural tension are FCM and should be designed and fabricated accordingly. Struts are primarily compression members subjected to biaxial bending and under some design load cases may see a small net tension stress. For gates where tension stresses are sufficiently low, the struts should not be considered fracture critical. Trunnion hub flanges are proportioned to resist the strut flexural, shear, and axial loads.

10.12 Design Details.

10.12.1 Seals. The seals used in Tainter gates follow standard details. However, there will be some differences based on operational requirements and the degree of water tightness required for the specific project. Devices for preventing the formation of ice, or for thawing ice adhering to the gates and seals, will be necessary for the gate to function during subfreezing weather. Operation in winter will be facilitated by the use of deicing systems and (as in all seasons) by clearing trash.

10.12.1.1. Side Seals. The seal attachment plate must have slotted bolt holes to allow for field adjustment of the seals. The seals are normally installed with a pre-compression against the side-seal plate, which allows for construction irregularities and creates a tighter seal under low heads. The standard side-seal configuration provides for an increase in the sealing force in proportion to increased head. Seals usually tend to leak under low heads rather than high heads.

10.12.1.2. Bottom Seals. The lip of the Tainter gate should form a sharp edge and the downstream side of the lip should be perpendicular to the sill (Figure 10.23). For most gates, the preferred seal configuration is provided by direct contact between the skin plate edge and the sill plate. The rubber seal may be eliminated where leakage can be tolerated. If leakage cannot be tolerated, a narrow rubber bar seal attached rigidly to the back side of the gate lip should be used, or a rubber seal can be embedded in the gate sill plate.

10.12.2 Drain Holes. Drain holes should be located at all locations where water could be trapped for all gate positions. This includes the webs of girders, end frames, strut-to-trunnion hub connection, and bracing members. Minimum hole size is 2 in. in diameter. Corner copes should be placed in stiffeners to avoid pockets of water between stiffeners. Holes in flanges should generally be avoided.

10.12.3 Gate Stops. Many gates are provided with gate stops to limit the gate movement. The machinery is designed to stop before the gate contacts the gate stops, but the stops keep the gate from over-traveling due to wind or water loading in unusual situations. Stops are more often used with the wire rope hoist system since the ropes offer no resistance to upward movement. The stops are usually a short section of steel beam embedded and anchored into the pier. The stops will contact a bumper on the gate if the gate travels beyond a certain position. See Figure 10.26 for example gate stop details.

10.12.4 Bumpers and Rollers. To help ensure that the gate moves smoothly between the piers, even if lifted from only one side, bumpers or rollers are generally located at the ends of the top and bottom horizontal girders near the upstream or downstream flanges. Bumpers can be fitted with a bronze or ultra-high molecular density plastic rubbing surface to reduce friction and prevent gate binding. Figure 10.27 and Figure 10.28 show some common bumper details. Rollers can be used in place of a rubbing surface but the bushings must be maintained to ensure the rollers function properly over time.

10.12.5 Dogging Devices. Some gates are provided with devices to temporarily support the gate in a full or partially raised position. These dogging devices will relieve the load on operating machinery and can facilitate maintenance or repair of the machinery or gate while the gate is raised.

10.13 Trunnion Assembly.

10.13.1 General Description. The trunnion assembly provides support for the Tainter gate while allowing rotation for operational use. Design of lubrication systems, tolerance and finish requirements, material selection, and determination of allowable stresses should be coordinated with a mechanical engineer for certain components.

10.13.2 Components. The trunnion assembly consists of the trunnion hub, trunnion yoke assembly, and a trunnion pin with a bushing or bearing. Bushings or bearings are provided to minimize friction and wear during rotation of the gate about the trunnion pin. The trunnion assembly is designed to transmit gate load directly to the trunnion girder. Figure 10.29 illustrates typical details for a cylindrical bushing assembly.

10.13.3 Spherical Bearing Considerations. Spherical bearings are generally more expensive than cylindrical bearings due to their complexity. However, spherical bearings will compensate for a degree of misalignment of gate arms, construction tolerances, thermal movement, and uneven gate lifting. Spherical bearings accommodate an angular rotation, transverse to the pin centerline, in the range of 6 to 10 degrees depending on bearing size. Gate arms associated with spherical bearing are usually heavier due to an increased buckling length. Figure 10.30 shows a spherical bearing configuration. Figure 10.31 shows the layout of components of the trunnion assembly. Compared to cylindrical bearings, spherical bearings:

10.13.3.1. Are narrower,

10.13.3.2. Reduce or eliminate stresses at the edge of the bearings,

10.13.3.3. Produce a more uniform pressure distribution over the trunnion pin, and

10.13.3.4. Can reduce trunnion pin moments and gate arm stresses due to misalignment.

10.13.4 Trunnion Yoke Assembly. The trunnion yoke assembly is fabricated of welded structural steel and consists of two parallel plates (yoke plates) welded to a stiffened base plate (Figure 10.32). The yoke plates are machined to receive the trunnion pin and associated components. The assembly is bolted to the trunnion girder.

10.13.4.1. Shear Transfer. Where shear resistance is needed between the base plate and trunnion, girder shear bars are welded to the base plate, or other means of shear transfer are provided. The trunnion yoke assembly is bolted to the trunnion girder. Consideration should be given to using partially prestressed high-strength stud bolts to minimize movement relative to the trunnion girder.

10.13.4.2. Positioning. Final adjustments are necessary after installation to ensure proper alignment. Adjustments are accomplished with horizontal and vertical jackscrews or other means.

10.13.5 Trunnion Hub. The hub can be fabricated of cast, forged, or structural steel. Castings and forged steel are typically more costly than welded steel construction. The inside bore is machined to tolerance for proper fit with the trunnion bushing or bearing. The hub is welded to the strut-to-trunnion trunnion hub transition flanges and is joined to the yoke with the trunnion pin. The hub is typically wider than the gate arm extensions to allow for a uniform distribution of stress and to provide clearance for a welded connection. A bushing or bearing is provided between the hub and trunnion pin to reduce friction. The trunnion hubs and yokes should be machined after fabrication welding is completed and the parts are stress relieved by heat treatment where heat treatment is needed.

10.13.6 Trunnion Pin. The trunnion pin transfers the gate loads from the hub to the yoke side plates. A retainer plate, welded to the trunnion pin, is fitted with shear pins to prevent the trunnion pin from rotating. The retainer plate and pin are connected to the yoke with a keeper plate. The trunnion pin is designed as a beam with simple supports at the centerlines of the yoke plates. The retainer plate and shear pins are designed to carry frictional loads produced when the Tainter gate is raised or lowered. The weld connecting the retainer plate to the trunnion pin (Figure 10.33) must be sized to prevent rotation.

10.13.7 Mechanical Features. See EM 1110-2-2610 for guidance on mechanical features including trunnion pins, bushings, and bearings.

10.13.8 Material Selection.

10.13.8.1. Trunnion Hub. Trunnion hubs should be corrosion resistant, weldable, and machinable. The trunnion hub is typically machined from cast steel (ASTM A27) or forged steel (ASTM A668).

10.13.8.2. Shear Pins or Keeper Plates. Material for shear pins should be corrosion resistant and machinable. The shear pins are typically machined from cast or forged steel.

10.13.9 Design Requirements. The trunnion yoke and trunnion hub will be designed for loads specified in this chapter. Stress limits and tolerances for bearings and bushings will be established by the mechanical Engineer. Provide loads to the mechanical engineer using the load combinations of this chapter using load factors of 1.0 for all loads. Use the trunnion pin friction coefficient specified in this chapter for the design of the trunnion assembly. The bearing stress between the yoke base plate and the trunnion girder should include both the pretensioning force of the anchorage stud bolts and global gate reaction forces.

10.13.10 Analysis and Design Considerations. The design of the trunnion assembly affects the end frame design and the required hoist capacity because of the trunnion pin friction. The centerline bearing of the trunnion hub is commonly offset with respect to the centerline of the gate arms (Figure 10.34). This offset is recommended so that a uniform bearing stress distribution occurs under maximum loading (gate is nearly closed and impact and gravity loads are applied). Other load conditions will produce non-uniform bearing stresses on the trunnion pin and bushing. These load conditions must be investigated individually. The transfer of forces between the trunnion pin, retainer plate, shear pins, and trunnion yoke assembly will be considered in design. Coordinate the bushing, trunnion pin, and trunnion bearing designs with the mechanical engineer.

10.13.10.1. Trunnion Yoke Assembly. The yoke side plates must be sized to resist trunnion pin bearing load and lateral gate loads. The base plate and stiffeners must be designed to resist contact pressure between the yoke bearing plate and trunnion girder based on gate reaction forces and bolt stressing loads, as shown by Figure 10.35. Analyze the base plate as a simple beam supported by the parallel yoke plates with a distributed load equal to the bearing pressure between the base plate and trunnion girder.

10.13.10.2. Trunnion Hub. The trunnion hub can be modeled as a cantilevered beam subjected to a distributed load from the trunnion pin as shown in Figure 10.34. The cantilevered portion of hub extends beyond the flange of the trunnion arm extension. Design checks for bending and horizontal shear are made along section A-A of Figure 10.34.

10.13.11 Trunnion Alignment.

10.13.11.1. Tolerances for the trunnion axis centerline, with respect to the piers, are based on clearance requirements between the side seal and seal plate embedded in the pier and between the gate bumper and the pier. If the trunnion centerline is not perfectly aligned, out-of-plane sweep (with respect to the pier) will occur when the gate is moved from the closed position to the fully raised position. Tolerance requirements may be relaxed if side-seal plates are terminated and a recess in the pier is provided above upper pool.

10.13.11.2. The centerlines of trunnions at each gate arm must pass through a common axis to avoid unintentional friction loads due to binding as the gate rotates through its operational range. Tolerance requirements should be determined based on gate size and should be included in project specifications. Horizontal and vertical jack screws are provided on the trunnion hub for setting and adjusting the trunnion yoke assembly so that the trunnion hub axes are on a common horizontal line. Second-placement concrete or grout can be used to fill the space between the trunnion girder and yoke and between the trunnion assembly and side bearing plate on the pier.

10.14 Gate Anchorage Systems.

10.14.1 General Description. The trunnion girder is held in place by an anchorage system that extends into the concrete pier. Anchorage systems can be classified as prestressed or non-prestressed. The prestressed system consists of a high-strength, post-tensioned anchor system while the non-prestressed system incorporates structural steel components.

10.14.1.1. Prestressed Anchors. Prestressed systems consist of groups of post-tensioned members that anchor the trunnion girder to the pier. The post-tensioning operation creates an initial compressive stress in the system that acts to maintain trunnion girder-to-pier contact and to reduce structural cracks in the pier concrete. Figure 10.42 and Figure 10.43 show a typical post-tensioned anchorage system.

10.14.1.1.1 The anchors, which are placed inside ducts embedded in the concrete, are tensioned after the concrete has set and cured. The annular space between the post-tensioning steel and duct is grouted for corrosion protection or other corrosion inhibiting compounds after the anchors are post-tensioned. Generally, two groups of anchorage steel are installed, one near each pier face. A limited bearing area is provided directly under each anchorage group.

10.14.1.1.2 A compressible material is placed between the anchorage bearing areas at the trunnion girder-to-pier interface to intentionally prevent the transfer of stress across this region. This detail enhances structural performance by reducing negative bending moments in the trunnion girder. A larger moment arm between anchorage groups is available to resist nonsymmetrical loads. The large bearing pressures between the girder and the pier create bursting, spalling, and edge tension stresses. Reinforcement must be provided to resist these stresses.

10.14.1.2. Non-Prestressed Systems. Non-prestressed systems may consist of embedded rolled steel beams, built-up sections, or large-diameter rods. Non-prestressed systems are relatively easy to design and install. However, non-prestressed systems allow greater deflections of the trunnion girder, allow tension and possibly structural cracking in concrete (bonded anchors), and require a large cross-sectional area of steel. This type of anchorage system is not recommended except for projects with small Tainter gates. This chapter provides criteria only for prestressed systems.

10.14.2 Components. A complete post-tensioned anchorage system includes tendons (bars or strands), anchorage devices or bearing plates, ducts, end caps, grout tubes, couplers, anchorage zones, and a corrosion protection system.

10.14.2.1. Anchorage Zones. Anchorage zones include a portion of pier in the vicinity of tendon anchorage at either end of the tendons. The anchorage zone is geometrically defined as the volume of concrete through which the concentrated tendon force applied at an anchorage device (or girder-to-pier-bearing area) dissipates to an area of more linear stress distribution. The area preceding the linear stress distribution is called the disturbed or D-region region (see AASHTO).

10.14.2.2. Tendons and Anchorage. Tendons can be high-strength, low-alloy steel bars or strands. The tendons are post-tensioned at the trunnion girder (referred to as the live end) to hold the trunnion girder to the pier. They pass through the trunnion girder and terminate at embedded bearing plates or anchorage devices (referred to as the dead end) within the pier. The embedded length of the tendons is typically 30 to 50 ft. Longer lengths provide better control of post-tensioned force and have higher pullout resistance since a larger area of concrete is effective in resisting the forces.

10.14.2.3. Couplers. Couplers are used to splice tendons. However, these are not usually required since anchors are produced in sufficient lengths to make the use of couplers unnecessary. The embedded ends of the tendons are supported by a positive means rather than by gripping devices, which are vulnerable to slippage if grout penetrates into the anchorage device.

10.14.2.4. Live and Dead Ends. The dead-end termination points of individual tendons are staggered from one another to distribute the anchorage forces over a larger area of the pier. Live-end anchorage devices may consist of a wedge, bell, or flat plate system and typically seat against the trunnion girder. Cable strands may also be continuous, extending from the live end to a fixed loop or 180-degree bend (that acts as the dead-end anchorage) back to the live end.

10.14.2.5. Tendon Ducts or Sheathing. Ducts encase the tendons to separate them from the surrounding pier and abutment concrete and allow tensioning after pier concrete has cured. The ducts also protect anchors during placement of concrete and act as part of the corrosion protection system. Ducts should be rigid or semi-rigid, and either corrugated metal or corrugated plastic. Include specification requirements that preclude the intrusion of water or other deleterious material prior to placement of concrete.

10.14.2.6. Corrosion Protection Systems. The corrosion protection system for tendons consists of tendon ducts, duct fittings, connections between ducts and anchorages, grout tubes, end caps, and grout. Gels may be used in lieu of grouts. A proper duct system will prevent infiltration of moisture into the duct. Special fittings are provided for duct splices and connections between ducts and anchor plates, using threaded or slip connectors and are provided with seals to prevent infiltration at these locations. Grout tubes extend from the sheathing to allow access for grouting. Grout end caps are placed over the live end of the anchors and anchor nuts or wedges after stressing is complete and excess tendon removed. The grout encapsulates the tendon to prevent corrosion. See also Federal Highway Administration (FHWA) Publication FHWA-NHI-13-026, Post-Tensioning Tendon Installation and Grouting Manual.

10.14.3 Material Selection.

10.14.3.1. Anchors. Post-tensioning bars are high-tensile alloy steel, conforming to the requirements of ASTM A722. When specifying greater than 1.375 in. diameter bars, verify bars are subjected to cold-stressing to not less than 80% of the minimum tensile strength, and then stress-relieved to achieve the required properties. Otherwise, specify material working to ensure sufficient ductility and relaxation properties are achieved. Cable strands conform to ASTM A416 with a minimum strength of 270 ksi.

10.14.3.2. Concrete. The minimum compressive strength of concrete in anchorage zones will be 4500 psi. The minimum compressive strength of concrete between the anchorage zones will be 4000 psi. Higher concrete strengths may be used if required due to bearing, spalling, or bursting stresses. The higher concrete strength may be used within and outside of the anchorage zone when the difference between the two is small (1,000 psi or less). Otherwise, the zones must be separated. This can be done through formwork or mixing of concretes within a lift. Consult a materials engineer when designing the concrete placement plan. The maximum concrete aggregate size should be selected to pass between ducts and reinforcing bars.

10.14.4 Design Requirements. A properly designed anchor system will prevent structural cracking of concrete; limit trunnion deflections; account for time-dependent stress losses; and safely accommodate specified loads for the life of the structure.

10.14.4.1. Design basis. Except as modified herein, the post-tensioned anchorage system must be designed according to current AASHTO LRFD Bridge Design Specifications. Other standards or specifications can be used if they can be shown to produce comparable results. The system will be proportioned such that the strength and serviceability limit states are met when the system is subject to load combinations as specified in this chapter. The anchorage zones, including spalling, bursting, and edge tension reinforcement, must be designed following procedures described by AASHTO using factored jacking forces. In general, the amount of anchorage steel required will be controlled by service limit states.

10.14.4.1.1 Strength Limit State. The strength limit states ensure that the anchorage system will resist all factored loading combinations without failing.

10.14.4.1.2 Service Limit State. Service limit states are provided to restrict stresses, deformations, and cracks that adversely affect performance under typical or normal loading conditions at specified stages of use.

10.14.4.1.3 Lift-Off. Excess deflections and structural cracking may occur if the trunnion girder loses contact with the pier (lift-off). Lift-off can be controlled by an effective anchorage force that maintains residual compressive stresses between the trunnion girder and pier. The design of the gate anchorage system is based on the strength and service limit states.

10.14.4.2. Loads. The maximum applied load will usually occur when one gate is raised just off the sill and the adjacent gate (if applicable) is unloaded.

10.14.4.2.1 Analysis Models. For the case where the trunnion girder is loaded on one side only, the pier concrete can be idealized as an eccentrically loaded tension member or as a post-tensioned beam assuming the residual compressive stress has been lost on the loaded side. For the case where both sides are loaded, it can be assumed that the compressive residual stress has been lost and all of the loads are transferred to the anchorage only. Other suitable models may be used.

10.14.4.2.2 Strength Limit State. The strength limit of the tendons will be as specified in AASHTO with a resistance factor of 1.0. Gate reactions from factored load combinations will be used. Resistance factors for the design of anchorage zones will be as specified in AASHTO. The initial jacking force is based on the required long-term effective stress and includes all expected prestress losses. Load combinations of this chapter will be applied for the design of tendons. The anchorage zone will be designed for the jacking load using only a load factor of 1.2.

10.14.4.2.3 Serviceability Limit State. The serviceability limit states are intended to limit tendon and pier concrete stresses at three stages of jacking (tendons only) after transfer of jacking loads and before losses, and after losses with trunnion service loads. Tendon stresses will be limited to those values provided in AASHTO. The tendons will be sized to maintain a minimum compressive bearing stress between the girder and the pier of 200 psi under service loads. For pier concrete, stresses will be limited as required in AASHTO, except tension stresses, which are not allowed.

10.14.5 Analysis and Design Considerations.

10.14.5.1. Anchorage Force. The trunnion girder is assumed to behave as a simply supported beam, with cantilevered end spans as shown in Figure 10.36. The gate anchorage represents the simple supports. The support position is assumed to lie at the centers of gravity of the anchorage tendon groups. Applied loads are the trunnion reactions. Bearing stresses should be calculated over the 2-D bearing surfaces, considering any eccentricity of loads compared to the bearing centroid. Figure 10.37 shows a typical design layout and assumed section properties of the bearing area.

10.14.5.2. Analysis Models . The bearing stress distribution for an unloaded gate and a loaded gate is shown in Figure 10.38. Anchorage forces are selected to maintain required pier contact pressures. Elastic analysis of the anchorage zone is discussed in AASHTO. Elastic analysis can be accomplished using 2-D or 3-D finite element models. Practical analyses are limited to linear behavior; however, models can be used to validate more simplified analyses and to determine required reinforcement in areas of complicated local conditions. The post-tensioned anchorage design must be coordinated with the engineer designing the dam pier.

10.14.5.3. Girder Movement. The minimum residual bearing stresses between the pier and trunnion girder due to the load combinations of this chapter must be sufficient to prevent sliding on the trunnion girder bearing surface due to trunnion reactions acting parallel to the bearing surface. See Figure 10.39. This condition is achieved when Equation 10 is satisfied. The right-hand side of Equation 10.15 represents the available shear strength due to frictional resistance. If this condition cannot be met, other means such as dowels or mechanical confinement by adjacent concrete must be provided.

10.14.5.4. Effective stresses. Estimated lump-sum losses (assumed values not supported by computation) may be used for the initial design of prestressed concrete. However, the final design should account for individual losses due to elastic shortening, shrinkage, concrete creep, steel relaxation, anchorage set, and friction. For straight tendons, losses due to friction may be neglected. The value of anchor set loss can be assumed or obtained from the manufacturer and should be verified prior to construction. A more refined incremental time-step analysis of losses is typically not required.

$$V_u \leq 0.85 \mu R \qquad \text{(Equation 10.15)}$$

Where:

V_u = factored shear force

μ = coefficient of friction for the interface

R = residual compressive force between the girder and pier

10.14.5.5. Anchor Depth. The depth of anchorage into the pier or abutment should be maximized to the greatest possible extent in order to maximize the area of concrete effective in resisting the anchorage forces. Anchorage tendons of approximately 80 to 90% of the gate radius have been used with satisfactory performance. Interference with embedded metals (side-seal plates) usually limits the anchorage depth.

10.14.5.6. Anchorage Zone Reinforcement. A portion of pier concrete adjacent to the trunnion girder and anchor plates will be subject to tensile stresses. Tension exists in a portion of concrete located at the center of the anchorage tendons (bars), ahead of the anchorage device or girder. This area is termed the bursting zone. At the end of the pier adjacent to the girder, edge tension forces may exist along the sides and end surface. The edge tension forces along the end surface are referred to as spalling forces. Reinforcement is provided where required in the tendon anchorage zones to resist bursting and edge tension forces induced by tendon anchorages.

10.14.5.7. Anchorage Zone Design Guidance. Design guidance regarding bursting, spalling, and edge tension is specified by AASHTO.

10.14.6 Serviceability. Corrosion of the tendons and anchorage components is the primary serviceability concern regarding design of the anchorage system. The anchorage system must be doubly protected against corrosion as detailed in FHWA-NHI-13-026.

10.14.7 Design Details.

10.14.7.1. Anchorage Layout. The post-tensioning anchors should be installed in two groups, with each group being located as close to the adjacent pier face as practicable, approximately 10 in. to provide sufficient room for conventional reinforcement. Anchor ends should be installed at staggered lengths so all anchorage loads are not transferred to one plane; this will significantly reduce the quantity of reinforcing steel that would otherwise be required to control vertical tension cracks in the concrete. A staggered spacing of 2 to 4 ft has been used successfully in the past to distribute anchorage forces. See Figure 10.43. Typical minimal pier widths to accommodate trunnion anchorages is 10 ft where a trunnion girder supports two gates and 8 ft at the ends where only one gate is supported. This ensures adequate room is provided for the placement of anchors.

10.14.7.2. Construction. Tendons can be placed horizontally if the gate trunnion is oriented so that the vertical component of thrust is negligible. With horizontal tendons, fewer lifts of anchorage zone concrete are required and construction is simplified. Prestressing tendons should be installed before installation of conventional reinforcement and forms. This will permit close inspection of the embedded ends to ensure proper construction. Pier reinforcing or steel frames may be used to support anchorages prior to placement of concrete.

10.14.7.3. Anchorage Systems. Recent failures of post-tensioned anchor bars have occurred. Embedded anchors are difficult to monitor over time due to lack of visual access. Designers should provide additional measures commensurate with consequences of failure. Additional measures include:

10.14.7.3.1 Visual access at anchor ends,

10.14.7.3.2 Improved corrosion protection systems,

10.14.7.3.3 Enhanced quality assurance during installation,

10.14.7.3.4 Internal monitoring using instrumentation,

10.14.7.3.5 Designing for replaceable anchors, and

10.14.7.3.6 Adding anchors for redundancy.

10.15 Trunnion Girder.

10.15.1 General. Trunnion girders can be constructed of post-tensioned concrete or steel. Selection is dependent on a variety of factors including availability of quality fabricators, site exposure conditions, economics, and designer preference. Post-tensioned concrete trunnion girders possess greater stiffness compared to steel girders, resulting in minimal deflections, and offer significant resistance to torsional loads. Steel girders are susceptible to corrosion and are more flexible than concrete girders, but are more easily retrofitted and repaired. Due to their higher flexibility, steel girders are more often limited to use with smaller gates. Concrete girders have historically been the preferred choice and have demonstrated acceptable performance.

10.15.2 Components.

10.15.2.1. Concrete. A concrete trunnion girder is post-tensioned longitudinally to increase girder resistance to flexure, shear, and torsion, and to control in-service deflections. Longitudinal ducts are provided for the post-tensioning tendons and recesses are commonly provided for second-placement concrete pours between the trunnion assembly and girder. Conventional reinforcement is provided to resist shear, torsion, bursting, and reverse loading forces, and to control spalling. Figure 10.40 shows the upstream face of a typical concrete trunnion girder.

10.15.2.2. Steel. Steel trunnion girders are typically welded I- or box-shaped girders. I-shaped members are primarily used where torsion is not a significant concern. Box-shaped girders are more difficult to fabricate, coat, and inspect than I-shape girders. Stiffeners are used to increase web stability where post-tension anchorage forces are applied and in areas of high shear. Stiffeners also increase torsional stiffness.

10.15.3 Material Selection. The minimum compressive strength of concrete should be 5000 psi. Post-tensioning bars should be of high-tensile alloy steel, conforming to the requirements of ASTM Designation A722. Post-tensioning strands are low-relaxation, high-tensile, seven-wire strands conforming to ASTM A416 with a minimum strength of 270 ksi. Steel trunnion girders are considered FCM and require appropriate weld quality and material properties.

10.15.4 Design Requirements. Trunnion girders must withstand combined forces of bending, torsion, shear, and axial compression due to trunnion reaction and anchorage forces. Girder torsion occurs due to trunnion pin friction and eccentric loads applied at the trunnion pin. However, torsion due to trunnion pin friction should not be considered if it counteracts torsion resulting from eccentricity. For multi-gate projects, the operating condition of adjacent gates must be considered when evaluating the loading condition on the trunnion girder (i.e., when one gate is closed and the other gate is open, closed, or dewatered for maintenance).

10.15.4.1. Concrete. Post-tensioned concrete will be proportioned and designed using loads and load combinations from paragraph 10.2. Design for the anchorage zones, including spalling, bursting, and edge tension reinforcement follows the guidelines for pier anchorage design, except that tension stress limits for concrete specified in AASHTO apply.

10.15.4.2. Steel. Steel trunnion girders will be proportioned and designed using loads and load combinations of paragraph 10.2. Include an additional load combination of the full post tension anchor load only using a load factor of 1.2 applied to the anchorage force.

10.15.4.2.1 Strength Limit State, Yielding and Buckling. The strength limit states of yielding and buckling will be evaluated following AISC, Chapter H (for members under combined torsion, flexure, shear and axial forces) and modified as required by Chapter 4. Resolved normal and shear stresses due to factored loads (i.e., required strength) will not exceed the factored yield strength, and normal stresses due to factored loads will not exceed the factored critical buckling stress of the member.

10.15.4.2.2 Strength Limit State, Combined Stresses. When shear and normal stresses are of comparable magnitude, the effect of combined stresses can be evaluated using Von Mises stresses. The factored Von Mises stresses should not exceed the factored yield strength. As Von Mises is a yield criterion, it does not apply directly to buckling and fracture. Therefore, these limit states should be checked independently of yield limits. In evaluating lateral torsional buckling, the unsupported length should be based on centerline supports of the anchorage groups as shown in Figure 10.36.

10.15.4.2.3 Serviceability Limit State. The serviceability limit state will be evaluated for maximum girder deformations. The fatigue limit state will generally not control the design of steel trunnion girders due to the low number of stress cycles typically encountered. However, steel trunnion girders typically are fracture critical and proper fracture control should be implemented (see Chapter 8).

10.15.5 Analysis and Design Considerations.

10.15.5.1. Trunnion Girder. The size of the trunnion girder is dependent on the magnitude of the flexural, shear, and torsional forces that result in a complex interaction of stresses. Shear and bending stresses can be significant; axial stresses are minimal except for post-tensioning. Torsional shear stresses may be significant; however, these stresses can be limited by orienting the trunnion girder to minimize eccentricity. Maximum torsion will usually occur in the girder when the gate is partially raised and the pool is at maximum level.

10.15.5.2. Trunnion Girder Stresses. A simple 2-D model for approximating trunnion girder stresses assumes the beam acts as a simply supported beam with cantilevered end spans (for cantilevered trunnion girders). The supports for this beam are located at the centerlines of trunnion girder anchorages. The girder support is assumed to be torsionally fixed at the anchorage point. This model is shown in Figure 10.41.

10.15.5.3. Stressing Sequence. Depending on the tendon stressing order, the controlling design stage may occur when only some tendons are stressed. Consider concrete stress limits and reinforcement requirements as each tendon is tensioned in sequence. Any special requirements regarding stressing order must be described in the contract specifications or noted on the contract drawings.

10.15.5.4. Steel Girders. Beam flanges and webs should be proportioned to satisfy compact section requirements to avoid local buckling. Where compact sections are not practical, non-compact sections are allowed. However, slender elements will not be used. Girders must be configured to resist the large anchorage post-tensioning loads. This will require stiffening members oriented parallel to the anchors.

10.15.6 Serviceability Requirements. Serviceability requirements for gates are provided in Chapter 4. Chapter 7 describes corrosion protection for steel girders. Bituminous fillers may be used to fill recesses and isolated pockets to promote proper drainage. If galvanizing is used, box girders must include access holes for penetration of galvanizing material to the interior of the girder.

10.15.7 Steel Girders. Steel girders must be proportioned to limit deflections so that design stresses for bearings are not exceeded, maximum allowable bearing rotations are not exceeded (for spherical bearings), gate seal contact surfaces are maintained within acceptable tolerances, and design assumptions are not compromised. Deflections should not be a problem if girder stiffness is comparable to a post-tensioned concrete trunnion girder. Alternatively, deflections may be calculated and the impact on operability determined, using techniques such as 3-D finite element methods.

10.15.8 Design Details.

10.15.8.1. Concrete Girders. It has been common practice to require the trunnion girder to be completely post-tensioned before placing adjacent pier concrete and tensioning the girder anchorage. This is done because shortening of the girder, due to post-tensioning, would be restricted by bond to the adjacent concrete at points of bearing. This requirement can cause delays in the construction schedule. The use of second-placement concrete can be incorporated in the area between the pier and girder to eliminate this concern.

10.15.8.2. Closed Stirrups. Closed stirrups are used for torsional reinforcement. To aid in construction, it is possible to assemble the conventional reinforcement as a cage with the web steel in a welded grid arrangement and welded to surrounding hoops and longitudinal steel. Longitudinal bar diameters should be limited since post-tensioning of the girders will have a tendency to cause buckling of these bars and larger bars may cause spalling of the concrete.

10.15.8.3. Tendon Spacing. Tendon spacing for the longitudinal post-tensioning steel must be offset with respect to the trunnion girder anchorage tendons, allowing adequate clearance for concrete placement between ducts for longitudinal and anchorage steel. A 7-in. grid spacing for both the longitudinal girder and main gate anchorage tendons has been used successfully in previous designs.

10.15.8.4. Steel Girders. I-shaped girders are easier to fabricate than box-shaped girders. Weld joints for flange-to-web welds and tendon support members are easily accessed. Box-shaped girders are more difficult to fabricate when anchor supports are incorporated. The top plate may be installed in sections if welding to intermediate plates is required. Allowance for welding access may control member selection and sizing so that adequate working room is provided and quality welding can be assured.

10.16 Operating Equipment. Hoisting equipment usually involves the use of wire ropes, roller chains, or hydraulic cylinders. Guidelines for operating equipment are provided in EM 1110-2-2610.

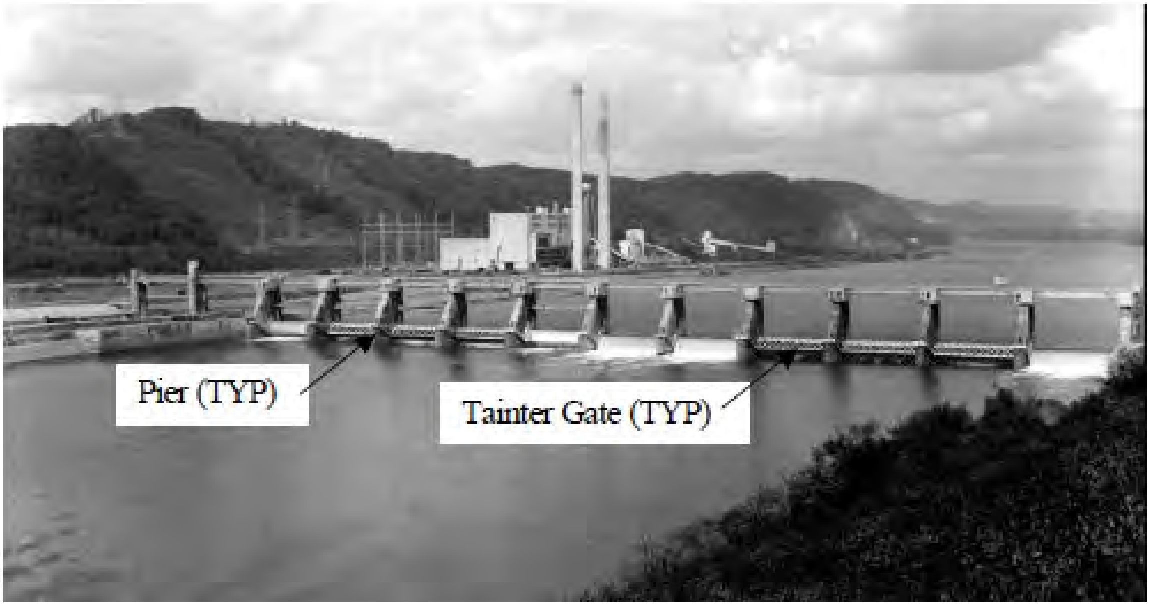


Figure 10.1. Overall View of Navigation Dam from Downstream

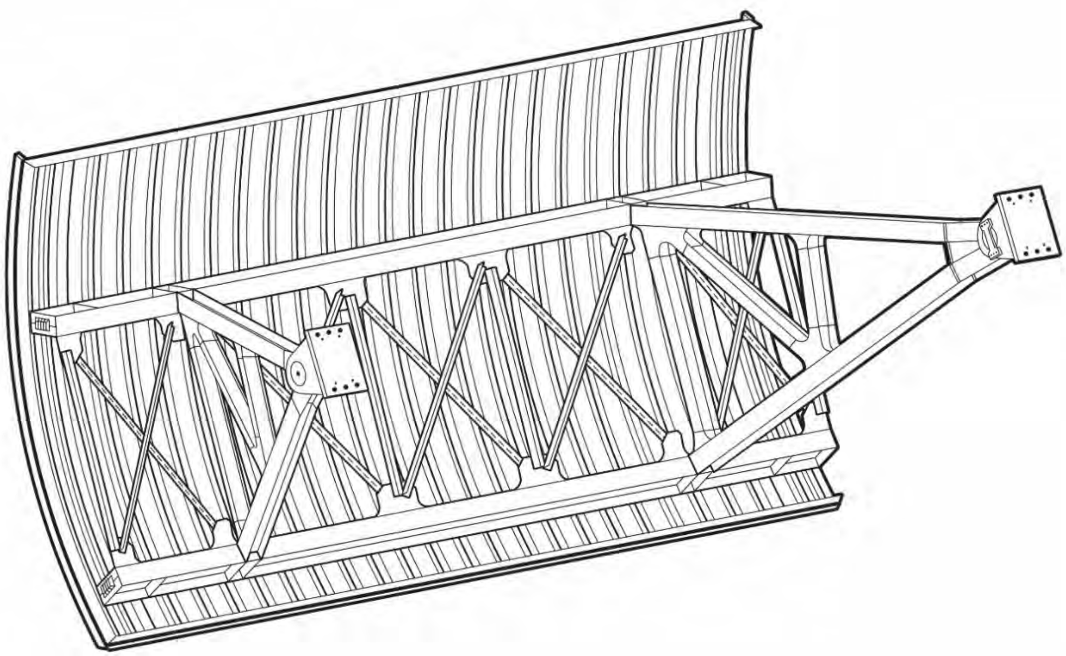


Figure 10.2. Downstream View of a Typical Tainter Gate

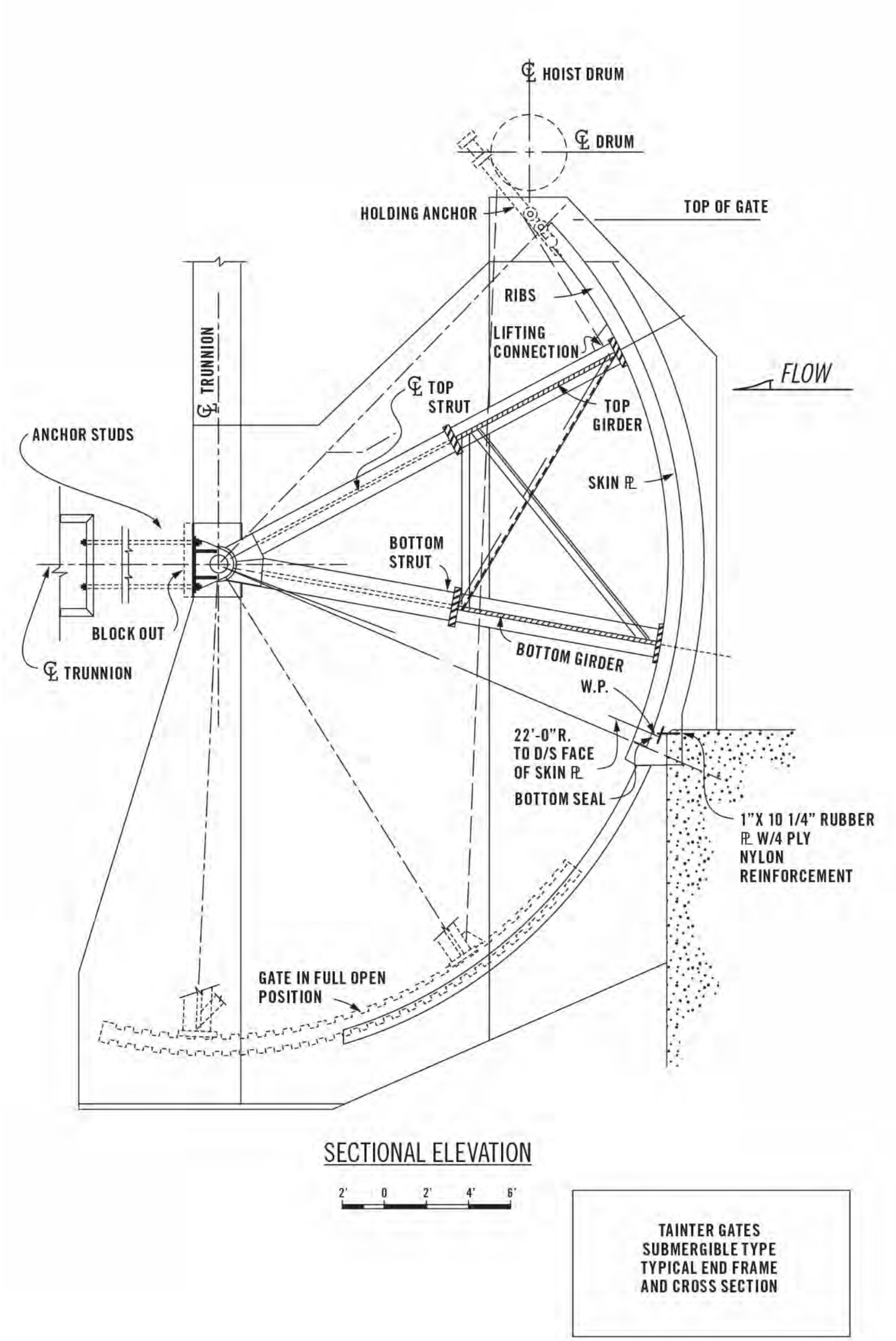


Figure 10.3. Submergible Tainter Gate

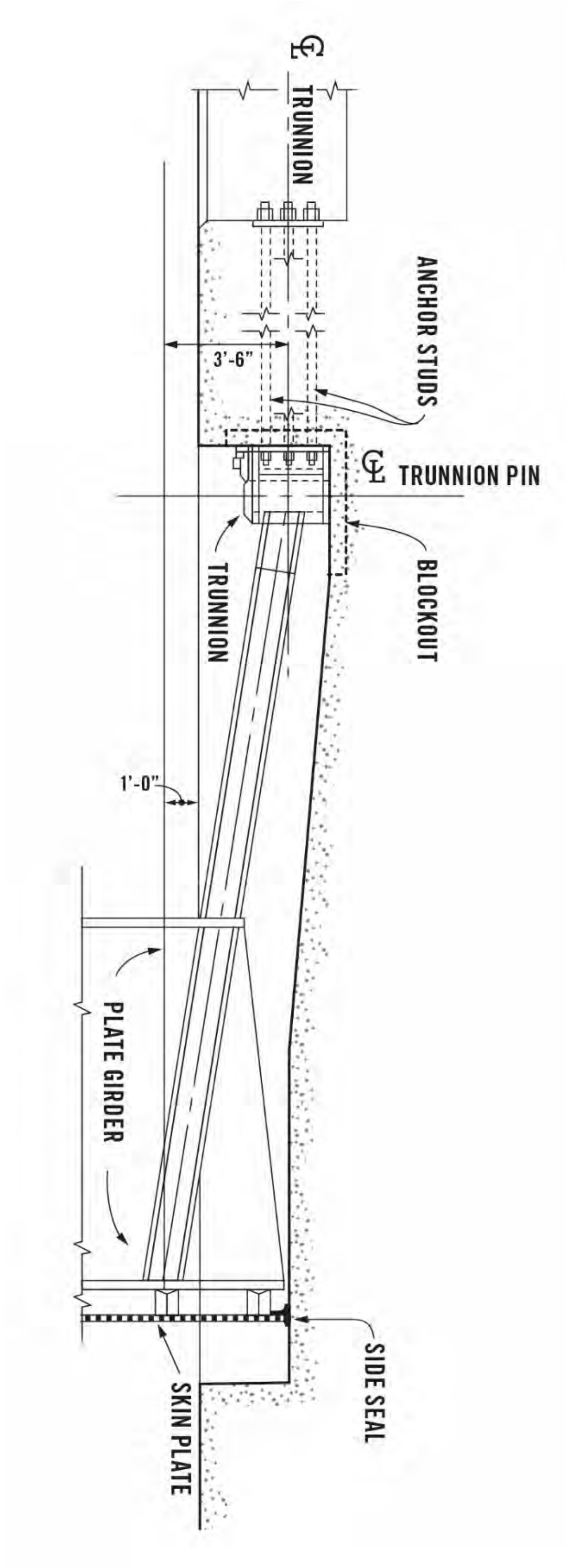


Figure 10.4. Submergible Tainter Gate Typical Recessed End Frame

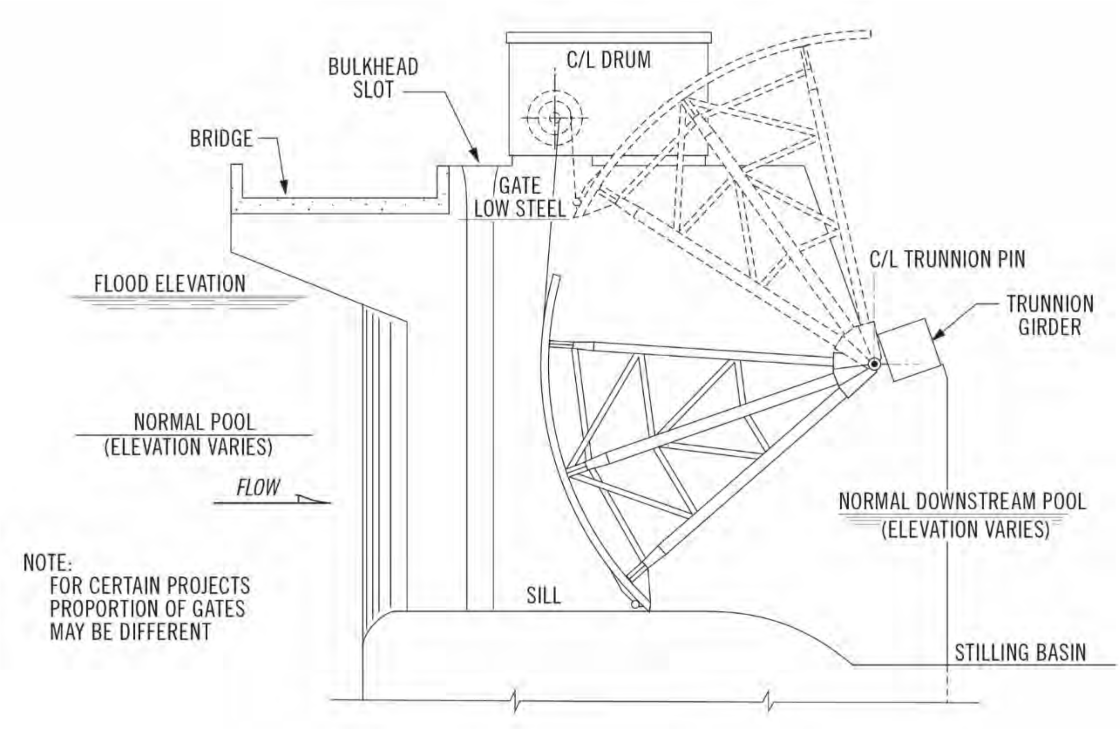


Figure 10.5. View of a Typical Tainter Gate

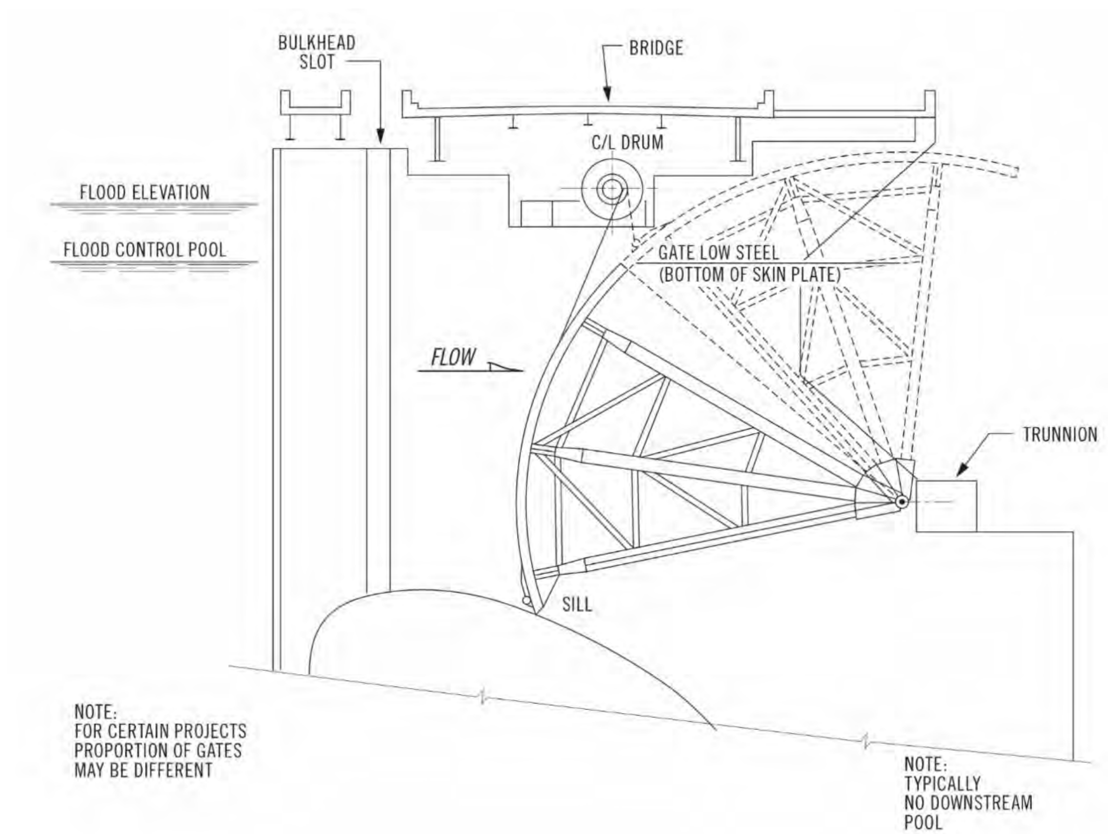


Figure 10.6. Wire Rope Hoisting System

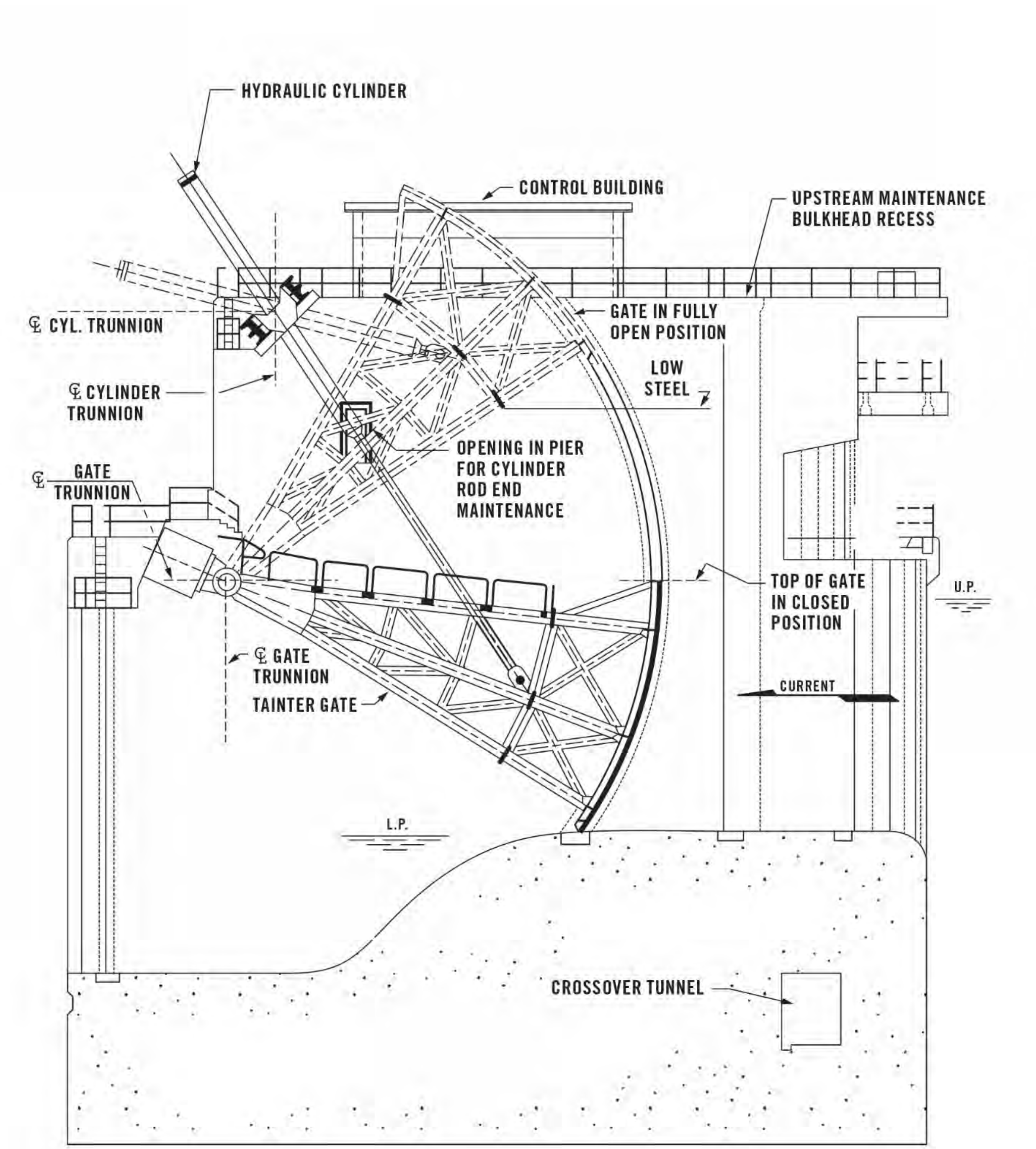


Figure 10.7. Hydraulic Hoisting System

Figure 10.8. Layout Variations of a Wire Rope Hoisting System

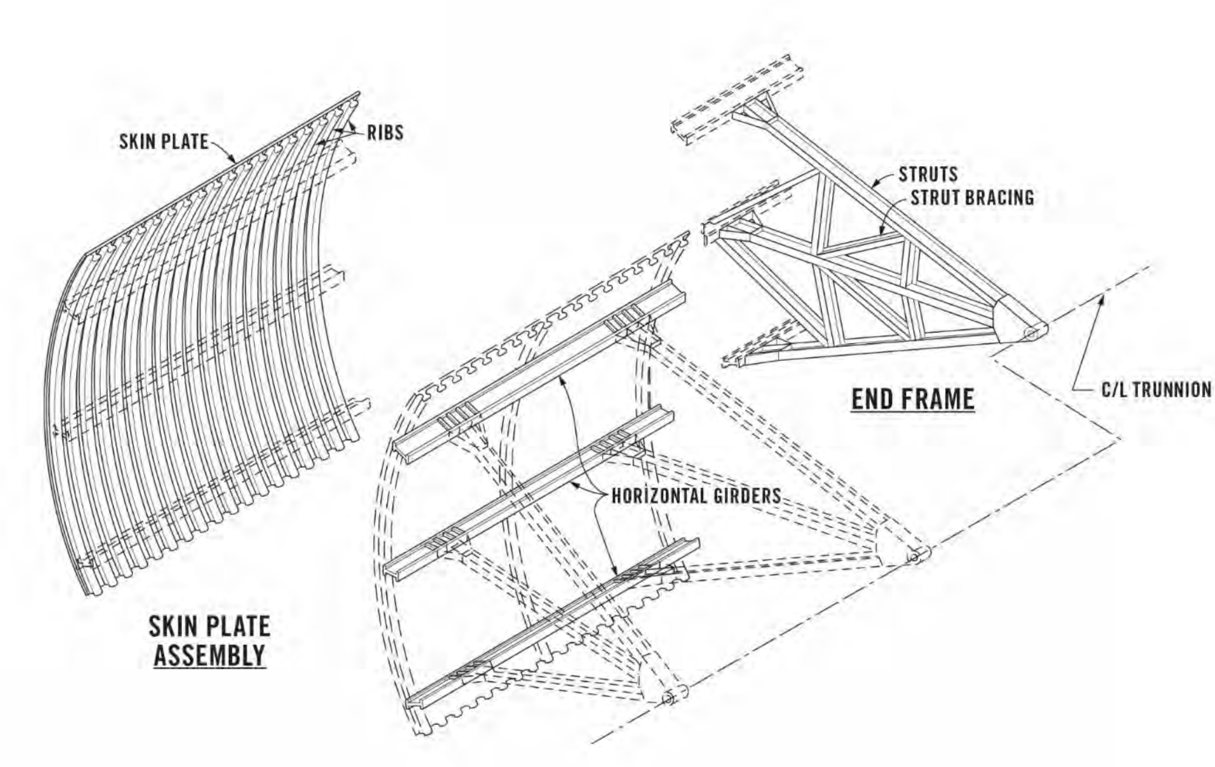


Figure 10.9. Primary Tainter Gate Components

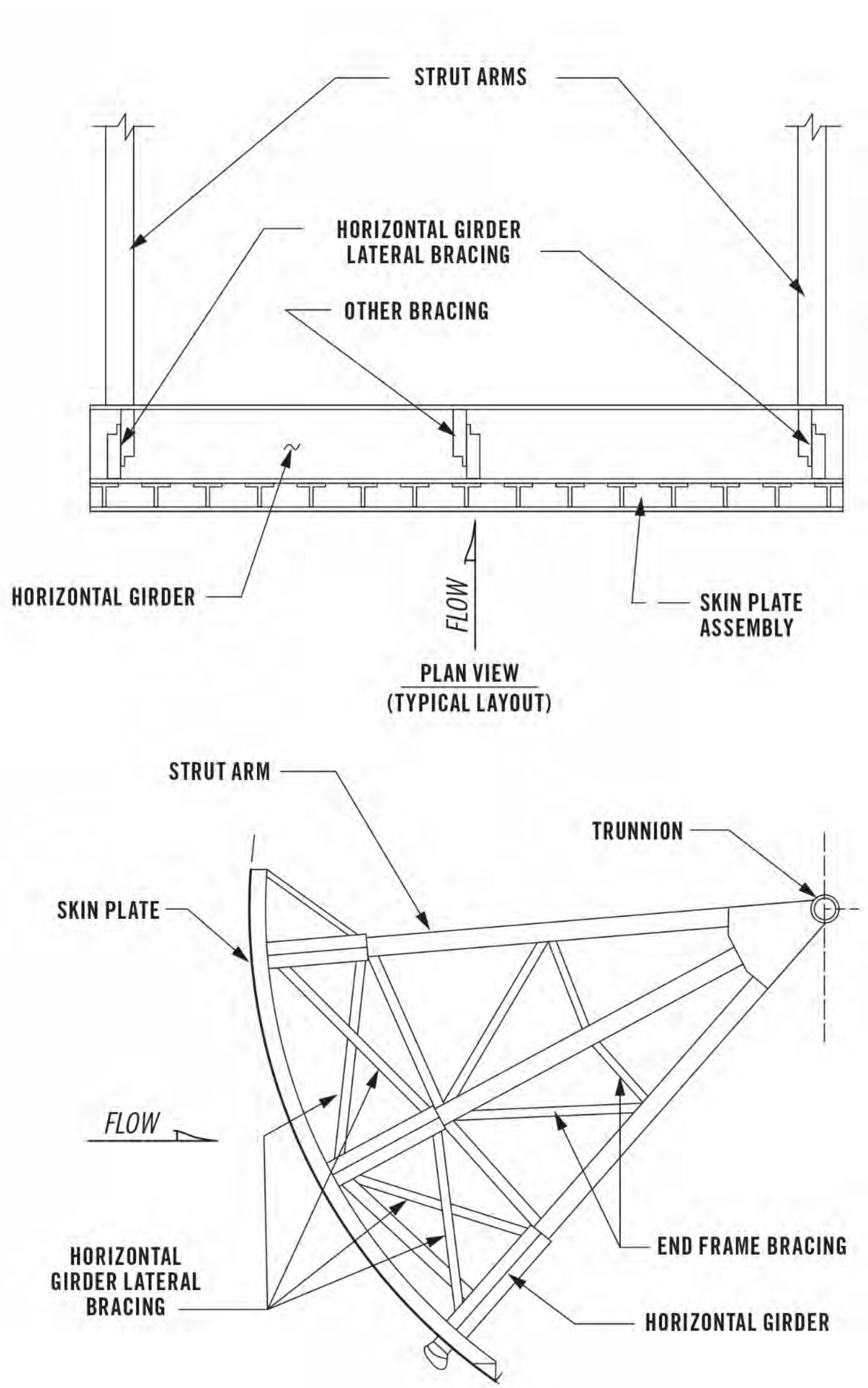


Figure 10.10. Horizontal Girder Lateral Bracing

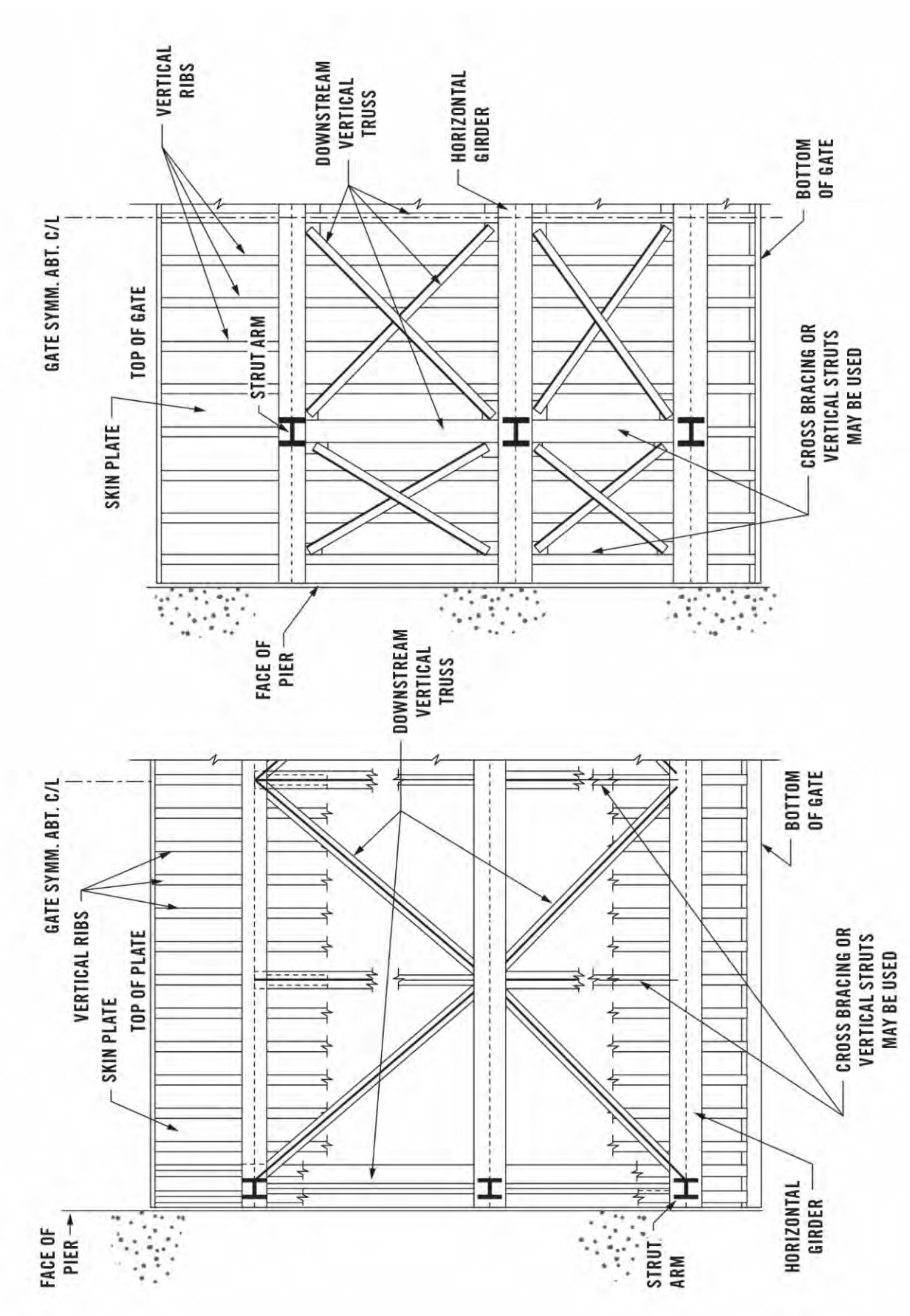


Figure 10.11. Downstream Vertical Truss

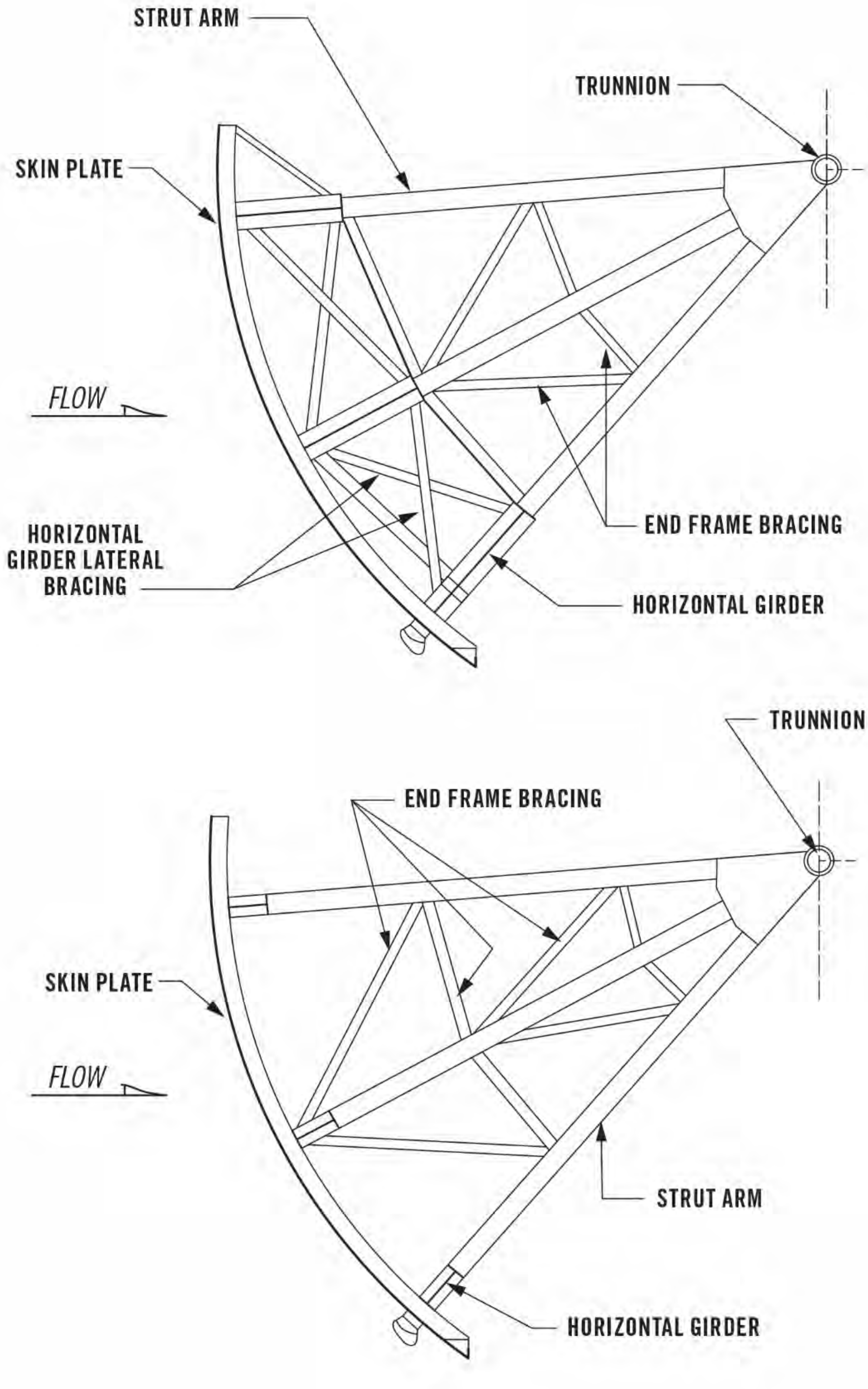


Figure 10.12. End Frame Bracing Examples

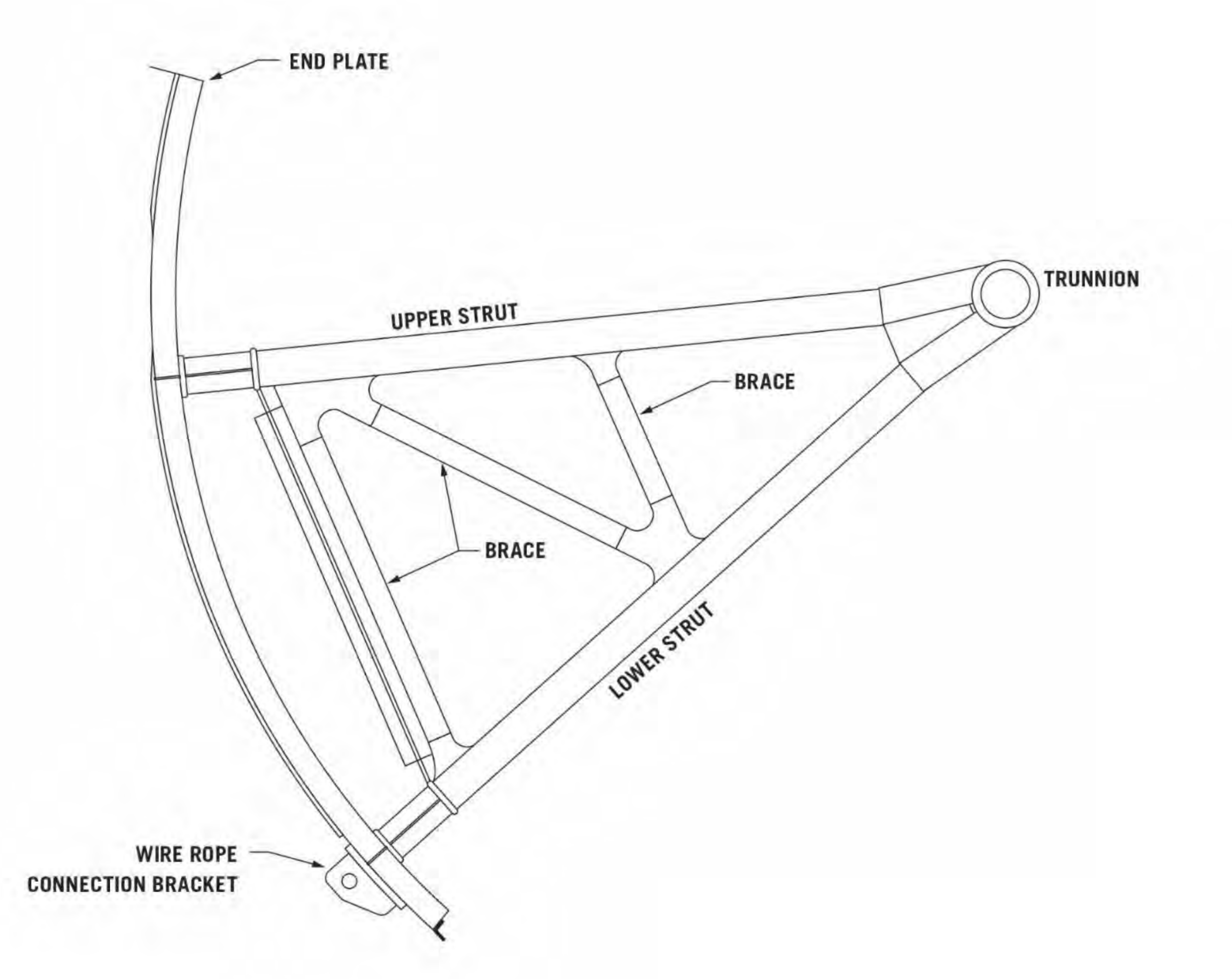


Figure 10.13. End Frame Bracing Example

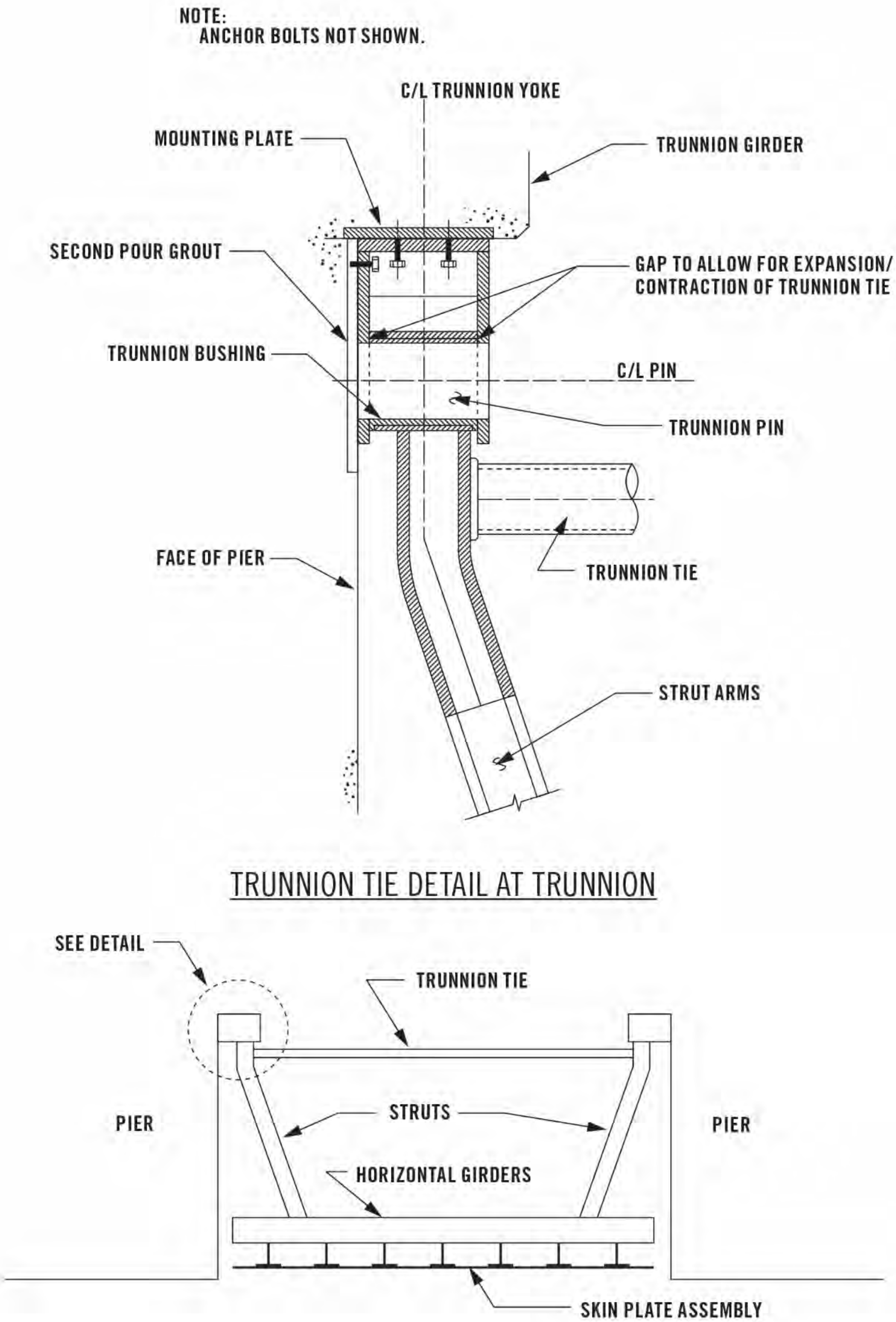


Figure 10.14. Trunnion Tie

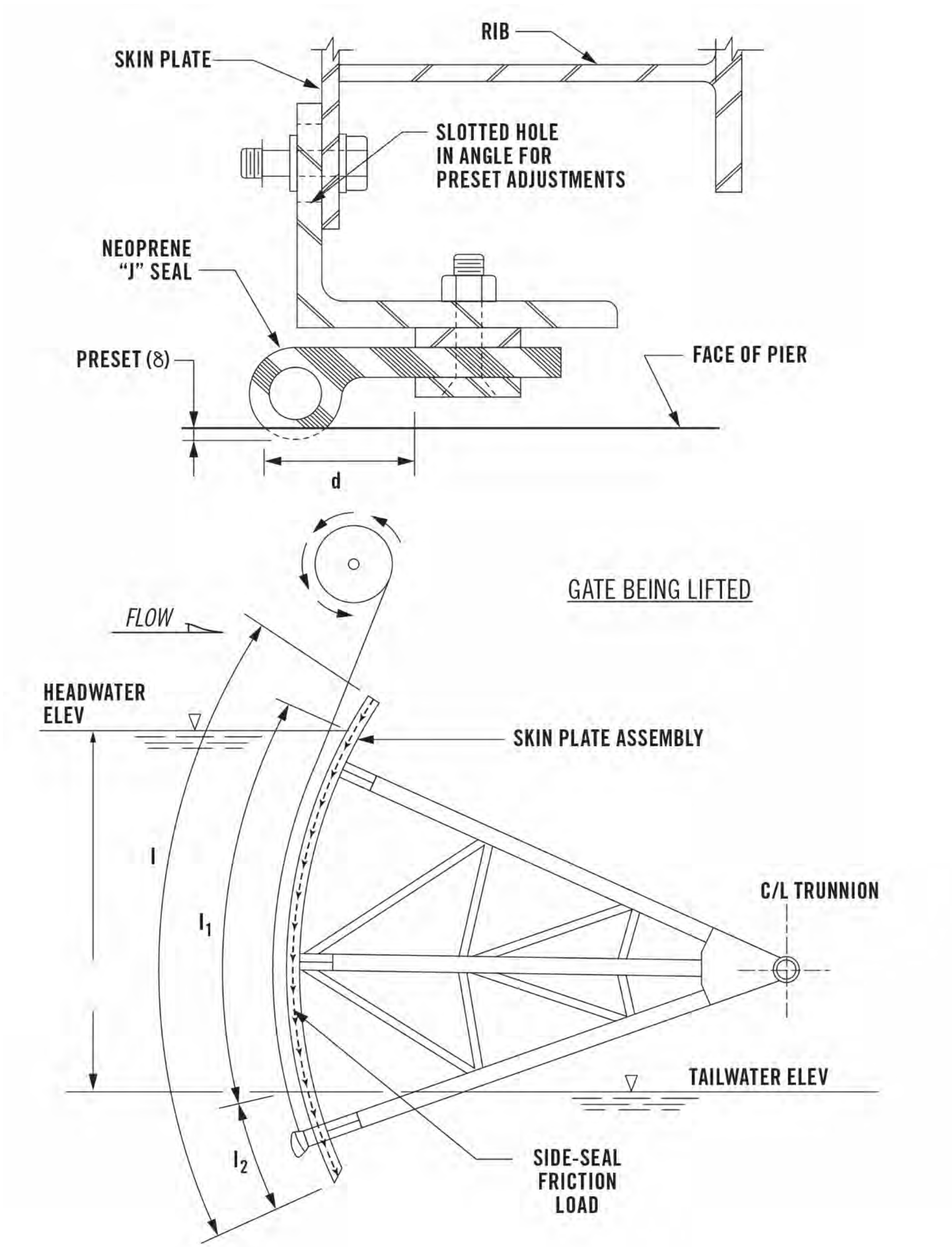


Figure 10.15. Side Seal Friction Variables

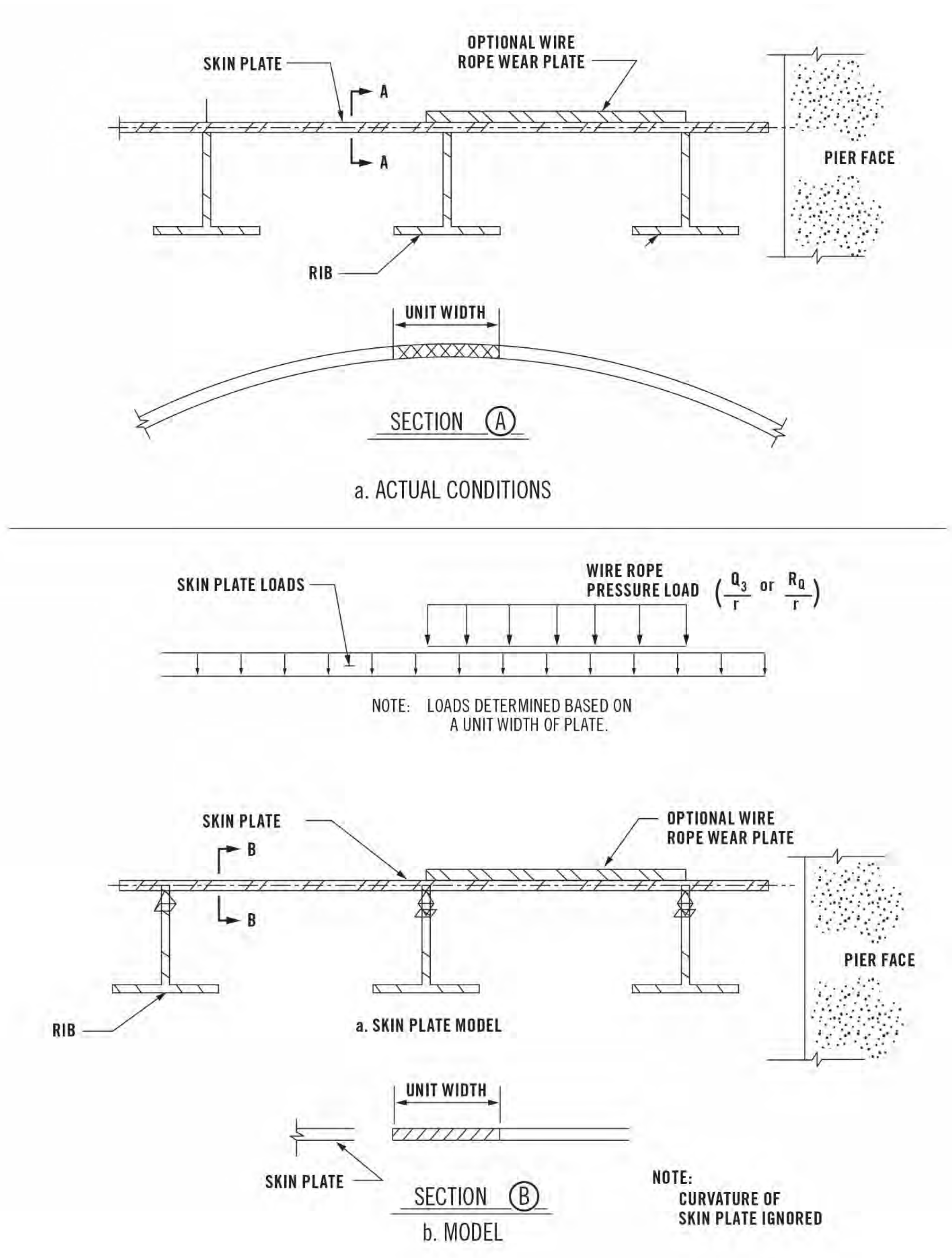


Figure 10.16. Skin Plate Model

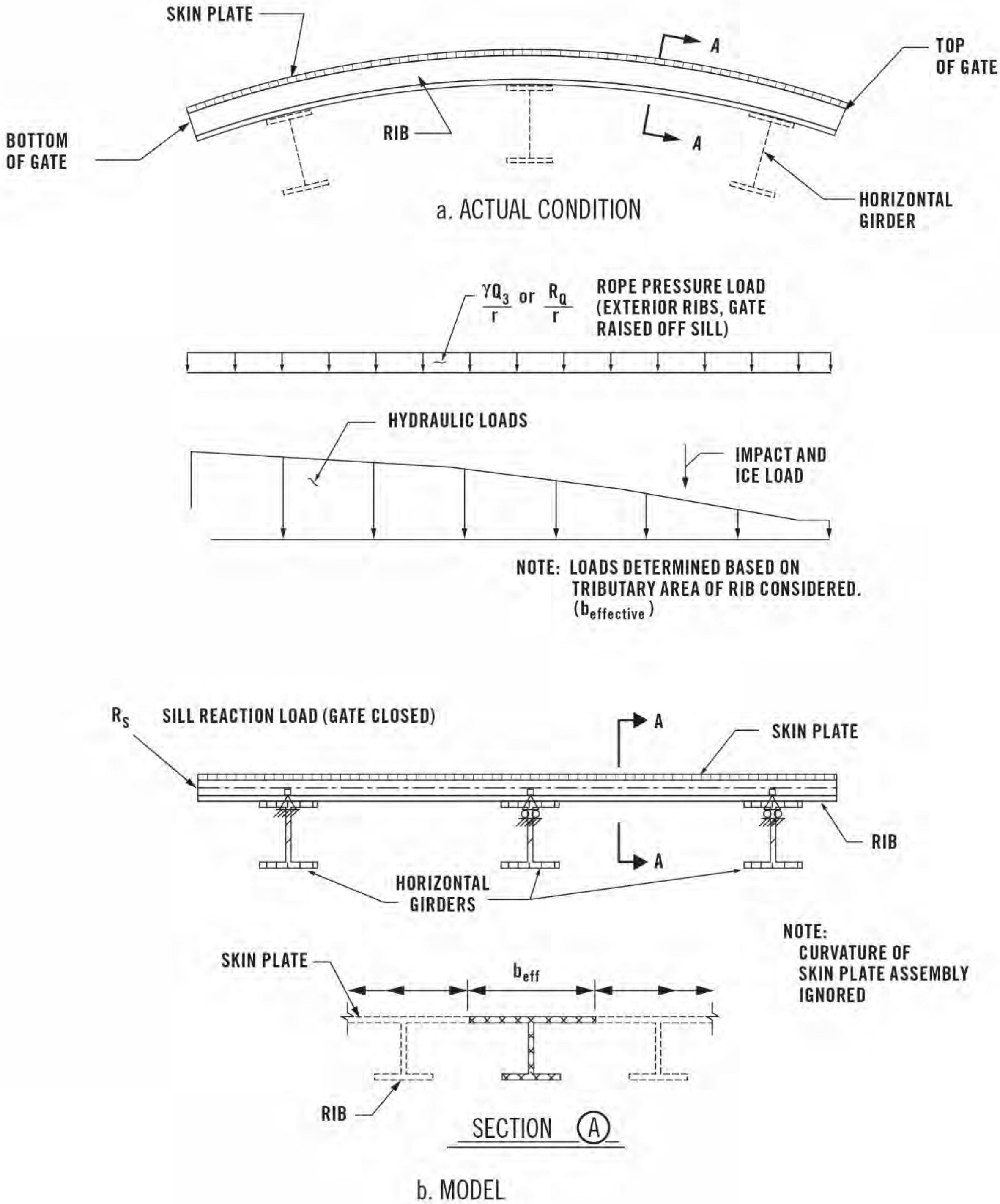


Figure 10.17. Rib Model

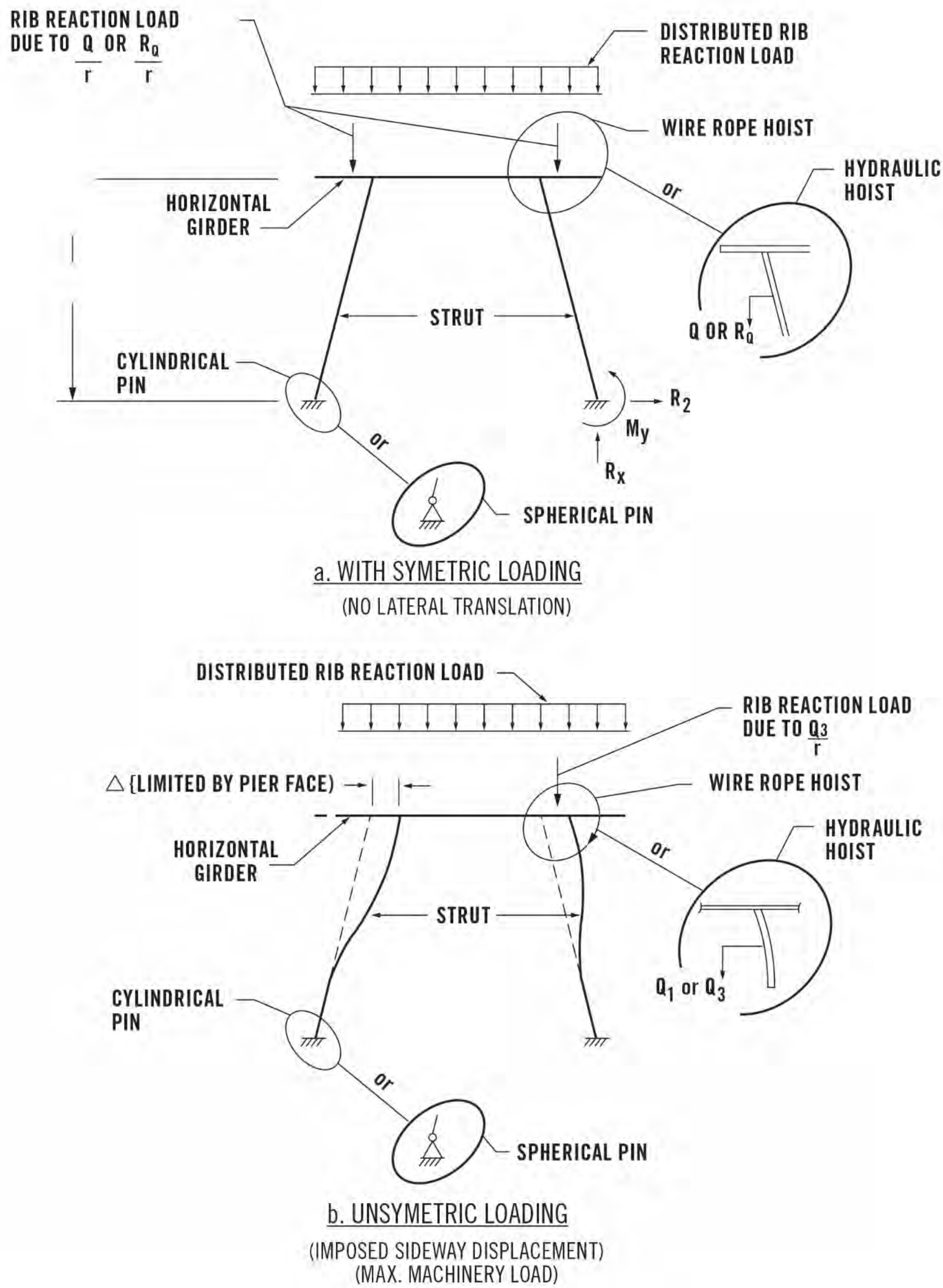


Figure 10.18. Girder Frame Model

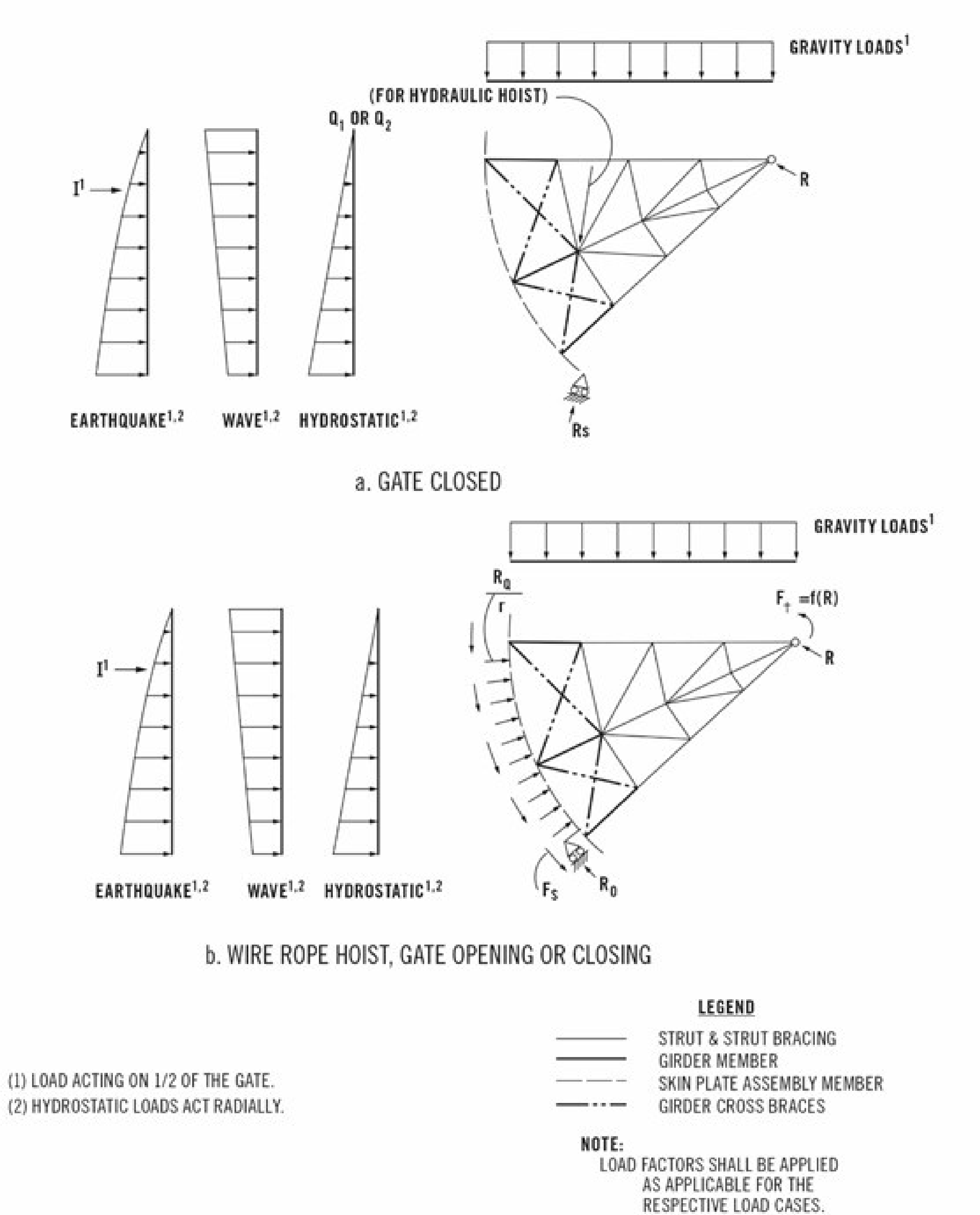


Figure 10.19. End Frame Model

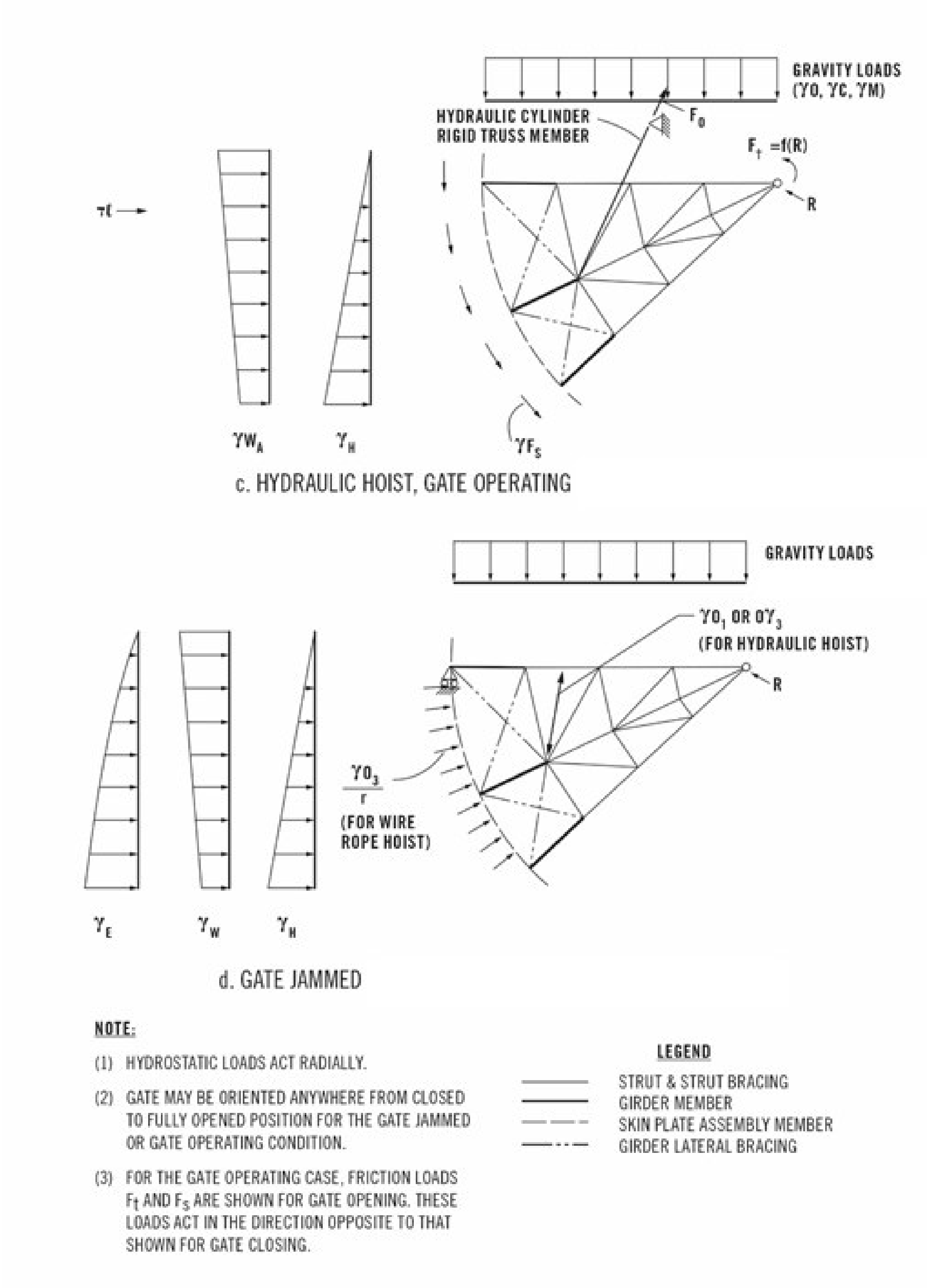


Figure 10.20. End Frame Model

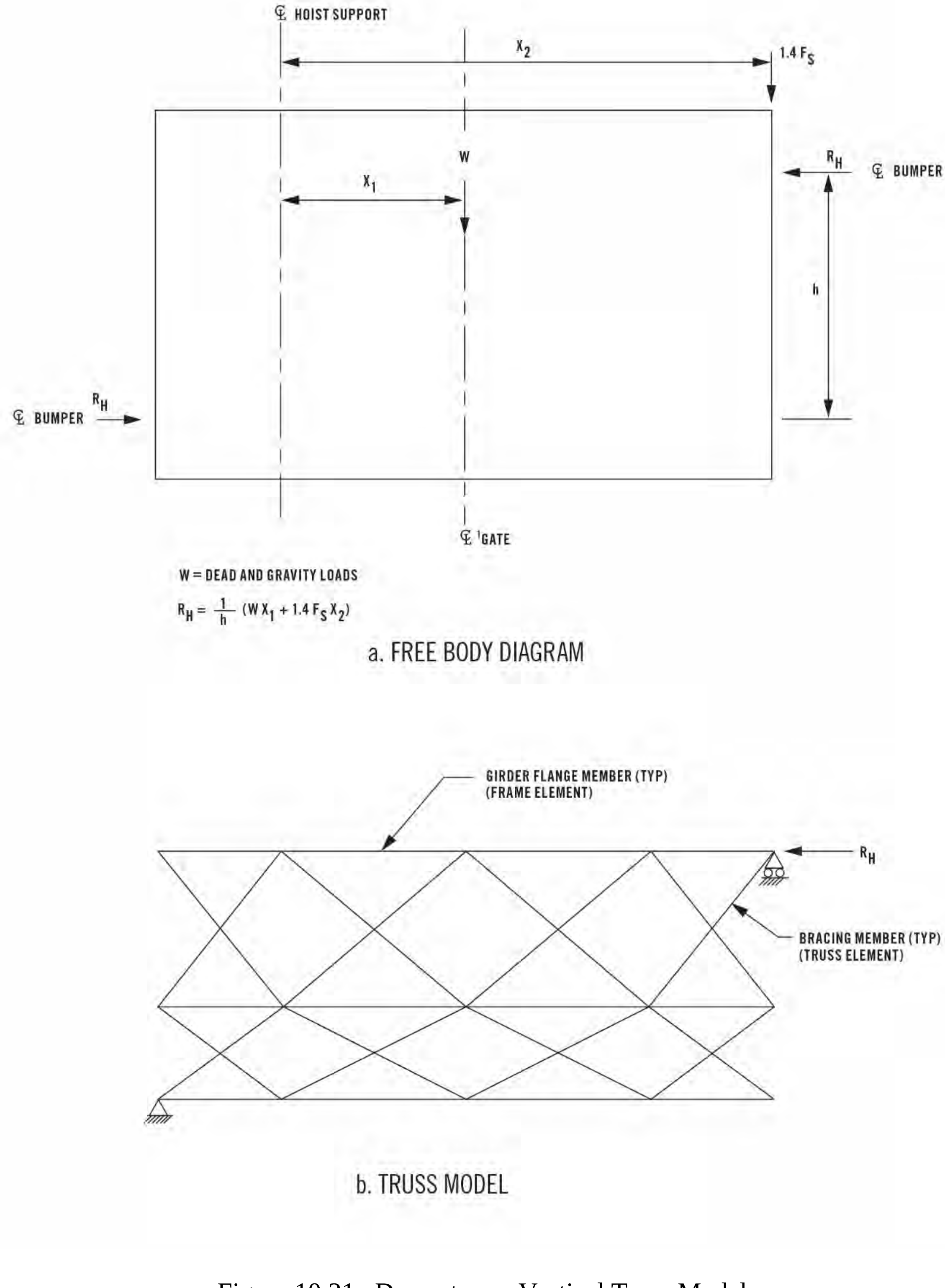


Figure 10.21. Downstream Vertical Truss Model

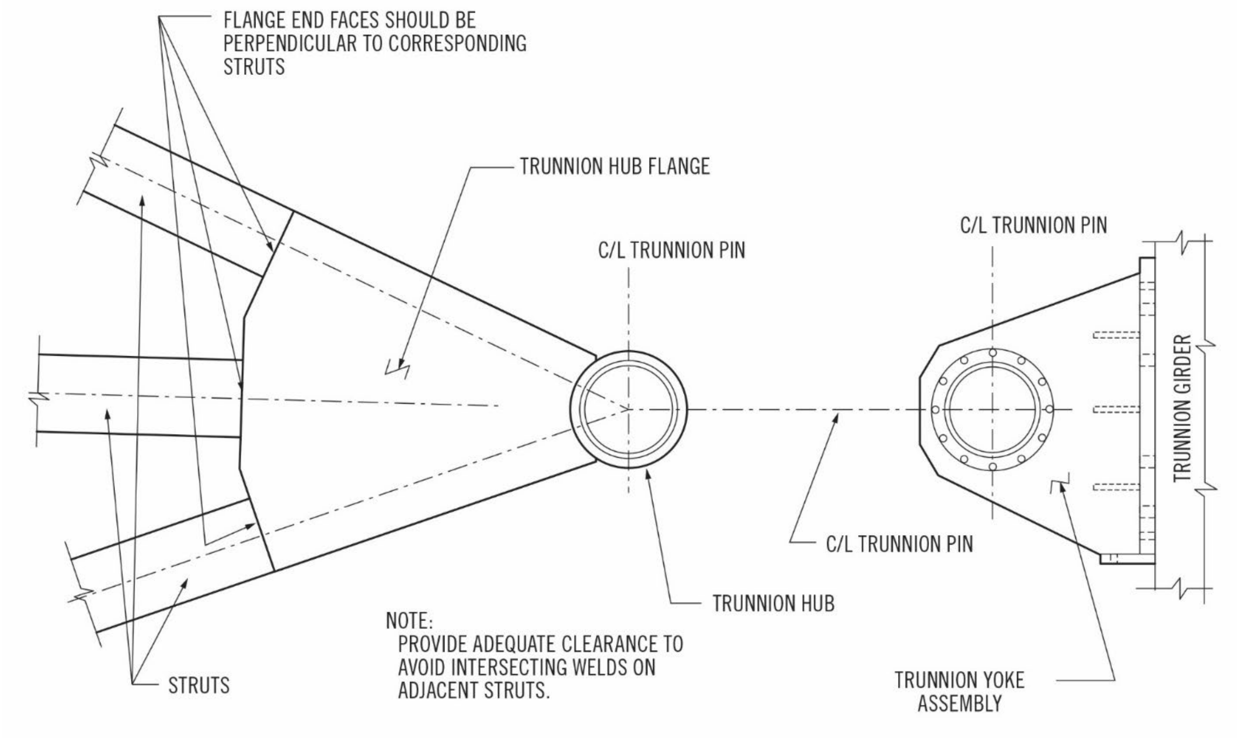


Figure 10.22. Trunnion Hub Assembly

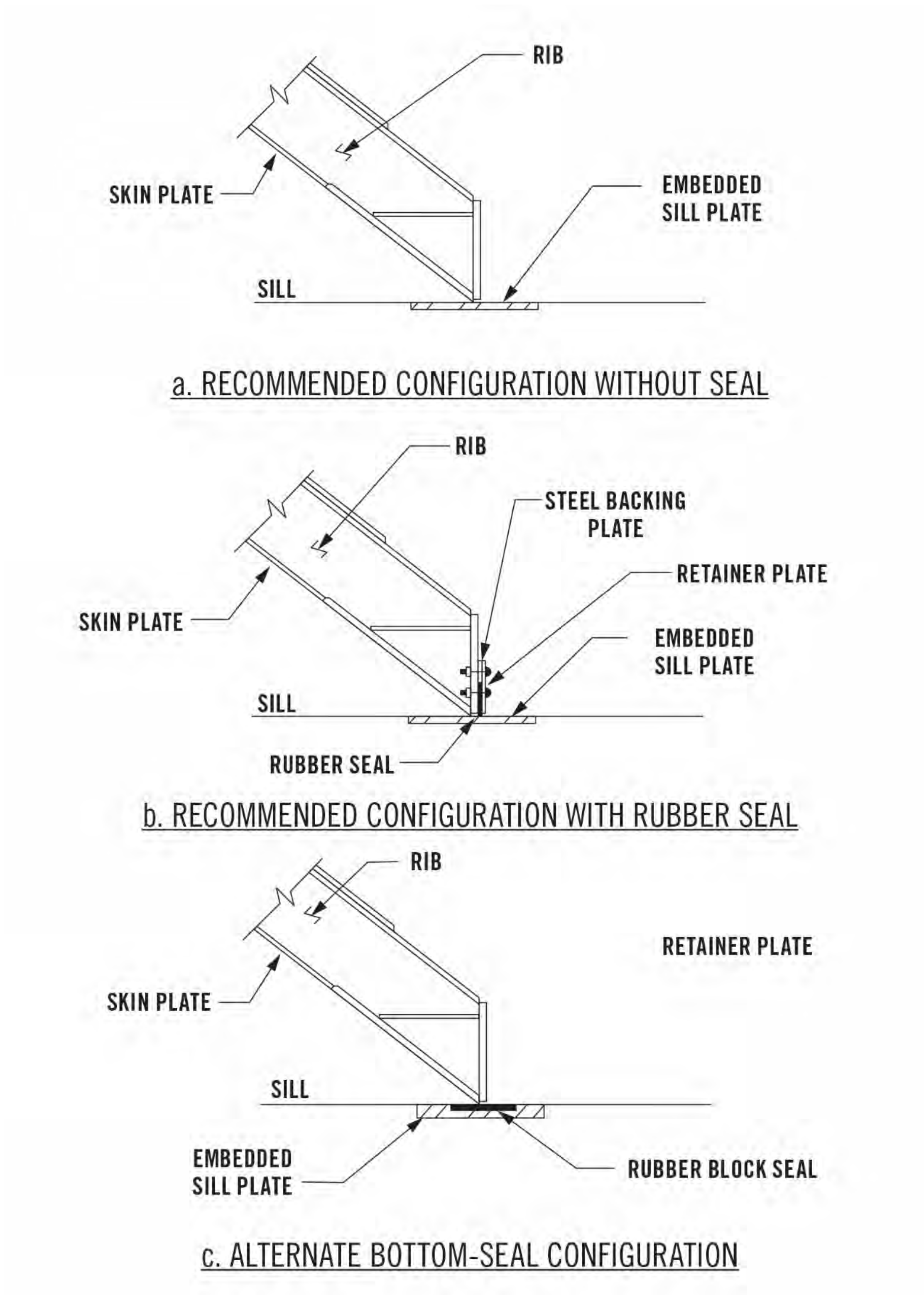


Figure 10.23. Seal Detail

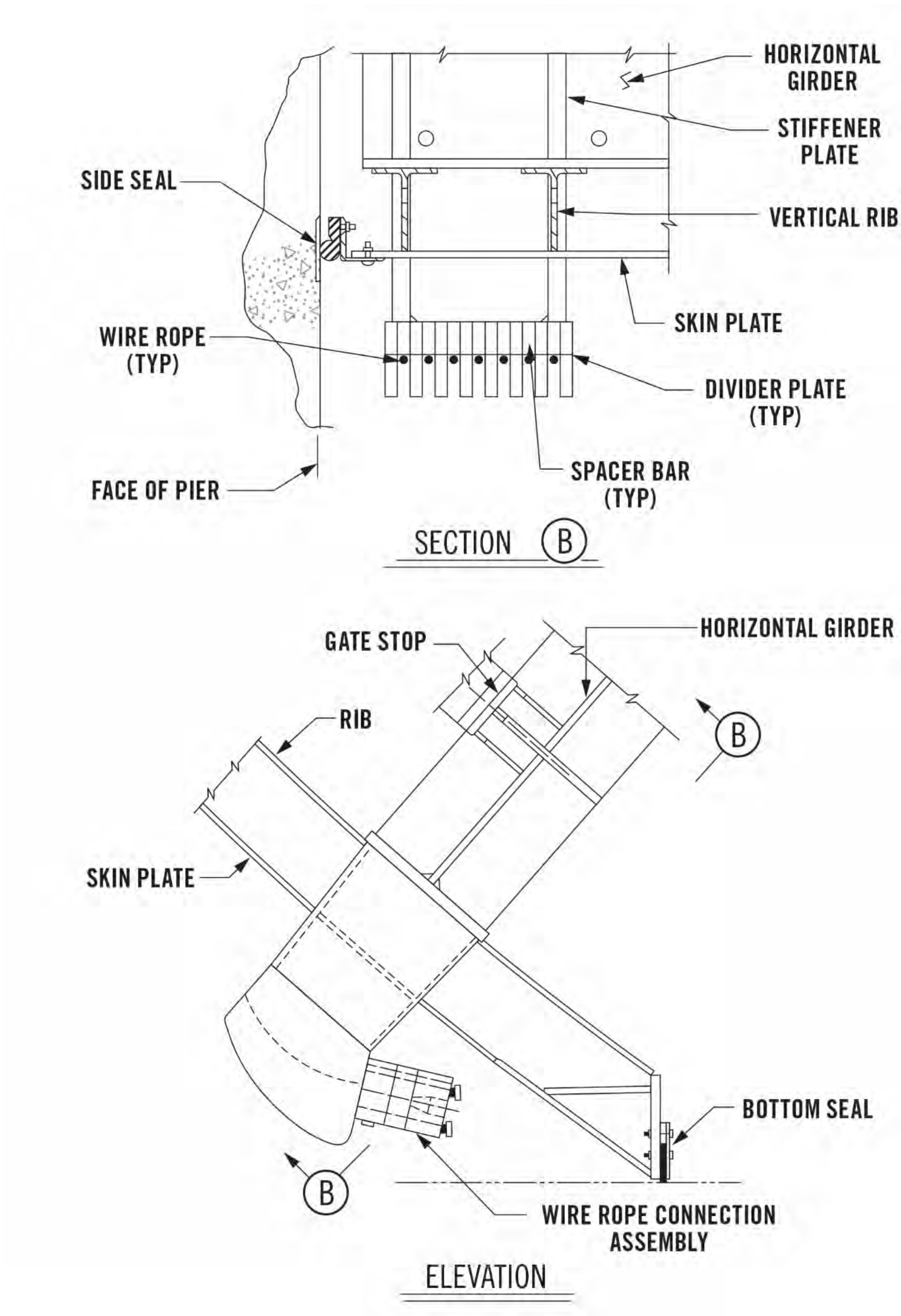


Figure 10.24. Wire Rope Attachment Detail

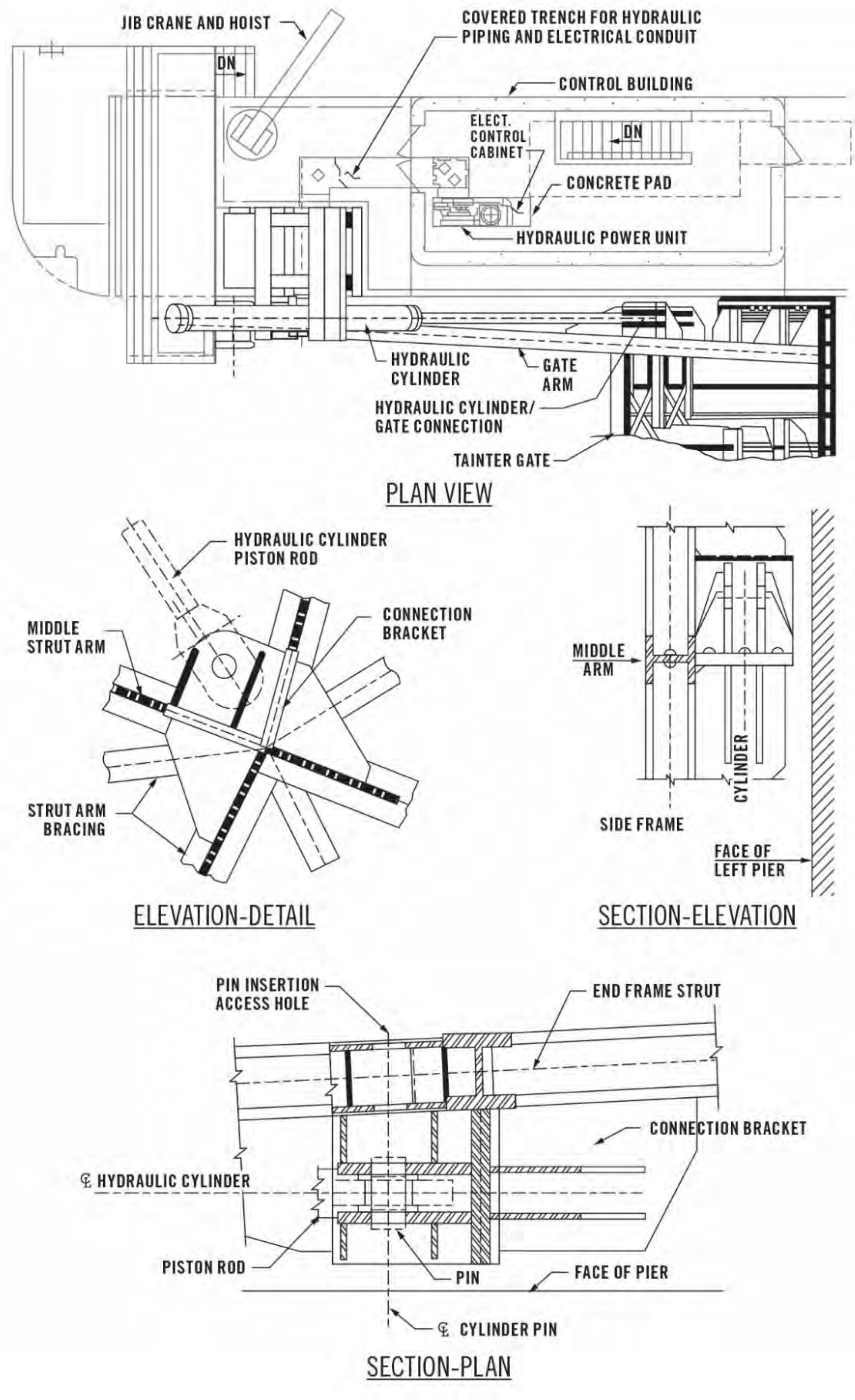


Figure 10.25. Hydraulic Cylinder Attachment Detail

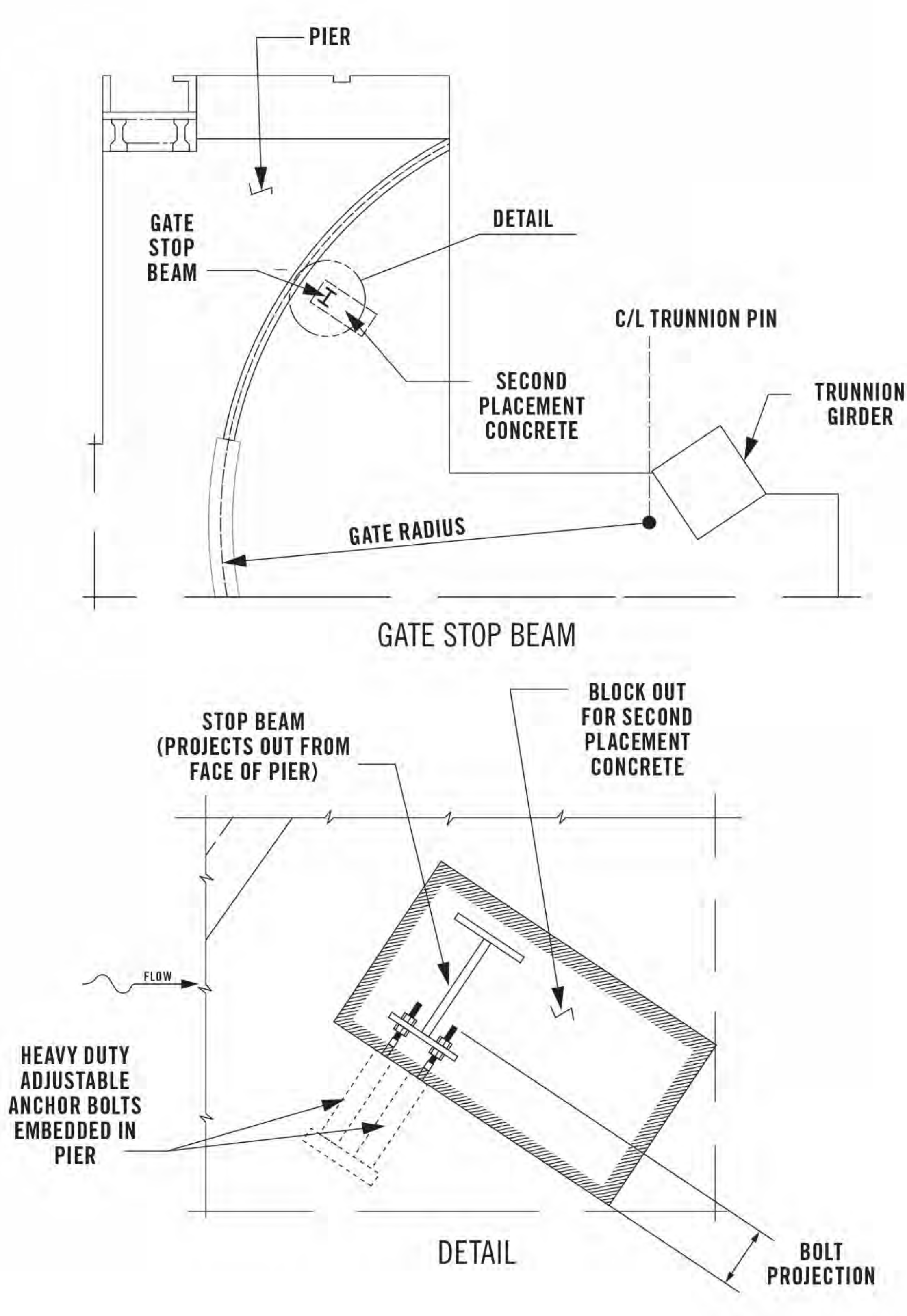


Figure 10.26. Gate Stop Details

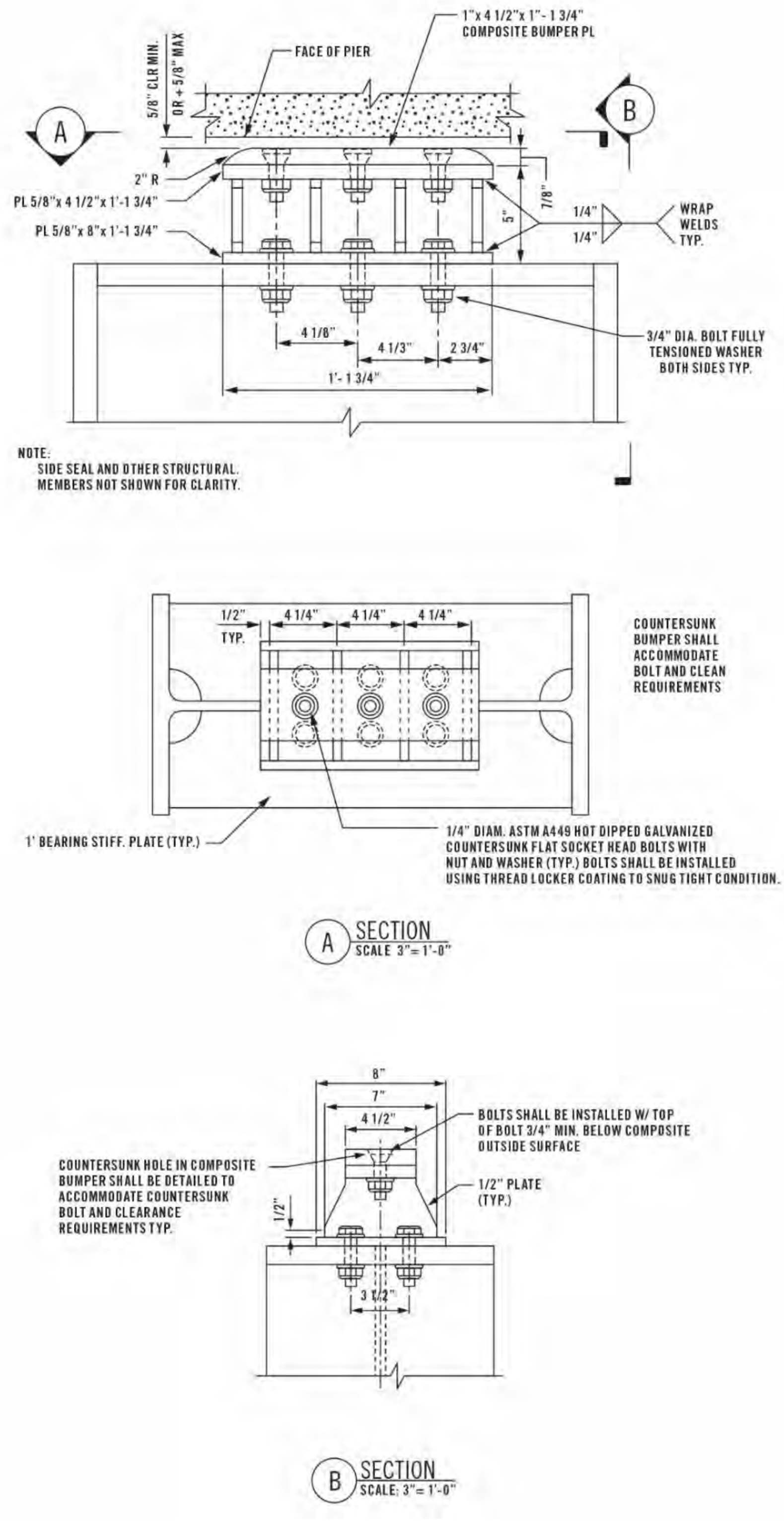


Figure 10.27. Bumper Details

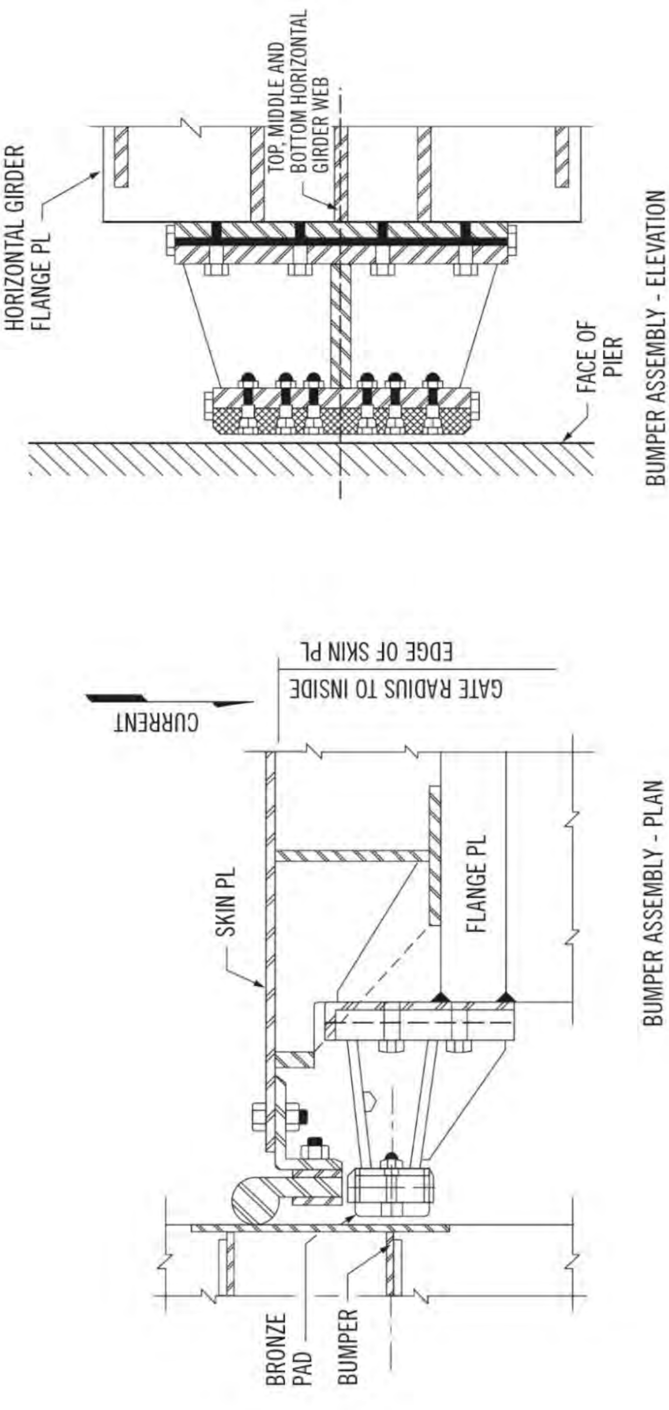


Figure 10.28. Bumper Details

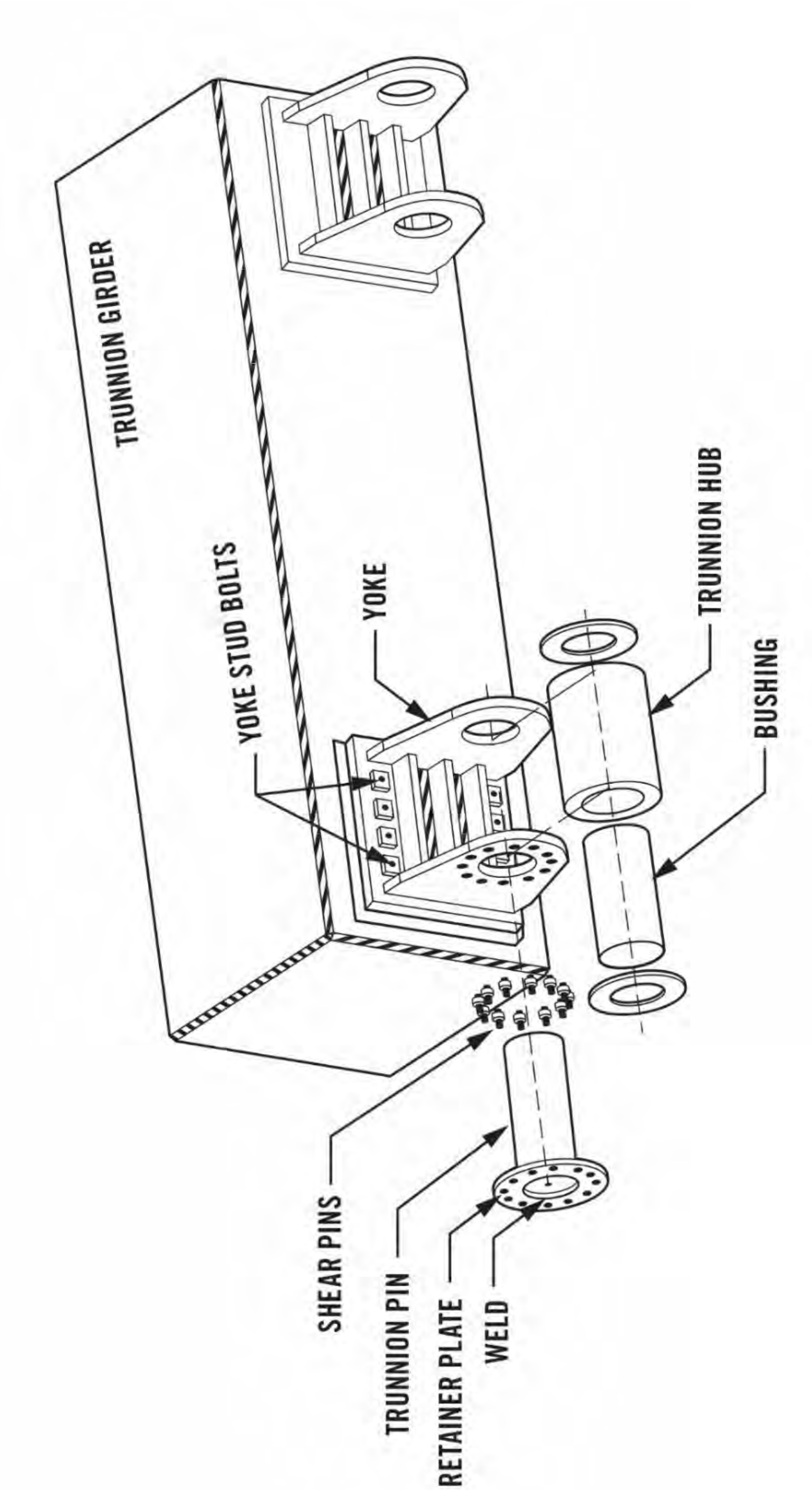


Figure 10.29. Trunnion Assembly with Cylindrical Bushing

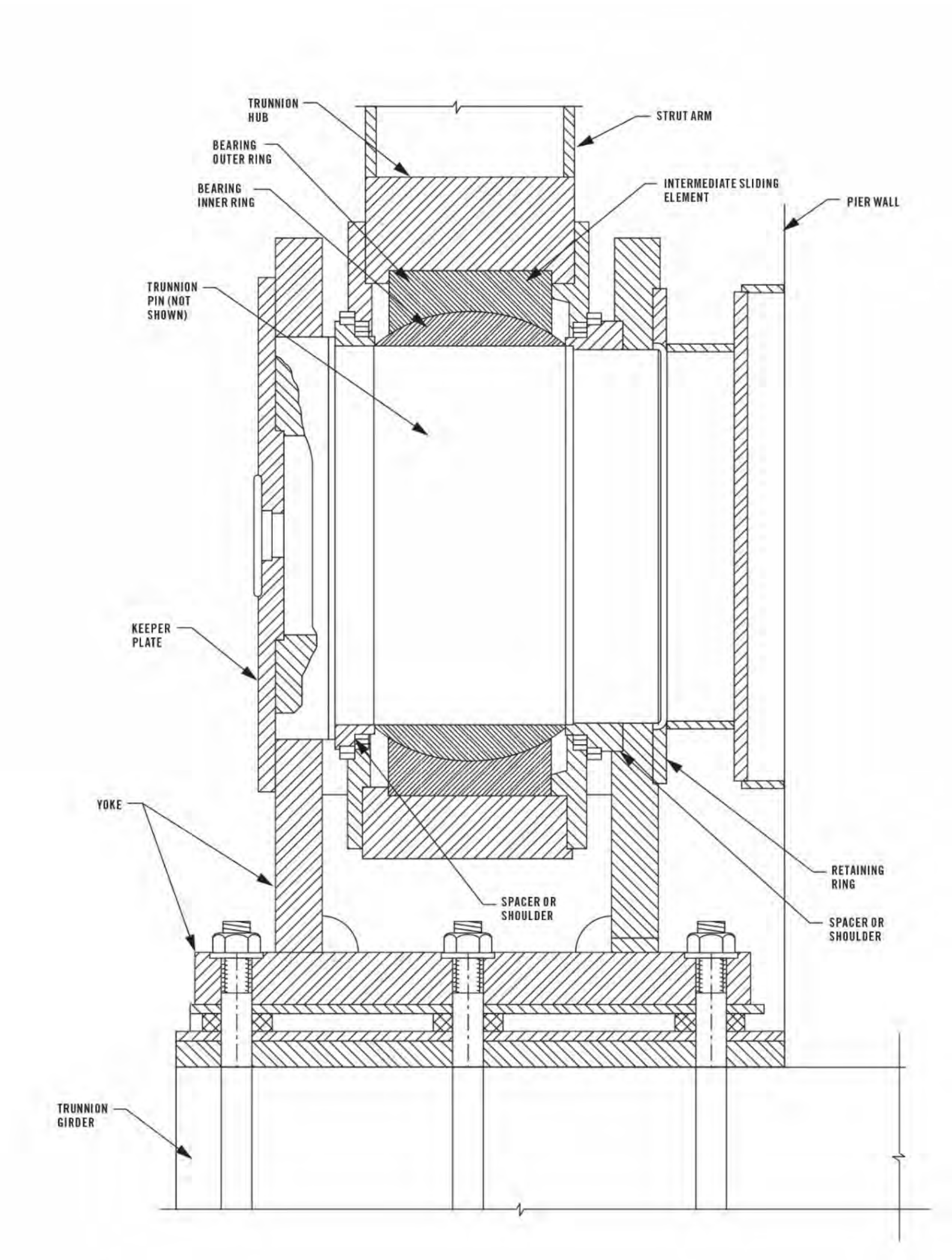


Figure 10.30. Spherical Bearing

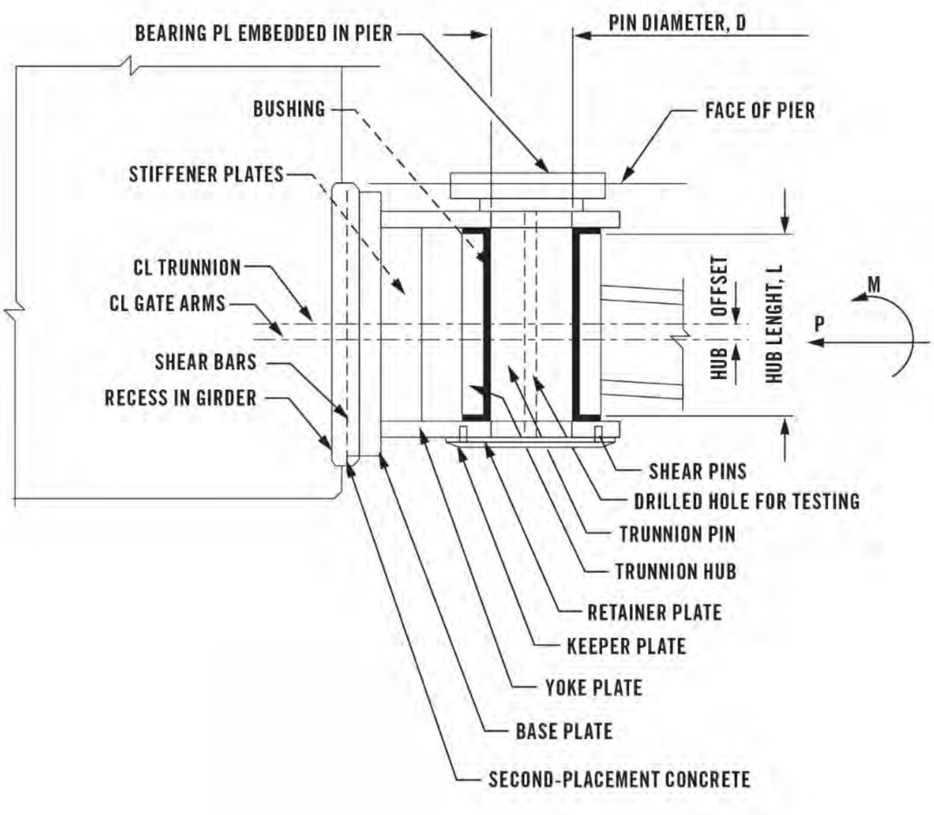


Figure 10.31. Trunnion Assembly Structural Components

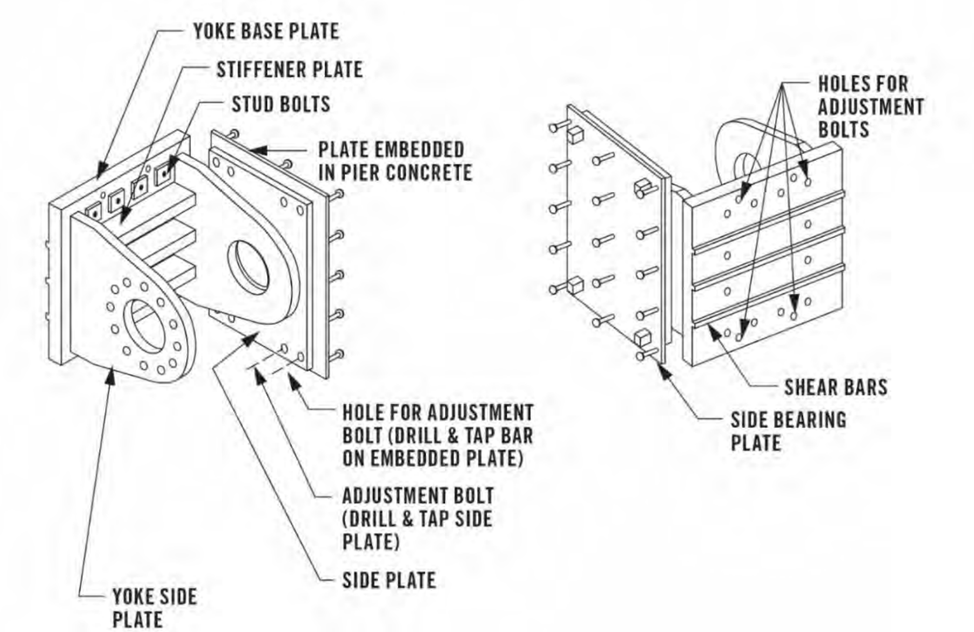


Figure 10.32. Trunnion Yoke Assembly

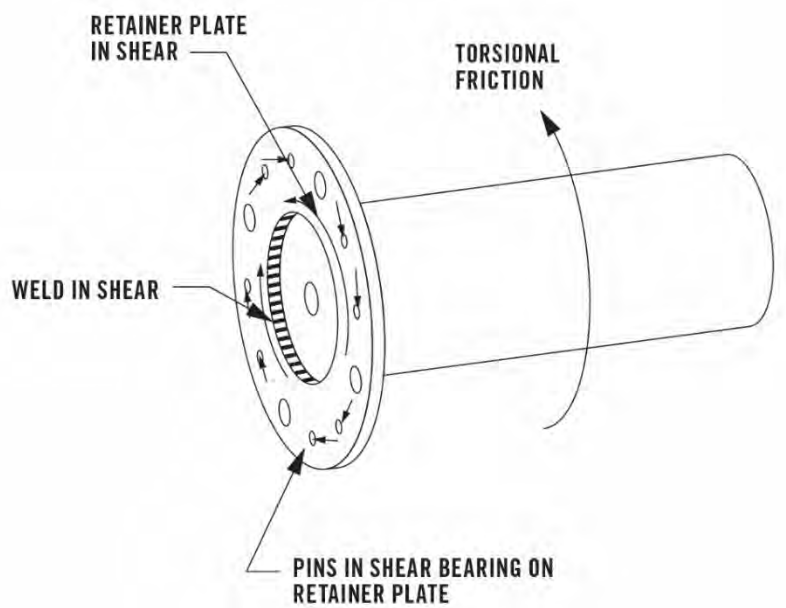


Figure 10.33. Generalized Forces on Trunnion Pin and Retainer Plate

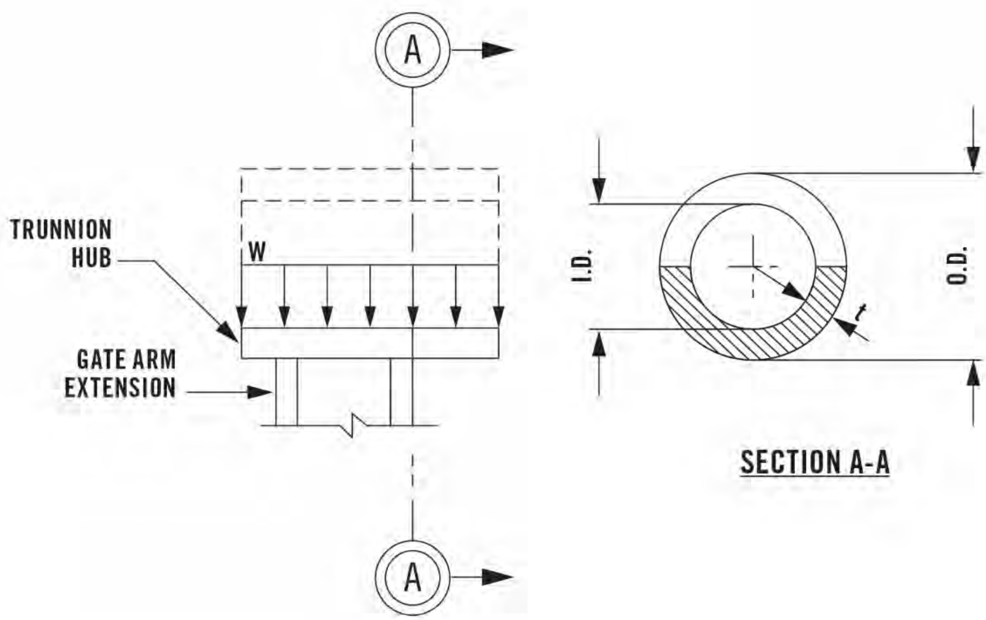


Figure 10.34. Trunnion Hub Design Assumptions

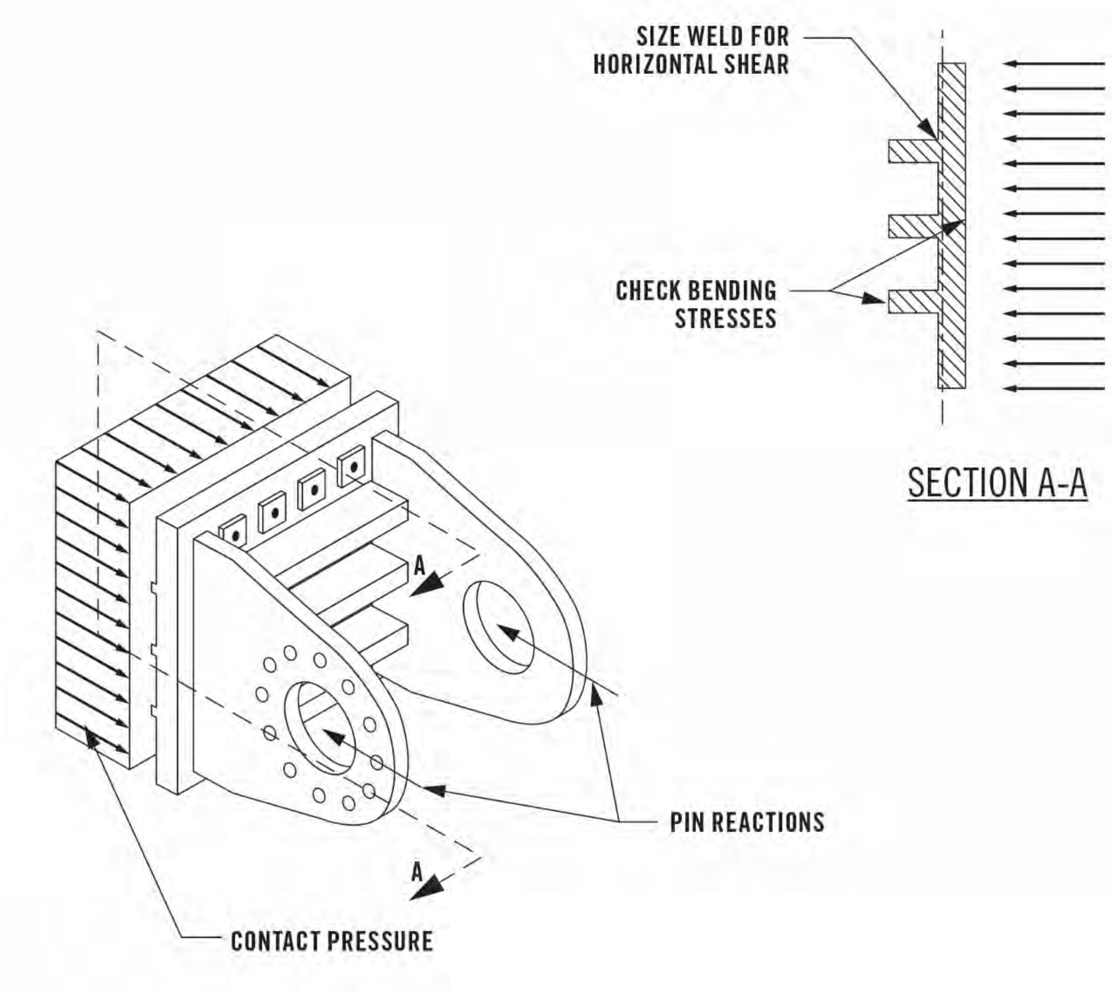


Figure 10.35. Design of Base Plate

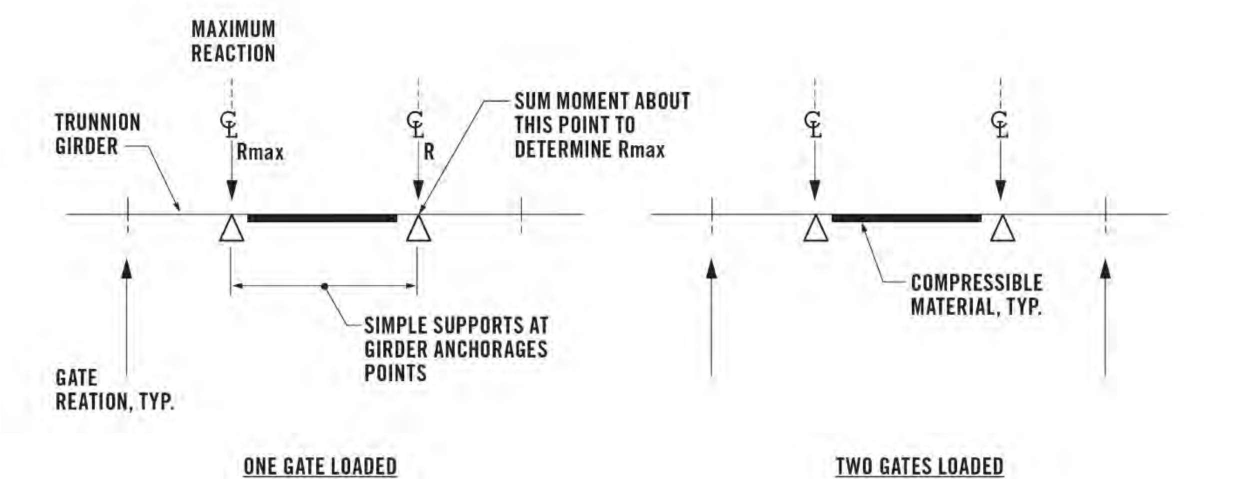


Figure 10.36. Trunnion Girder Analytical Model

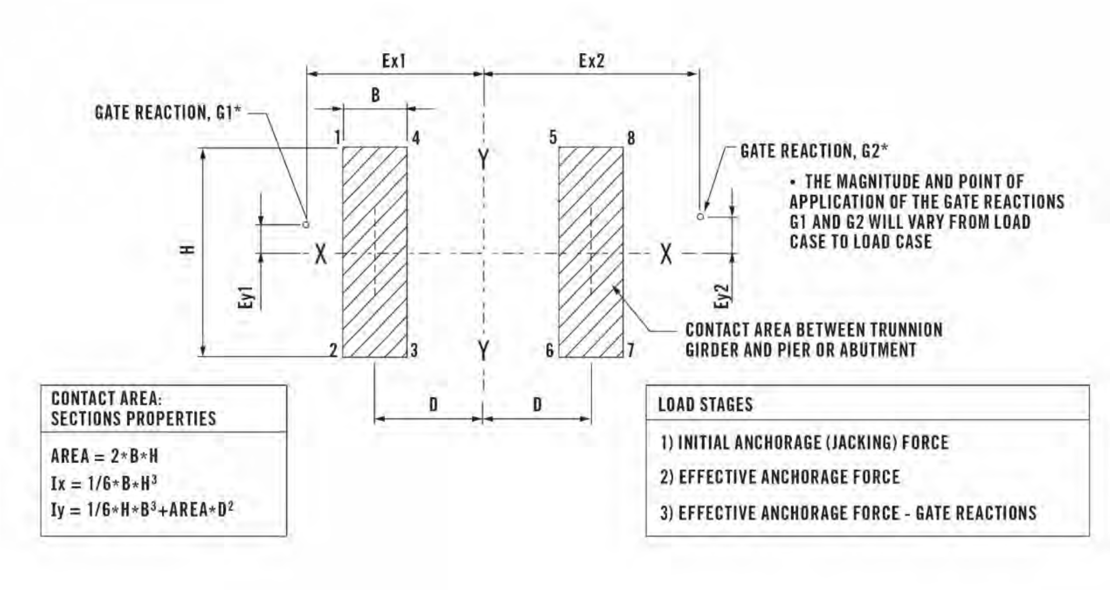


Figure 10.37. Analytical Model to Evaluate Anchorage Bearing Stresses

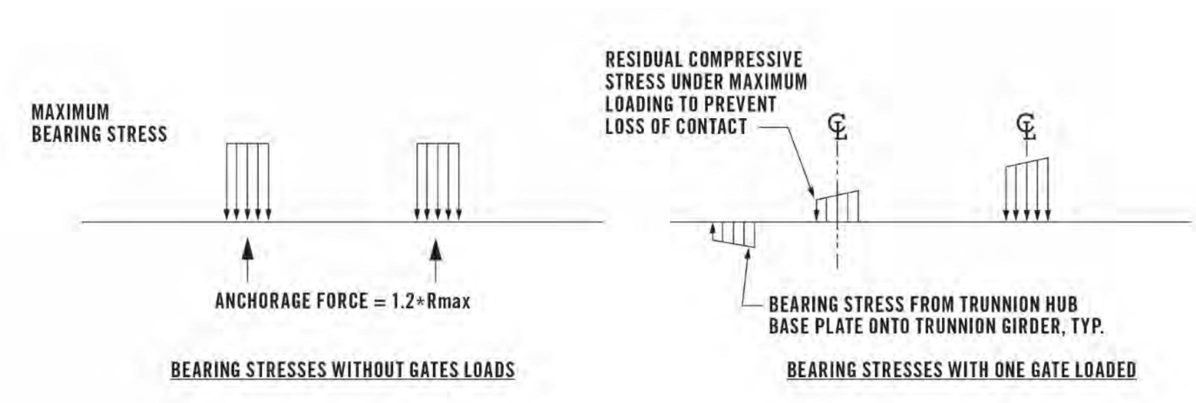


Figure 10.38. Stress Distributions Between Pier/Trunnion Girder Interface

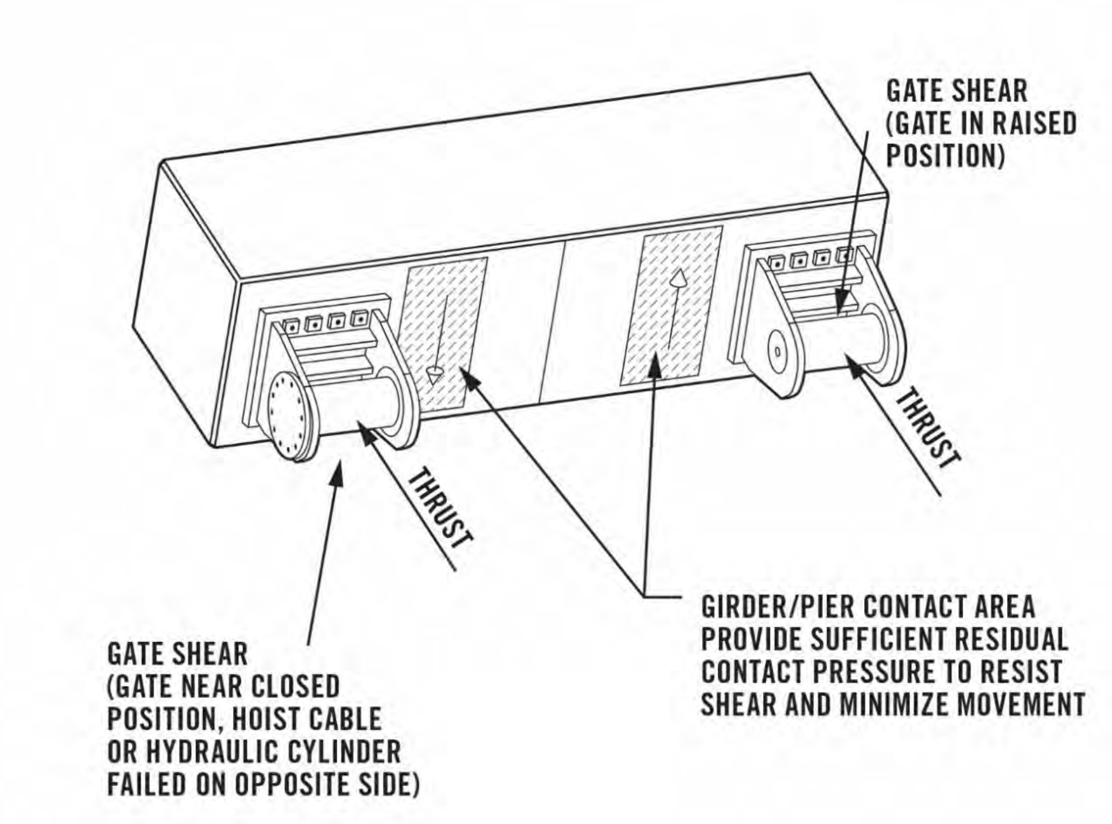


Figure 10.39. Trunnion Girder Movement

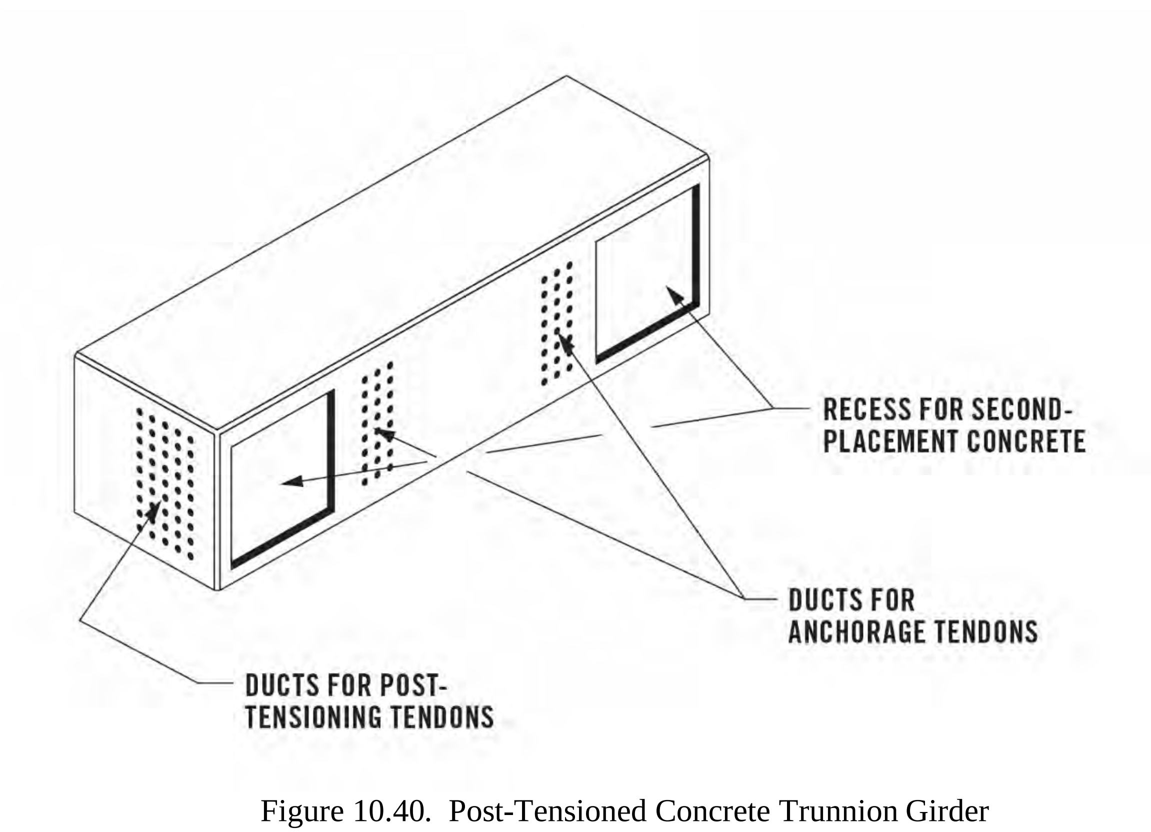


Figure 10.40. Post-Tensioned Concrete Trunnion Girder

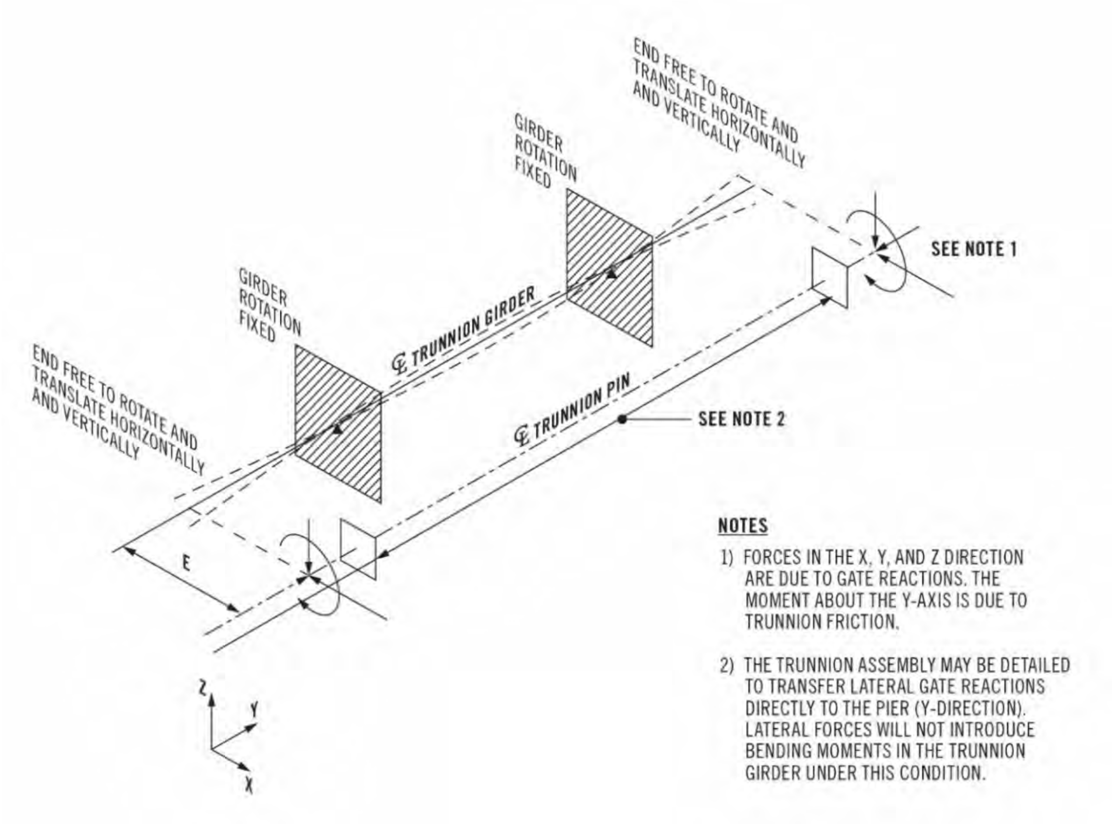


Figure 10.41. Trunnion Girder Analytical Model

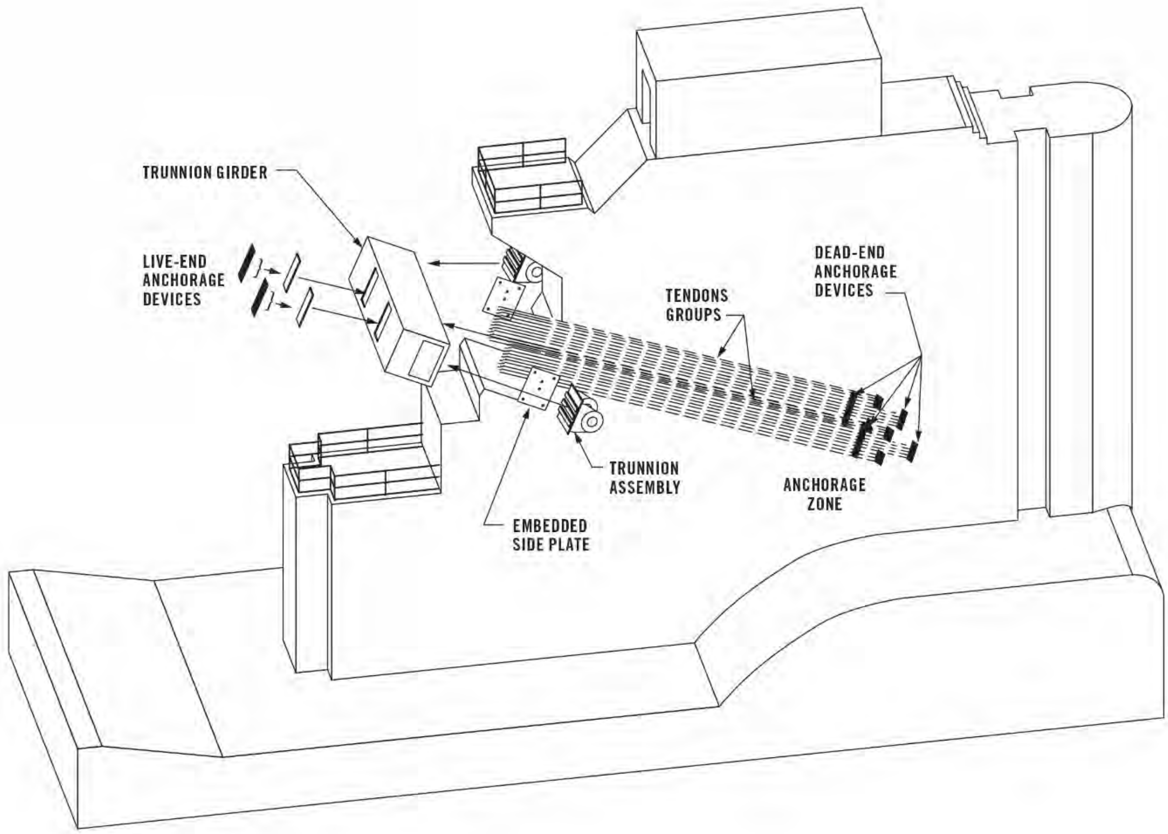


Figure 10.42. General Arrangement of Trunnion Girder Anchorage

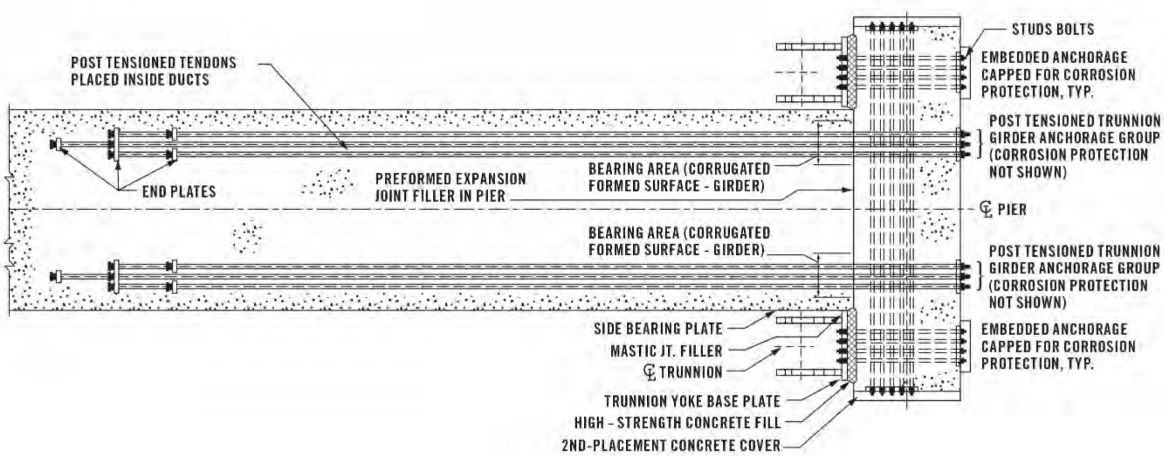


Figure 10.43. Post-Tensioned Anchorage System

Chapter 11
Lock Tainter Gates

11.1 General.

11.1.1 Submergible Tainter Gates. Submergible Tainter gates can be used as a lock gate, typically at the upstream end of the lock. For lock gate applications, the gate is raised to close the lock chamber and lowered into the lock chamber to open it. The end frames are recessed into the lock wall so the end frames do not project into the lock width. This type of gate might be less expensive than a double-leaf miter gate, and it permits the length of the approach channel to be reduced slightly. There are two potential problem areas in the operation of this type of gate: skewing of the gate during opening and closing, and vulnerability to damage if hit by lock traffic. However, with good design and operational practices, these problems can be minimized. See Chapter 10 for other considerations for designing Tainter gates.

11.1.2 Navigation Lock Gates. Navigation lock gates are usually wider and might have lower heads compared to spillway gates. Because of the greater lock widths, the gates main horizontal structural members will be trusses or plate girders. Because of lock width clearance requirements, struts, trunnions, and lifting devices might all be located in recesses in the lock walls. Figure 11.1 shows a modern lock Tainter gate design and Figure 11.2 shows a more traditional design. Figure 10.3 shows an example submergible Tainter gate and Figure 10.4 shows a recessed end frame.

11.1.3 Configuration. This manual describes a conventional Tainter gate configuration. Lock Tainter gates are comprised primarily of structural steel and are generally of welded fabrication. Structural members are typically rolled sections; however, welded built-up girders may be required for large gates in order to control deflections, eliminate skewing, and eliminate torsion concerns. Various components of the trunnion assembly and operating equipment may be of forged or cast steel, copper alloys, or stainless steel. There are typically three framing methods utilized in construction: truss girders, plate girders, or tub girders.

11.1.3.1. Advantages of Lock Tainter Gates. Advantages of Tainter gates include: efficient transfer of hydrostatic loads to the trunnion due to the radial shape; a lower hoist capacity is required; efficient operation; and side seals are used, so gate slots are not required.

11.1.3.2. Disadvantages of Lock Tainter Gates. Disadvantages of Tainter gates include the following:

11.1.3.2.1 Embedded trunnion anchorages are problematic to inspect.

11.1.3.2.2 Substantial blockouts are required to accommodate end frames and trunnion anchorage.

11.1.3.2.3 Seal details are challenging due to deflections.

11.1.3.2.4 May not be feasible for extremely wide navigational locks.

11.2 Loads and Load Combinations.

11.2.1 Design Requirements Using LRFD. Design requirements using LRFD are provided in chapter 3 and 4. This section provides information specific to design of Lock Tainter Gates for strength and serviceability.

11.2.2 Loads. Chapter 4 describes loads for all gates. General descriptions of loads applied to Tainter gates are provided in Chapter 10. Fatigue loads are described in Chapter 9.

11.2.3 Load Combinations. Lock Tainter gates will be designed for the strength or fatigue limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in paragraph 4.3.3. These cases assume the gate is submergible. For non-submergible gates, the load cases of Chapter 10 are applicable with the addition of the fatigue limit state load cases below. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

11.2.3.1. Load Combination 1: Strength Limit State.

11.2.3.1.1 Gate in the fully lowered position with no differential hydrostatic pressure. Loads consist of dead load, gravity loads, maximum hydraulic cylinder pressure, QD_{pr} if a sill is present.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, QD_{pr} \qquad \text{(Equation 11.1)}$$

11.2.3.1.2 The hydraulic cylinder load should be considered an extreme load (QD_X) with unknown return period ($\gamma_{pr} = 1.3$).

11.2.3.2. Load Combination 2: Strength Limit State. Gate Supported by Two Hoists in Closed Position.

11.2.3.2.1 Load Combination 2a. Upper gate subjected to maximum differential hydrostatic loading, Hs_{pr}, with companion wave, Hw_c, or debris impact, IM_c, if applicable. For locks without an upstream dewatering system, the gates will be designed for hydrostatic pressure from the dewatered condition. Operating machinery forces are a reaction. The hydrostatic principal load factor is selected according to paragraph 4.3.3 based on the return period of the maximum loading.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, Hs_{pr} + 1.0\, (Hw_c \text{ or } IM_c) \qquad \text{(Equation 11.2)}$$

11.2.3.2.2 Load Combination 2b. Loads consist of barge impact (both upper and lower gates), or extreme wave, debris impact, or thermal ice expansion (upper gates if applicable), as applicable plus companion hydrostatic loading, Hs_c, dead load and gravity loads. Operating machinery forces are a reaction:

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, (IX_X \text{ or } IM_X \text{ or } BI_X \text{ or } Hw_X) + 1.0\, Hs_c \qquad \text{(Equation 11.3)}$$

11.2.3.2.3 Unless site specific data is available, the return period of the maximum thermal expansion ice load and debris and ice impact load are not known and γ_{pr} = 1.3. For barge impact γ_{pr} = 1.3. Wave loads for return periods meeting the requirements of paragraph 4.3.4.1 can be estimated from wind data and therefore γ_{pr} = 1.2 for waves (principal load condition 1).

11.2.3.3. Load Combination 3: Strength Limit State. Gate Operating on Two Hoists. Loads consist of maximum hydrostatic, Hs_{pr}, with gate subjected to dead load, gravity loads, and side seal and trunnion friction. Operating machinery forces are a reaction:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs_{pr} + 1.4\ Fs + 1.4\ Ft \qquad \text{(Equation 11.4)}$$

11.2.3.4. Load Combination 4: Strength Limit State. Gate Open. Gate Operating on One Hoist. Loads consist of unusual hydrostatic loading, Hs_N, as a principal load with gate subjected to dead load, gravity loads, side seal and trunnion friction, and sidesway friction load, Fb, (if present). Operating forces are reactions.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.4\ Hs_N + 1.4\ Fs + 1.4\ Fb + 1.4\ Ft \qquad \text{(Equation 11.5)}$$

11.2.3.5. Load Combination 5: Strength Limit State. Gate Jammed.

11.2.3.5.1 Loads consist of maximum operating equipment force, QU_{pr}, plus companion hydrostatic loading, Hs_c, with gate subjected to dead load, and gravity loads:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ QU_{pr} + 1.0\ Hs_c \qquad \text{(Equation 11.6)}$$

11.2.3.5.2 The maximum operating load should be considered an extreme load (QU_X) with unknown return period (γ_{pr} = 1.3) unless site specific information is available.

11.2.3.6. Load Combination 6: Strength Limit State. Gate Fully Opened. Gate Supported by Two Hoists. Loads consist of dead load and gravity loads plus wind where wind is the principal load.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.0\ W \qquad \text{(Equation 11.7)}$$

11.2.3.7. Load Combination 7: Strength Limit State. Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load and gravity loads. The gate may be closed or open.

11.2.3.7.1 For standard and site-specific OBE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c \qquad \text{(Equation 11.8)}$$

11.2.3.7.2 For standard MDE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.25\ EQ + 1.0\ Hs_c \qquad \text{(Equation 11.9)}$$

11.2.3.7.3 For site specific MDE and MCE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ EQ + 1.0\ Hs_c \qquad \text{(Equation 11.10)}$$

11.2.3.8. Load Combination 8: Fatigue Limit State. Lock Tainter gates will be designed for fatigue for the stress range described in Chapter 5. Design must satisfy requirements for either Infinite Life or Finite Life. See section 5.1 for more information on load factors for fatigue.

11.2.3.8.1 Load Combination 8a. Fatigue Limit State I. Infinite Life:

$$2.0\ Hs + 2.0\ Hd \qquad \text{(Equation 11.11)}$$

11.2.3.8.2 Load Combination 8b. Fatigue Limit State II. Finite Life:

$$1.0\ Hs + 1.0\ Hd \qquad \text{(Equation 11.12)}$$

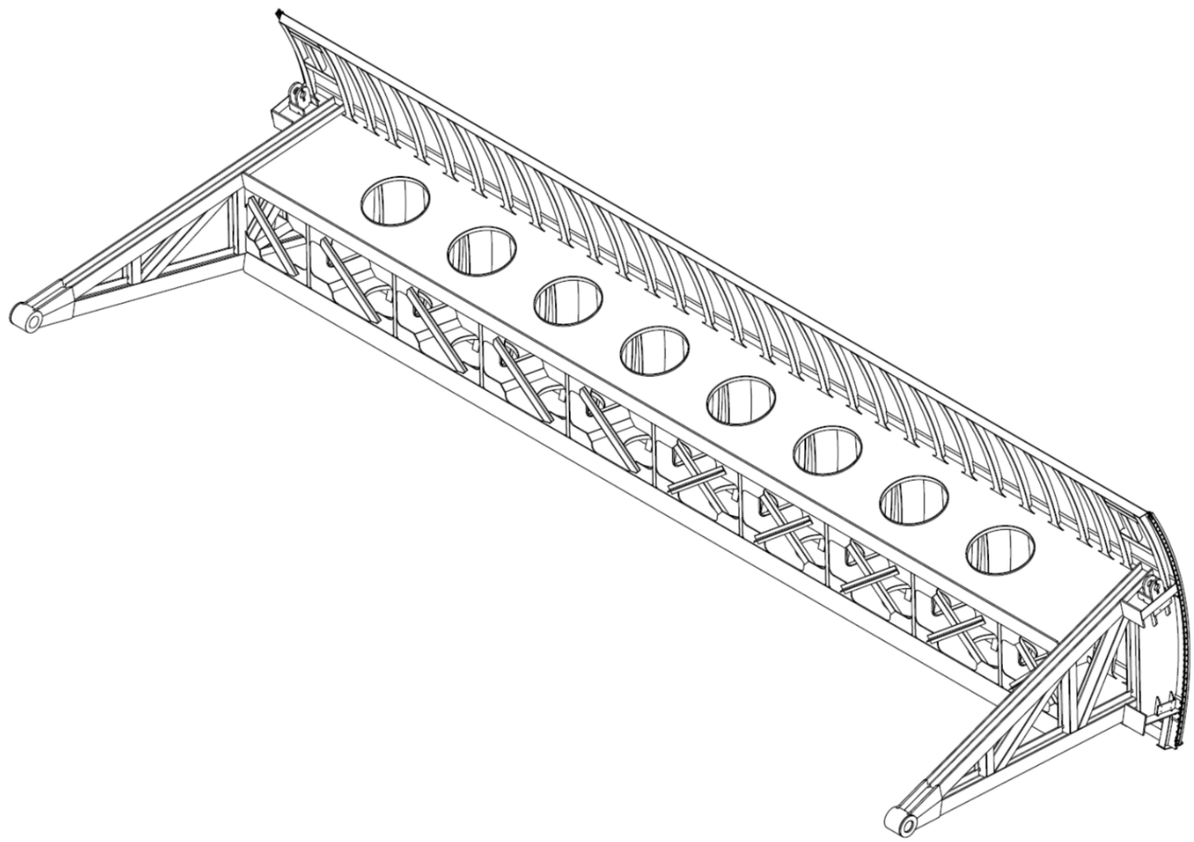


Figure 11.1. The Dalles Lock and Dam Navigation Lock Tainter Gate

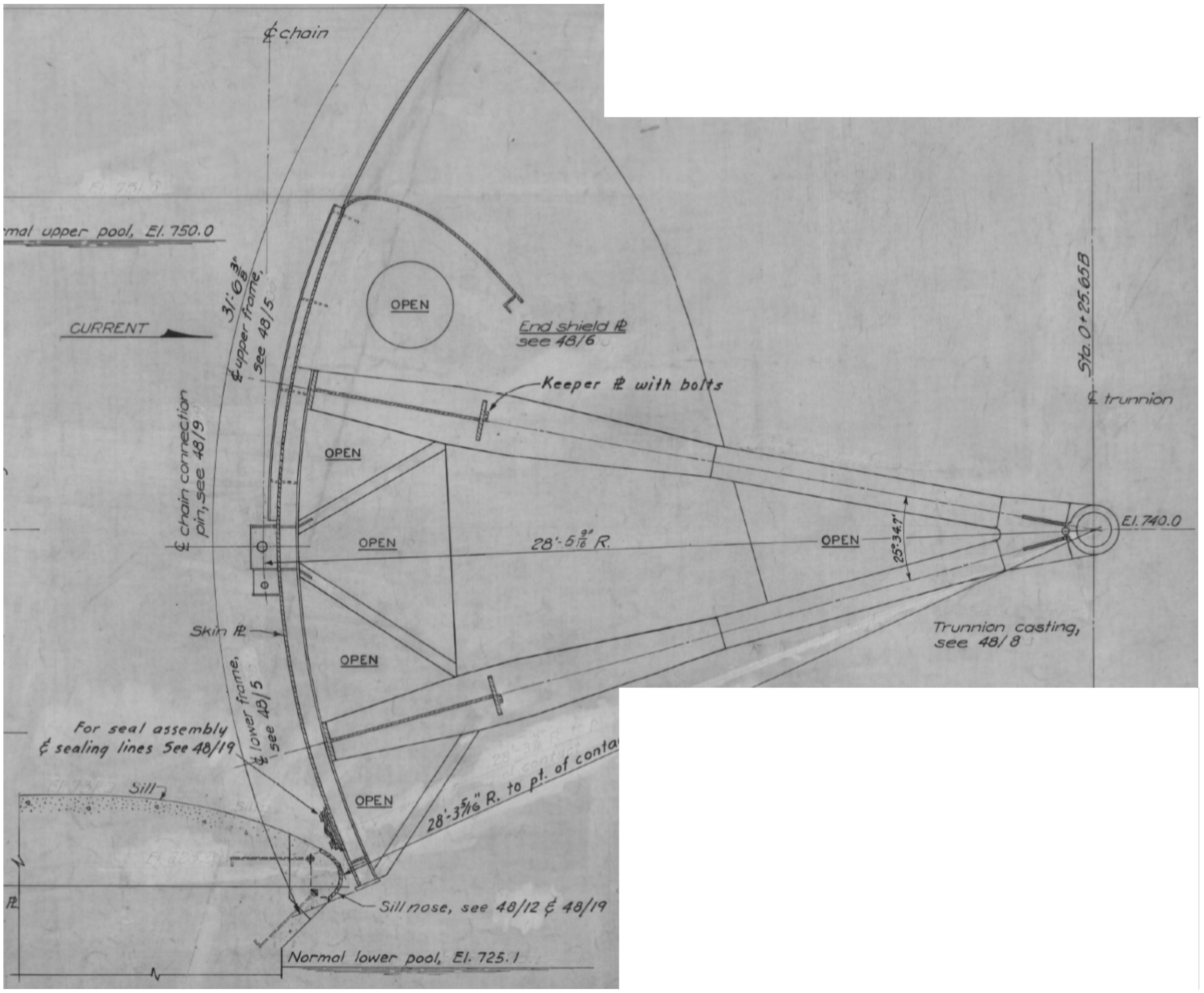


Figure 11.2. Lower Saint Anthony Falls Lock and Dam Navigation Tainter Gate

Chapter 12
Tainter Valves

12.1 General.

12.1.1 Tainter Valves. Tainter valves are typically used to fill and empty lock chambers. Tainter valves will be designed according to the guidance of Chapter 10 for Tainter Gates, with the exceptions to guidance on framing practice, loads, and load combinations provided in this chapter. Additional information is provided in EM 1110-2-1610.

12.1.2 Orientation. Tainter valves have been widely used in locks in North America since the 1930s. Initially, Tainter valves were oriented in the conventional manner, with the trunnions downstream of the skin plate. It was determined through model testing that for high-head applications or those instances where air can be drawn into the culvert, orientating the valve in the reverse manner proves beneficial. The type of valve to be used for filling and emptying lock chambers is determined by hydraulic considerations. See EM 1110-2-1610 for information on hydraulic performance of Tainter valves.

12.1.3 Fatigue Considerations. The cantilevered arched rib and horizontal girder connection can be subjected to high cycles of fatigue loading during operation of the valve, particularly at small valve openings or during slow opening and closing cycles. The operation of the valve through slow opening and closing cycles is often necessary to prevent concrete damage. This prolonged exposure to hydrodynamic loading on the cantilever rib connection results in infinite significantly high number of stress cycles with each valve operation. These connections are in direct tension or bending tensile stress and prone to fatigue cracking and fracture. When detailing these connections, particular attention should be paid to proper fatigue and fracture control techniques to improve the performance of this connection.

12.1.4 Configuration. This manual describes a reverse Tainter valve configuration. See Figure 12.1 for an example of a reverse Tainter valve configuration. Valves are comprised primarily of structural steel and are generally of welded fabrication. Structural members are typically rolled sections. Various components of the trunnion assembly and operating equipment may be of forged or cast steel, copper alloys, or stainless steel.

12.1.5 Framing Methods. There are three framing methods utilized in construction: Horizontal framing, vertical framing, and double-skin plate. The latter is used in order to generate a smooth hydraulic profile, where all upstream surfaces are typically covered with a smooth skin plate. The disadvantage of this framing type is it does not allow for the inspection of internal members and results in poor fatigue detailing. Selection of the framing system should be made using the detailed information of the effects of framing configuration on performance that are described in EM 1110-2-1610. See Figure 12.2 for examples of Tainter valve framing.

12.1.6 Advantages and Disadvantages of Tainter Valves. See Chapter 11 for advantages and disadvantages. Additional disadvantages for Tainter valve applications are sizable anchors are required to resist tensile forces because of the reverse configuration; trunnion anchorages are problematic to inspect; and flow induced oscillations can create a significant amount of stress cycles.

12.2 Loads and Load Combinations.

12.2.1 Design Requirements Using LRFD. Design requirements using LRFD are provided in Chapters 3 and 4. This section provides information specific to design of Tainter valves for strength and serviceability.

12.2.2 Loads. General loads and loading combinations are described in Chapter 4. Loads that are applicable to Tainter valve design include dead load, gravity loads, hydrostatic and hydrodynamic loads, operating loads, and earthquake loads. Specific loads for Tainter gates described in Chapter 10 are also applicable to Tainter valves, except for hydrodynamic loads described below. Fatigue loads on lock gates are described in Chapter 9.

12.2.3 Hd, Hydrodynamic Loads. See EM 1110-2-1610 for description of hydrodynamic loads applied to Tainter Valves. Hydrodynamic loads must be derived from model studies. Because of uncertainty in the hydrodynamic load, a load factor of 1.5 will be applied when the hydrodynamic load adds to the load effect and 0.0 when the hydrodynamic load reduces the load effect. Depending on the load combination, the maximum hydrodynamic load may be applied as principal, loads, Hd_{pr}. Or, the normal, usual, hydrodynamic load may be applied as a companion load, Hd_c.

12.2.4 Load Combinations. Tainter valves will be designed for the strength and fatigue limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in paragraph 4.3.3. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. The serviceability limit state is addressed in Chapter 4. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

12.2.4.1. Load Combination 1: Strength Limit State. Valve Closed. Loads consist of maximum differential hydrostatic head, $Hs_{pr.}$ The hydrostatic principal load factors are selected according to paragraph 4.3.3 based on the return period of the load.

$$(1.2 \text{ or } 0.9)\,D + \gamma_{pr}\,Hs_{pr} \qquad \text{(Equation 12.1)}$$

12.2.4.2. Load Combination 2: Strength Limit State. Valve Open. Valve Operating on Two Hoists (or when only one hoist is present with symmetrical lift). Loads consist of maximum combination of hydrostatic loading, Hs_{pr}, and hydrodynamic force, Hd_{pr}, with valve subjected to dead load, gravity loads, and side seal, trunnion friction. The hydrostatic and hydrodynamic principal load factors are selected according to paragraph 4.3.3 based on the return period of the load.

$$(1.2 \text{ or } 0.9)\,D + (1.6 \text{ or } 0)\,G + \gamma_{pr}\,Hs_{pr} + (1.5 \text{ or } 0)\,Hd_{pr} + 1.4\,Fs + 1.4\,Ft \qquad \text{(Equation 12.2)}$$

12.2.4.3. Load Combination 3: Strength Limit State. Valve Open. Valve Operating on One Hoist with Unsymmetrical Lift. Loads consist of unusual hydrostatic loading, Hs_N, acting as a principal load with valve subjected to dead load, gravity loads, side seal and trunnion friction, side sway contact drag load, Fb, (if present) and companion hydrodynamic load, Hd_c, (if present and acting in the direction of the hydrostatic load).

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.3\ Hs_N + 1.4\ Fs + 1.4\ Fb + 1.4\ Ft + (1.5 \text{ or } 0)\ Hd_c$ (Equation 12.3)

12.2.4.4. Load Combination 4: Strength Limit State. Valve Jammed.

12.2.4.4.1 Loads consist of maximum operating equipment forces, QU_{pr}, with companion hydrostatic loading Hs_c, dead load, and gravity loads as applicable.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ QU_{pr} + 1.0\ Hs_c$ (Equation 12.4)

12.2.4.4.2 The maximum operating load should be considered an extreme load (QU_X) with unknown return period ($\gamma_{pr} = 1.3$) unless site specific information is available.

12.2.4.5. Load Combination 5: Strength Limit State. Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load and gravity loads. The gate may be closed or open.

12.2.4.5.1 For standard and site-specific OBE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c$ (Equation 12.5)

12.2.4.5.2 For standard MDE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.25\ EQ + 1.0\ Hs_c$ (Equation 12.6)

12.2.4.5.3 For site specific MDE and MCE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ EQ + 1.0\ Hs_c$ (Equation 12.7)

12.2.4.6. Load Combination 6: Fatigue Limit State. Tainter valves will be designed for fatigue for the stress range described in Chapter 5. Design must satisfy requirements for either Infinite Life or Finite Life. See section 5.1 for more information on load factors for fatigue.

12.2.4.6.1 Load Combination 6a: Fatigue Limit State I. Infinite Life.

$2.0\ Hs + 2.0\ Hd$ (Equation 12.8)

12.2.4.6.2 Load Combination 6b: Fatigue Limit State II. Finite Life.

$1.0\ Hs + 1.0\ Hd$ (Equation 12.9)

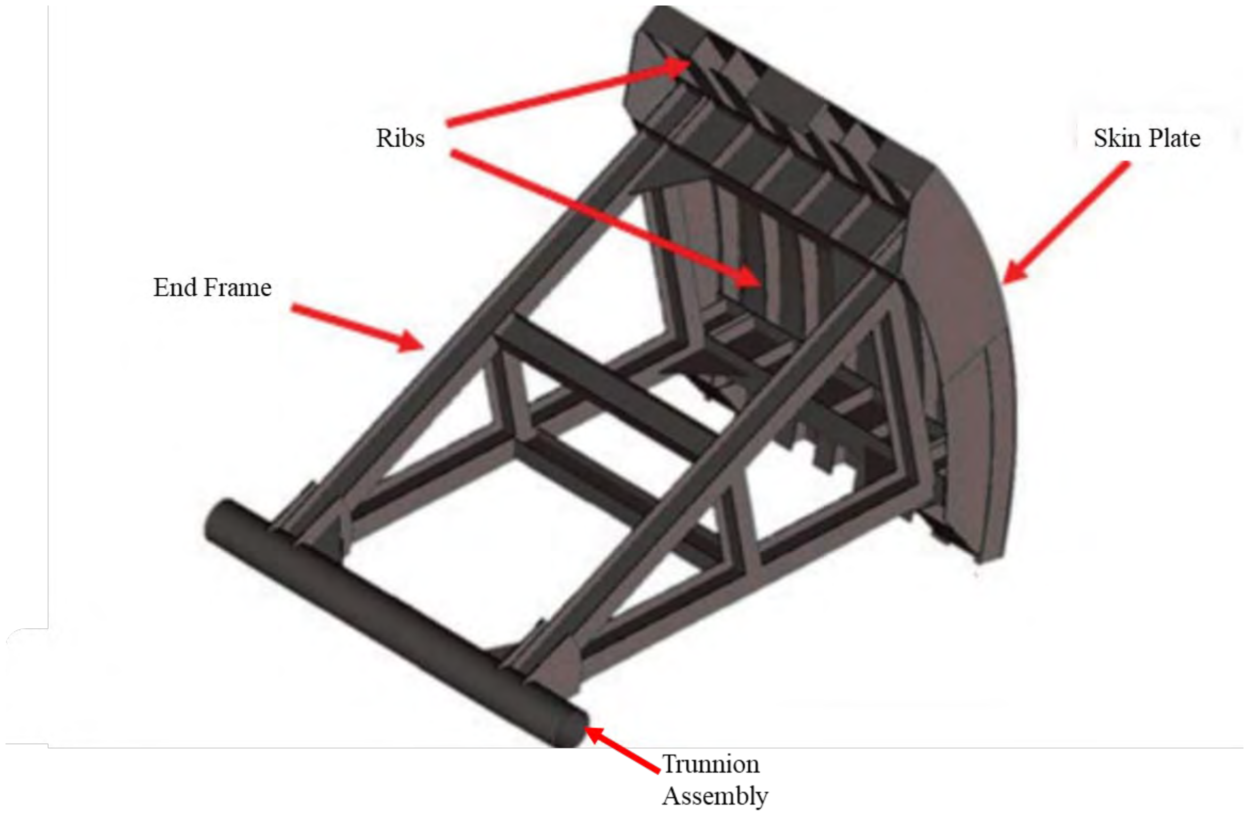


Figure 12.1. Typical Tainter Valve Components

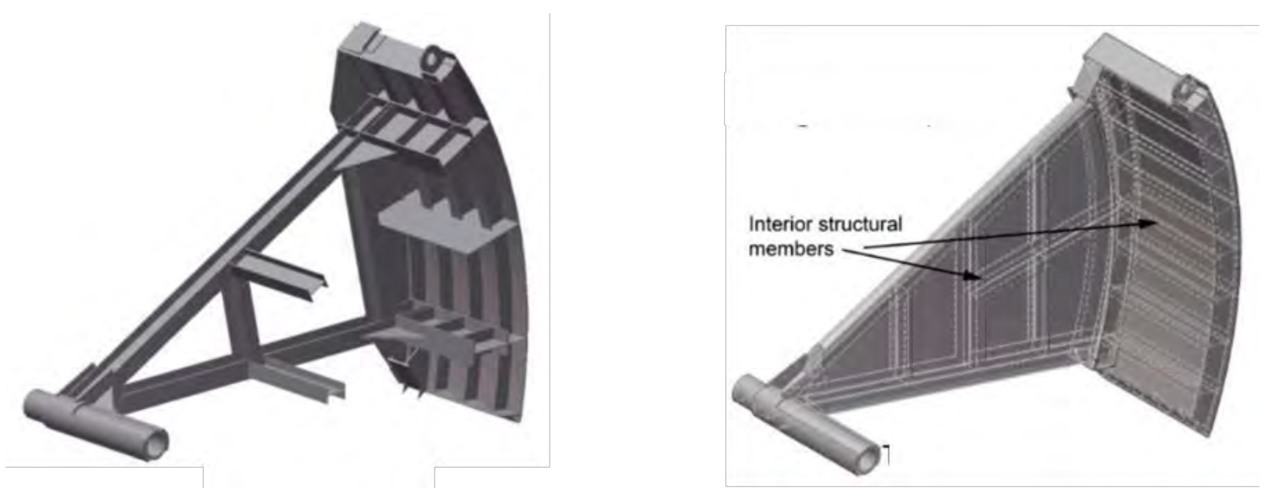


Figure 12.2. Tainter Valve Configurations – Vertically Framed (Left) and Double Skin (Horizontally Framed) (Right)

Chapter 13
Vertical Lift Gates

13.1 Introduction. This chapter presents criteria for the design of vertical lift gates used for water retention for routine or emergency operation in navigation projects, powerhouses, spillways, outlet works, and coastal hurricane protection or tide gates. For other types of gates such as sluice gates, bonnet type gates, and slide gates, specific load combinations have not been developed. Loads and load combinations of this chapter can be used on these gates if dynamic loading is equivalent. Otherwise, model studies should be conducted to determine hydrodynamic loading.

13.2 Description and Application.

13.2.1 General. Vertical lift gates are used for navigation lock chamber gates, emergency closure gates for powerhouse intakes and outlet works, and spillway crest gates. For each gate usage type, the gate is designed to accommodate special requirements for closure and retention of hydraulic head.

13.2.2 Types.

13.2.2.1. Overhead Gates. This type of gate uses a tower with overhead cables, sheaves, and bull wheels to support the gate during its operation and counterweights to assist hoisting machinery. The tower height is governed by the lift required to pass barge traffic. These gates can be a plate girder, horizontal tied arch, or horizontal truss. Plate 13.3, Plate 13.4, and Plate 13.5 show examples of a horizontal truss and tied arch. This type of gate would be used for the following applications: where it is not practical to use submersible gates (as with high-head applications); when sufficient support cannot be provided for transferring thrust from miter gates; where the available area to place the gate monolith is limited and will not permit the use of miter gates; or when the gate is used as a hurricane or tide gate and is subject to reverse hydrostatic or hydrodynamic loadings.

13.2.2.2. Submersible Gates. A submersible gate can be used as the upstream gate for a navigation lock, where the submersible leaf rests below the upstream sill. There are two types of submersible gates: single leaf and multiple leaf. The double-leaf arrangement is most common. It is composed of a downstream leaf, used for normal lock operation, and an upstream leaf, used infrequently as a movable sill or as an operating leaf in an emergency. This is referred to as the emergency leaf.

13.2.2.2.1 Downstream Leaf. Plate 13.6 and Plate 13.7 show an example of a downstream leaf. Both leaves are normally constructed of horizontal girders with an upstream skin plate. The hoist components at either side of the lock are mounted above the high water in a concrete recess with a removable roof section. The powered hoist component is mounted on a structural steel frame anchored to a concrete structure on one side of the lock. The nonpowered component is then mounted on the opposite wall. For the normal open or stored position, the leaves are lowered into the sill.

13.2.2.2.2 Emergency Leaf. The emergency leaf is used for lock closure in the event of an accident or damage to the gate that would otherwise result in loss of the navigation pool. This type of gate is useful when it is necessary to skim ice and drift from the lock approaches or to open the lock gates to pass flood flows.

13.2.3 Applications.

13.2.3.1. Navigation Locks. Overhead or submersible lift gates may be used as operating gates for lock chambers. Lift gates may be used at both ends of a lock, or at only one end in combination with another gate type at the other end. They can be raised or lowered under low to moderate heads, and may be used to skim ice or debris through the lock chamber. However, they are not used when there is reversed head. Plate 13.1 shows a general configuration for a double-leaf submersible gate.

13.2.3.1.1 Upstream Gates. Upstream gates are typically submergible and can be single or double leafed. A single leaf gate rises vertically to close off the lock chamber from the upper pool. When the lock is filled, the gate is opened by sliding the leaf vertically downward until the top of the leaf is at or below the top of the upper sill. In some cases, a double-leaf lift gate may be used. The upper leaf can be designed with a curved crest, which permits overflow to skim ice and debris, or to supplement flow from the primary filling system when the lock chamber is nearly full.

13.2.3.1.2 Downstream Gates. Downstream gates are single leafed and raised vertically to a height above the lower pool level so that vessels can pass underneath. The gate leaf is suspended from towers on the lock walls and may be equipped with counterweights to reduce the machinery requirements. Lock gates of this type are practical only for very high locks and where required vertical clearance can be provided under the gate in its raised position. Plate 13.2 shows an overhead lift gate.

13.2.3.2. Spillway Gates. A spillway gate may be used instead of a Tainter gate where a shorter length of pier is desired. These gates are usually raised by using the gantry crane or fixed hoists for each gate located on the spillway deck or operating platform. Dogging devices may be used to hold the gate at the proper elevation.

13.2.3.2.1 Single-Section Spillway Gates. This gate consists of one section that provides a variable discharge between the bottom of the gate and the sill.

13.2.3.2.2 Multiple-Section Spillway Gates. A multiple-section gate consists of two or more sections in the same slot with variable discharge between the sections or between the bottom section and the sill. Multiple-section gates can be equipped with a latching mechanism to allow use as a single-section gate. As the required discharge increases beyond the capacity of the largest opening between sections, top sections are removed from the service slots and are dogged above the pool level in emergency slots. Plate 13.8 shows the top section of a multiple-section gate.

13.2.3.2.3 Double-Section Spillway Gates. This gate consists of two sections in adjacent slots with variable discharge over the top section or beneath the bottom section. The double-section gate is used less frequently because removing the gate from the slot is more cumbersome, sealing is more complicated, and additional length of pier is required. This type is useful for skimming ice and debris. However, that function can also be performed by shallow top sections of a multiple-section gate that are lifted clear of the pool.

13.2.3.3. Outlet Gates. Lift gates may be used for emergency closure of water intake systems, turbine intakes, or outlet works. Their normal operation is in the open position. They are not used for controlling flows but are used to stop flow under operating conditions. They are supported by dogging devices during normal operation. Rollers may be required under high-head or hydrodynamic conditions. See Plate 13.9 for an example emergency closure gate.

13.2.4 Diaphragms. Diaphragms are used to transfer vertical loads from the hoists. The hoisting system uses either hydraulic cylinder or wire ropes. The type of hoisting system will be based on economics and governing criteria for closure times under emergency conditions. The hoisting system for wire ropes may be deck mounted or placed in recesses above the high pool elevation. Cylinders for the hydraulic system are mounted below the deck in the intake gate slot.

13.2.5 Types of End Supports. End supports for vertical lift gates may be classified according to the method used to transfer the loads to the gate guides. The gate guides receive the main reaction component from horizontal loads. Certain end supports are used to reduce friction which reduces the hoisting effort required for controlling flow and reduces wear and related maintenance.

13.2.5.1. Fixed-Wheel. Wheels revolve on fixed axles, which are either cantilevered from the body of the gate or supported at each end by the web of a vertical girder attached to the gate frame. The wheels may also be mounted by pairs in trucks that carry the wheel loads through center pins to end girders attached to the gate frame. When gate hoisting occurs with no static head, this type of end support will usually be most economical. The fabrication is generally less costly than using tractor type end supports, described below.

13.2.5.1.1 Outlet Works. When the gate is used for outlet works, this type of end support will receive higher point loads. This will cause a much higher bearing stress to the wheel and guides, as well as shear, bearing, and bending forces to the center pins and end girder. This type of end support is normally used in navigation lock gates or in situations where the gate is used to control flows while under low static head, as with spillway gates or emergency closure gates.

13.2.5.1.2 Navigation. When used for navigation lock gates, the wheels normally rest in a wheel recess to prevent them from transferring hydrostatic loads. With the wheels in the recess, horizontal loads are transferred through an end-bearing shoe to the pier-bearing surface. Hence, the wheels carry no hydrostatic load. Hydrostatic load is then transferred from end-bearing shoes on the gate to the gate guides. Refer to EM 1110-2-2610 for design and detailing information.

13.2.5.2. Tractor (Caterpillar). This type of end support has at each side of the gate one or more endless trains of small rollers that are mounted either directly on the vertical end girder, or on members attached to the vertical end girder. Plate 13.9 shows this type of end support system. Plate 13.10 shows chain and roller details. These are more commonly found on emergency closure gates or gates that control flow under high head. Because load transfer is achieved by uniformly distributed bearing through the small rollers, they are able to withstand large horizontal loads while being lowered under full hydrostatic head. The main advantages over fixed wheels include a lower friction component while hoisting under load, lower bearing stresses transferred to the guides and gate framing, and shear and bending not transferred to the gate through the axle.

13.2.5.3. Slide. Slide gates use metal-to-metal contact for end support. A machined surface that is mounted to the front face of the gate bears directly against a machined guide surface in the gate slot. The two bearing surfaces also serve as the gate seal. Materials for the gate seal surface may include aluminum, bronze, or stainless steel.

13.2.5.3.1 Application. These types of gates are normally used in intake/outlet tunnels where a head cover (bonnet) is used to seal off the guide slot from the gate operator for submerged flow installations. They can be used for high heads. The head during flow control in combination with the width and height of the inlet/outlet tunnel will determine the feasibility for using slide gates.

13.2.5.3.2 Bearing Surface Requirements. The bearing surfaces of the guides and slide bearings must be machined to tight tolerances to maintain a seal for the gate. This requires tighter construction tolerances for installation of the guides and slide bearings than with tractor gates and fixed-wheel gates.

13.2.5.4. Stoney. Similar to a tractor gate, a Stoney gate uses a small train of rollers. However, the fundamental difference is that the roller axles are held in position by two continuous vertical bars or angles on either side of the roller. The load is transferred from a bearing surface on the gate, through the rollers, to the guide-bearing surface on the monolith. The entire roller train is independent from the gate and the guide, which allows free movement of the roller train.

13.2.5.4.1 Supports. To maintain the roller train in its proper vertical position, it is common to use a wire rope support. The rope is fixed to a point on the gate, passes around a sheave fixed to the roller train, and is fixed to a point on the pier or monolith. Lateral movement is prevented by vertical bars or axles along the guide surfaces.

13.2.5.4.2 Advantages. A unique feature of this type of load transfer system, as in tractor gates, is that axle friction is not developed. Hence, there is a much lower friction component attributed to rolling friction. The main advantages of this type of gate support system are the same as those for the tractor supports.

13.3 Framing Systems.

13.3.1 General. Horizontal framing systems are preferred over vertical framing systems because they transfer loads more efficiently to the supports. Vertical framing systems are not recommended for new vertical lift gates, except where being replaced in kind. Framing members consist of girders, trusses, or tied arches. The framing system selected will depend on span, hydrostatic head, and lift requirements. Trusses and tied arches are typically used for wide or tall structures. The magnitude of design forces will dictate the framing type.

13.3.2 Girders. Horizontal plate girders are the main force-resisting members of the gate. They consist of built-up plate elements forming the stiffened webs and flanges of the girder. The spacing of the girders will depend on the head requirements, the height of the gate, and the clear span. Varying of girder spacing is economical for taller gates but may not be for intermediate to shorter height gates. Varying spacing is more economical than varying member sizes due to the economy in fabrication of multiple identical members. The girders frame into end posts that transfer end shear from the girders to bearing, either on the gate guides or through the end supports. Intercostals are used to support the skin plate. Diaphragms are used to distribute horizontal and vertical loads. Plate 13.6, Plate 13.7, Plate 13.8, and Plate 13.9 show examples of horizontal girder framing.

13.3.3 Trusses. Truss spacing and member sizing considerations are identical to those for girders. Plate 13.5 shows a typical use of horizontal trusses for navigation lock framing. Common members used for the trusses are wide flanges and structural Ts. It may be economical to construct trusses from all plate material. The main trusses frame into end posts supported by an end-bearing similar to girders. Special framing requirements need to be considered for the roller guides in the upstream-downstream and lateral directions. As with girders, other framing members include intercostals, diaphragms, end posts, stiffeners, and skin plates.

13.3.4 Tied Arches. The members can be made of rolled shapes, built-up members, solid plates, or plate girder members. The front arch may be framed with structural Ts, with the webs welded continuously to the skin plate. Plate 13.3 and Plate 13.4 show a vertical lift gate of this type. As with girders, other framing members include intercostals, diaphragms, end posts, stiffeners, and skin plates.

13.3.5 Vertical Framing. Vertically framed gates typically use stiffened plate girders. Loads applied to the skin plate are transferred through the supporting vertical girders into horizontal girders at the top and bottom of the gate. The horizontal girders transfer the loads to wheels at the guide recess.

13.3.6 Outlet Gates. Outlet gates require a sloping bottom or flat bottom with lip extension on the downstream side to reduce downpull forces while operating with high-velocity water flows.

13.4 Loads.

13.4.1 Design Requirements Using LRFD. Design requirements using LRFD are provided in Chapters 3 and 4. This section provides information specific to design of Lift Gates for strength and serviceability.

13.4.2 General. General loads and loading combinations are described in Chapter 4. Loads that are applicable to lift gate design are described in the following paragraphs. Reactions are not listed below or in the load cases. Reactions are not factored since they are determined from equilibrium with factored loads applied. As a result, reaction forces include the load factors of the applied loads.

13.4.3 D, Dead Load. Dead load is defined in Chapter 4. A load factor of 1.2 is used when dead load adds to load effects and 0.9 when it reduces load effects.

13.4.4 G, Gravity Loads. Gravity loads include mud weight (M), and ice weight (C), and will be determined based on site-specific conditions.

13.4.5 Hs, Hydrostatic Loads, General. Hydrostatic load consists of hydrostatic pressure on the gate considering both upper and lower pools. This load is applied with a load factor of 1.6 when it adds to load effects. When it reduces load effects it is not applied.

13.4.5.1. Strength Design. For strength design, hydrostatic loads consist of hydrostatic pressure on the gate considering both upper and lower pools. Hydrostatic loads are described in Chapter 4. For Hs as a principal load, Hs_{pr}, Hs is the maximum hydrostatic loading from differential head. Depending on the return period of the load it may be usual, unusual, or extreme.

13.4.5.2. Companion Loads. For companion hydrostatic loads, Hs_c is the normal operating condition, with a return period of 10 years as defined in paragraph 3.3.3.3.

13.4.5.3. Fatigue Loads. For fatigue design, the fatigue stress range, as described in Chapter 5, will be computed considering load variation due to Hs.

13.4.5.4. Hs, Lock Gates.

13.4.5.4.1 Submersible Gates. For submersible gates, consideration must be given to the operation of a multiple-leaf gate, with the gate seals effective and ineffective. Figure 13.1 shows a typical double-leaf submersible gate configuration with seals noted. With this arrangement, the two leaves will be subject to differing hydrostatic loads. This arrangement should consider normal operation, using the downstream leaf as the operating leaf.

13.4.5.4.2 Downstream Leaf Operation. Operation of the downstream leaf when skimming ice or debris (hydrodynamic load described below) and use of the upstream leaf during emergency gate operation should the operating leaf fail. Figure 13.2 and Figure 13.3 show the case where the downstream leaf is used for normal operation, with the gate seal between the upstream and downstream leaf effective and ineffective, respectively. In this case, Hs is hydrostatic loading from the differential head between upstream and navigation lock pool elevations.

13.4.5.4.3 Normal Operation. During normal operation, Figure 13.4 and Figure 13.5 show the hydrostatic load to the submerged (upstream) leaf with the seal between the upstream and downstream leaves effective and ineffective, respectively. For this condition, Hs is loading from the differential head from the upstream and navigation lock pools. When the upstream leaf is used for lock operation, the same loadings must be applied to it, as in the case of the downstream leaf during normal operation.

13.4.5.4.4 Overhead Gates. Figure 13.6 and Figure 13.7, respectively, show the hydrostatic load Hs and water seal arrangements for overhead gates with and without a crossover gallery. For both conditions, Hs_{pr} is the maximum hydrostatic loading from differential head between the navigation lock pool and downstream tailwater. For the case where an overhead gate is used for an upstream navigation lock gate, the loading conditions would be the same as for a single leaf submersible gate, where Hs is the hydrostatic loading from differential head between the upstream pool elevation and tailwater pool elevation, or upstream sill.

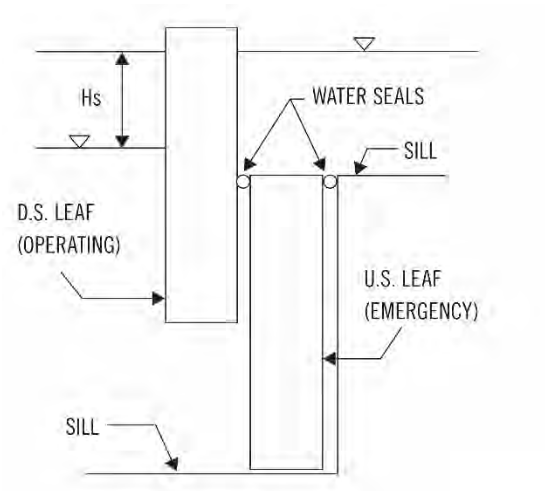


Figure 13.1. Submersible Lift Gate, Normal Operation

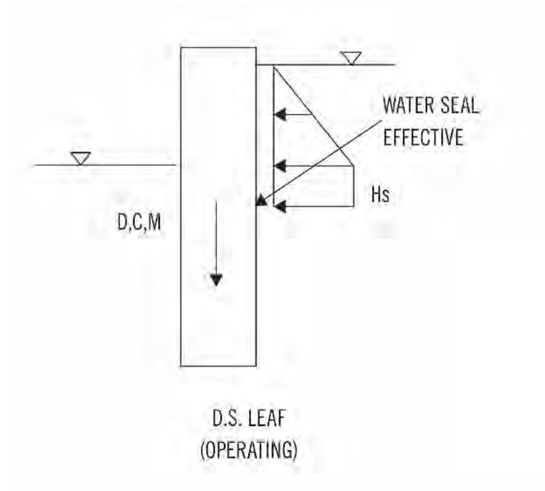


Figure 13.2. Submersible Lift Gate, Hydrostatic Loading Diagram, Downstream Leaf, Seals Effective

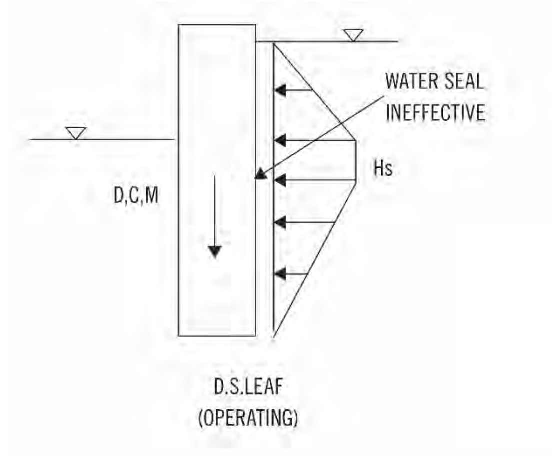


Figure 13.3. Submersible Lift Gate, Hydrostatic Loading Diagram, Downstream Leaf, Seals Ineffective

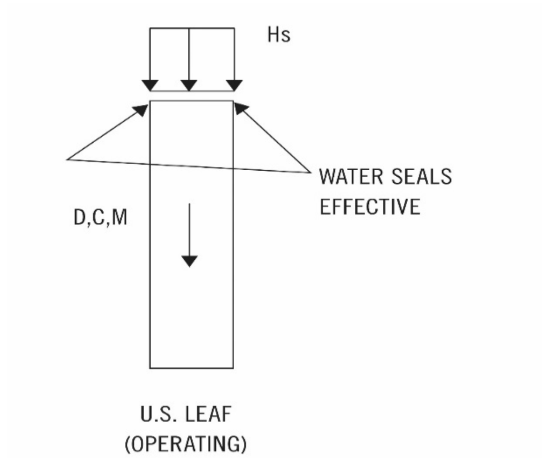


Figure 13.4. Submersible Lift Gate, Hydrostatic Loading Diagram, Upstream Leaf, Seals Effective

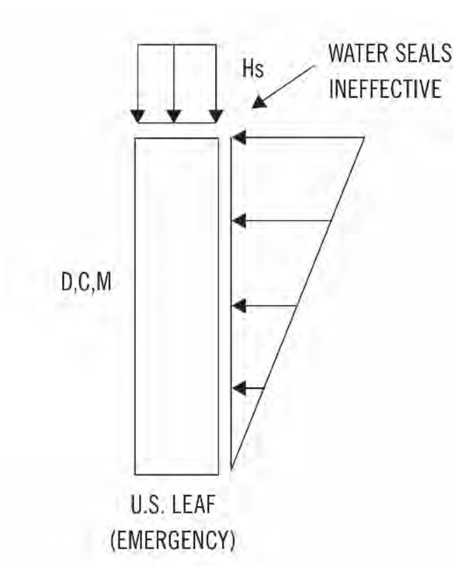


Figure 13.5. Submersible Lift Gate, Hydrostatic Loading Diagram, Upstream Leaf, Seals Ineffective

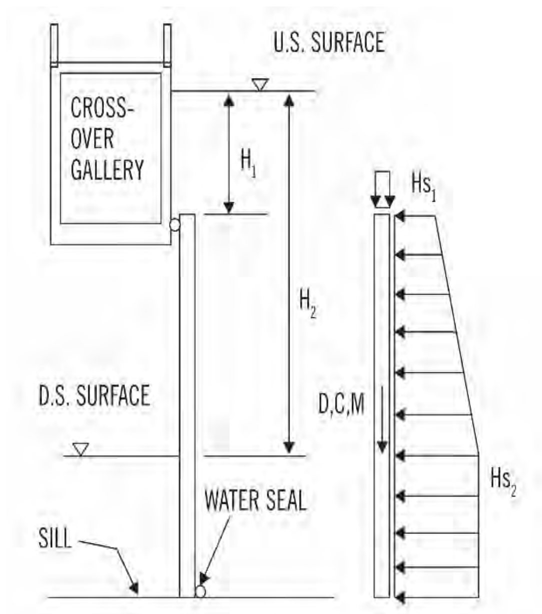


Figure 13.6. Overhead Lift Gate with Crossover Gallery, Hydrostatic Loading

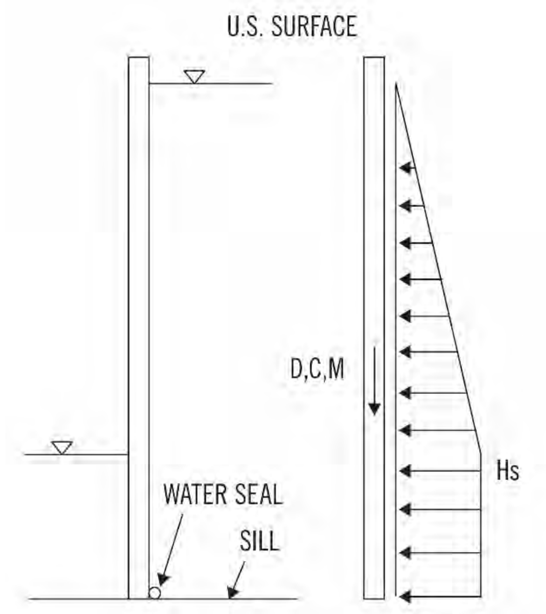


Figure 13.7. Overhead Lift Gate Without Crossover Gallery, Hydrostatic Loading

13.4.5.5. Hs, Spillway Crest Gates.

13.4.5.5.1 Single-Section Gates. For single-section gates, flow is under the gate. No consideration is given to water passing over the top of the gate. Hs represents hydrostatic loading from differential head between headwater and the sill bearing at the spillway crest (Figure 13.8 and Figure 13.9). In addition, Hs acts as an uplift on the bottom of the gate when passing flows through the spillway.

13.4.5.5.2 Multiple-Section Gates. For multiple-section gates, consideration must be given to water passing over the top sections of the gate because the gate can be split to allow flow at various sections. For each section, Hs is the hydrostatic loading from differential head between headwater and the bottom of each section, with the bottom section at the sill, bearing at the spillway crest (Figure 13.10 and Figure 13.11). These gates may be used as a single-section gate.

13.4.5.5.3 Double-Section Gates. For double-section gates, consideration must be given to flow over the top section. The amount of hydrostatic head flowing over the top section of the gate is determined from hydraulic studies and operational criteria for the reservoir. Operation of the bottom section should consider uplift (buoyant effects) on the bottom of the gate (included in Hd). Hs is the hydrostatic loading from differential head between headwater and the bottom of both sections, with the bottom section at the sill, bearing at the spillway crest (Figure 13.12 and Figure 13.13).

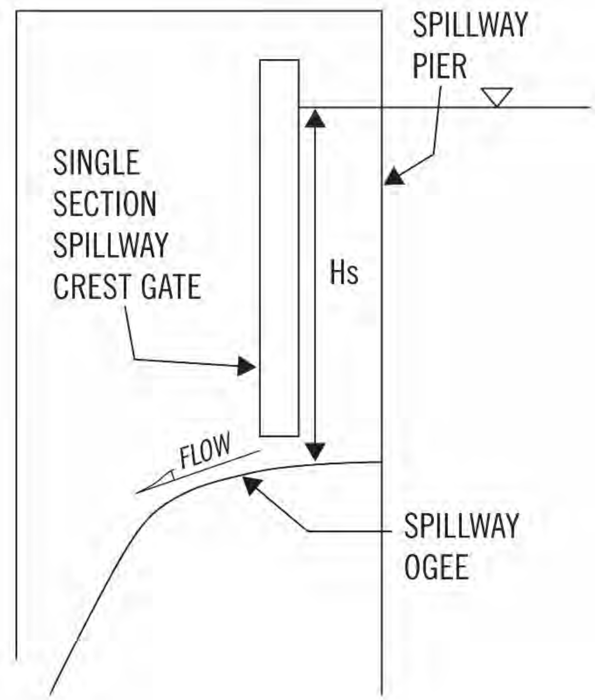


Figure 13.8. Single-Section Spillway Crest Gate

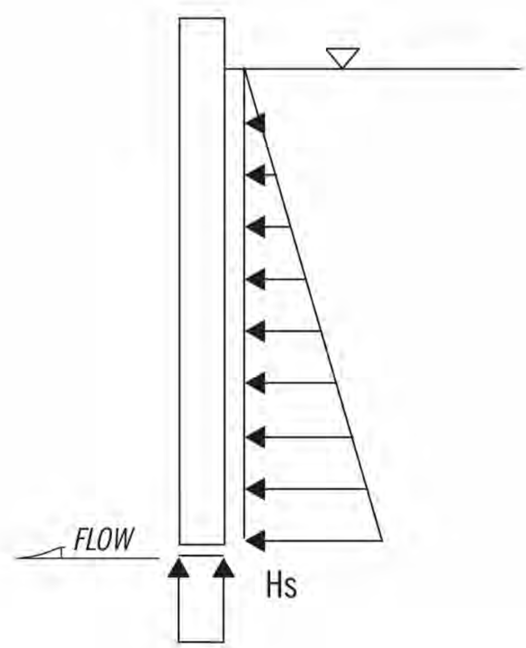


Figure 13.9. Single-Section Spillway Crest Gate, Hydrostatic Loading Diagram

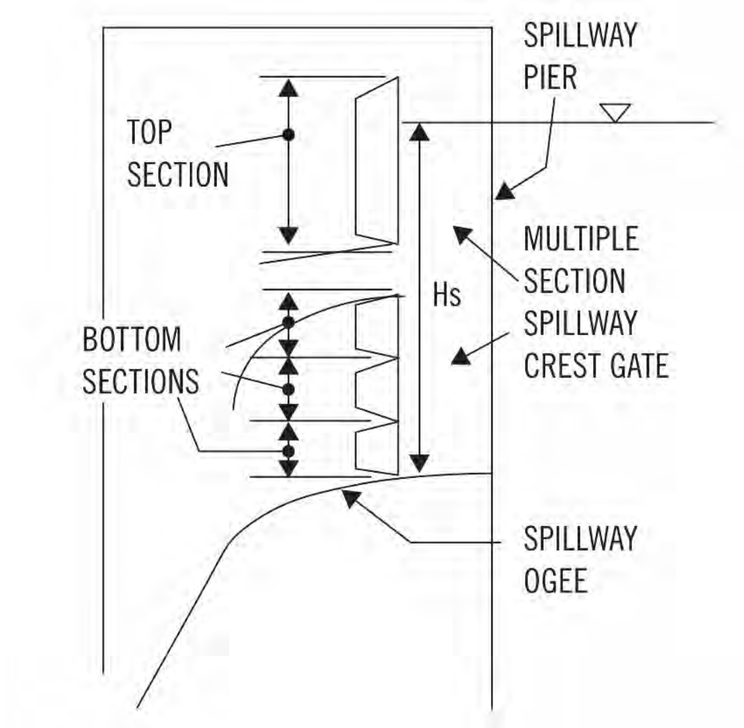


Figure 13.10. Multiple-Section Spillway Crest Gate

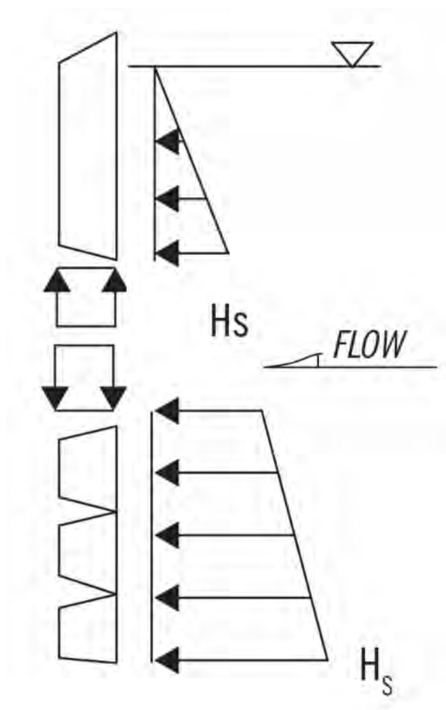


Figure 13.11. Multiple-Section Spillway Crest Gate, Hydrostatic Loading Diagram, Top and Bottom Sections Split

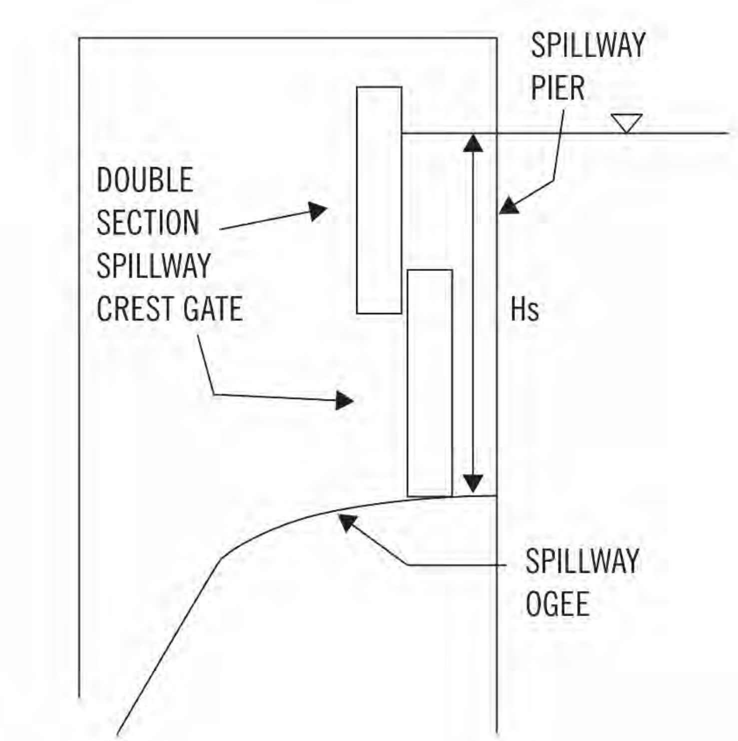


Figure 13.12. Double-Section Spillway Crest Gate

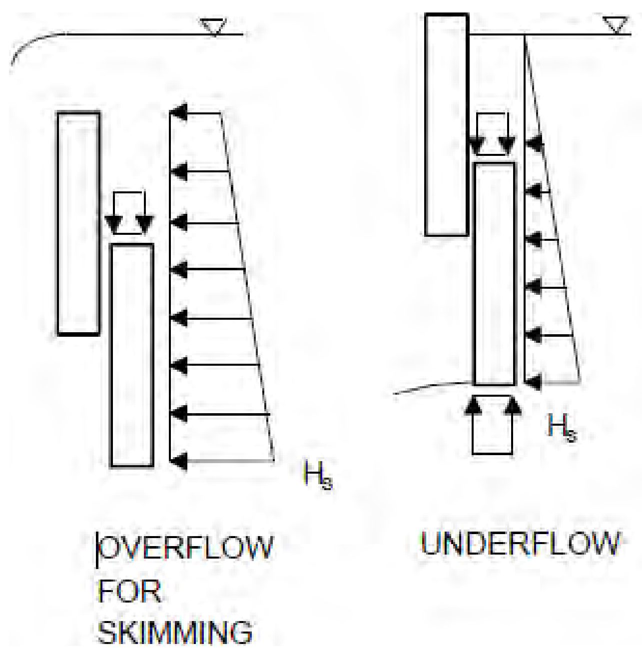


Figure 13.13. Double-Section Spillway Crest Gate, Hydrostatic Loading for Overflow and Underflow Operation

13.4.5.6. Hs, Outlet Gates. For outlet gates Hs consists of hydrostatic pressure on the gate considering both upper and lower pools. Figure 13.14 and Figure 13.15 show loading diagrams for hydrostatic loading of an outlet gate with a downstream seal and an upstream skin plate. Figure 13.16 and Figure 13.17 show loading diagrams for hydrostatic loading of an outlet gate with an upstream seal with an upstream skin plate.

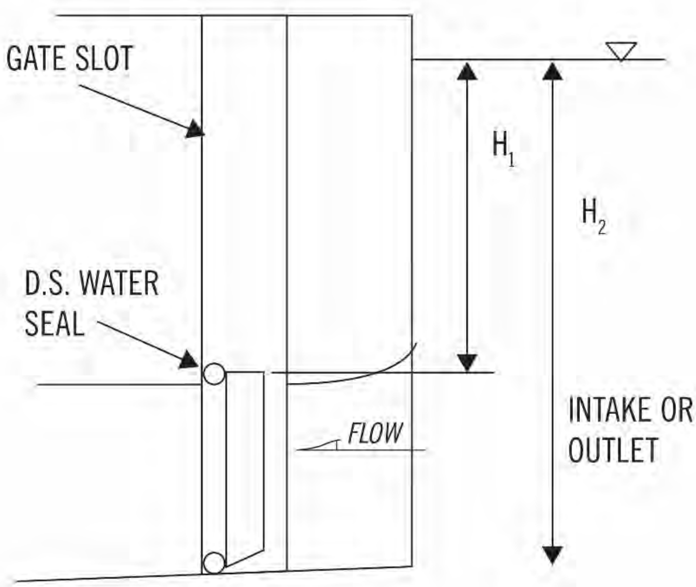


Figure 13.14. Outlet Gate with Downstream Seal with an Upstream Skin Plate

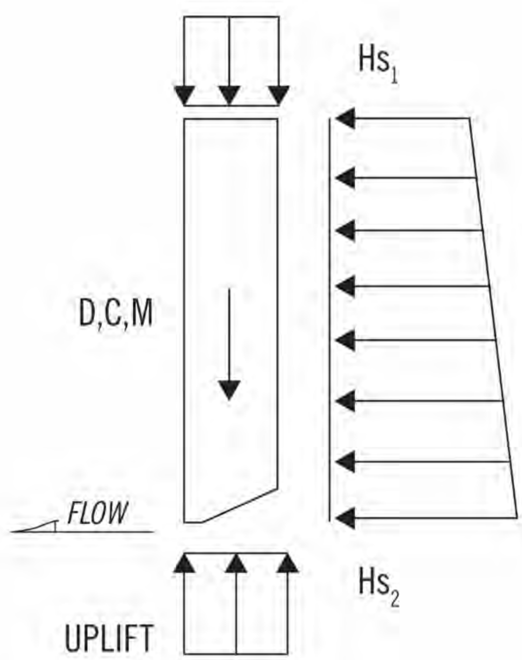


Figure 13.15. Outlet Gate, Hydrostatic Loading, Downstream Seal with an Upstream Skin Plate

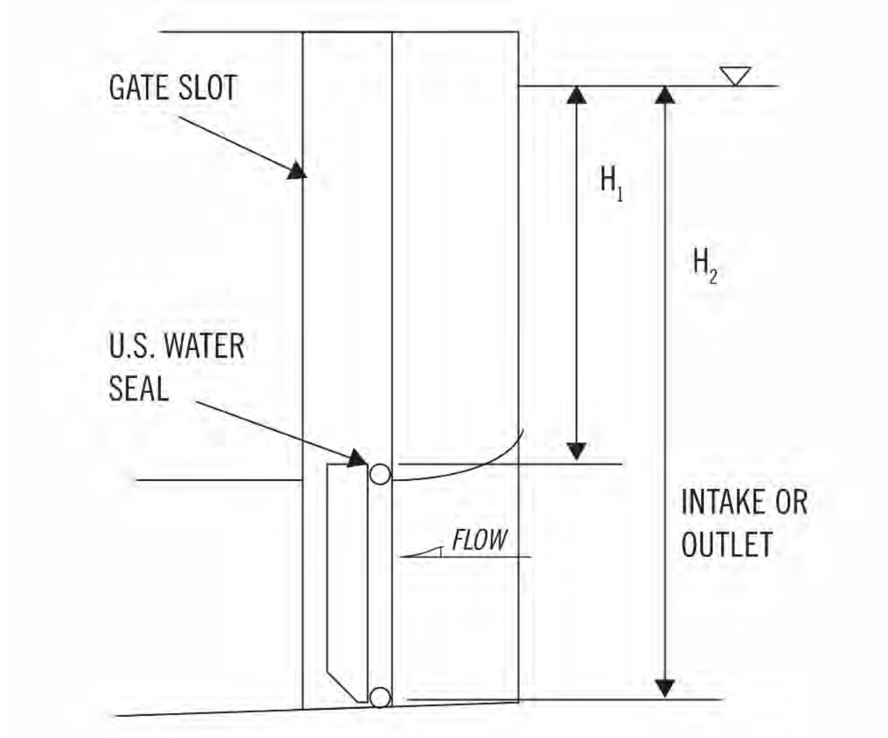


Figure 13.16. Outlet Gate with Upstream Seal with an Upstream Skin Plate

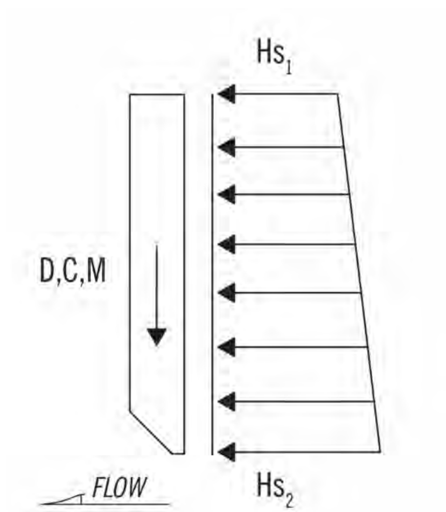


Figure 13.17. Outlet Gate, Hydrostatic Loading, Upstream Seal with an Upstream Skin Plate

13.4.6 Hd, Hydrodynamic Loads.

13.4.6.1. General. Hydrodynamic loads on lift gates include water flowing over submerged gates, uplift from water flowing under the gate, downpull (also known as downdrag) from water flowing down along the gate, and water hammer. The net uplift must be determined from combined effects of downpull forces and hydrostatic pressure under the gate.

13.4.6.2. Hd, Lock Lift Gates. For submersible gates, Figure 13.18 shows the operation of the downstream leaf when passing ice and debris. For lock lift gates Hd is the head from the flow overtopping the downstream leaf. Downpull and uplift and water hammer forces are not applicable for navigation lock gates because, except for occasional passing of ice and debris, they are raised and lowered in neutral head conditions.

13.4.6.3. Downpull Forces. Downpull force on a gate is the result of a reduction of pressure on the bottom of the gate from the static head, or may be viewed as a reduction in upthrust or reduction in buoyancy. These forces contribute to the hoisting requirements as well as the vertical load capacity of the gate. EM 1110-2-1602 and its referenced publications may be used to determine these loads acting on the gate. Other published data for methods of determining the effect of downpull forces may be obtained from USACE Hydraulic Design Criteria (HDC) Sheets 320-2 to 320-2/3, Sagar and Tullis (1979), and Sagar (1977a, 1977b, 1977c). Factors that affect the amount of downpull include:

13.4.6.3.1 The location of the gate seals (upstream or downstream);

13.4.6.3.2 Gate seal friction;

13.4.6.3.3 Upward thrust component due to energy head acting on the bottom of the gate;

13.4.6.3.4 Shape of the bottom of the gate; and

13.4.6.3.5 Flow over the top of the gate.

13.4.6.4. Water Hammer. Water hammer may develop depending on the type of application to which the gate will be subjected. Variables associated with the magnitude of pressure change include the rate of change of the flow (closure time), the velocity of the water, and length of penstock or conduit. EM 1110-2-3001 provides information to determine the effects of water hammer and suggests that the hydraulic system be modeled using computer analysis for various operating conditions. Water hammer associated with emergency closure is considered an extreme event.

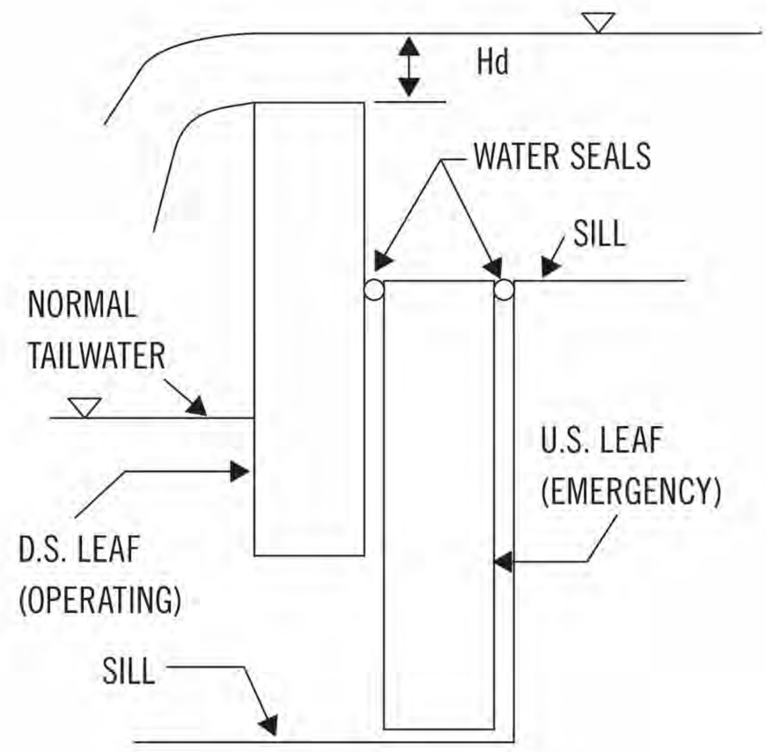


Figure 13.18. Submersible Lift Gate, Hydrodynamic Loading for Passing Ice and Debris

13.4.6.5. Hd, Dam Crest Gates. Hydrodynamic loads consist of overtopping of submersible gate leaves, similar to that shown in Figure 13.18, and downpull.

13.4.6.6. Hd, Outlet Lift Gates. For outlet gates, Hd includes water hammer, uplift and downpull. Hydrodynamic forces from flow either under or over the top of this type of gate are accounted for in downpull forces. Downpull forces are covered under Crest Gates.

13.4.7 Hw, Wave Loads. See Chapter 4 for determining wave loads.

13.4.8 Q, Operating Loads. Under normal operating conditions, operating loads are treated as reactions to all opposing forces including D, G, Hs, Hd, and friction, F. In the case of gate binding, the operating load, Q_{pr}, will be the maximum load that can be exerted by the operating machinery (obtained from the mechanical engineer who designed the machinery). See Chapter 4 for further discussion on operational loads.

13.4.9 IM, Debris Impact Load. This load accounts for the impact of debris (timber, floating ice, and other foreign objects). For sites where floating ice or debris present, IM is specified as a uniform distributed load of 5,000 lbs/ft that acts in the down-stream direction and is applied along the width of the gate at the upper pool elevation. Sites without floating ice may be designed for lesser values but design values should represent the upper bound of expected loads. IM must be placed to produce maximum effects. The probability of loading is unknown for IM and therefore the principal load condition 3 (Extreme) load factor applies.

13.4.10 IX, Thermally Expanding Ice Load. The thermally expanding ice load is specified to account for lateral loading due to thermal expansion of ice sheets on sites where this load is possible. Thermally expanding ice is a temporary load that is 5,000 lbs/ft across HSS members exposed to ice. The thermally expanding ice load will be applied at the upper pool elevations to produce maximum effects in each member. The probability of loading is unknown for IX and therefore the principal load condition 3 (extreme) load factor applies.

13.4.11 BI, Barge Impact. Barge impact load for lock lift gates is specified as a point load and is applied to main framing members exposed to barge impact at locations that produce the maximum effects in the primary members of the gate. For gates in navigable waterways, the minimum design barge impact load is equal to 5 kips/ft multiplied by the width of the gate opening. Gates at locations in which failure may result in loss of life from uncontrolled release of water or high economic or environmental consequences may require higher design loads. See section 4.2.6.3 for additional guidance on selection of barge impact loads.

13.4.12 W, Wind. Determination of wind loads is described in paragraph 4.2.4.2. Wind load must be applied as normal to the projected surface of the gate. Wind is applied to the exposed portion of the gate and while wind forces may not control gate member sizing, it can affect stability in the raised condition and the design of supporting members such as latching mechanisms. For submersible gates, wind loads need not be applied.

13.4.13 L, Live Loads. Lift gates occasionally have access ways on the gate designed for live loads. Live loads are defined in Chapter 4.

13.4.14 T, Self-Straining. Self-straining loads from extreme temperature differentials caused by ambient air and water temperatures adjacent to the exposed faces of the gate must be determined based on the navigation lock at full pool. At full pool, the skin plate is exposed to the pool temperature and the downstream girders or tension ties to ambient air conditions and tailwater. This will include temperature differentials related to seasonal ambient and water temperatures. For moderate climates, the ambient temperature range is from 0 to 120 °F, and for cold climates from –30 to 120 °F. Pool temperatures will be based on observed or recorded data and applied to the season during which the maximum ambient temperatures are predicted to occur.

13.4.15 EQ, Earthquake Loads. See Chapter 4 for earthquake loading.

13.5 Load Combinations. General loads and loading combinations for gates are described in Chapter 4. Lift gates must be designed for the strength, and fatigue limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in Chapter 4. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. The serviceability limit state is addressed in Chapter 4. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

13.5.1 Load Combination 1: Strength Limit State. Gate Closed. Maximum hydrostatic plus wind. Loads consist of dead, gravity, maximum hydrostatic loading from differential head (apply in both directions for gates that can be loaded by both direct and reverse head), and wave. The hydrostatic principal load factor is selected according to paragraph 4.3.3 based on the return period of the maximum hydrostatic loading.

13.5.1.1. Inland Lift Gates. Waves are created by wind events independent of water level.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr} Hs_{pr} + 1.0\ Hw_c \qquad \text{(Equation 13.1)}$$

13.5.1.2. Coastal Lift Gates. Hydrostatic, wave, and wind loads are correlated. Peak design wind gusts are unlikely to coincide with peak wave. A companion wind load is applied to the structure above the top of the wave pressure diagram.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ [Hs + Hw]_{pr} + 0.5\ W \qquad \text{(Equation 13.2)}$$

13.5.2 Load Combination 2: Strength Limit State. Gate Closed or Opened and set in typical operating positions. Maximum impact forces.

13.5.2.1. Loads consist of dead, gravity, companion hydrostatic along with companion barge impact, ice and debris impact, or thermally expanding ice force, as applicable.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr}\ (Hw_X \text{ or } IM_X \text{ or } IX_X \text{ or } BI_X) + 1.0\ Hs_c \qquad \text{(Equation 13.3)}$$

13.5.2.2. Unless site specific data is available, the average annual return periods of the extreme thermal expansion ice load and debris and ice impact load are not known and $\gamma_{pr} = 1.3$. For barge impact, $\gamma_{pr} = 1.3$. Wave loads for return periods meeting the requirements of paragraph 4.3.4.1 can be estimated from wind data and $\gamma_{pr} = 1.2$ for waves.

13.5.3 Load Combination 3: Strength Limit State. Gate Open. Loads consist of dead load and gravity loads plus maximum hydrostatic and companion hydrodynamic loading.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr}\ Hs_{pr} + 1.0\ Hd_c \qquad \text{(Equation 13.4)}$$

13.5.4 Load Combination 4: Strength Limit State. Gate Open.

13.5.4.1. Loads consist of dead load and gravity loads, maximum hydrodynamic loading plus companion hydrostatic loading.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr}\ Hd_{pr} + 1.0\ Hs_c$ (Equation 13.5)

13.5.4.2. Unless site specific data is available, the maximum dynamic load should be considered an extreme load (Hd_X) with unknown return period ($\gamma_{pr} = 1.3$).

13.5.5 Load Combination 5: Strength Limit State. Gate Open. Loads consist of dead load and gravity loads plus wind with wind applied in upstream or downstream directions.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ W$ (Equation 13.6)

13.5.6 Load Combination 6: Strength Limit State. Gate Jammed with load on one side.

13.5.6.1. Loads consist of dead load and gravity loads maximum operating loads plus companion hydrostatic loads.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G\ \ \gamma_{pr}\ Q_{pr} + 1.0\ Hs_c$ (Equation 13.7)

13.5.6.2. The maximum operating load should be considered an extreme load (Q_X) with unknown return period ($\gamma_{pr} = 1.3$).

13.5.7 Load Combination 7: Strength Limit State. Live Load. Gate Closed. Loads consist of live load as the principal load, dead, gravity, and companion hydrostatic, Hs_c.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.6L + 1.0\ Hs_c$ (Equation 13.8)

13.5.8 Load Combination 8: Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load and gravity loads. The gate may be closed or open.

13.5.8.1. For standard and site-specific OBE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c$ (Equation 13.9)

13.5.8.2. For standard MDE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.25\ EQ + 1.0\ Hs_c$ (Equation 13.10)

13.5.8.3. For site specific MDE and MCE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ EQ + 1.0\ Hs_c$ (Equation 13.11)

13.5.9 Load Combination 9: Fatigue Limit State. Lift gates will be designed for fatigue for the stress range described in Chapter 5. Design must satisfy requirements for either Infinite Life or Finite Life. See section 5.1 for more information on load factors for fatigue.

13.5.9.1. Infinite Life.

$2.0\ Hs + 2.0\ Hd$ (Equation 13.12)

13.5.9.2. Finite Life

1.0 Hs + 1.0 Hd (Equation 13.13)

13.6 Design Analysis and Detail Requirements.

13.6.1 Tied Arch Members. Design of the main tension tie requires the consideration of weak axis bending and torsion. Mud and ice resting on the web as well as diagonal and vertical bracing make a significant contribution to this type of loading. The secondary bracing may have a direct load path to the tension ties, which will also induce weak axis loading. Proper detailing of fracture critical connections to the tension tie is important to the service life of the gate.

13.6.2 End Posts and Bearings. The end-bearing transfers the girder reactions from bearing shoes, wheels, and rollers to bearing plates or tracks on the pier. Horizontal girders transfer load through shear into end posts. End posts may be single girders supporting cantilevered wheels or rollers, or double girders with wheels mounted on pins bearing on both sides. Two types of bearing conditions can occur:

13.6.2.1. One type provides bearing directly from the rollers as a series of point loads or from wheels as a single point load.

13.6.2.2. The other type relies on a bearing shoe mounted to the gate to transfer loads to the bearing plate. This type requires a recess in the guides to prevent the wheels from transferring hydrostatic load, allowing the bearing shoe to transfer hydrostatic load to the bearing plate. This becomes advantageous when loads are too great for wheel point bearing transfer.

13.7 Operating Equipment. Hoisting equipment includes wire ropes, roller chains, or hydraulic cylinders. See EM 1110-2-2610 for guidelines for operating equipment.

13.8 Dogging Devices.

13.8.1.1. General. Dogging devices are usually mounted on supports recessed in the piers opposite the gate end posts. They pivot to permit retraction for clearance of the gate. They are operated with push rods. Two or more dogging devices are required at each end of the gate slot. The number and location are determined by the operating requirements for discharge regulation and gate storage. The gate sections require dogging seats fabricated with structural or cast steel welded or bolted on the end posts. The treads of cantilevered wheels may be used as dogging seats.

13.8.1.2. Beam Type. This type consists of a cantilevered mild steel H-beam that retracts inside the gate at each end between the top and second girder web. The beam is located at the center of gravity of the gate in the upstream-downstream direction and runs through the end post to a reaction point at an interior diaphragm. The dogging beam is extended and retracted by using a bar as a lever extending through a hole in the top web and into a row of holes in the top of the dogging beam. The cantilevered end of the beam rests on bearing pads recessed in the piers. This type of dogging device is preferred for powerhouse gates because they can also be dogged at the intake or draft tube deck-level and because there are no mechanical devices to be lubricated or maintained. Dogging devices should be designed to support twice the calculated dead load to allow for impact.

13.9 Corrosion Control. See Chapters 5, 6, and 7 for guidelines on corrosion control.

13.10 Maintenance Considerations. See Chapter 7 for maintenance and inspection considerations.

13.11 Serviceability Requirements. Chapter 4 provides serviceability requirements for HSS.

13.12 Fatigue and Fracture Control. Chapter 5 provides gate design requirements related to fatigue and facture control.

13.13 Material Selection. Material selection guidelines for gates are provided in Chapter 3. Material information for wheels and axles is provided in EM 1110-2-2610.

13.14 End Support Design Details. See EM 1110-2-2610 for roller, wheel, and track design requirements.

13.14.1 Guides. Structural steel guide members should be provided to limit the movement of the gate horizontally, either in the upstream or lateral direction. The maximum upstream movement may be determined by the allowable deflection of the seal, the depth of wheel flange, the clearance in the lifting or latching devices, or an established nominal clearance for handling.

13.14.1.1. Clearances. The clearance in the upstream direction is usually from 1/4 to 3/8 in. Side clearance between the edge of the gate and the slot should allow for thermal expansion and contraction of the gate body, fabrication clearance in the lifting or latching mechanism, permissible deviation of centerline of wheels or rollers from centerline of track, and deflection of the seal, if mounted with sealing surface parallel to the pier. Accurate installation of the guides is accomplished by leaving blockouts in the structural concrete. Double-nutted anchor bolts are installed in the piers to allow for guide adjustment in two directions.

13.14.1.2. Adjustments. After the guide steel has been accurately aligned, it is grouted in place using nonshrink grout. Sills should be wide steel flanges set in a blockout. Accurate adjustment to line and slope is accomplished with anchor bolts through the bottom flange, with nuts top and bottom. This also prevents movement while the nonshrink grout is cast in the blockout. The bearing surface of the top flange of the sill should be corrosion-resistant steel or have a stainless steel plate welded to it.

13.14.2 Seals on High-Head Gates. The bottom rubber seal is normally a wedge seal that relies on the weight of the gate to provide the seal compression for sealing. For high-head installations (200 ft), pressure-actuated seals are used for the other sides of the gate. The pressure source is usually the head pressure of the reservoir. Designers should consult with the seal manufacturer for proper use of these seals. Observations of rubber seals indicate that the rubber has sometimes become extruded into the space between the clamp bar and the seal plate. To prevent this, brass-clad, or fluorocarbon-clad seals have been used. The fluorocarbon-clad seal has proven to be superior to the brass-clad because of its lower coefficient of friction (0.1) and greater flexibility and resiliency. The lower coefficient of friction reduces the load to hoisting equipment.

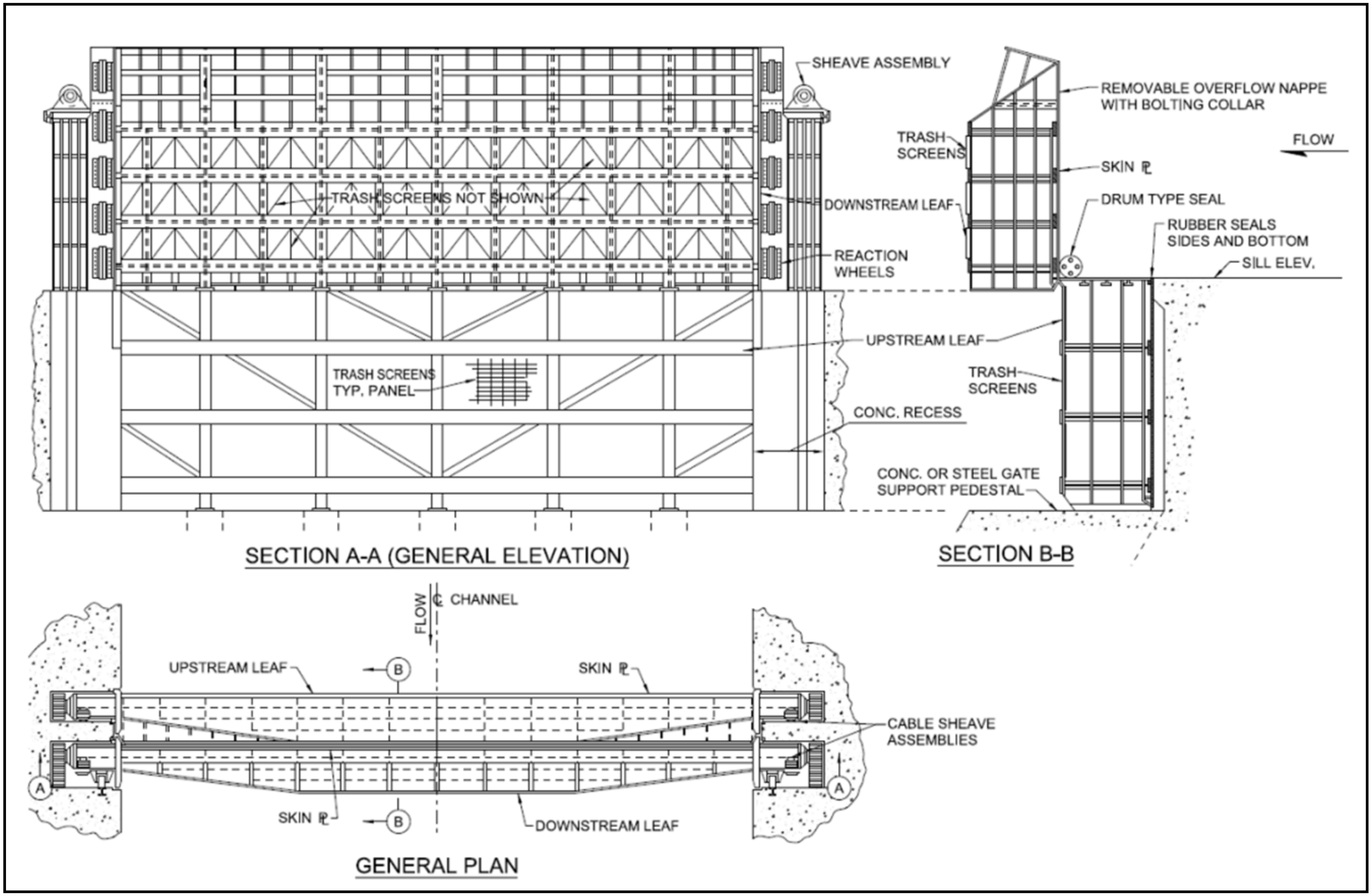


Plate 13.1. Vertical Lift Gates Double-Leaf General Plan and Elevation

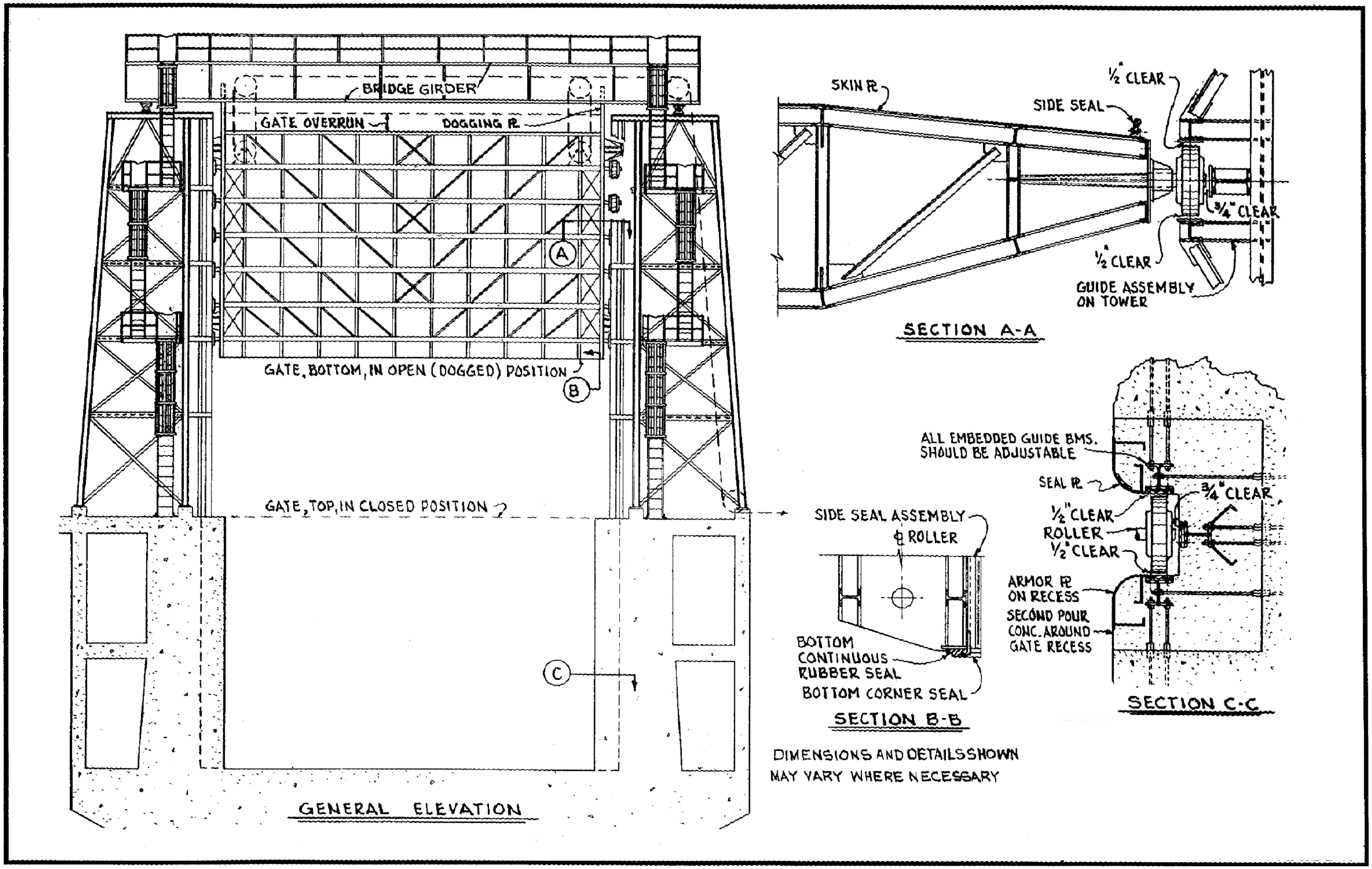


Plate 13.2. Vertical Lift Gates Single with Towers General Elevation and Typical Details

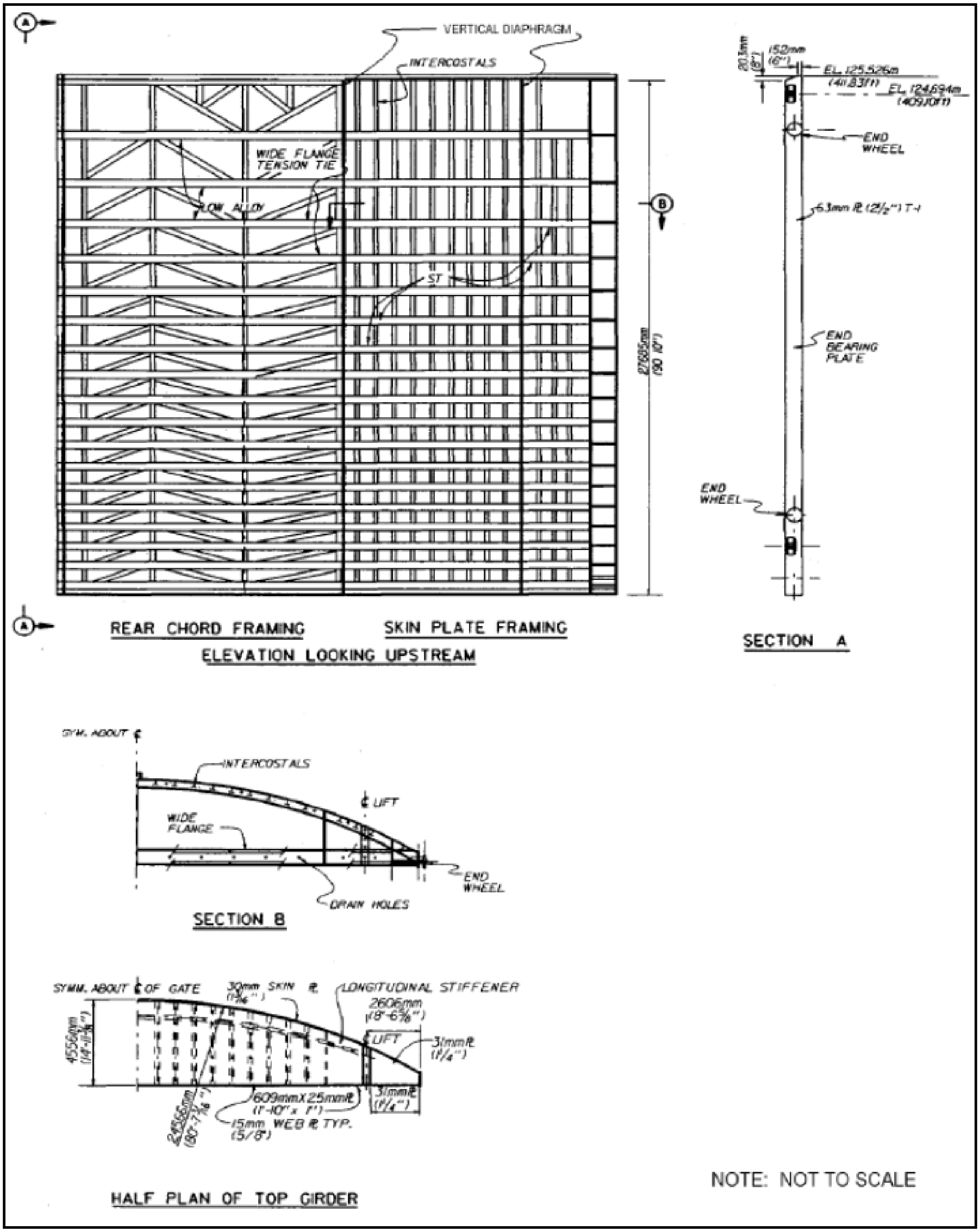


Plate 13.3. Ice Harbor Lock and Dam Navigation Lock (Old Downstream Lift Gate), Snake River, Oregon, Idaho, Washington

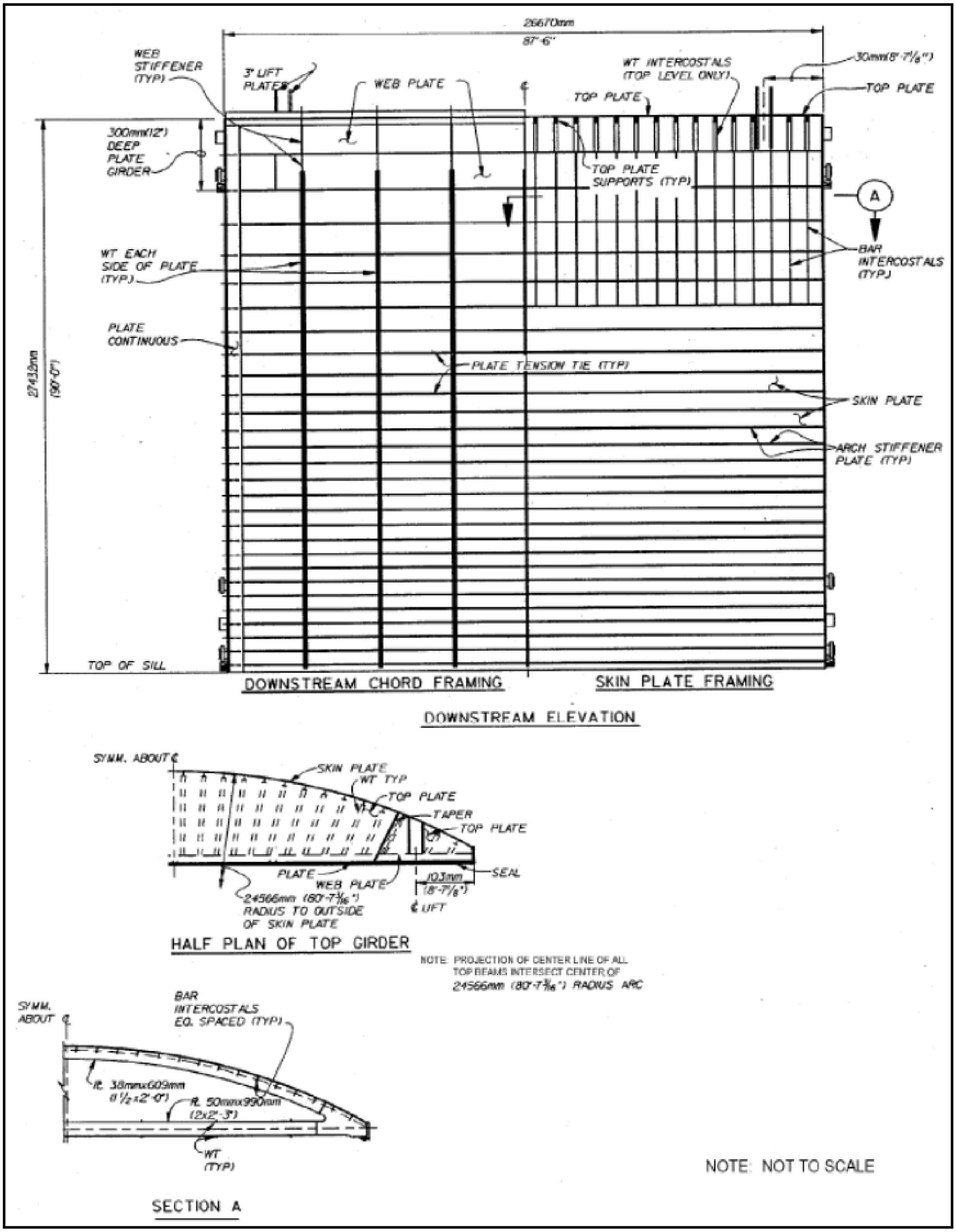


Plate 13.4. Ice Harbor Lock and Dam Navigation Lock Replacement Downstream Lift Gate, Snake River, Oregon, Idaho, Washington

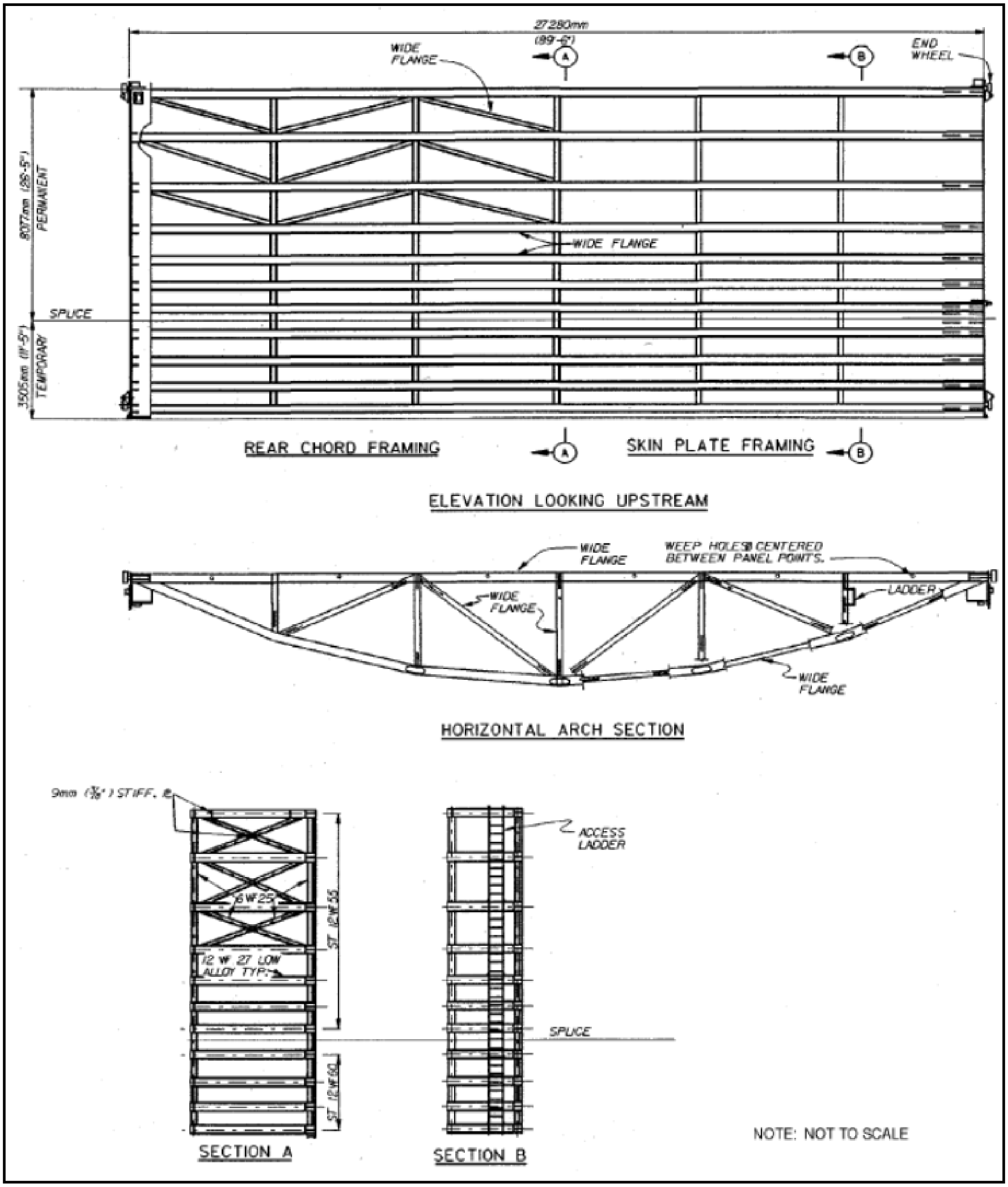


Plate 13.5. John Day Lock and Dam Navigation Lock Upstream Lift Gate, Columbia River, Oregon and Washington

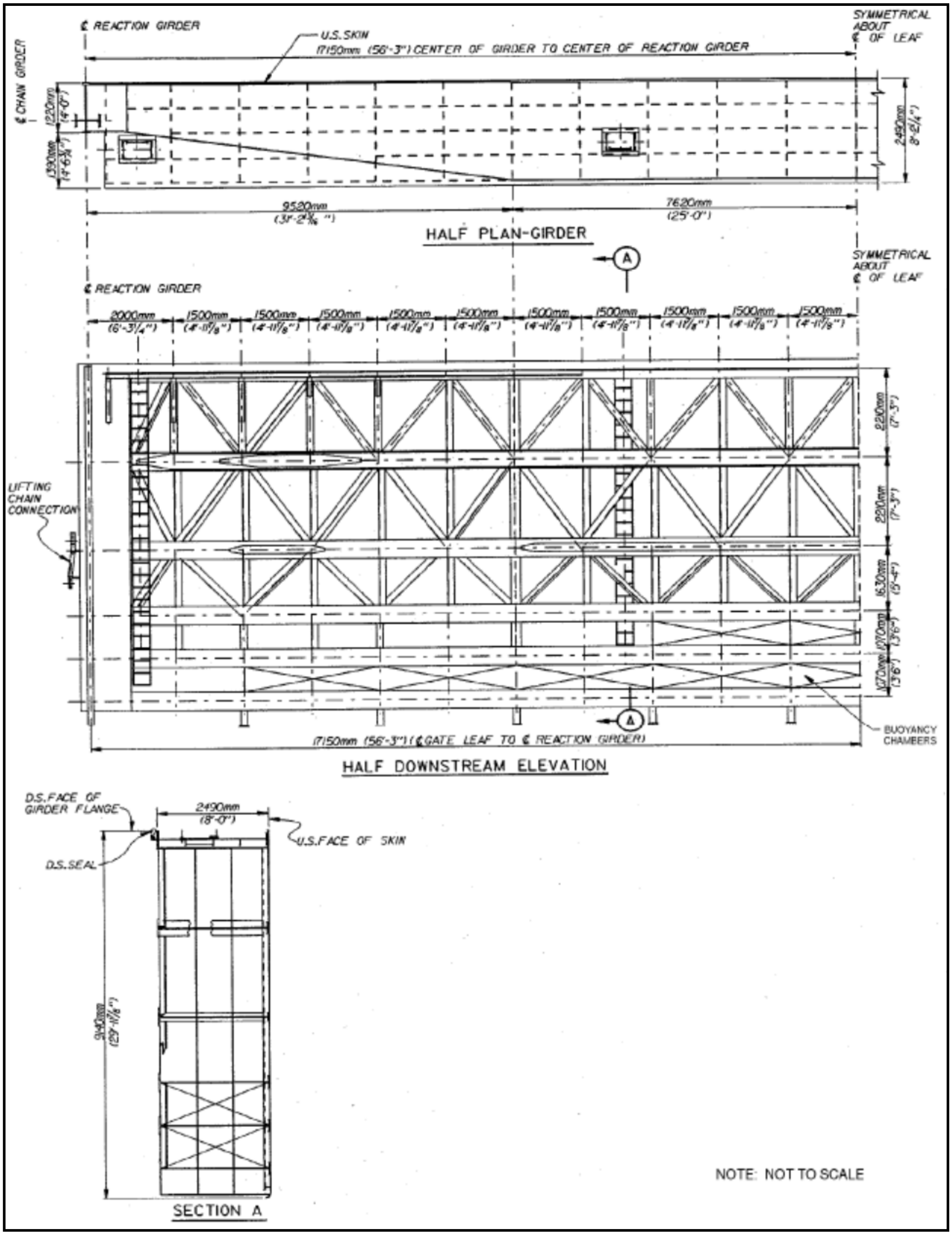


Plate 13.6. Old Lift Gate Upstream Leaf, Upper Mississippi River Basin, Mississippi River, Granite City, IL, Lock No. 27, Main Lock

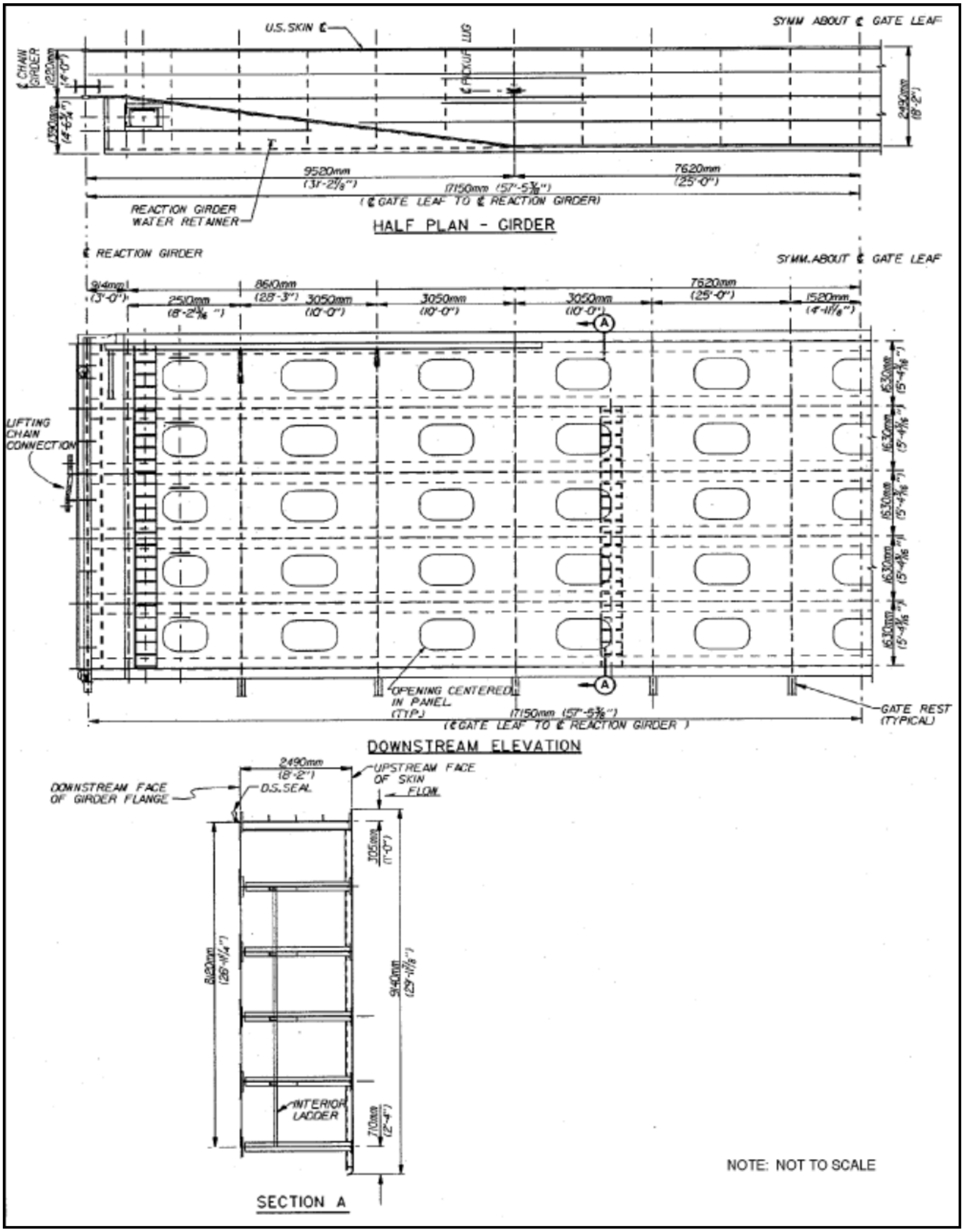


Plate 13.7. Replacement Lift Gate Upstream Gate, Upper Mississippi River Basin, Mississippi River, Granite City, IL, Lock No. 27, Main Lock
(Note improved fatigue detailing of the replacement gate)

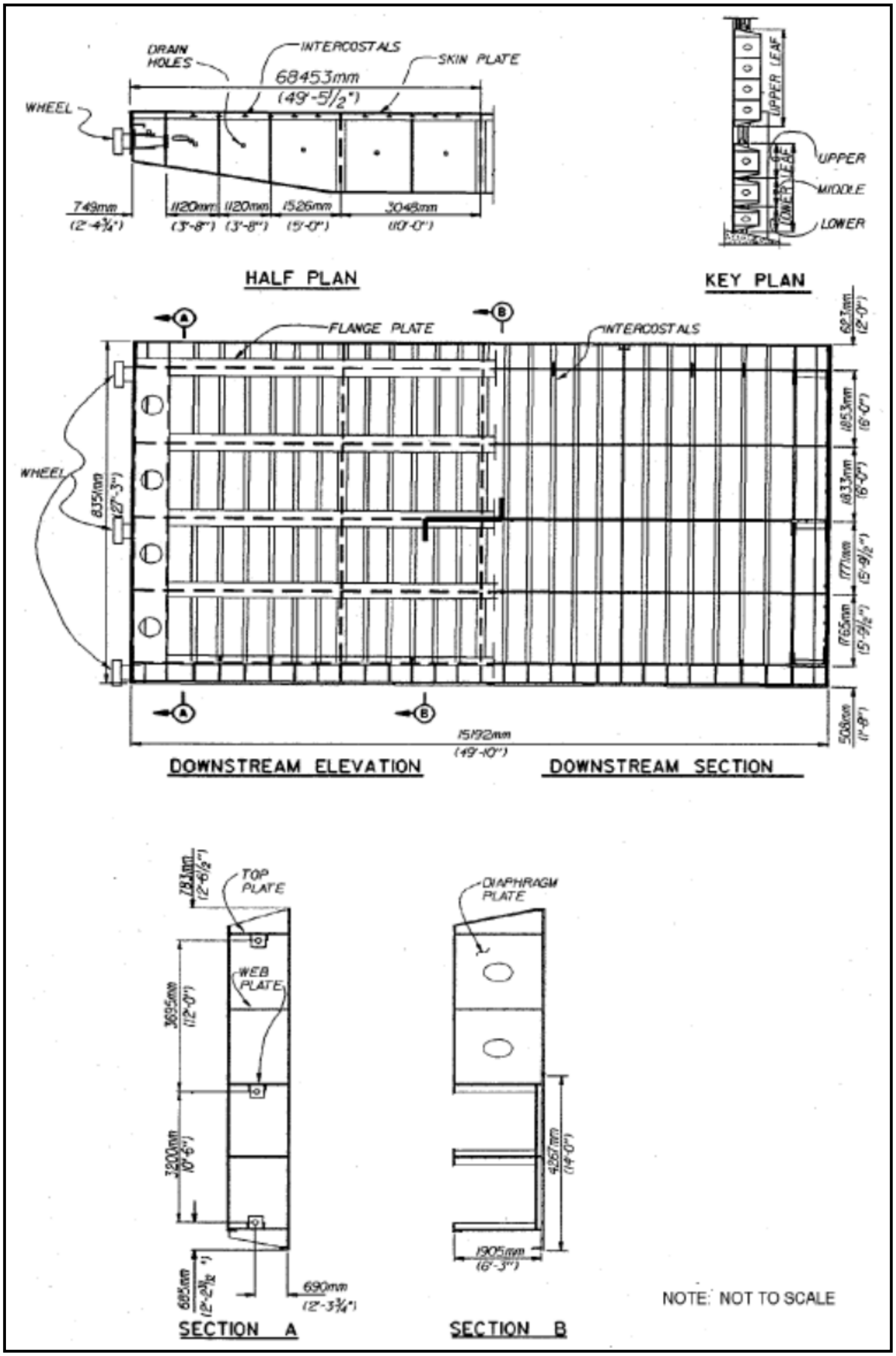


Plate 13.8. Spillway Crest Gate Upper Leaf, McNary Lock and Dam, Columbia River, Oregon, Idaho, and Washington

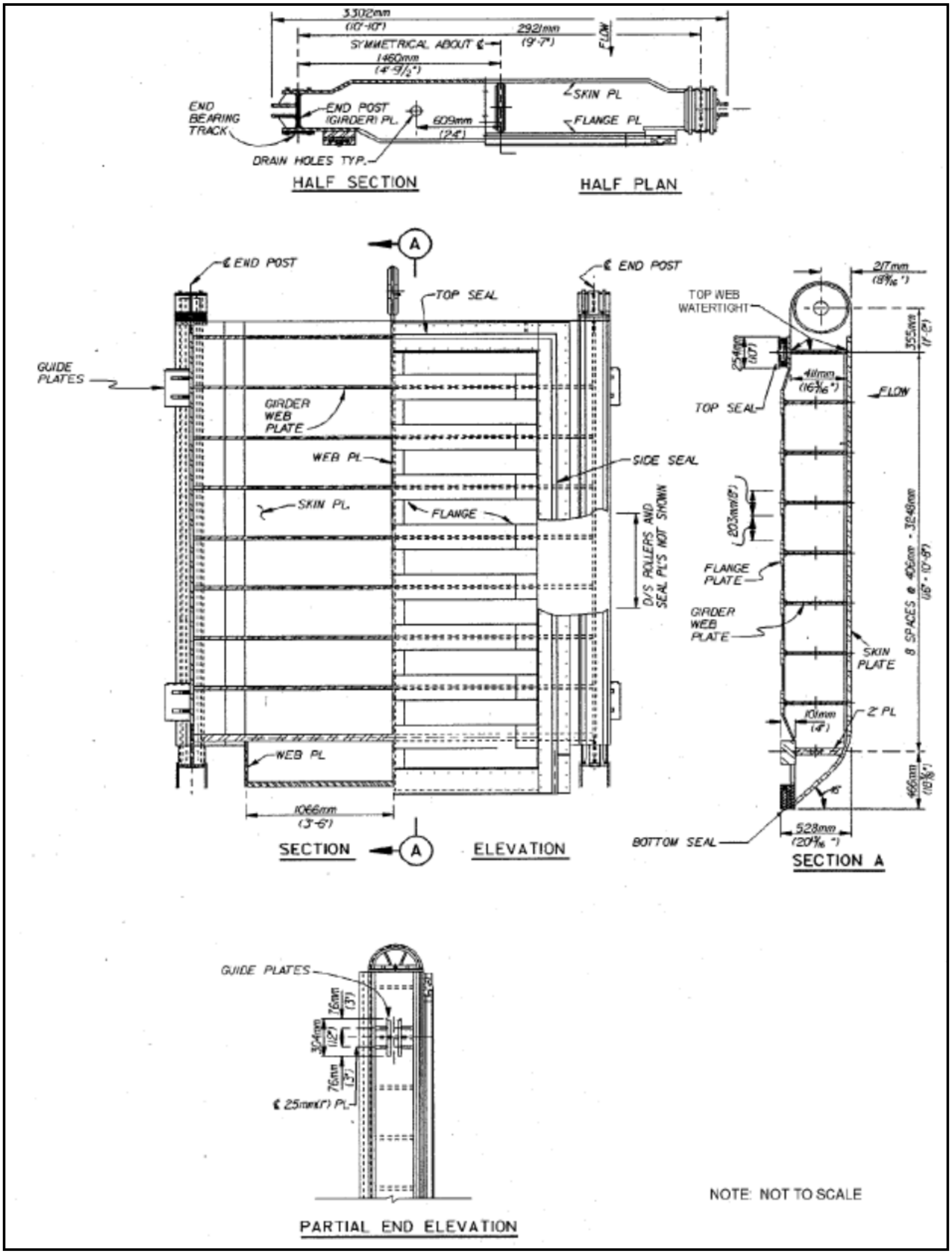


Plate 13.9. Emergency Closure Gates, Mud Mountain Dam, White River, Washington

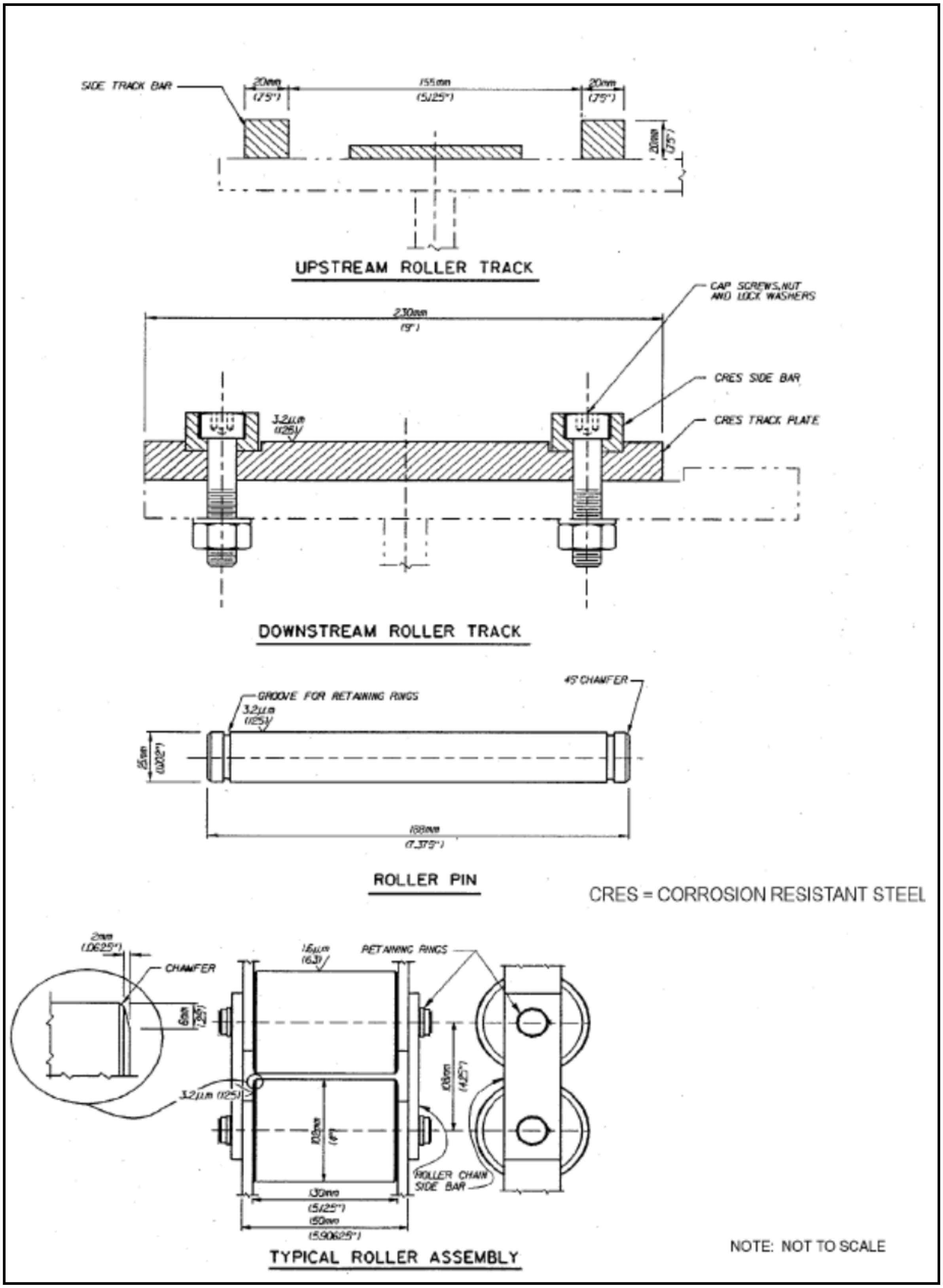


Plate 13.10. Tractor Chain and Roller Track Details

Chapter 14
Closure Gates for Levee Systems

14.1 Introduction.

14.1.1 General. Closure structures are often required at openings in levee systems for traffic or water to pass through below the top of the levee or floodwall. While the need for closures should be avoided when possible, openings might be necessary for normal road, railroad, or pedestrian traffic, navigation, drainage, or tidal flow.

14.1.2 Scope. This chapter provides guidance for selection and design of levee system closure gates for roads, railroads, and pedestrians. It discusses advantages of various gate configurations, provides some typical details, and identifies design requirements. See EM 1110-2-2502 for design of the foundation that supports the closure gates.

14.1.3 Drainage and Tidal Flow. Flow of water through levee systems may be through a pipe with a gatewell or through a channel connecting to a gate structure. Unless allowance for vessels is required, sluice gate, slide gates, and flap gates are usually used to control flow through the levee system. Other gate types may also be used. See EM 1110-2-2902 for more information on pipes and gatewells. See the applicable chapters for design of gates covered in this manual.

14.1.4 Openings for Navigation. In many coastal levee systems, openings are required for vessels to pass through the levees. To close the levee system during floods, sector gates, lift gates, stoplogs or bulkheads are commonly used. Because of their quick, reliable operation, lack of overhead obstructions, and ability to be operated under head, sector gates are the most common. For design of these gates, refer to the applicable chapters in this manual. Also see the general design requirements in this chapter for additional considerations.

14.2 Design.

14.2.1 Design Coordination. The design of closure structures for openings in levees and floodwalls must address operation, function, maintenance, aesthetics, safety, security, construction, and economics. Since ownership is usually transferred to a local sponsor, maintenance and operation will be the responsibility of that sponsor. The design team must coordinate effectively with the sponsor to ensure that the completed project is compatible with the sponsor's needs and capabilities.

14.2.2 General Design Requirements.

14.2.2.1. Operation. It is necessary to complete closure of the gates before floodwaters rise above the gate sill. Failure to close the gates in time can result in flooding; however, closing the gates too early can unnecessarily inconvenience traffic that normally passes through. To select a gate size and type that can be closed in sufficient time, the designer must be aware of the available water level forecasting for the site, and the equipment and capabilities of the sponsor's operations staff.

14.2.2.2. Function.

14.2.2.2.1 The normal function of these gates is to remain open to enable traffic to pass through the opening. Sight distance and clearance requirements for roadways and railroads are primary functional considerations that must be incorporated into the design. Opening widths for roadways must comply with AASHTO and local requirements, and opening widths for railways must be coordinated with the owner.

14.2.2.2.2 The width of closure openings should not be less than 30 ft for roadways with two lanes of traffic. The minimum vertical clearance between the crown of roadways and fixed overhead components of closures must be coordinated with local highway authorities to ensure that vertical clearance needs are met, but they should not be less than 14 ft. The normal minimum opening for railroads is approximately 20 ft for each set of tracks involved in the closure. Clearances should be coordinated with and approved by the facility owner.

14.2.2.3. Maintenance. Proper maintenance of closure structures is essential to continued satisfactory performance. The required maintenance provisions, including inspection requirements, must be included in the agreement with the local sponsor. Designs should use materials, systems, and features that are economically feasible and require minimal maintenance.

14.2.2.4. Aesthetics. Aesthetics is usually not a major consideration for flood closures. However, EM 1110-2-38 provides guidance for aligning flood risk management channels, landscaping along channels, and the aesthetic treatment of channel linings. This EM might provide some insight on aesthetic treatments, where necessary. For a few levee system projects, an open view of the waterway has been an aesthetic requirement.

14.2.2.5. Safety. The design of closures must include safety provisions for the public and the operations personnel. Local sponsors are responsible for the safe operation of closure structures. Therefore, designers must coordinate with sponsors so the appropriate design provisions are incorporated to ensure safe operation. General safety provisions include providing railings on the top of the gates and adjacent walls for public protection and ladders for access by operations personnel. Other safety features could include warning signs and barriers that prevent access by unauthorized persons. Compliance with appropriate traffic safety standards is also necessary.

14.2.2.6. Security. The design of closure structures must include security provisions that prevent vandalism and impairment of operating capability. Locked storage facilities, which are inaccessible to the public, should be provided for the storage of stoplogs, removable posts, and other unsecured parts of closure structures. There should be latching devices that hold gates in the open and closed positions, and these should be provided with adequate locks.

14.2.2.7. Construction. Construction of closure gate sills can interfere with normal traffic and local businesses and residents. This might have some effect on selecting the type or size of gate. Traffic interruption issues must be coordinated with the sponsor and the users. This is especially important for railroad closures. Transportation restrictions might be another key construction issue for larger gates.

14.2.2.8. Seals. Gates are generally located on the flood side of the supporting structure so that floodwaters force the gate closed. For most gates, rubber J-seals form a continuous watertight seal between the gates and supporting walls and sill of the opening. The most difficult sealing area is along the bottom of the gate, where there might be railroad tracks or sloping roadways. In some cases, it might be necessary to provide a permanent or retractable bottom sill to accommodate uneven sill surfaces.

14.3 Selection of Closure Types.

14.3.1 Stoplogs.

14.3.1.1. General. Stoplog closures usually consist of one or more sets of horizontal aluminum or steel beams stacked vertically to close the opening. Aluminum stoplogs weigh less than steel stoplogs of the same size, but have a higher strength to weight ratio. For narrow openings, one set of beams or logs may span between support slots constructed at the edge of openings. For wider openings, intermediate, removable support braces are required (Figure 14.1).

14.3.1.2. Design. Stoplog closures may be designed in-house, or proprietary systems may be acquired using performance specifications. In either case, the system must be designed according to the requirements of this manual.

14.3.1.3. Installation. Proprietary stoplogs can be acquired with integrated seals between each log to limit seepage. In the case where stoplogs are not fitted with seals, other means like plastic sheeting or sandbags can be used to reduce leakage through the stoplog closure. Secure storage facilities must be provided for the stoplogs, removable posts, and accessories. Stoplogs should be secured (held down and tight against any seals) in the installed condition.

14.3.1.4. Stoplog Advantages. Advantages of stoplog closures are: fabrication methods are simple and economical; initial cost is usually less than for other gated closures; and easy stoplog placement for narrow and low openings.

14.3.1.5. Stoplog Disadvantages. Advantages of stoplog closures are: a storage building is required to prevent vandalism or theft; intermediate support posts might be required for wide openings; larger stoplogs require special lifting equipment for installation; installation time is longer than for other gated closures, due to mobilization of personnel and equipment for installation, number of individual pieces to install, and time to allow cleaning of the post sockets, where used, during installation; and because of the longer installation time, accurate long-range weather forecasting is needed to provide that time.

14.3.2 Swing Gates. Swing gates are composed of two or more horizontal girders, vertical intercostals, vertical end diaphragms, a skin plate, and diagonal braces. Swing gates are supported on one side by top and bottom hinges attached to a support structure (Figure 14.2). In most cases, swing-gate closures consist of a single gate leaf for openings up to about 40 ft. However, double-leaf gates are used for wide openings. Double-leaf gates must be stabilized by a removable center post or diagonal tieback linkages (Figure 14.3). When using a linkage rod, a support jack is provided beneath the gate to withstand the vertical component of load from the rod. Provisions should be made for the use of winches or motor vehicles to accomplish closure during strong winds.

14.3.2.1. Swing Gate Advantages. Advantages of swing gate closures are: no special skills or equipment are required for operation except when removable intermediate support posts are used with double-leaf gates; and can be closed quickly except when removable intermediate support posts are used with double-leaf gates.

14.3.2.2. Swing Gate Disadvantages. Disadvantages of swing gate closures are: requires right-of-way area for operating; hinges require complex shop fabrication with machine work; a storage facility is required when removable intermediate support posts are used with double-leaf gates; and they can be difficult to operate during high winds.

14.3.3 Miter Gates. Miter gates consist of two leaves that form a three-hinged arch when the gates are in the closed position. Each gate leaf is composed of horizontal girders, vertical intercostals, vertical end diaphragms, a skin plate, and adjustable diagonal tension rods. The gate leaves are attached to support piers by top and bottom hinges (Figure 14.4). The diagonal tensioning rods are required to prevent twisting of the gate leaves due to their dead load and must be properly tensioned after the gates are installed so that the gates hang plumb and miter properly.

14.3.3.1. Design Considerations.

14.3.3.1.1 Components. For miter gates with two horizontal girders, the three-hinged arch reactions are resisted by the top and bottom hinges at the supports and spot bearing blocks at the miter ends of the horizontal girders. The magnitude of loading on large miter gates requires the use of three or more horizontal girders, quoin posts with bearings attached to the support piers, and continuous miter posts at the miter ends of the gates to accommodate the large forces. Also, hemispherical pintles and top linkages, similar to navigation lock gates, may be required instead of hinges.

14.3.3.1.2 Adjustments. EM 1110-2-2610 includes provisions for the design of pintles and top linkages. Hinges and miter blocks or bearing posts must be adjustable to accommodate construction tolerances and allow the gates to miter properly. Support structures for miter gates are usually more difficult to design and cost more than support structures for other types of gates.

14.3.3.1.3 Other Considerations. The supporting structures and their foundations must be designed to minimize the deflections at the gate hinges or quoin posts so that the gates will function as designed. Latches are provided to secure the gates in the stored and closed position. Seal, hinge, and latch details for miter gates are similar to those used for swing gates. Closure provisions should include the use of winches or motor vehicles to accomplish closure in high winds.

14.3.3.2. Miter Gate Advantages. Advantages of miter gate closures include: they are suitable for large openings; the closure can be made quickly without the use of skilled personnel; a storage building is not required; a center support is not required; and the gate weighs less than other types of gates designed for large openings.

14.3.3.3. Miter Gate Disadvantages. Disadvantages of miter gate closures include: hinges require complex shop fabrication with machine work; they require rights-of-way area for operating; the support structure is larger and more expensive than for other closure gate types; and they can be difficult to operate in high winds.

14.3.4 Rolling Gates – General. Rolling gates are constructed similarly to swing gates. The gates are supported by wheels that roll on tracks embedded in the sill across the closure opening and storage area. The gates are sometimes operated by a cable attached to a motorized winch. However, the gate can also be connected directly to a truck that pulls the gate open or closed. Gates along fast rising streams may be designed to be opened or closed from the protected side of the floodwall. Latches should be provided to secure the gates in the stored and closed positions.

14.3.4.1. Rolling Gate Advantages. Advantages of rolling gate closures include: they are adaptable to wide openings; closure can be made quickly without the use of skilled personnel; a storage building is not required; and storage space requirements are small.

14.3.4.2. Rolling Gate Disadvantages. Disadvantages of rolling gate closures include: jacks are required to lift the wheel assemblies from the tracks when the gate is in the closed position (unless wheel assemblies are designed to accommodate the lateral bottom girder deflection); and a level storage area is required immediately adjacent to the closure opening.

14.3.4.3. Rolling Gates – Two Lines of Wheels. Rolling gates can be stabilized with two lines of wheels. Figure 14.5 shows this type of gate. The wheels support and stabilize the gate against overturning. The wheels are usually V-grooved castings and roll on tracks that are usually inverted angles with embedded anchorages. The depth of the bottom girder is usually governed by the required transverse spacing between the supporting wheels rather than the hydrostatic load. A girder depth of 30 to 36 in. is normally required to accommodate the spacing between the two lines of wheels to provide stability of the gate during opening and closing operations, but this depends on the height of the gate and wind speed.

14.3.4.4. Rolling Gates – Single Line of Wheels. These gates are usually composed of a trussed steel frame covered with skin plate or bridge planks. The gates are supported at the bottom by a single line of wheels and are stabilized laterally by an extended top girder supported by trolleys attached to the top of the floodwall. This extended girder makes this type of gate practical only for openings up to about 30 ft. Girder depths are usually governed by the hydrostatic loading on the gate.

14.3.4.5. Rolling Gates – L-Frame. These gates are usually composed of a series of L-shaped structural steel frames interconnected by horizontal and diagonal members. The gates are supported at the bottom by two lines of wheels (Figure 14.6). Hooks attached to the heel of each of the L-frames engage anchorages embedded in the concrete sill structure to stabilize the gate against hydrostatic loadings. This differs from other rolling gates, which span horizontally between supports. These gates can be fabricated in sections to simplify handling and storage and requires a wide sill to accommodate the installation of tracks and hook anchorages.

14.3.4.6. Trolley Gates. Trolley gates are fabricated similar to rolling gates. Trolley gates are suspended from trolleys running on an overhead rail and a beam supported by the floodwall (Figure 14.7). The gates are opened and closed by a winch arrangement similar to that used for rolling gates. These gates are suitable for railroad closures because required vertical clearances for railroads are fixed. The gates may be rendered inoperative due to overhead support members being damaged. A guide member at the base of the gate may be required to support the gate against wind loads during opening and closing operations.

14.4 Structural Design. Design requirements using LRFD are provided in Chapters 3 and 4. This section provides information specific to design of closures for strength and serviceability.

14.4.1 Loads. General loads and loading combinations are described in Chapter 4. The following loads must be considered in the design of closure structures.

14.4.1.1. D, Dead load. Dead load is defined in Chapter 4. A load factor of 1.2 is used when dead load adds to load effects and 0.9 when it reduces load effects.

14.4.1.2. G, Gravity. Ice and mud are determined on a site-specific basis, but generally can be neglected unless the Engineer has reason to believe they will exist.

14.4.1.3. Hs, Hydrostatic Loads. Hydrostatic loads consist of hydrostatic pressure on the closure. For strength design of closure gates, for Hs as a principal load, Hs_{pr}, Hs is the maximum hydrostatic loading from differential head. Depending on the return period of the load it may be usual, unusual, or extreme. For companion hydrostatic loads, Hs_c is the normal operating condition, with a return period of 10 years as defined in paragraph 3.3.3.3. Hydrostatic loading is normally not present for closure gates when other loads are at their maximum.

14.4.1.4. Hw, Wave Load. See Chapter 4 for determining wave loads.

14.4.1.4.1 Riverine Closure Gates. Riverine closure gates are normally dry. Wave loads are applied as companion loads to flood loads, Hw_c, and computed using average annual maximum wind velocities. In many cases fetch length and direction and shallow water depths make wave loads insignificant in riverine situations.

14.4.1.4.2 Coastal Closure Gates. For coastal situations with correlation between surge and wave, annual exceedance (return period) of combined loads must be computed using a coupled analysis. The surge level and wave force computed as a function of the annual exceedance probability will be provided by the hydraulic engineer.

14.4.1.5. Q, Operating Machinery. Under normal operating conditions, operating loads are treated as reactions to all opposing forces including D, G, and friction, F. When applied as a principal load Q_{pr} is the maximum machinery load that can be applied to the closure. Consult with the mechanical engineer for the project to determine this load.

14.4.1.6. IM, Ice or Debris Impact. The ice and debris impact load are specified to account for the impact of debris (timber, ice, and other foreign objects). For closure gates, which are normally dry, this load is a companion action load and should be computed following the principles in Chapter 3.

14.4.1.7. BI, Barge Impact. Barge impact is applied in locations where aberrant BI are possible. This load is usually not applied to riverine levee systems but may be if project conditions warrant it. It is used in coastal areas where barges and other vessels may become loose in high winds that accompany high water events and can impact the structure. See section 4.2.6.3 and EM 1110-2-3402 for more information on determining design aberrant barge impact loads.

14.4.1.8. F, Friction. Frictional forces on hinges, casters, trolleys, and other moving parts during gate opening and closing.

14.4.1.9. EQ, Earthquake. Earthquake is not considered for the design of closure gates but should be considered for support columns and walls.

14.4.1.10. W, Wind, According to ASCE 7-22. See Chapter 4. For riverine structures, design wind loads are unlikely during closure operations.

14.4.2 Load Combinations. Closure gates will be designed for the strength limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in Chapter 4. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. The serviceability limit state is addressed in paragraph 14.4.4.7. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

14.4.2.1. Load Combination 1: Flood Plus Wave:

14.4.2.1.1 Inland. Waves are created by wind events independent of water level.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs_{pr} + 1.0\ Hw_c \qquad \text{(Equation 14.1)}$$

14.4.2.1.2 Coastal. Hydrostatic, wave, and wind loads are correlated. Peak design wind gusts are unlikely to coincide with peak wave. A companion wind load is applied to the structure above the top of the wave pressure diagram.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ (Hs + Hw)_{pr} + 0.5\ W$ (Equation 14.2)

14.4.2.2. Load Combination 2: Flood Plus Impact: Maximum hydrostatic pressure, Hs_{pr}, with companion debris impact or barge impact associated with the flood event.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs_{pr} + 1.0\ (IM_c \text{ or } BI_c)$ (Equation 14.3)

14.4.2.3. Load Combination 3: Maximum Impact.

14.4.2.3.1 Loads consist of ice, debris, or barge impact, plus companion hydrostatic loading, dead load, and gravity loads. Impact loads will be correlated with water levels and the companion hydrostatic loading so high water levels are used.

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ (IX_X \text{ or } IM_X \text{ or } BI_x \text{ or } Hw_X) + 1.0\ Hs_c$ (Equation 14.4)

14.4.2.3.2 Unless site specific data is available, the average annual return periods of the extreme ice or debris are not known and $\gamma_{pr} = 1.3$. For extreme barge impact load $\gamma_{pr} = 1.3$.

14.4.2.4. Load Combination 4: Closure in Normal Storage Position. Closure gate subjected to design wind pressure according to ASCE 7-22:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.0\ W$ (Equation 14.5)

14.4.2.5. Load Combination 5: Closure Gate Opening or Closing.

14.4.2.5.1 Closure gate subjected to dead and maximum operating load with gate jammed, Q_{pr}:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Q_{pr}$ (Equation 14.6)

14.4.2.5.2 The operating load should be considered an extreme load (Q_X) with unknown return period ($\gamma_{pr} = 1.3$).

14.4.2.6. Load Combination 6: Closure Gate Opening or Closing With Wind. Closure gate subjected to dead load, friction forces, and wind load. Operating load is treated as a reaction:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + 1.0\ W + 1.4\ F$ (Equation 14.7)

14.4.2.7. Load Combination 7: Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load, and gravity loads.

14.4.2.7.1 For standard and site-specific OBE ground motion analysis:

$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c$ (Equation 14.8)

14.4.2.7.2 For standard MDE ground motion analysis:

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.25\, EQ + 1.0\, Hs_c \qquad \text{(Equation 14.9)}$$

14.4.2.7.3 For site specific MDE and MCE ground motion analysis:

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.0\, EQ + 1.0\, Hs_c \qquad \text{(Equation 14.10)}$$

14.4.3 Design of Stoplog Closures. Some considerations for design of stoplog closures are as follows:

14.4.3.1. Member Thickness. Stoplogs systems must be stored in a secure location and usually are protected from the weather. As such, there are no minimum member thickness requirements beyond those required to meet strength, stability, and stiffness (serviceability) requirements. Stoplog systems stored outdoors must be designed with consideration for corrosion and steel members should be a minimum of 5/16 in. thick.

14.4.3.2. Fracture Critical Members. Bracing members may have components that are fracture critical. Anchorage components in bracing may also be fracture critical. Fracture critical members must be identified and be designed according to Chapter 5.

14.4.3.3. Anchorage. Anchorage components that are embedded in the foundation are subject to weather and potentially to deicing chemicals and other corrosive substances. Pavement around the anchorages may be damaged or degraded by traffic and weathering. The design of the anchorage must account for the environment. Factors to consider in the design are material selection, member sizing, detailing, corrosion protection, and using configurations that reduce tensile forces.

14.4.4 Design Assumptions and Considerations. The following paragraphs include a brief description of design assumptions and design considerations for rolling gates, miter gates, and swing gates.

14.4.4.1. Skin Plate. Skin plates must be sized such that the maximum calculated stress is less than the yield limit state of $\alpha\varphi Fy$. Skin plates will be designed for hydrostatic loading only. More than one thickness of plate may be desirable for taller gates. The minimum plate thickness should be 5/16 in. Chapter 9 provides additional guidance on intercostal design.

14.4.4.2. Intercostals. Intercostals will be sized so the maximum calculated moment is less than the nominal bending strength of $\alpha\phi_b M_n$. Intercostals are designed using Equation 14.1. They may be flat bars or plates, tee sections, or angle sections. Chapter 9 provides additional guidance on intercostal design.

14.4.4.3. Girders. Horizontal girders for swing gates that support components of the diagonal loads are designed for flexure due to hydrostatic loading plus flexure and axial load induced by dead load in the diagonals. Chapter 9 provides additional guidance on girder design for miter gates.

14.4.4.4. Diagonals. Diagonals may be required to resist gate torsion due to dead load and operation using Equation 14.3. Chapter 9 provides additional guidance on diagonal design.

14.4.4.5. Vertical Diaphragms. Vertical diaphragms for hinge gates are designed to resist diagonal loads as well as flexure loads. Vertical diaphragms for wheel gates need be designed to resist flexure loads only, except for those diaphragms that are in line with wheels or trolley hangars, which include axial and bending due to the forces from the wheels or trolley hangars. The minimum thickness of any diaphragm element should be 5/16 in.

14.4.4.6. Stabilizing Systems. These consist of hinges, wheels, trolleys, latching devices, closing links, gate tie-down assemblies, gate hooks, or other stabilizing systems. Components of the system are designed as individual units. The force applied to the units may be from hydrostatic, dead, operating, wind, or a combination of these loads. Components being used to stabilize the gate in the closed position with hydrostatic load will be designed using Equation 14.1. Other gate components are designed to resist dead, operating, or wind load as applicable.

14.4.4.7. Serviceability Requirements. Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, details for ease of maintenance, details for ease of operation, and ensuring the gate is not damaged in the latched open position) are chosen so that the closure functions properly throughout its service life.

14.5 <u>Gate Operating Equipment</u>. Gate operating equipment includes motorized vehicles, winches, latches, wire rope, hooks, sheaves, snatch blocks, and other appurtenances. Guidelines for operating equipment are provided in EM 1110-2-2610.

14.6 <u>Corrosion Protection</u>. See Chapter 7.

Figure 14.1. Stoplog Closure Structure with Center Post


Figure 14.2. Swing-Gate Closure Structure

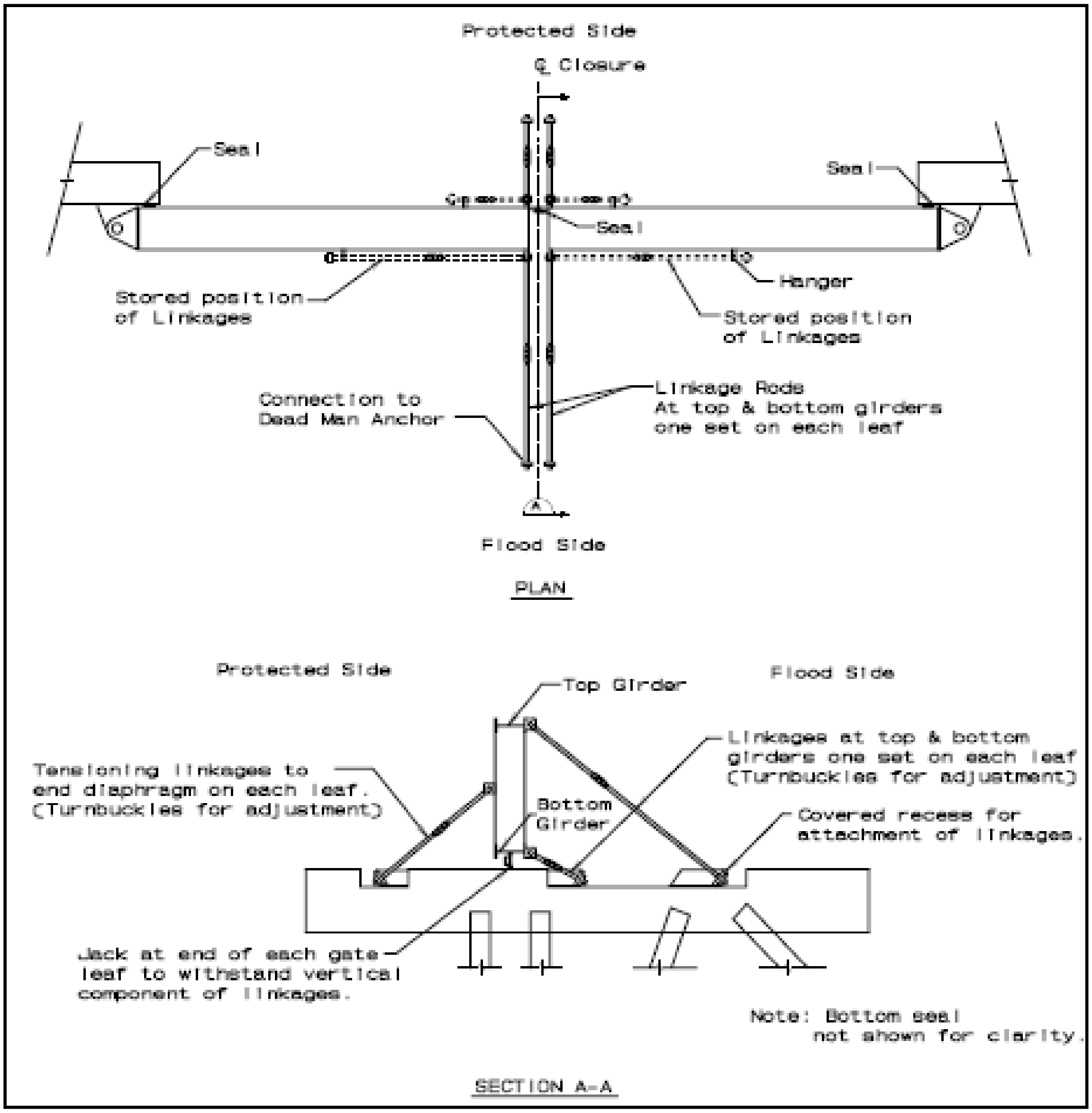


Figure 14.3. Tieback Linkage for Double-Leaf Swing Gate

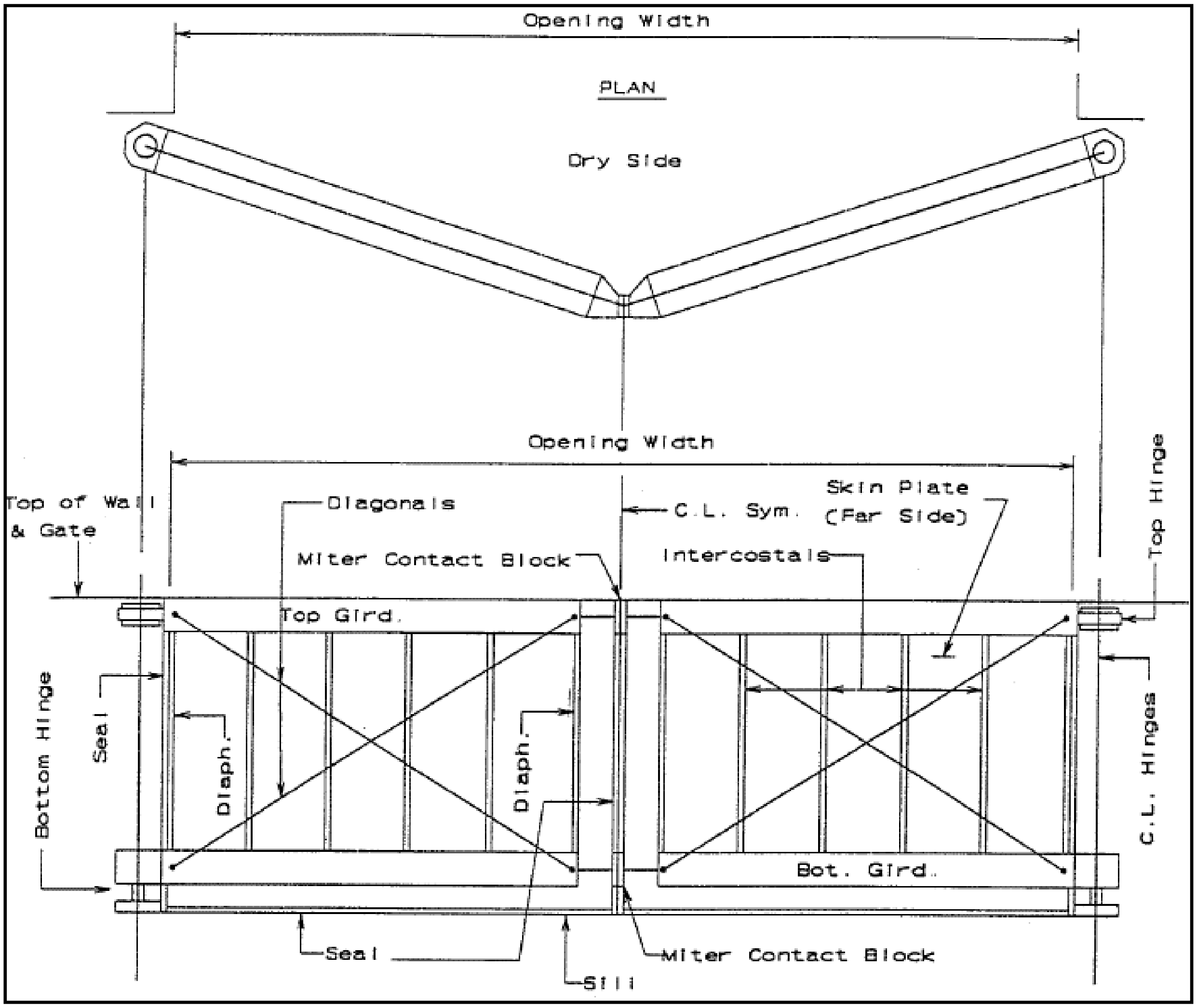


Figure 14.4. Miter Gate Closure Structure

Figure 14.5. Rolling Gate Closure Structure

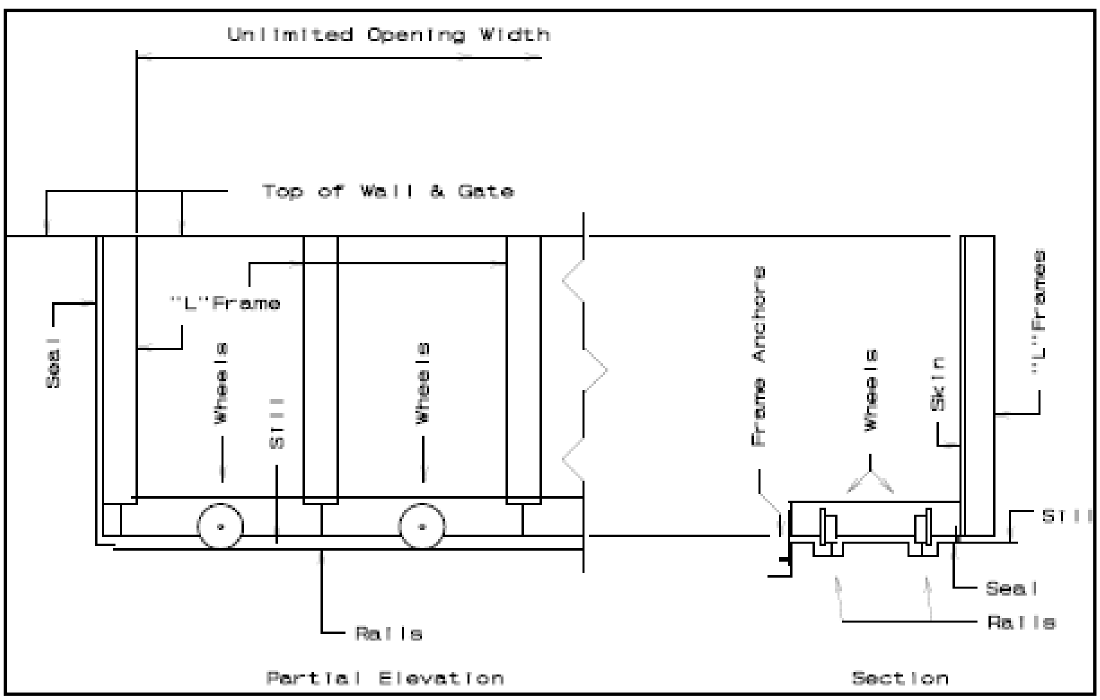


Figure 14.6. Rolling Gate Stabilized By L-Frame and Hooks

Figure 14.7. Trolley Gate Closure Structure

Chapter 15
Bulkheads, Stoplogs, and Lifting Beams

15.1 General. This appendix provides guidance for the design of bulkheads, stoplogs, and lifting beams. There is no specific number of stoplogs or bulkheads required for any project; that depends on the type of project, the number of gates, and the maintenance plan for the project. One or a few stoplogs or bulkheads might be used for maintenance in a large number of gate bays. If sizes are compatible, multiple sites can be serviced by one set of stoplogs and bulkheads. Chapter 2 includes examples of bulkheads and stoplogs. For the purposes of design, bulkheads and stoplogs are considered the same in this chapter.

15.2 Bulkhead Types.

15.2.1 One-Piece Bulkheads. For narrower openings, such as some spillway crests or hydropower discharges, a single full-height bulkhead is common. Typical framing consists of horizontal girders with a skin plate attached to girder flanges or stems of Tees. Girders can be made from rolled shapes or built up plates. Since a one-piece bulkhead is similar to a vertical lift gate, much of the information contained in Chapter 13 should be applicable to this type of bulkhead.

15.2.2 Stackable Unit Bulkheads. For wider openings, such as locks or between dam piers on navigation projects, bulkheads usually consist of stacked units because the weight of a one-piece bulkhead would make it difficult to place and remove. Stackable units, sometimes called stoplogs, are typically several feet high, consisting of two horizontal built-up girders or trusses with a skin plate between the trusses.

15.2.3 Poiree Dams.

15.2.3.1. General. Some older projects use poiree dams to permit maintenance dewatering. This type of bulkhead can be used for wider openings and shallower water depths. A poiree dam consists of removable frames that are installed at intervals across the opening and attached to the concrete base. To provide the damming surface, needles or panels are supported by the frames and form the damming surface. The frames are typically attached to anchor points embedded into the concrete floor.

15.2.3.2. FCM. Poiree dams contain several FCM including frame tension members and the embedded anchors. This configuration is highly unreliable because of the presence of FCM. The embedded anchorage is continually submerged and the above concrete portion of the anchorage is only visible after dewatering or by conducting diving operations. The below concrete portions of the anchorage can only be inspected by using NDT methods. Because of these issue, many projects have replaced this system with one that is more reliable.

15.2.4 Floating Bulkheads. Some dams were constructed without slots for maintenance bulkheads. For some dams, this does not pose a problem since reservoir levels are frequently below the spillway crest. However, where water levels were continuously above the sill, it was not possible to lift or remove the gate for maintenance. Floating bulkheads have been used successfully for some projects.

15.2.4.1. Construction. Floating bulkheads are similar to one-piece bulkheads, but with a skin plate on both faces to provide airtight chambers. The bulkhead is equipped with pipes and valves to permit controlled filling of selected chambers. This permits the bulkhead to be floated in an upright position and lowered to various depths. The unit is floated from the storage area into place on the face of the dam, then the dam gate can be opened slightly to lower water behind the bulkhead. The water pressure forces the bulkhead against the face of the dam. After use, the bulkhead can be floated into position at the next gate bay for additional maintenance.

15.2.4.2. Multiple Units. A variation of the floating bulkhead is to use several smaller individual units that can be connected to form the required height. This permits individual units to be small and light enough to be transported by truck, and then to be assembled in the water to form a single unit.

15.2.5 Emergency Bulkheads. Emergency bulkheads are used when flow needs to be controlled immediately. The bulkhead is installed to prevent excess release of water, which could cause flooding or loss of pool. The emergency bulkhead may need to be installed in rapidly flowing water resulting in significant vertical and horizontal forces on the gate while it is being positioned. Design of emergency bulkheads must consider hydrodynamic forces. The lower lip of the gate must be configured to minimize vertical forces. This is done in consultation with the hydraulic engineer. Typically, wheels or rollers are attached to the bulkheads to minimize friction forces during installation. If the friction and vertical forces are larger than the weight of the unit, it might be necessary to have equipment that can force the bulkhead down into position. Chapter 14 includes the design of emergency bulkheads configured as a lift gate.

15.3 Bulkhead Design. Bulkheads are typically designed similar to lift gates. See Chapter 14 for guidelines on construction and detailing of bulkheads.

15.3.1 Stoplogs. Stoplogs are designed as simple spans and can be constructed as trusses or built up beams. Trusses are tapered to fit into wall slots. Fixed, welded joints, eccentric connections and loading on chord members will create moments in truss members. These need to be considered in member design and joint detailing. When fully or partially fixed welded joints are used in a stoplog design that induce moments in the joints, the stoplog should be designed as a frame (fixed joints) instead of a truss (pinned joints). A fully or partially fixed joint is defined as a joint where full or partial continuity is provided at the flanges that results in rotational restraint in the joint, allowing transfer of bending moment forces. This restraint is typically provided by welding flange-to-flange, flange-to-web, or gusset plate-to-flange at adjoining members.

15.3.2 Water Loads. Water loads vary with the location of a stoplog in a stack. Therefore, stoplogs near the top of the stack have lower stresses than those near or at the bottom of the stack. Individual or groups of stoplogs can be designed to be placed at certain locations in the stack to provide some economy in fabrication. However, the preferred practice is to design one stoplog based on the location of highest loading so that the set of stoplogs can be placed at any location without concern for loading.

15.3.3 Boundary Conditions. Each stoplog is usually designed to act independently, assuming no horizontal load transfer between units. Since deflections should be similar for each unit, this is a reasonable assumption. However, the bottom stoplog rests on a concrete sill, possibly with embedded steel bearing plates, and friction forces between the sill and stoplog significantly change the boundary conditions compared to design assumptions. The effect from these boundary conditions are unknown but there have been no reported issues from this condition and it is recommended to ignore this effect for design.

15.3.4 Gravity Loads. Gravity loads on stoplogs are transferred vertically through the stack to the concrete sill. This transfer is accommodated by the use of bearing surfaces such as elastomeric blocks, attached to stoplog members. When a unit is being lifted, this causes vertical bending in the members. The vertical bending can be resisted by the skin plate and by vertical bracing or diaphragms between upper and lower trusses or beams.

15.3.5 Skin Plate. Bulkheads can be designed for use with the skin plate toward or away from the dewatered area. When attached to the compression side or watered side of the bulkhead, skin plate provides continuous vertical bracing along the compression chord and the lateral torsional buckling limit state is avoided. However, this causes the bulkhead to protrude into the dewatered space and sufficient gate clearance or working room must be provided. When the skin plate is attached to tension or the unwatered side of the bulkhead, workspace is maximized while minimizing the amount of water to remove and the amount of uplift on the dewatered floor. However, the skin plate becomes an FCM and all requirements associated with FCM must be implemented.

15.4 Lifting Equipment. Projects may not have dedicated lifting equipment at each project where bulkheads will be installed and crane rental or contracted crane services may be required. Other projects may have cranes dedicated to the project for bulkhead operations. Equipment may be shared between districts or regions.

15.4.1 Lifting Beams. Lifting beams are often required for positioning bulkheads, especially for stacking stoplogs. Stoplogs might be underwater as they are lowered into position and then removed. A lifting beam is used to connect from the crane to the unit. This beam includes mechanisms that can connect to and release from specific attachment points on each unit. The mechanisms can be activated from above the water, thus simplifying crane hook up and release.

15.4.2 Transportation. For bulkheads located on a project feature, with a permanent crane to place the bulkheads, the size and weight of each unit may not be an issue. Other bulkheads may need to be transported from their storage location to the point of use, and then put into place using non-dedicated equipment. In some cases, bulkheads must be transported over long distances. In these cases, the size and weight of each unit must be compatible with transportation options. Along major rivers, barge transportation can usually accommodate the largest units. If movement by road is necessary, length, width, and weight limits must be considered.

15.5 Seals. Seals are required to minimize leakage through the bulkheads and associated pumping of the dewatered area. J-bulb seals are typically used for single unit bulkheads. Elastomeric pads or blocks are typically used to seal between stoplogs and J-bulbs used at the end to seal against the slots.

15.6 Bulkhead Maintenance. Bulkheads are usually stored out of the water and are used infrequently. Thus, they are less subject to wear, damage, and corrosion than most other types of gates, and require less maintenance. Bulkheads require a good paint system to prevent long-term corrosion damage. Any moving parts might need periodic lubrication, or at a minimum inspections, to ensure that they are still functioning properly.

15.7 Life Safety. Bulkheads represent a life safety risk since maintenance workers occupy the dewatered area. A bulkhead failure would endanger the workers. The same inspection requirements applicable to other gates also apply to bulkheads. Many bulkhead components can be classified as FCM (e.g., truss tension chords on stackable units) and these members must be inspected according to ER 1110-2-8157.

15.8 Storage Areas. Some bulkheads are stored directly on other project features, on a lock wall, or in a slot in the upper portion of a dam pier. Others require storage at some distance from their point of use. There are few specific requirements for storage areas. They should be dry, not subject to high-velocity flows during flood periods, accessible for required transportation equipment, and secure. The smaller the bulkhead unit, the more subject it is to theft.

15.9 Loads and Load Combinations.

15.9.1 Design Requirements Using LRFD. Design requirements using LRFD are provided in Chapters 3 and 4. This section provides information specific to design of bulkheads. General guidance is provided. Bulkheads are typically used for temporary purposes and the load determination should account for conditions during use.

15.9.2 Loads. General loads and loading combinations are described in Chapter 4. The following loads will be considered in the design of bulkheads.

15.9.2.1. D, Dead Load. Dead load is defined in Chapter 4. A load factor of 1.2 is used when dead load adds to load effects and 0.9 when it reduces load effects.

15.9.2.2. G, Gravity. Ice and mud are determined on a site-specific basis except the minimum mud load of Chapter 4 is applied.

15.9.2.3. Hs, Hydrostatic Loads. Hydrostatic loads consist of differential hydrostatic pressure on the gate. For strength design: When the hydrostatic load is a principal load, Hs_{pr}, it is the hydrostatic loading from differential head with maximum possible effect. For companion hydrostatic loads, Hs_c is the normal operating condition, with a return period of 10 years as defined in paragraph 3.3.3.3.

15.9.2.4. Hw, Wave Load. See Chapter 4 for determining wave loads. When combined with maximum hydrostatic loads, wave loads are applied as companion loads and determined according to Chapter 3. When applied as principal loads, wave loads are extreme loads. For stoplogs or bulkheads used for coastal levee systems, wave loads are correlated with high water levels.

15.9.2.5. IM, Impact Load. The impact load is specified to account for impact of debris (timber, ice, and other foreign objects). When combined with maximum hydrostatic loads, impact loads are applied as companion loads and determined according to Chapter 3. When applied as principal loads, impact loads are extreme loads. For stoplogs or bulkheads used for levee systems, impact loads may be correlated with high water levels that are more likely to carry debris.

15.9.2.6. BI, Barge Impact. Impact from vessel or barges operating near the bulkheads, or from aberrant barges bulkheads used for coastal flood risk management projects. When combined with maximum hydrostatic loads, barge impact loads are applied as companion loads and determined according to Chapter 3. When applied as principal loads, impact loads are extreme loads. For stoplogs or bulkheads used for coastal levee systems, barge impact loads may be correlated with high water levels. See section 4.2.6.3 for more information on determining barge impact loads. EM 1110-2-3402 has guidance for calculating barge impact loads.

15.9.2.7. Q, Operating Load. Operating loads are caused by friction, suction, or stickage that prevents the bulkhead from being removed. Q_{pr} is the maximum lifting load of the machinery or crane assuming the bulkhead is jammed in the slots. The maximum lifting load is the maximum upward load that can be applied. The maximum lifting load is applied by the machinery used in lifting and should be controlled by use of a load limiting device. Where a limiting device is not provided, the maximum lifting load should be specified and monitored during operations. (Some cranes may not have limiting devices). Include the specified lifting load in the operations manual.

15.9.2.8. EQ, Earthquake. Bulkheads are typically used for temporary purposes which reduces the probability of earthquake while they are in place under load. Earthquake loading can be neglected for bulkheads under design loading less than an average of 3 months in 10 years except for bulkheads used in high seismic zones. For bulkheads under design load more than an average of 3 months in 10 years or located in FEMA Seismic Design Categories (SDC) C-E https://www.fema.gov/emergency-managers/risk-management/earthquake/hazard-maps), design for earthquake loads according to Chapter 4. Design bulkheads for EQ when the bulkhead will be installed for greater than six months.

15.9.3 Load Combinations. Bulkheads will be designed for the strength limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in Chapter 4. The hydrostatic principal load factor is selected according to paragraph 4.3.3 based on the return period of the maximum hydrostatic loading. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

15.9.3.1. Load Combination 1: Maximum Hydrostatic Load Plus Companion Wave. The load category and corresponding load factor of the maximum hydrostatic loading is dependent on the characteristics of the project.

15.9.3.1.1 Inland. Waves are created by wind events independent of the water level.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, Hs_{pr} + 1.0\, Hw_c \qquad \text{(Equation 15.1)}$$

15.9.3.1.2 Coastal. Hydrostatic, wave, and wind loads are correlated. Peak design wind gusts are unlikely to coincide with peak wave. A companion wind load is applied to the structure above the top of the wave pressure diagram.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, (Hs + Hw)_{pr} + 0.5\, W. \qquad \text{(Equation 15.2)}$$

15.9.3.2. Load Combination 2: Maximum Hydrostatic Load Plus Companion Impact (if applicable). The load category and corresponding load factor of the maximum hydrostatic loading is dependent on the characteristics of the project.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, Hs_{pr} + 1.0\, (IM_c \text{ or } BI_c) \qquad \text{(Equation 15.3)}$$

15.9.3.3. Load Combination 3: Maximum Impact.

15.9.3.3.1 Loads consist of extreme wave, impact, or thermal ice expansion, as applicable, plus companion hydrostatic loading, dead load, and gravity loads. For coastal gates, impact loads will be correlated with water levels and the companion hydrostatic loading will be equal to design values from Load Combination 1.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, (IX_X \text{ or } IM_X \text{ or } BI_x \text{ or } Hw_X) + 1.0\, Hs_c \qquad \text{(Equation 15.4)}$$

15.9.3.3.2 Unless site specific data is available, the average annual return periods of the extreme thermal expansion ice load and debris and ice impact load are not known and $\gamma_{pr} = 1.3$. Wave loads for return periods meeting the requirements of paragraph 4.3.4.1 can be estimated from wind data and $\gamma_{pr} = 1.2$ for waves.

15.9.3.4. Load Combination 4: Removal.

15.9.3.4.1 Dead, gravity, and maximum lifting load assuming the stoplog is stuck or jammed in the slots:

$$1.2\, D + 1.6\, G + \gamma_{pr}\, Q_{pr} \qquad \text{(Equation 15.5)}$$

15.9.3.4.2 The maximum operating load should be considered an extreme load (QU_X) with unknown return period ($\gamma_{pr} = 1.3$) unless site specific information is available.

15.9.3.5. Load Combination 5: Dead Load. Forces on bottom bulkhead under maximum stack height.

$$1.4\, D + 1.6\, G \qquad \text{(Equation 15.6)}$$

15.9.3.6. Load Combination 6: Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load and gravity loads.

15.9.3.6.1 For standard and site-specific OBE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c \qquad \text{(Equation 15.7)}$$

15.9.3.6.2 For standard MDE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.25\ EQ + 1.0\ Hs_c \qquad \text{(Equation 15.8)}$$

15.9.3.6.3 For site specific MDE and MCE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.0\ EQ + 1.0\ Hs_c \qquad \text{(Equation 15.9)}$$

15.10 Lifting Beams. Lifting beams must be designed following American Society of Mechanical Engineers (ASME) B30 and ASME Below-the-Hook (BTH)-1-2014. Consult with a mechanical engineer for the design of mechanical components. The provisions of Chapter 5 must be applied to FCM. The provisions of Chapter 8 must be applied for fabrication of lifting beams.

Chapter 16
Sector Gates

16.1 General. Sector gates are similar in shape to Tainter gates except they are oriented to rotate about a vertical axis and are supported at the top and bottom with a hinge and pintle, in a manner similar to a miter gate. Sector gates are used as lock gates and as waterway closure gates, usually in coastal areas because of their structural ability to withstand reverse head water loadings caused by tidal, inland flooding, or storm surge fluctuations. Like miter gates, sector gates are most often used in pairs, meeting at the center of the channel in the closed position and swinging into recesses in the walls for the open position. The trunnions are located in the walls and the skin plate face in the direction of the normally higher pool level.

16.1.1 Application. Sector gates are used at both ends of locks that are located in tidal reaches of rivers or canals where the lifts are low and where the gates may be subjected to reversal of heads. Since these gates can be opened and closed under head, they can also be used to close off flow in an emergency. The gates swing apart and water flows into or out of the lock through the center opening between the gates. In some cases, flow is admitted through filling and emptying culverts to improve filling characteristics or where ice or debris may not permit adequate flow between the gates.

16.1.2 Limitations. Because the turbulence area at the upper end of a lock filled by a sector gate is not effective for lockage of vessels, the length of the lock chambers must be increased proportionately. Model tests indicate that about 100 ft of additional length is required. Like other end-filling systems, sector gates cannot be used for filling and emptying high-lift locks unless the filling and emptying rates are greatly reduced. The practical lift limitation is usually about 10 ft, although gates with higher lifts have been built. Sector gates have high construction costs, long opening and closing times, and larger wall recesses.

16.2 Components.

16.2.1 Skin Plate. The skin plate is designed as a continuous member supported by vertical ribs composed of angles or tees.

16.2.2 Vertical Ribs. The skin plate is attached to vertical ribs, usually angles or tees, by continuous welds. These ribs are designed as continuous members supported by the horizontal beams. The skin plate is considered as an effective part of the vertical ribs, with the effective width of skin plate determined according to the AISC 360 (see also Chapter 10). The minimum depth of ribs should be 8 in. to facilitate fabrication and maintenance, with design using load combination 16-1 only.

16.2.3 Horizontal Beams. The typical gate leaf has three horizontal beams supporting the vertical ribs and skin plate. Each beam is designed for water load and a combined water and boat impact load. The beam is designed as a continuous member supported by the horizontal struts. The curve of the beam can be neglected with the length used for design equal to the arc length along the center line of the beam. The minimum depth of horizontal beams is 24-in. out-to-out of flanges, which permits adequate clearance between the channel-side vertical truss and the skin plate to reduce operating forces required during opening of the sector gate under reverse head conditions.

16.2.3.1. Vertical Members. In order to reduce the effect of dead-load eccentricity on the horizontal beams, the vertical members of the center and recess-side vertical trusses may be framed into the webs of these beams as shown in Figure 16.1. The vertical member of the channel-side vertical truss should be attached to the downstream flanges of the horizontal beams, as shown in Figure 16.2, to reduce operating forces required during opening of the sector gate under reversed head conditions. Based on model test results published in Oswalt (1970) and Oswalt and Murphy (1971), this framing method is recommended for new sector gates.

16.2.3.2. Horizontal Beams. Numerous existing sector gates are framed as shown in Plate 16.5 where horizontal ribs are used to support the skin plate. The horizontal beams consist of straight members, with a length equal to the chord length determined by one-half the interior angle of the gate leaf. The beam is assumed to be a continuous beam of two equal spans, with the center support being braces from the horizontal struts. This type of framing is not recommended for new construction due to higher operating forces when opening the gate from the closed position.

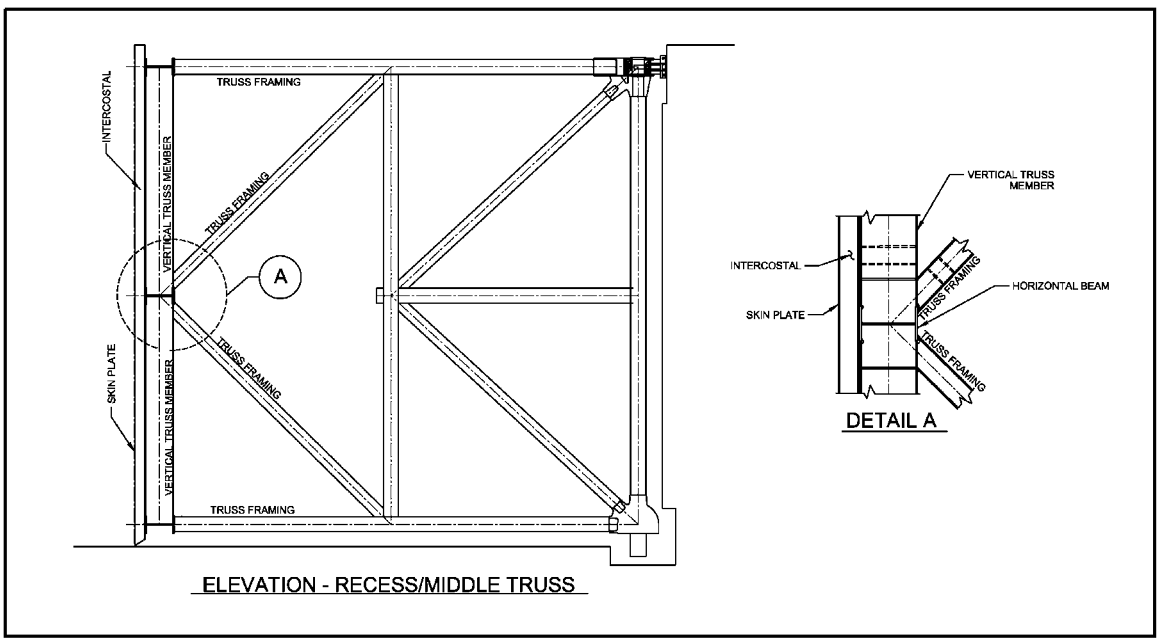


Figure 16.1. Framing of the Recess/Middle Truss Vertical Member
into the Web of the Horizontal Beam

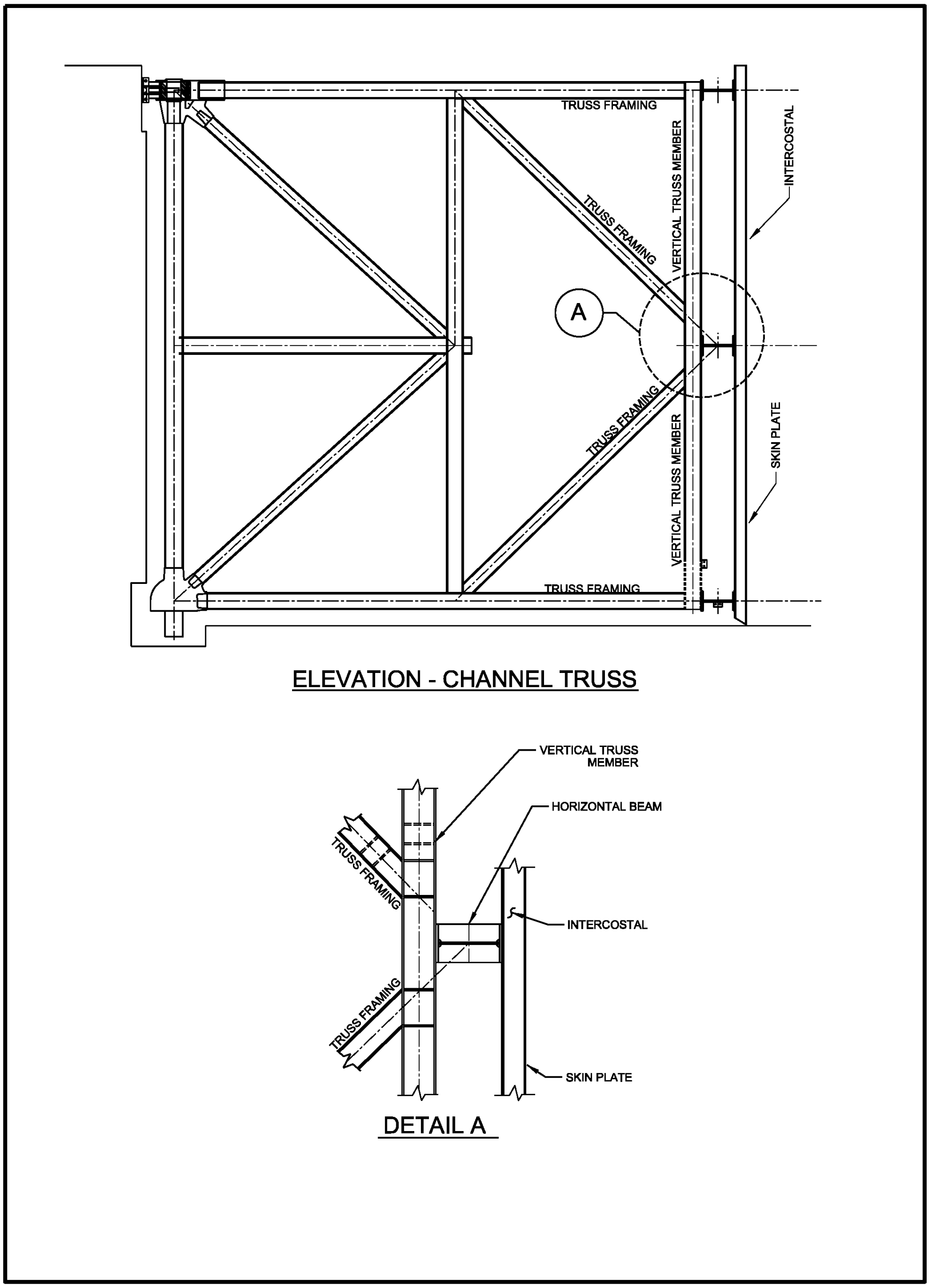


Figure 16.2. Framing of the Channel-Side Vertical Member to the Horizontal Beam Downstream Flange (to allow more water flow between the skin plate and the vertical member when opening gate from closed position)

16.2.4 Framing. The typical sector gate leaf framing consists of three horizontal trusses and three vertical trusses, with horizontal and vertical trusses having some common members. The typical horizontal truss framing consists of three horizontal struts and related bracing. Tubular members are often used for large sector gates. Tubular connections must be designed according to American Petroleum Institute (API) standards (RP-2A LRFD). AISC is used for the main members. For tubular members, ensure the proper welding code is used (e.g., AWS D1.5 does not apply to tubular members). It is recommended to seal all tubular elements.

16.2.4.1. Vertical Trusses. The vertical trusses are designed for all load combinations including barge impact. The top horizontal members of the channel and recess side vertical trusses are also designed to support the walkway loads.

16.2.4.2. Horizontal Frames. The interior angle of the horizontal frames typically varies from 60 to 70 degrees.

16.2.4.3. Buoyancy Chambers. Buoyancy chambers may need to be utilized on large sector gates to reduce the hinge and pintle loading as well as ease operation. Buoyancy chambers have had maintenance issues with leakage so it is recommended to include a monitoring plan or equipment as well as a maintenance plan. Alternatively, the consequences of chamber flooding should be addressed in the design.

16.2.5 Hinge Assembly. The hinge bracket and the hinge bracket support are made of cast steel or welded plates. The hinge bracket support is connected to the lock wall with bolts prestressed to slightly more than the maximum tension load obtained from dead load and the maximum reverse head. It is recommended to provide the prestressing force on the contract documents. See Plate 16.2 for hinge assembly details. See EM 1110-2-2610 for hinge design details.

16.2.6 Pintle Assembly.

16.2.6.1. Pintle. The spherical pintle has proved to be the most satisfactory type for sector gates. This type of pintle has the advantage of allowing the gate leaf to tilt slightly without binding and also facilitates the replacement of the gate leaf after it has been removed for maintenance or repair. The pintle is designed for the maximum reaction, consisting of the combined water, boat, and gate dead loads. For the bolts that connect the pintle socket to the gate, it is recommended to provide bolt tensioning requirements. See EM 1110-2-2610 for pintle design details. A means of pressurizing the pintle and pintle base may aid maintenance removal of the gate.

16.2.6.2. Pintle Base and Anchorage. The pintle base and anchorage have the same function as the pintle base of miter gates, to transfer the horizontal and vertical forces of the gate leaf to the mass concrete. The pintle shaft fits into a recess in the pintle base, and through a combination of direct stress, bending, and shear, the force is transferred from the pintle to the pintle base. The base, in turn, transfers the force into the concrete.

16.2.6.2.1 Use of Grillage Beams. A grillage of small beams, normally in the range of an 8 in. wide flange, is used to transfer the shear and distribute the bearing into the concrete. Anchor bolts are placed in first placement concrete with the base placed in second placement. The base may be made of cast steel or built-up welded plates. In some cases, anchor bolts, prestressed to compensate for slightly more than the design forces, may be used to hold the pintle base in contact with the concrete. These bolts will be so located and prestressed that a compressive force will exist between all parts of the pintle base and the concrete under all loading conditions.

16.2.6.2.2 Use of Prestressed Bolts. Where prestressed bolts are used the grillage of beams may be eliminated. For pintle designs that bolt both into the floor slab and into the vertical wall, it is recommended to make the bolt holes slotted on the pintle plate that backs into the vertical wall. Without having slotted bolt holes on the vertical wall, the ability to properly adjust the pintle vertical elevation with the pintle plate bolts and adjustment nuts in the slab is negated. See Plate 16.2 for typical details of pintle and pintle base.

16.2.6.2.3 Recess Considerations. For the recess in the concrete wall surrounding the pintle, design a high enough block out in the wall to allow the gate to be set down on rollers and slide with the pintle attached to the gate for installation and maintenance purposes.

16.2.7 Seals.

16.2.7.1. Types. The vertical seals on sector gates usually consist of a pair of J-bulb seals for the gate closure at the center of the lock and a single J-bulb seal attached to the corner of the gate recess. The seals at the gate closure, with one seal on each leaf, are preset for 1/8 in. of interference at each seal, for a total of 1/4 in. to assure a minimum amount of leakage when the gate is closed. The recess seal is also set so as to have 1/4 in. compression when the gate is closed.

16.2.7.2. Fabrication and Installation. The recess seal can be mounted to the gate or the concrete wall. If the seal is attached to the wall and a plate or angle bracket extends from the gate's skin plate to mate up with the seal, the stick-out distance of the plate or angle bracket should be minimized (Plate 16.3). The bottom seal utilizes an offset "J" seal, with the bulb offset upstream, or away from the convex side of the skin plate. Normal procedure is for the corner, where the bottom seal meets the vertical seal at the miter point, to be fabricated integrally with the bottom seal (See Plate 16.3). For gates with larger opening widths, floating seals are recommended for the bottom seal (see Plate 16.4).

16.2.8 Fenders, Lifting Supports, and Gate Stops.

16.2.8.1. Fenders. Fenders should be utilized on the channel side of gate leaves to facilitate the distribution and absorption of barge impact. Fenders can be wood, rubber, metal, or plastic composite. For salt water and brackish environments, plastic composites are recommended. See Foltz and Lampo (2017) for more in-depth coverage of fenders and considerations. Fenders should be attached to the framing utilizing bolted connections to facilitate the removal and replacement of damaged components.

16.2.8.2. Lifting Supports. Jacking pads should be provided on the bottom of the gate leaf, located at panel points of the vertical trusses. The pads should be located so that the full weight of the gate can be supported by the pads while maintaining the gate in a stable position. A minimum of three pads should be used on each leaf. Additional pads may be necessary to provide blocking of the gates if jacks need to be adjusted to support higher jacking of gate leaf.

16.2.8.3. Lifting Supports. Lifting lugs on top of the gate should be considered for complete removal of the gate from the lock. Lugs should be located on the vertical truss panel points if possible. If welded, the lugs and welds will be fracture critical.

16.2.8.4. Gate Stops. Bumpers should be provided to prevent damage to the gate leaf caused by the leaf being forced against the wall of the recess. Bumpers should be placed on the center line of the respective horizontal trusses or frames. Each bumper on the leaf has a companion bumper attached to the wall of the recess, with each pair of bumpers having matching alignment.

16.2.9 Walkway. It is recommended to include walkway plates that miter over each other to form a walkway when the gate is in the closed position (see Plate 16.6). Care should be taken that the walkway miter plates do not stick out over the machinery pit when the gate is in the open position.

16.2.10 Embedded Metals. Embedded metals include hinge anchorage, pintle base and anchorage, seal beam for the bottom seal, and the embedded plate which supports the side seal beam.

16.2.10.1. Seal Beam. The seal beam for the bottom seal normally is made up of a rolled beam with a corrosion-resisting plate attached to the top flange. The top of the corrosion resisting plate is flush with the floor of the lock. The beam should be placed in second placement concrete with anchor bolts, also used for adjustment first placement concrete. The anchor bolts are used for seal adjustment.

16.2.10.2. Side Seal Beam. The embedded plate, which supports the side seal beam, is located at the corner of the gate recess. This plate should be made of structural steel and should be anchored with bolts set in the first placement concrete. The side seal beam should be bolted to the embedded plate with corrosion-resisting bolts. The seal contact of the beam should be clad with corrosion-resisting material.

16.2.11 Corrosion Protection. For salt water and brackish environments, a coal tar epoxy paint system is recommended (e.g., USACE coal tar 6-A-Z system). However, coal tar epoxy is a detriment for inspection purposes and should only be used where necessary. For freshwater environments, a vinyl paint system is recommended (e.g., USACE vinyl 3-A-Z or 5-E-Z). In salt water and waters with high conductivity, sacrificial anode cathodic protection is typically used. Sacrificial anodes do require periodic maintenance and replacement. Another cathodic protection system is impressed current. Impressed current must be carefully monitored to ensure the current is at the correct level such that it does not inadvertently accelerate corrosion. For guidance on cathodic protection, please see Chapter 7 and consult the USACE Corrosion Control and Cathodic Protection Technical Center of Expertise (TCX) at Mobile District.

16.2.12 Thermal Expansion. Typically, sector gates are operated with rack and pinion type drives with the rack of gear teeth being mounted horizontally along the skin plate near the top of the gate. There have been several instances at locks where thermal expansion has caused the gears to bind, rendering the gate inoperable. Similarly, the gate seals can bind due to thermal expansion. Therefore, thermal effects need to be considered when designing the gate and mechanical systems to ensure that any contraction or expansion is accommodated. Likewise, seals should also be designed to allow for thermal expansion and contraction.

16.2.13 Erection and Testing. The same general procedures that were discussed for miter gates should apply to sector gates although miter gates have tighter tolerances in fabrication and installation. Each sector gate leaf should have the same shop assembly and match marking as well as the same general allowable tolerances. The clearances of the gate leaves above the lock floor may preclude the use of temporary concrete pedestals for erection. Centering the location of the top hinge directly over the top of the pintle is critical to proper operation.

16.3 Loads and Load Combinations.

16.3.1 Design Requirements for LRFD. Design requirements for LRFD are provided in Chapters 3 and 4. This section provides information specific to design of sector gates.

16.3.2 Loads. Chapter 4 describes loads for all gates. Loads that are applicable to sector gate design include dead load, gravity loads, hydrostatic and hydrodynamic loads, operating loads, barge and other impact loads, ice loads, wave loads, and earthquake loads.

16.3.2.1. D, Dead Load. Dead load is defined in Chapter 4. A load factor of 1.2 is used when dead load adds to load effects and 0.9 when it reduces load effects.

16.3.2.2. G, Gravity Loads. Gravity loads include mud and ice weight, and must be determined based on site-specific conditions.

16.3.2.3. Hs, Hydrostatic Loads. Hydrostatic loads consist of hydrostatic pressure on the gate considering both upper and lower pools. Hydraulic loading on sector gates is produced from direct heads and reverse heads. A direct head is a differential head across the gate with the highest water elevation on the convex side of the skin plate. A reverse head is a differential head across the gate with the highest water surface on the concave side of the skin plate.

16.3.2.3.1 Strength Design. For strength design, the hydrostatic load is from differential hydrostatic pressure on the gate considering both upper and lower pools. Hydrostatic loads are described in Chapter 4. For Hs as a principal load, Hs_{pr}, Hs is the maximum hydrostatic loading from differential head. Depending on the return period of the load it may be usual, unusual, or extreme.

16.3.2.3.2 Companion Loads. For companion hydrostatic loads, Hsc is the normal operating condition, with a return period of 10 years as defined in paragraph 3.3.3.3.

16.3.2.3.3 Fatigue Loading. For fatigue design, the fatigue stress range as described in Chapter 5 will be computed considering load variation due to Hs.

16.3.2.4. Hd, Hydrodynamic Loads. When gates are opened and closed with differential head, lateral forces on the end frame are created from water flowing parallel with the gate face. These forces are more important to the machinery design than to the structural design where they rarely control member selection.

16.3.2.4.1 Use of Model Studies. When determining lateral hydraulic loads, Oswalt (1970) and Oswalt and Murphy (1971) should be used as a guide. The model tests in these reports measured torque at the pintle. The actual load path is from the end frame to the opening and closing mechanisms acting at the top of the skin plate. If a gate design varies considerably from the type shown in the report, model studies must be performed to determine hydrodynamic loads.

16.3.2.4.2 Model Uncertainties. Oswalt and Murphy (1971) state "Due to the nature of the model, force measurements were expected to be slightly erratic. Thus, one particular measurement should not be used for design; instead, the data should be cross-plotted and average curves drawn for design purposes." Because of uncertainty in these loads, a load factor of 1.5 is applied.

16.3.2.5. Q, Operating Loads. Under normal operating conditions, operating loads are treated as reactions to all opposing forces including D, G, Hs, Hd, and friction, F. When applied as a principal load, operating load, Q_{pr}, will be the maximum load that can be exerted by the operating machinery (obtained from the mechanical engineer who designed the machinery). This assumes a gate jam that prevents movement. See Chapter 4 for further discussion on operational loads.

16.3.2.6. BI, Barge Impact.

16.3.2.6.1 Barge impact load can occur from navigation incidents or from aberrant barges during storm events. The minimum value for BI from navigation is a concentrated load of 125 kips. The load is applied at any point on the horizontal trusses and at any panel point on the channel-side end frame truss where barge impact is possible while the gates are in both the open and closed positions. The impact value of 125 kips was determined based on past design practice for sector gate structures along and nearby to the Gulf and Intracoastal Waterway. A study is needed to determine design impact forces for sites with different characteristics.

16.3.2.6.2 Gates at locations in which failure may result in loss of life from uncontrolled release of water or high economic or environmental consequences may require higher design loads than stated in the previous paragraph. In addition, sector gates that are used for coastal flood risk management and potentially subject to aberrant barge impact in storm events will require an assessment to determine design impact loads based on site characteristics. See section 4.2.6.3 for additional guidance on selection of barge impact loads.

16.3.2.7. IM, Ice and Debris Impact Load. The ice-impact load is specified to account for impact of debris (timber, ice, and other foreign objects). For sites with navigation or where ice is present, IM is specified as a uniform distributed load of 5,000 lbs/ft. It acts in the down-stream direction and is applied along the width of the gate at the upper pool elevation. Sites without ice or navigation may be designed for lesser values but design values should represent the upper bound of expected loads. IM will be placed to produce maximum effects. The return period of loading is unknown for IM and therefore the load factor principal load condition 3 (Extreme), applies (see Chapter 4).

16.3.2.8. IX, Thermally Expanding Ice Load. The thermally expanding ice load is specified to account for lateral loading due to thermal expansion of ice sheets on sites where this load is possible. Thermally expanding ice is applied as described in Chapter 4 across HSS members exposed to ice. The thermally expanding ice load will be applied at the applied water elevations to produce maximum effects in each member.

16.3.2.9. EQ, Earthquake. See Chapter 4 for earthquake loading.

16.3.2.10. F, Friction from various components as follows:

16.3.2.10.1 Ft, Pintle and Hinge Friction. During opening or closing of gates, friction loads exist around the surface of the pintle and the hinge between the bushing and the ball. These friction loads result in a friction moment Ft about the pintle and the hinge that must be considered in design.

- The friction moment is a function of a coefficient of friction, the reaction forces component R that acts normal to the surface of the pintle and hinge, and the radiuses of the pintle and hinge.

- A coefficient of friction of 0.3 will be used. This is a reasonable value that applies for any bushing material that may be slightly worn or improperly maintained.

16.3.2.10.2 Fs, Bottom Seal Friction. Loads exist along the bottom of the gate because of friction between the bottom seal and the bottom-seal plate when the gate is opening or closing. The friction force is equal to the preset force in the seal plus product of the coefficient of friction and normal force between the sill plates and the bottom seal.

16.3.2.10.3 Friction Coefficient. For rubber seals, a coefficient of friction of 0.5 is recommended. Seals that have Teflon rubbing surfaces provide a lower coefficient of friction and are recommended for serviceability. However, wear of the Teflon is a concern, and applying a lower coefficient of friction for design purposes is not recommended.

16.3.2.10.4 Pre-Set Forces. Initially, the seals on a sector gate are set with approximately 0 to 1/32 in. of clearance. A design pre-compression of 0.25 in. accounts for gate sag, hinge and pintle wear, and variations in gate temperature between submerged members and non-submerged members. See Chapter 10 for the determination of seal friction forces.

16.3.2.11. W, Wind. The main environmental load considered for sector gate design is wind. See Chapter 4. Wind loads are small when compared to hydrostatic loads and only affect gate reactions when the gate is in an open position.

16.3.2.12. T, Self-Straining. Self-straining loads from temperature differential are not normally considered for sector gates because they are largely unrestrained and water and operation loads predominate. Self-straining loads should be considered if the geometry of the gate may provide restraint for strains from temperature change.

16.3.2.13. Hw, Wave Loads. See Chapter 4 for determining wave loads. Wave loads applied as principal loads are extreme loads, Hw_X. For sector gates resisting storm surge, the water height and wave height are correlated and the combined hydrostatic and wave force should be computed using a correlated analysis. Wave loads applied as companion loads, Hw_c, are usual loads computed according to Chapter 4.

16.3.2.14. L, Live Loads. Sector gates usually have access ways on top of the gate to cross the structure when the gates are closed. Live loads are defined in Chapter 4.

16.3.3 Load Combinations. Sector gates will be designed for the strength and fatigue limit states for each of the following load combinations. Principal load factors, γ_{pr}, and companion loads are defined in Chapter 4. The serviceability limit state is addressed in Chapter 4. The following load combinations are required but other load combinations may be needed for specific applications. Loads are combined according to Equation 4.2.

16.3.3.1. Load Combination 1: Strength Limit State. Gate Closed. Maximum Hydrostatic Plus Wave. Loads consist of dead, gravity, maximum hydrostatic loading, Hs_{pr}, (apply in both directions for gates that can be loaded by both direct and reverse head) and wave. The hydrostatic principal load factor is selected according to paragraph 4.3.3 based on the return period of the maximum hydrostatic loading. Where maximum and minimum load factors are shown such as for dead and gravity loads, the factors must be applied for greatest effect. For gates with independence of water level and wind event, the waves are calculated as companion loads, Hw_c.

16.3.3.1.1 Inland.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + \gamma_{pr}\, Hs_{pr} + 1.0\, Hw_c \qquad \text{(Equation 16.1)}$$

16.3.3.1.2 Coastal. For coastal sites where water levels, waves, and wind are correlated, the correlated hydrostatic and wave load is used. Wind is unlikely to be at a maximum design gust pressure simultaneous with maximum wave loading. A companion wind load is applied to portions of the structure that extend above the wave pressure diagram.

$$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0)\, G + \gamma_{pr}\, [Hs + Hw]_{pr} + 0.5\, W \qquad \text{(Equation 16.2)}$$

16.3.3.2. Load Combination 2: Strength Limit State. Gate Closed. Maximum Hydrostatic Plus Impact. Loads consist of dead, gravity, maximum hydrostatic loading, Hs_{pr}, (apply in both directions for gates that can be loaded by both direct and reverse head) and companion impact loads.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs + 1.0\ (IM \text{ or } BI)_c \qquad \text{(Equation 16.3)}$$

16.3.3.3. Load Combination 3: Strength Limit State. Gate Closed. Maximum Impact Forces.

16.3.3.3.1 Loads consist of extreme barge impact, BI_X, ice and debris impact, IM_X, or thermally expanding ice force, IX_X, as applicable, with dead, gravity and companion hydrostatic loading, Hs_c. For inland gates principal wave loads are also possible under this case. For coastal gates, the impact load will be correlated with water levels and the companion hydrostatic loading will be equal to design values from Load Combination 1.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr}\ (BI_X \text{ or } IM_X \text{ or } IX_X \text{ or } Hw_X) + 1.0\ Hs_c \qquad \text{(Equation 16.4)}$$

16.3.3.3.2 Unless site specific data is available, the average annual return periods of the extreme thermal expansion ice load and debris and ice impact load are not known and $\gamma_{pr} = 1.3$. For barge impact, $\gamma_{pr} = 1.3$. Wave loads for return periods meeting the requirements of paragraph 4.3.4.1 can be estimated from wind data, allowing $\gamma_{pr} = 1.2$ to be used for waves.

16.3.3.4. Load Combination 4: Strength Limit State. Gate Operation. Normal or Reverse Flow. Either leaf subjected to dead, gravity, or maximum differential hydrostatic, Hs_{pr}, during gate opening, corresponding companion hydrodynamic load, Hd_c, pintle and hinge friction, and bottom seal friction. Hydrostatic and operation load is considered a reaction.

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr}\ (Hs_{pr}) + 1.5\ Hd_c + 1.4\ Ft + 1.4\ Fs \qquad \text{(Equation 16.5)}$$

16.3.3.5. Load Combination 5: Strength Limit State. Gate Jammed.

16.3.3.5.1 Loads consist of and maximum operating force, Q_{pr}, with dead, gravity, and companion differential hydrostatic, Hs_c:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + \gamma_{pr}\ Q_{pr} + 1.0\ Hs_c \qquad \text{(Equation 16.6)}$$

16.3.3.5.2 The maximum operating load should be considered an extreme load (Q_X) with unknown return period ($\gamma_{pr} = 1.3$).

16.3.3.6. Load Combination 6: Strength Limit State. Live Load. Gate Closed. Loads consist of live load as the principal load, dead, gravity, and companion hydrostatic, Hs_c:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.6\ L + 1.0\ Hs_c \qquad \text{(Equation 16.7)}$$

16.3.3.7. Load Combination 7: Strength Limit State. Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading, Hs_c, dead load and gravity loads.

16.3.3.7.1 For standard MDE ground motion analysis:

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0.0)\ G + 1.5\ EQ + 1.0\ Hs_c \qquad \text{(Equation 16.8)}$$

16.3.3.7.2 For standard MDE ground motion analysis:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.25\, EQ + 1.0\, Hs_c$ (Equation 16.9)

16.3.3.7.3 For site specific MDE and MCE ground motion analysis:

$(1.2 \text{ or } 0.9)\, D + (1.6 \text{ or } 0.0)\, G + 1.0\, EQ + 1.0\, Hs_c$ (Equation 16.10)

16.3.3.8. Load Combination 8: Fatigue Limit State. Sector gates will be designed for fatigue for the stress range described in Chapter 5. Design must satisfy requirements for either Infinite Life or Finite Life. See section 5.1 for more information on load factors for fatigue.

16.3.3.8.1 Load Combination 8a: Fatigue Limit State I. Infinite Life.

$2.0\, Hs$ or $2.0\, Hd$ (Equation 16.11)

16.3.3.8.2 Load Combination 8b: Fatigue Limit State II. Finite Life.

$1.0\, Hs + 1.0\, Hd$ (Equation 16.12)

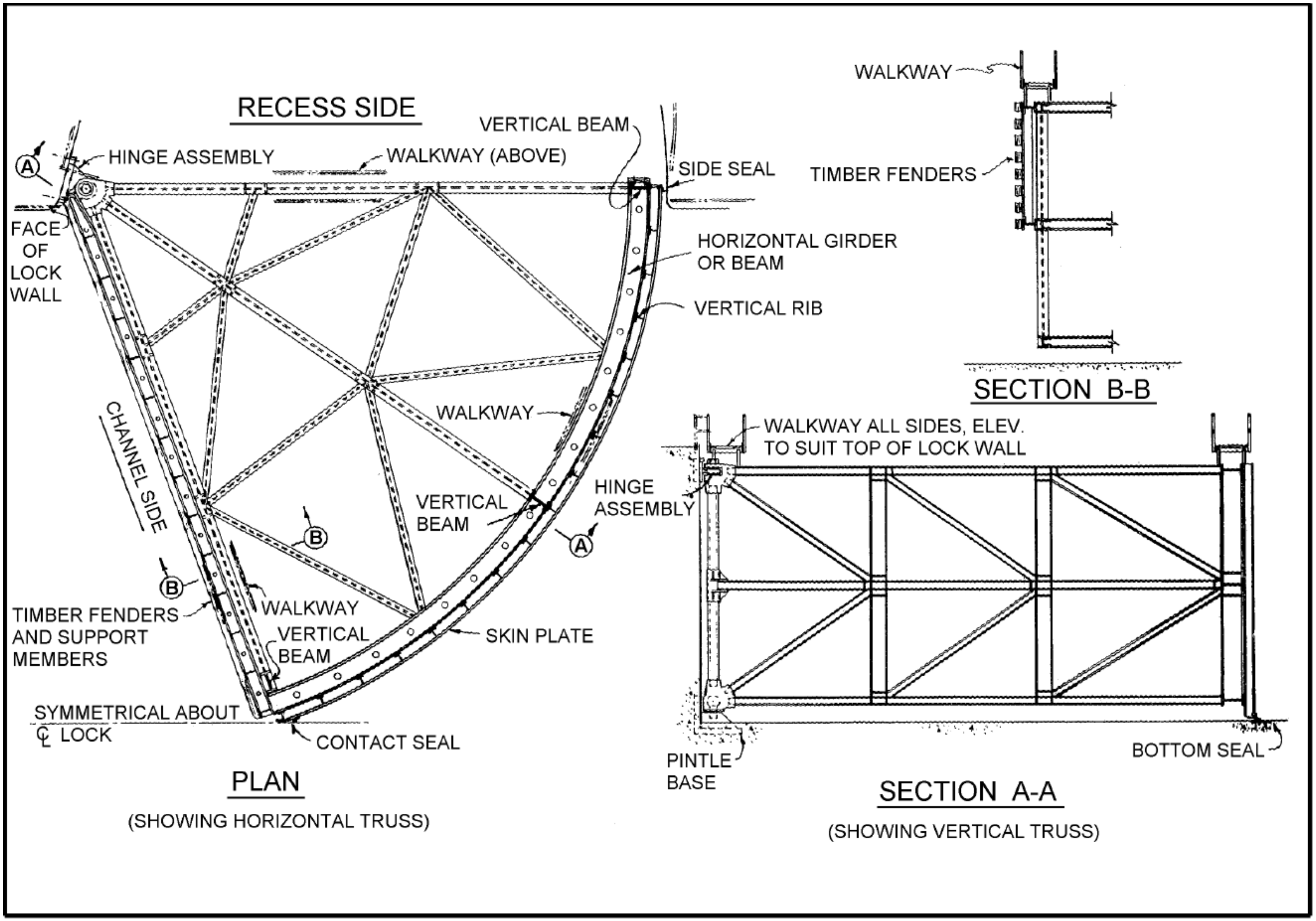


Plate 16.1. Sector Gate – General Plan and Sections

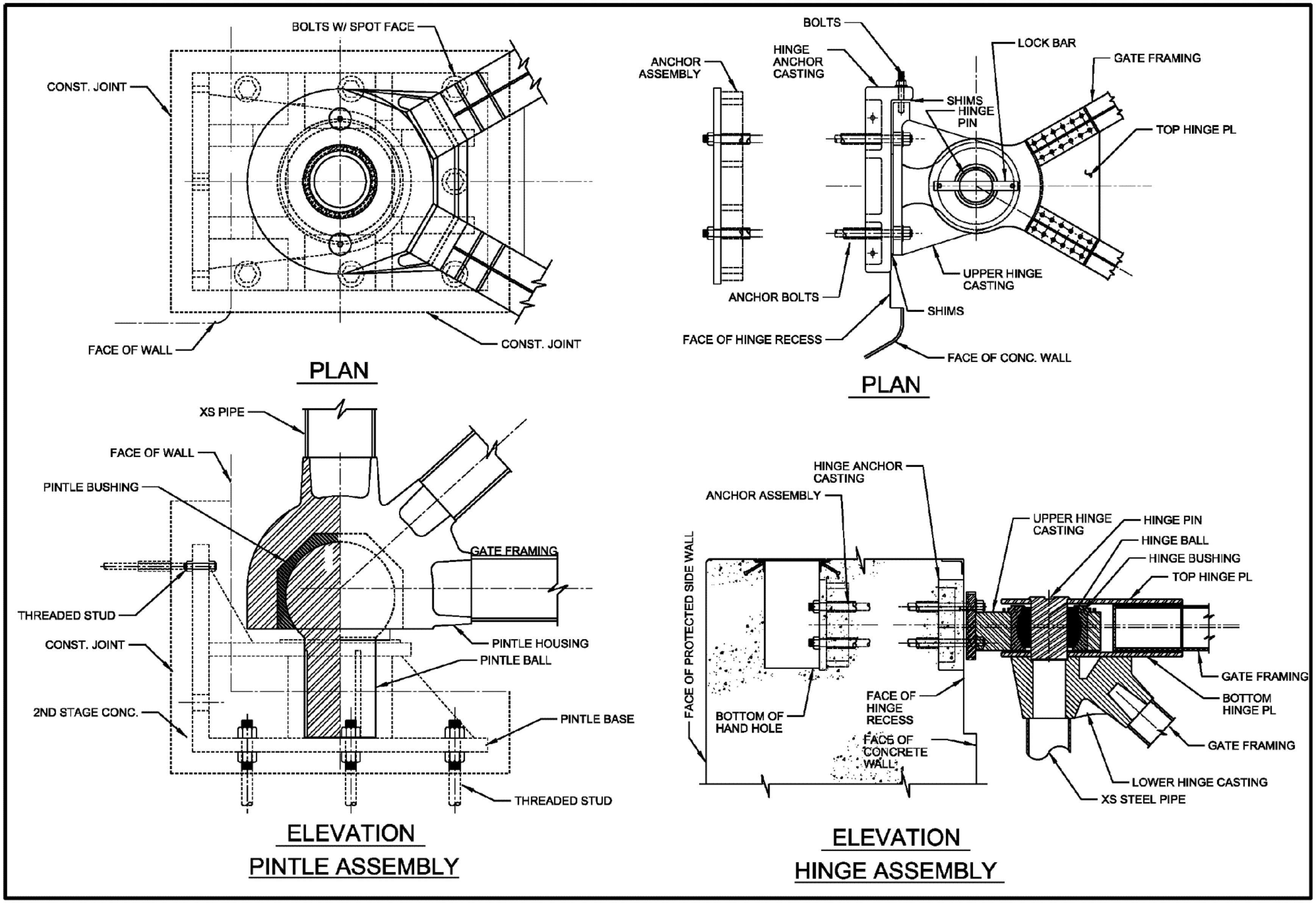


Plate 16.2. Sector Gate – Hinge and Pintle Assemblies

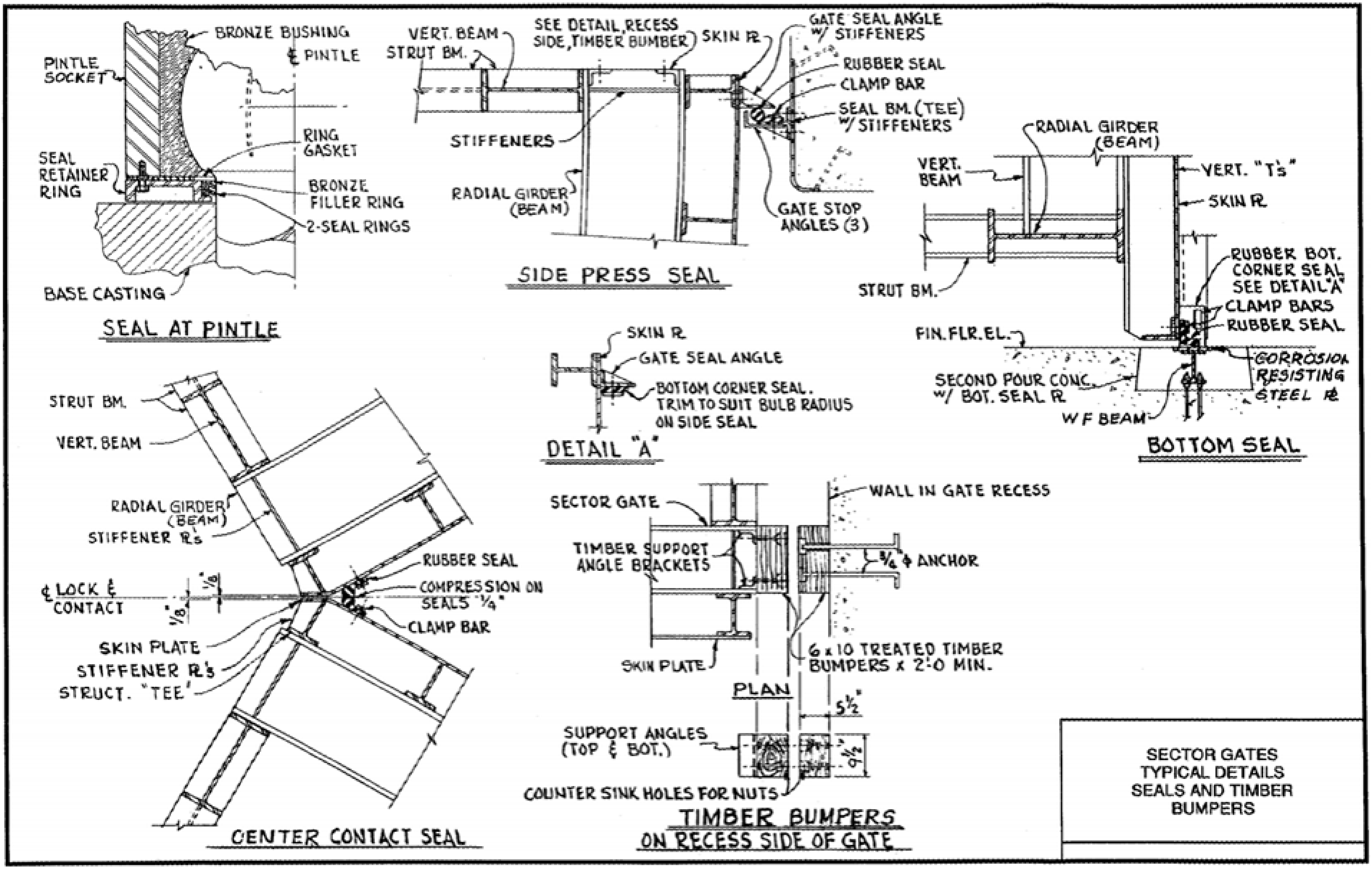


Plate 16.3. Sector Gate Seal Details

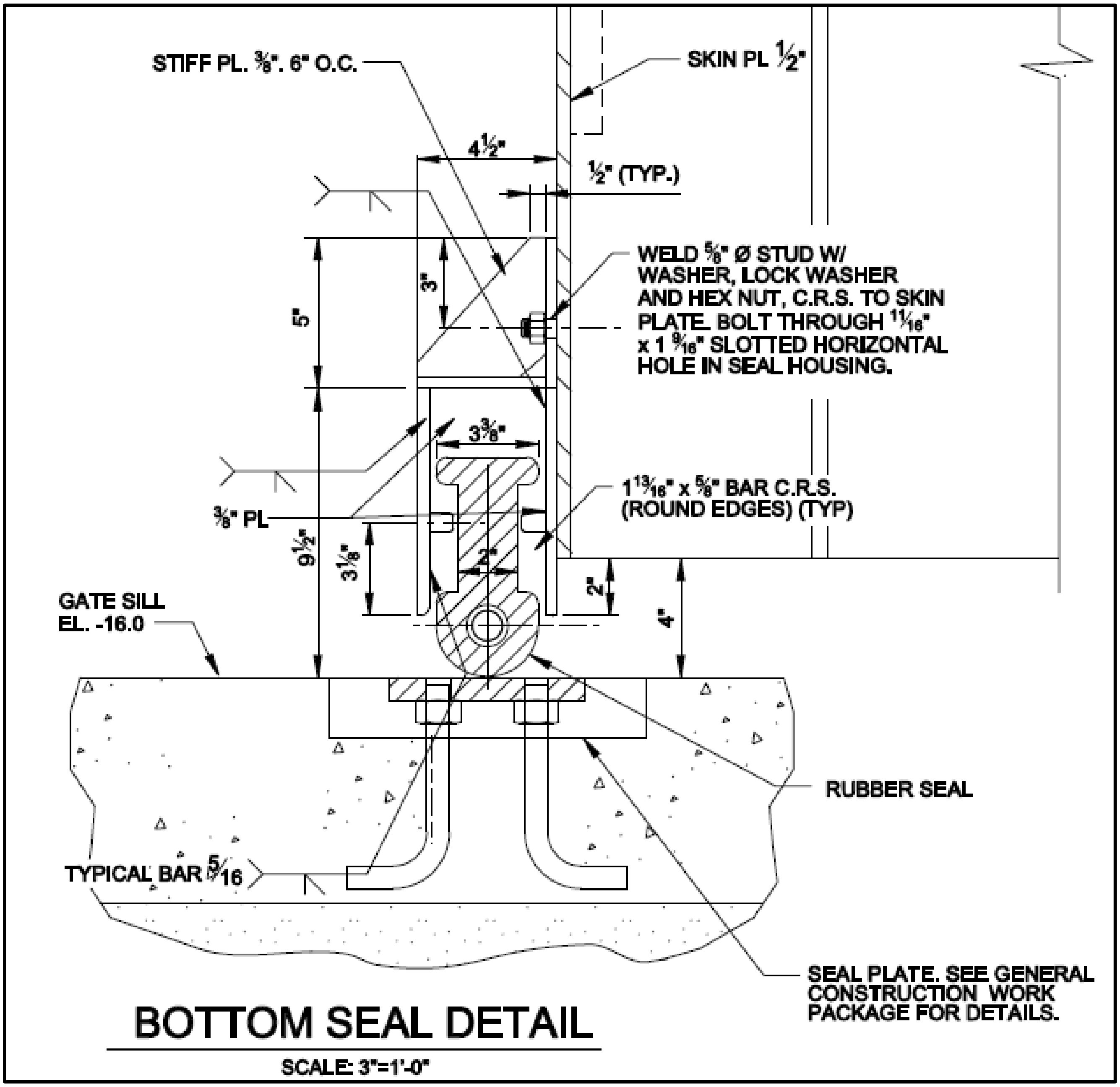


Plate 16.4. Bottom Floating Seal

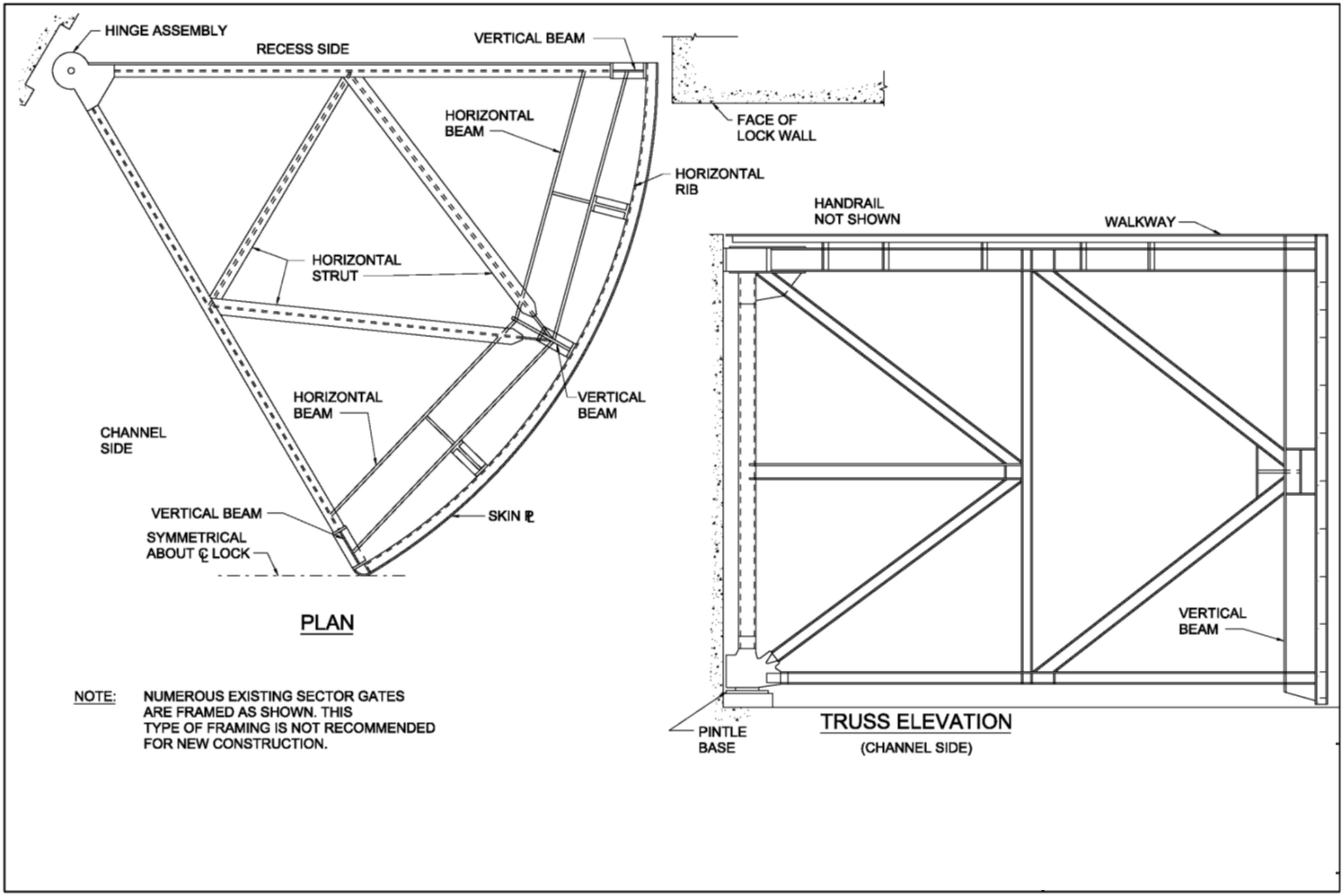


Plate 16.5. Sector Gate – Alternate Framing System

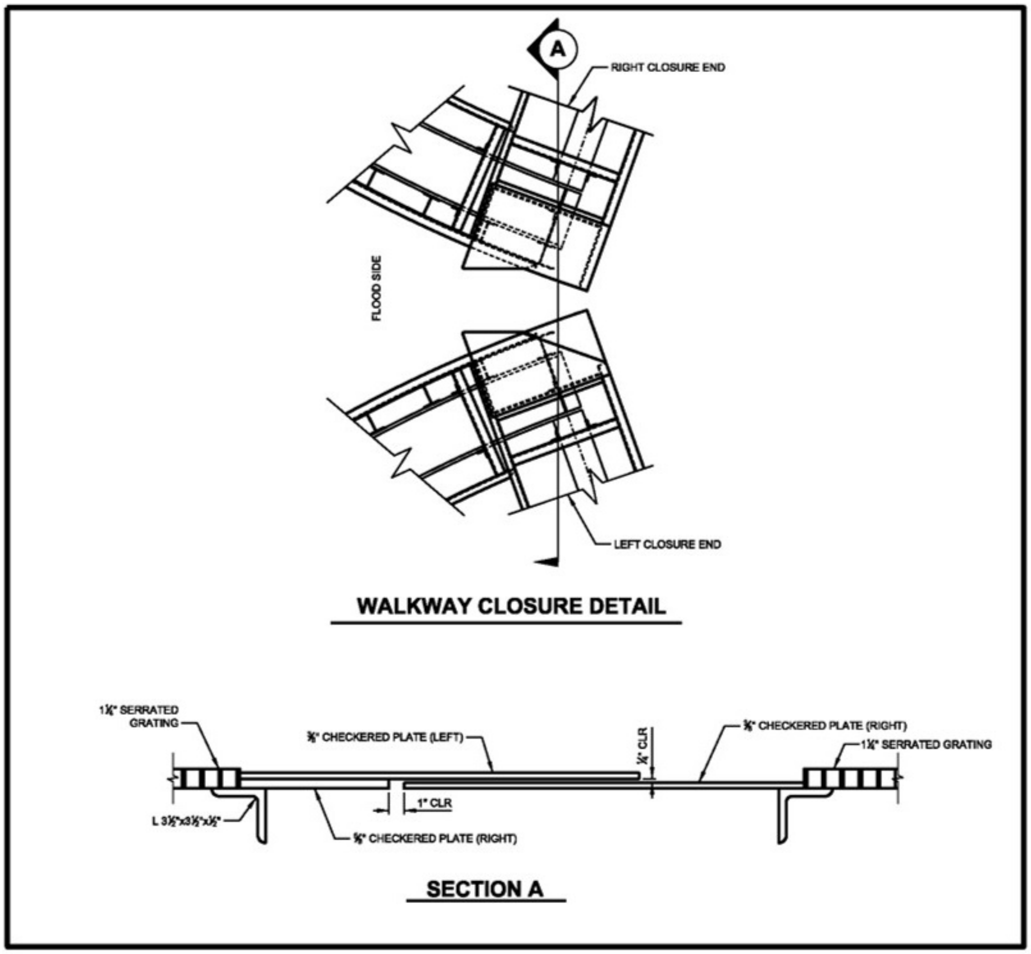


Plate 16.6. Walkway Detail at Miter Location

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Appendix B
Commentary

B.1. Commentary to Chapter 1.

B.1.1. Commentary to paragraph 1.1, Purpose. The LRFD design criteria was initiated within USACE in 1993. The load and resistance factors provided are developed considering the specific magnitude and frequency of loading events anticipated during the design life of the structure and material properties and condition associated with new construction.

B.1.2. Commentary to paragraph 1.7, Purpose. Design of HSS is unique when compared to other types of steel structures because of the unique loadings and environment to which HSS are exposed. This manual prescribes design requirements developed to provide safe structures that must operate in these unique conditions.

B.1.3. Commentary to paragraph 1.8.2, Discussion. Application of this manual alone is insufficient to ensure that all design limit states are adequately addressed. The use of this manual must be supplemented with proper training, experience, or exercise of judgment by the Engineer.

B.1.4. Commentary to paragraph 1.10, Reuse of Existing Designs. Existing designs may not comply with current design requirements such as material properties, standard detailing practice, and fatigue and fracture requirements, and therefore must be modified to meet current requirements.

B.1.5. Commentary to paragraph 1.11, Design Guidance. The load and resistance factors provided in this guidance do not take into account existing conditions and evaluation life for the evaluation and repair of existing HSS. These factors were not considered in the calibration process used to develop this guidance, however until procedures are developed specifically for evaluation of existing HSS, the design criteria outlined in this manual may be used for evaluation. If the HSS does not meet the criteria of this manual, risk assessment procedures may be used. ER 1110-2-1156 provides risk assessment guidance. EM 1110-2-6054 also provides information for evaluation.

B.2. Commentary to Chapter 2. No commentary.

B.3. Commentary to Chapter 3.

Commentary to paragraph 3.1, Purpose. Chapter 3 lays the foundation for the remaining chapters and presents the basis for HSS design.

B.3.1. Commentary to paragraph 3.2, Design Philosophy. The limit state philosophy is employed in these design requirements. The ductility requirement is to ensure ductile failure modes, which tend to redistribute loads under member failures and give warning time as opposed to sudden, brittle failures that can be catastrophic with no warning. Redundancy is a contributing factor in the redistribution of load and is encouraged in design. However, redundancy in HSS, particularly load path redundancy, may be uneconomical. Where redundancy is not provided, additional measures are necessary to ensure ductile failure. See discussions on Failure and Fracture Critical Members (FCMs).

B.3.2. Commentary to paragraph 3.2.1, Failure Modes. Failure is defined as loss of strength or serviceability that threatens the safety of the user or public, affects function or performance to unacceptable levels, or reduces the reliability to unacceptable levels.

B.3.3. Commentary to paragraph 3.2.2, Limit States. The limit state design philosophy is employed within this guidance using the Load and Resistance Factor (LRFD) method. LRFD is:

- Rationally based (based on scientific methods)
- Based on probabilistic models of loads and resistance
- Calibrated through statistical modeling, by comparison with some standard to achieve a desired level of reliability
- Verified by judgment and past experience, and by comparisons of previous or existing designs with satisfactory performance history.

The result is a set of load and resistance factors for various load combinations for various limit states where:

- The Load Factor is a statistically based multiplier applied to force effects accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.
- The Resistance Factor is a statistically based multiplier applied to nominal resistance that accounts primarily for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance.

Load and resistance factors are also related statistically through the calibration process.

B.3.4. Commentary to paragraph 3.2.2.1, Strength. The Strength Limit State is applied to ensure that strength and stability are provided to resist applied loads with an acceptably low chance of the load exceeding the capacity. Some localized damage such as yielding or buckling may occur, but the structural integrity of the member or system is maintained.

B.3.5. Commentary to paragraph 3.2.2.3, Fatigue. The Fatigue Limit State imposes restrictions on stress range that may occur due to a single design load applied at the number of expected stress range cycles. This is a Serviceability Limit State as cracks degrade the structure over time, but may not lead to failures. It could be considered a Strength Limit State if it does lead to fracture or if it does reduce the strength of a member by an amount sufficient enough to compromise function or safety.

B.3.6. Commentary to paragraph 3.2.2.4, Fracture. Tensile rupture and ductile fracture are addressed in the Strength Limit State. Brittle fracture is addressed by imposing material toughness and detailing requirements.

B.3.7. Commentary to paragraph 3.2.3, Critical and Normal Structures. The determination of project hazard potential should include consideration of the population at risk, the downstream flood wave depth and velocity, and the probability of fatality of individuals within the affected population. Environmental and economic consequences may also be considered in the determination of the critical classification.

B.3.8. Commentary to paragraph 3.2.4. Reliability is a measure of an HSS to perform or function consistently for the given variability in loads and resistances. Service life may require maintenance and repair. Service life is defined in Chapter 3.

B.3.9. Commentary to paragraph 3.2.5, Reliability. The LRFD loads and load factors are calibrated for a 100-year service life. HSS are designed to maintain an acceptable level of performance assuming a reasonable amount of maintenance. To assure the desired service life, sufficient maintenance is required. Future deterioration is not considered in the calibration process.

B.3.10. Commentary to paragraph 3.2.6, Constructability and Quality Assurance. In general, the Engineer should not dictate how HSS will be fabricated or erected except where specific procedure, process, or sequence affects the design. Because HSS are complex structures that exhibit 3-dimensional (3-D) behavior, improper sequencing can create residual stresses that can have negative implications on design performance. Therefore, the Engineer should consider fabrication by specifying or excluding certain sequencing if required.

B.3.11. Commentary to paragraph 3.4.2, Material Selection. ASTM A709 is the preferred structural steel due to its availability, wide range of strengths, weldability, superior toughness, and fine-grained characteristics. High Performance Steels (HPS), available in 70 and 100 ksi grades, are good alternatives when higher strength steels are required due to improved (compared to other steel specifications of comparable strength) toughness properties and welding characteristics. However, since these are weathering steels, they should only be used where members are infrequently submerged. Structural bolts conforming to ASTM F1325 Grade A325 are generally the preferred to Grade A490 due to their higher ductility, easier installation, and because they can be galvanized. See Unified Facilities Guide Specifications (UFGS) 05 50 15 for guidelines on selecting forgings, castings, seals, and other materials related to fabrication of HSS.

B.3.12. Commentary to paragraph 3.5, Member Types. A distinction is made among the various member types to better apply the design criteria.

B.3.13. Commentary to paragraph 3.5.1, Primary and Secondary Members. Under different loading conditions, secondary members may act as primary members or they may impose effects on primary members. For example, a diaphragm supporting a skin plate may act as the web of a beam with a portion of the skin plate acting as the flange; walkway supports on a bulkhead are secondary in terms of bulkhead loading, but primary for those using the walkway; bracing members in a stoplog are secondary members when subjected to hydraulic loads, but may act as primary members when the bulkhead is lifted; attachments for access ladders and catwalks are secondary members, but may impact the fatigue resistance of primary members; and, bracing members can cause distortion or out-of-plane bending in primary members.

B.3.14. Commentary to paragraph 3.5.2, Redundancy. Redundant structures have the ability to share load or redistribute load to other members without causing severe damage, partial or complete collapse, or loss of operability or function. Load path defines the path or paths through which the load flows from point of application to the supports. Load path redundancy should be determined considering a global perspective or system response. A structure is load path redundant if one of the paths is severed or otherwise disabled and the load is able to flow to the supports through the other members. Capacity may be severely reduced and function impacted, but the structure should survive. Load path is a function of geometry, member stiffness, continuity, connection stiffness, boundary conditions, and other influencing factors.

B.3.15. Commentary to paragraph 3.5.2.1, Failure Critical Members. A Failure Critical Member is any member for which failure would cause collapse, partial collapse, or loss of functionality of the structure. Because current guidelines to determine the level of scrutiny needed do not exist, the Engineer should determine additional requirements when needed. Although the requirements for FCMs could be applied Failure Critical Members, specifying these requirements in all instances (i.e., non-Failure Critical Members) may result in uneconomical and overly conservative designs. Failure Critical Members can include components loaded in compression, shear, or torsion. Failure critical behavior must be considered under all load cases.

B.3.16. Commentary to paragraph 3.5.2.2, Fracture Critical Members. FCMs are subsets of Failure Critical Members. Truss members and girders in non-redundant structures are examples of FCMs where failure may result in structure collapse. The gudgeon and diagonals on miter gates, the tension chord of a truss-formed stoplog, lifting eyes, and tension components of a simple span or cantilevered trunnion girder are examples of FCMs where failure may result in a significant loss of functionality or collapse of the structure.

B.3.17. Commentary to paragraph 3.6, Analysis. HSS have traditionally been analyzed on a member-by-member basis using simple 2D models and by combining 2D models to simulate 3D behavior. While this is generally conservative and adequate for design of most HSS, there are instances where this approach is not conservative and does not adequately represent the response of the HSS. In these instances, analyses that are more refined and consider the system response may be warranted.

B.3.18. Commentary to paragraph 3.6.1, System Response to Loads. HSS are generally complex structures with complex interaction between members, connections, and moving parts. Simplified analysis techniques are often employed in their design and the resulting design is based more on individual member loading rather than overall system response. While this is often an acceptable approach to design, an understanding of overall behavior, including the influence of the fixity of connections, boundary conditions, and relative stiffness between members, is necessary to produce the most functional designs.

B.3.19. Commentary to paragraph 3.6.2, Simplified Analysis. Simplified analysis techniques have been used successfully to produce designs that perform as intended. However, problems (cracking, member overloads) have occurred due to the inability of these methods to adequately depict overall system response.

B.3.20. Commentary to paragraph 3.6.3, Advanced Analysis. Simplified models may not represent 3D behavior adequately and lack of consideration for this interaction can lead to poor prediction of performance over time. Advanced analysis techniques provide a useful tool to better estimate realistic behavior and more fully predict member interaction in the design. However, the designer must fully understand the process that is being used and have sufficient experience to understand behavior of HSS for these analyses to be meaningful. Complete documentation of the analysis, including the information described in paragraph 3.6.3, provides information others can use to better understand the modeling and validity of results.

B.3.21. Commentary to paragraph 3.7, Corrosion Control. HSS can be subjected to severe environmental and operating conditions. Corrosion on structural members leads to section loss and eventually loss of strength if left unabated. Corrosion on moving parts such as pins, increases friction forces and resulting stresses in members. Design for corrosion control should include consultation with Mechanical Engineers and coating specialists. EM 1110-2-2704 provides design guidance for HSS cathodic protection systems, and EM 1110-2-3400 provides guidance on painting systems.

B.3.22. Commentary to paragraph 3.8, Inspection and Maintenance. Inspection access includes platforms, ladders, and other means for physical access to HSS members for inspection and maintenance. Access to connections and critical members should take precedence over other members. Maintenance access includes physical access for greasing, painting, and other maintenance activities. Good detailing practices include avoiding sharp edges, detailing to provide access for painting, and fastener and connection repairs. The Engineer should seek input from operations personnel during the design process to help assure all operation and maintenance needs are considered in the design.

B.3.23. Commentary to paragraph 3.9, Plans and Specifications. The plans and specifications should not dictate how the contractor will construct the HSS except where it is necessary to ensure that the HSS functions as intended in the design.

B.3.24. Commentary to paragraph 3.10, Fabrication and Erection. Fabrication and erection can induce significant stresses in HSS. These processes must be understood to properly account for them in design. Assumptions are made during the plans and specifications phase to address these loads and adjustments are made as needed for the fabrication and erection methods proposed by the contractor. The designer should assure appropriate tolerances exist in the plans and specifications to effectively fabricate and erect HSS.

B.4. <u>Commentary to Chapter 4.</u>

B.4.1. Commentary to paragraph 4.1, Design Basis. LRFD is a method of proportioning structures such that no applicable limit state is exceeded when the structure is subjected to all appropriate design load combinations. The expression $\Sigma\gamma_i Q_{ni}$ is the required strength and the product $\alpha\phi R_n$ is the design strength. Load factors and load combinations for structural steel design are based on limit states of steel structures. Description of the methodology used in developing load factors and load combinations for buildings and other structures may be found in ASCE (1990), Ellingwood et al. (1982), Galambos et al. (1982), and McCormac (1990) and the commentary of AISC.

The magnitude of a particular load factor is primarily a function of the characteristics (predictability and variability) of the load to which it is assigned and the conservatism with which the load is specified. A well-known load with little variability or a conservatively specified load usually results in a relatively low load factor. Dead loads and static hydraulic loads are in this category.

B.4.2. Commentary to paragraph 4.1.1, Performance Factor for HSS. Performance factor is applied to AISC resistance factors for HSS design. This is to reflect the exposure conditions to which HSS are subjected and the limited ability to inspect and maintain some HSS. The variables that require additional consideration for HSS include:

- Inspection accessibility;
- Maintenance and repair or replacement (may require dewatering or submerged work by divers);
- Possibility of corrosion (water may be fresh, polluted, brackish, or saline); and
- Possibility of severe vibrations or repeated stress reversals (hydraulic flow may cause vibrations and operating procedures may cause stress reversals).

For these reasons, performance factors are applied to the resistance factors specified by AISC.

B.4.3. Commentary to paragraph 4.1.2, Design for Corrosion. Historically, the potential for corrosion has been addressed by adding a 1/8 in. material thickness beyond that needed to satisfy the design limit states.

B.4.4. Commentary to paragraph 4.1.3, Strength Limit State. Strength Limit States are related to safety and load-carrying capacity (e.g., the limit states of plastic moment and buckling). The controlling limit state is the one that results in the lowest design strength. Formulas for calculating member strengths are given in AISC design specifications.

B.4.5. Commentary to paragraph 4.1.4, Serviceability Limit State. Serviceability is a state of acceptable performance in which the function and operability of an HSS are preserved under normal service.

Deflections should be limited to ensure that bearings and other moving parts are not overstressed, seals function properly, machinery loads are not exceeded, and design assumptions are not compromised.

Seals and the members to which they are attached should provide proper flow characteristic and have adequate stiffness to limit vibrations.

Limiting values of structural behavior (maximum deflections, vibrations) are chosen to ensure serviceability with regard to the intended function of the structure.

B.4.6. Commentary to paragraph 4.1.5, Fatigue Limit State. The Fatigue Limit State is applied to cyclically loaded structures. See Chapter 5 for further discussion on evaluation of fatigue life.

B.4.7. Commentary to paragraph 4.1.6, Fracture Limit State. See Chapter 5 commentary for further discussion on fatigue and fracture control.

B.4.8. Commentary to paragraph 4.2.1, D, Dead Load. The nominal section of a member alone, without regard to connections and coatings, will be sufficient for design of most individual members unless a member contains an unusually large number of welds or fasteners. The total dead load, including connections and coatings, is necessary for designing moving parts (bearings), machinery design, and various support members (diagonals, anchor bars, etc.). Because the dead load is constant throughout the life of the structure, excluding effects from corrosion, it is treated as a usual load.

B.4.9. Commentary to paragraph 4.2.2, G, Gravity Loads. Mud, M, includes silt and debris. Silt can accumulate on any horizontal or nearly horizontal surface. Examples include horizontal girders in miter and Tainter gates. Presence of diaphragms and stiffeners can further compartmentalize these members and contribute further to accumulation. The presence of drain holes should not exclude the consideration for silt accumulation as drain holes may not function sufficiently throughout the life of the HSS. The actual thickness of the accumulation of silt will be a function of the turbidity of the waterway and maintenance and cleaning of the HSS. Debris includes log, vegetation, and garbage and can accumulate on or become embedded in HSS members. Other types of ice accumulation include that which occurs due to fluctuation of pool and spray from leaking seals.

B.4.10. Commentary to paragraph 4.2.3, Hydraulic Loads. Hydraulic loads are typically the predominant loads on HSS in terms of magnitude and frequency.

B.4.11. Commentary to paragraph 4.2.3.1, Hs, Hydrostatic Loads. The hydrostatic load can be a function of hydrodynamic conditions, particularly the tailwater under flowing conditions. In this case, the tailwater is reduced resulting in a higher net hydrostatic load. A Hydraulic Engineer should be consulted to quantify tailwater effects.

B.4.12. Commentary to paragraph 4.2.3.2.2, Hd, Hydrodynamic Loads. Some HSS are designed and operated to be overtopped (submerged Tainter gates, crest gates) either on a regular basis or randomly and must be designed accordingly. Some HSS may be overtopped under an extreme event.

B.4.13. Commentary to paragraph 4.2.3.3, Hw, Wave Loads. Wave loads must be considered for all HSS subject to significant wind and fetch and will be additive to the coincident hydrostatic load.

B.4.14. Commentary to paragraph 4.2.4, T, Self-Straining. Self-straining loads will generally not govern the design of HSS except for possibly localized members. However, prying loads may be significant for miter gates, particularly at the anchorages.

B.4.15. Commentary to paragraph 4.2.6, IM, BI, Impact . Impact loads are highly variable and are site specific. The Engineer should determine site specific loads when the values prescribed in this manual are not appropriate. These loads should be applied to any location on the gate where they have a reasonable chance of occurrence. Impact loads are not considered in skin plate design because load effects are generally localized and the added weight and corresponding machinery requirements resulting from accommodating these loads are not justified. Where debris loading is a concern from over-topping of an HSS, these loads will be considered in member design or provisions must be made to protect members from these loads.

B.4.16. Commentary to paragraph 4.2.7, IX, Thermally Expanding Ice. This load is highly site dependent and uncertain. Experience has shown that the design load of 5,000 lb/ft will result in HSS that are not damaged by thermally expanding ice loading. This load can be reduced or neglected where the maximum ice sheet thickness is less than six inches.

B.4.17. Commentary to paragraph 4.2.10,, L, Live Load. Operational loads include all actions resulting from operating the HSS. These include machinery loads applied directly to the HSS, any loads resisting operation of the HSS, and lifting for erection, storage, or inspection purposes. In some load cases, machinery loads are reactions to other applied loads while in other cases (e.g., a jammed gate condition) they are an applied load.

A machinery load is that expected under normal operation with hydrostatic and hydrodynamic load conditions and design friction forces. This would not include loads required for additional machinery load due to impacts from debris or a jammed gate. See EM 1110-2-2610 for more detail on machinery design loads.

The machinery load limit is one that is determined through design, in consultation with the project Mechanical and Electrical Engineer.

B.4.18. Commentary to paragraph 4.3.1, Load Factors. Load factors account for load variation with time, uncertainty of load location on HSS, design idealizations such as assumption of load distribution, and uncertainties in structural analysis. Different load combinations may control the design of different elements of HSS. For a given member, different limit states may be controlled by different load combinations. The load combination method is based on the observation that the maximum of a combination of structural actions (load effects) generally occurs when one of the loads attains its maximum value while the other loads assume their "point-in-time" values.

B.4.19. Commentary to paragraph 4.3.2, Equations. Applicable loads to be combined are site specific. Design equations for individual HSS are provided in Chapters 9–16.

Loads should be combined only when it is possible for them to occur at the same time. Because of the duration, dynamic loads can occur only one at a time.

B.4.20. Commentary to paragraph 4.3.3, Permanent Load Factors (γp).. Permanent load factors for D and G are from ASCE 7-22 and past design.

B.4.21. Commentary to paragraph 4.4.6.2, Acceleration. The term in square brackets in Equation 4.4 is intended to approximate the peak acceleration at the location of the HSS. This acceleration is therefore instantaneous. In order to estimate a more sustained loading on the HSS, the pseudo static correction factor (C) is introduced. A value of 0.75 was selected to be within the range of values used in other publications for this purpose. EM 1110-2-2100 utilizes 0.67 for stability analysis and the U.S. Bureau of Reclamation Best Practices in Dam Safety Risk Analysis utilizes 0.85 for spillway gates. While this factor is uncertain and depends on the structure height, HSS type, water loading, etc., a factor of 0.75 is considered reasonably conservative in the context of the simplified methods used to develop Equation 4.4.

B.5. Commentary to Chapter 5.

B.5.1. Commentary to paragraph 5.1, Design for Fatigue. HSS that are routinely operated are subject to cyclic loading. The likelihood of fatigue cracking depends on the number operating cycles anticipated throughout the life of the structure and number of stress cycles and stress magnitude associated with the operating cycles. The Stress Life approach consists of evaluating stress ranges, stress cycles, and detail category to determine the potential for fatigue. Due to high stress ranges typically encountered in HSS, the Finite Life condition will generally control, but Infinite Life will govern where high cycles and low stresses exist (e.g., gate vibration due to hydrodynamic effects). The AASHTO and AISC procedures are intended for load induced fatigue (i.e., members subjected to tension considering the direction of primary stress (axial and flexural tension or the principal tension component from shear)).

A fatigue design is acceptable when the combined stress range and number of stress cycles exceeds the fatigue strength (plots below the sloping line) for the detail category selected (see Figure B.1).

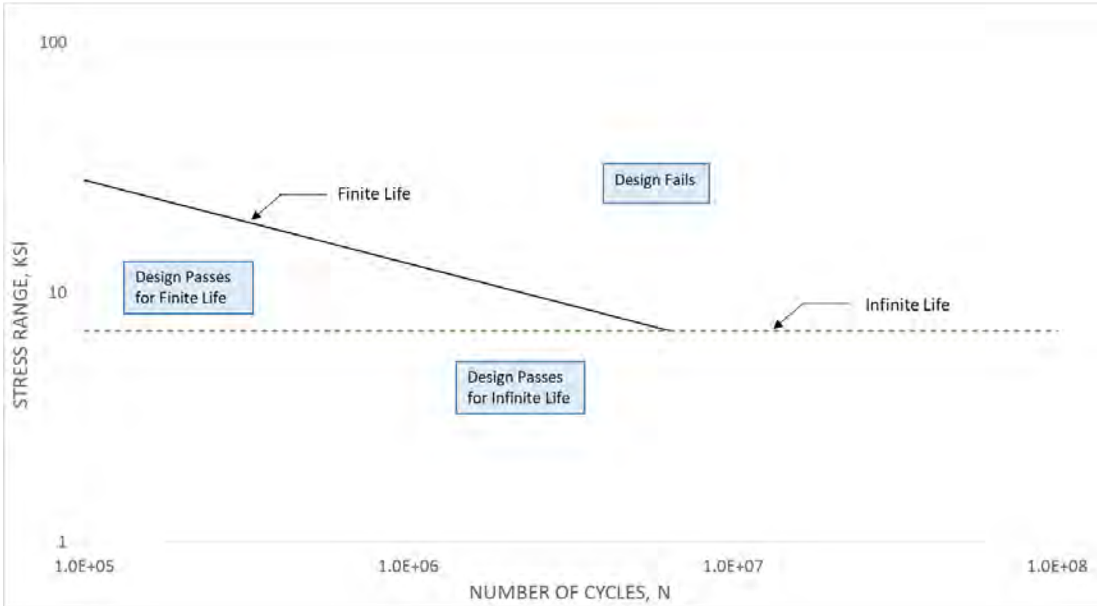

Figure B.1. Example Fatigue Design, Category D S-N Curve Shown

Details subjected to stress ranges below the CAFL, the dashed horizontal line in Figure B.1, are considered to have infinite fatigue life; i.e., fatigue cracks will not occur. If the CAFL is exceeded more than about 0.01% of the time, damage will occur and the detail reverts to finite life. A load factor must be applied to ensure damage does not occur and infinite life is maintained. A load factor of 2.0 should ensure the CAFL is rarely exceeded. This factor may be overly conservative in many cases; thus, the alternate values are allowed. In the case of hydrodynamic loading, multiple stress ranges of varying magnitudes may be encountered within one operating cycle. The highest stress range that exceeds the CAFL more than 0.01% of the time should be used for evaluating infinite life.

B.5.2. Commentary to paragraph 5.1.1.2, Infinite Life. Detail categories A through C have been shown to provide sufficient fatigue resistance except under unusual loading conditions. Detail categories D through E` may provide sufficient fatigue strength under low stress ranges, but may perform poorly when subjected to unanticipated or unpredicted stresses. Therefore, these detail categories should be avoided when possible. In instances where fatigue design cannot be satisfied for the service life or poor performing details cannot be avoided (e.g., cope holes are Category D), the design should include an instrumentation plan to measure stress magnitude and cycles to determine the actual fatigue life. Additional options include design to facilitate replacement of damaged members and to improve fatigue resistance through techniques such as weld toe peening. See the AASHTO LRFD Bridge Construction Specifications for weld toe improvement using Ultrasonic Impact Treatment (UIT).

The Fatigue Detail Categories presented in AISC and AASHTO were developed specifically for connection details encountered in the industries (buildings and bridges) subject to these specifications. Therefore, HSS specific details may not be addressed and some judgment may be necessary in selecting the proper detail category.

B.5.3. Commentary to paragraph 5.1.3, Selecting Stress Ranges. Fatigue cracks propagate under live load tensile stresses. Therefore, if the live load tensile stress does not exceed the dead load compressive stress (net compressive stress), then a crack will likely not propagate. To ensure the net compressive stress remains in a compressive state, a factor of 2 is applied to the live load stress. This factor accounts for unintended loading and modeling inaccuracies that can lead to higher tensile stresses than assumed.

Conservative design assumptions should lead to conservative fatigue design. The simplified load models are generally considered a conservative representation of load distribution. However, actual load distribution may be quite different from that assumed for design. It may change over time, for example, due to change in boundary conditions or change in member or connection stiffness, so that the simplified models may not be adequate. In some cases, it may be desirable to conduct refined analyses to better account for changes in boundary conditions and to more accurately determine stress distributions.

The AASHTO/AISC fatigue provisions are based on a constant amplitude loading to better facilitate laboratory testing, where in reality, stresses are variable. To apply the fatigue provisions, a constant amplitude loading representing an equivalent stress range for all stress cycles must be assumed. Both stress range and stress cycles impact fatigue behavior with stress range predominating (stress has a cubic relationship whereas as number of cycles is linear). A reasonably assumed value is based on loads that occur normally (the mean value). A conservative value is greater than normal, say one or two standard deviations above the mean value (Hs_N or $Hs_{U)}$. Any coincident operating of hydrodynamic load, such as wave, pulldown, or flow induced vibration, should be included when computing stress ranges. Variable amplitude stress ranges can be converted to constant amplitude through use of stress histograms and summing techniques, like Miner's Rule.

B.5.4. Commentary to paragraph 5.1.4, Selecting Number of Cycles. The number of stress cycles can be calculated as follows:

$$N = 365 * Y * n * AD\mathrm{OC} \qquad \text{(Equation 5.1)}$$

Where:

Y = design life in years

ADOC = the Average Daily Operating Cycles

n = the number of stress cycles per operating cycle, recognizing that each operating cycle may have multiple stress ranges

B.5.5. Commentary to paragraph 5.1.5, Distortion Induced Fatigue. Distortion induced fatigue is caused by out-of-plane forces. These effects can be reduced or minimized through proper detailing. Alternatively, the details can be evaluated (e.g., Hot Spot analysis) and the stress ranges compared to an equivalent Fatigue Detail Category. The category selected should possess similar fatigue characteristics, in terms of stress flow and stress concentration, associated with the detail evaluated. See Appendix A for a reference on hot spot analyses.

B.5.6. Commentary to paragraph 5.2, Design for Fracture. Cracking is undesirable in HSS due to impacts to performance and safety and because it can reduce service life. The potential for fracture can be reduced significantly through simple means. Parameters that influence fracture include temperature, loading rate, material toughness, average or nominal stress in a member, flaws or discontinuities that induce stress concentrations, and constraint.

In design of HSS, temperature and loading rate are generally a given, material toughness can be specified, member stress is controlled through member design and selection of cross-section, discontinuities and stress concentrations are controlled through fabrication specifications and proper detailing, and constraint is avoided. Good Fatigue Detail Categories, A–C, are also good for fracture design because they limit the amount of stress concentration in the detail. In addition, details that inherently provide crack arresting mechanisms should a crack develop, are considered to be favorable.

B.5.7. Commentary to paragraph 5.2.1, Fracture Critical Member Determination. Certain FCM are commonly identified using engineering judgment or by default. For example, tension members in two truss or two girder stoplogs or HSS with two lifting points are typically considered FCM. Other members are obvious, like miter gate gudgeon anchors or single lifting points. These structures have little or no load path redundancy under the loads they are intended to carry and the cost to conduct an analysis to identify absence of FCM might not be justified.

HSS with multiple members will be analyzed to determine extent or lack of load path redundancy. Simple analytical methods, like simple beam or plate theory, can be employed. For example, a single piece bulkhead constructed of a skin plate welded to multiple beams can be analyzed by assuming one beam has failed and evaluating the stress in the remaining beams. Each applicable load case of Chapter 15 is evaluated using load factors of 1.0 applied to each load. The loads are then redistributed in a reasonable fashion to adjacent beams. If any of the bending stresses in the adjacent beams exceed 90% of yield in tension, then the failed member will be labeled FCM. If the skin plate in this example is located on the tension side, then consideration must be given for damage to propagate along the skin plate and adjacent beams by fracture, ductile tearing, or other mechanisms.

A Fitness for Service evaluation may be conducted to identify the potential for these failure mechanisms or the extent of damage. All remaining intact members will be evaluated under the applicable limit states and if limits are exceeded, the member is identified as an FCM. If serviceability is a consideration, the deformations (or other limits) resulting from the failed member(s) and impacts on operability should be evaluated. If the HSS is unable to function as needed, then the member(s) is (are) labeled FCM.

As another example, strut arms of a Tainter gate are evaluated under load case 6 with wind to the back of the gate creating tension in the strut arms. The resulting stresses are significantly under yield and the strength limit state is adequately addressed. Because the members are in tension, fracture is a potential failure mode and the fracture limit state must be addressed. If it is determined that member stresses are sufficiently low and stress concentrations are minimized due to presence of fatigue resistant details and good fabrication quality, the Engineer may judge that fracture potential is sufficiently low to avoid the FCM label.

In all cases, sufficient members should be evaluated to ensure all potential FCM will be identified. Note in this process that there is no consideration given for the likelihood of the failed member failing (i.e., a member is assumed to fail regardless if it satisfies any limit state). The purpose of the analysis is to determine lack of redundancy. The issue is fracture because fracture failures occur suddenly and without warning. Once identified as FCM, additional measures are taken (see Fracture Control Plan) to lessen the potential for fracture and increase the reliability of the structure to a level approaching redundant structures.

For design purposes, labeling a member FCM does not appreciably increase the overall cost of the HSS. There may be slight increases in material and fabrication costs, but with small impact on total cost. However, once labeled FCM, that label will likely be carried through the life of the HSS and with that, associated increased costs in operation and maintenance. These costs, including increased inspection frequencies and inspection access, can be significant. Therefore, the application of the FCM label must be used judiciously.

The AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members provides analysis guidelines for identifying FCM in bridges. These guidelines may provide useful information that can be applied to the determination of FCM in HSS.

B.5.8. Commentary to paragraph 5.2.2, Fracture Control Plan. The FCP is contained within Clause 12 of AWS D1.5. The FCP ensures an acceptable material toughness, minimizes discontinuity sizes, and limits the potential for hydrogen cracking. The FCP is a collaborative effort between the Engineer and the fabricator. The Engineer is responsible for identifying material requirements, acceptance criteria, and process limitations. The fabricator is responsible for selecting weld procedures, preparing weld procedure specifications, and ensuring quality is maintained throughout the fabrication.

AWS D1.5 is a workmanship document (i.e., fracture potential is minimized by ensuring adequate quality is maintained during the fabrication process). The AWS D1.5 FCP was developed to reflect this. It does not explicitly address fracture mechanisms by use of fracture mechanics or other evaluation methods. An FCP can be developed applying fracture mechanics in terms of toughness and acceptable discontinuities for a given set of design stresses. Additional considerations should be given for hydrogen cracking potential and residual stresses inherent in the fabrication processes.

B.5.9. Commentary to paragraph 5.2.3, Constraint. Constraint is induced through presence of intersecting welds, intersecting members, thick weldments, and any other situation that does not allow members to deform freely through Poisson's effect, and can lead to sudden and brittle failures.

B.6. Commentary to Chapter 6.

B.6.1. Commentary to paragraph 6.1, Design of Connections. Connections consist of connecting elements (e.g., stiffeners, gusset plates, angles, brackets) and connectors (bolts or welds). Connections for HSS are usually in a more severe environment than connections for buildings. AISC 360 can be used as guidance on design of connections, but should be supplemented with AASHTO 2018 since many HSS members have more in common with bridges (sizes, types of connections, environment, and loads) than with steel building frames. The forces may consist of any combination of axial or shear loads, and bending or torsional moments. Connections may also provide stiffness to limit relative movement between members.

B.6.2. Commentary to paragraph 6.1.1, Design Considerations. For comparing connection types, there are several considerations including their performance, advantages, and disadvantages. The overall cost of the two types of connections is similar.

Bolting Advantages:

- In bolted connections, a crack rarely propagates through the connection due to the circular bolt-hole crack-arresting geometry;
- Ease of maintenance – structural elements can be unbolted, repaired and reinstalled;
- Internally redundant connection – if one or more bolts fails, the connection may continue to perform (the failure mechanism is not sudden and catastrophic); and
- The quality control of bolted connections is generally less complex to implement.

Bolting Disadvantages:

- In hydraulic (submerged) applications, bolted connections are more susceptible to corrosion; and
- May require larger member sizes to account for the net section.

Welding Advantages:

- Provides a sealed connection that prevents water from migrating into mating surfaces of connecting elements; and
- Facilitates simpler splicing and fit up of elements can achieve the full strength of the structural element.

Welding Disadvantages:

- Significant quality control and assurance testing is required;
- Quality can be more difficult to achieve with difficult field conditions (temperature, wind, access); and
- Lacks internal redundancy.

B.6.3. Commentary to paragraph 6.1.1.3, Eccentricities. A concentric connection is detailed so that the centers of gravity of all members framing into the connection pass through a common point. This ensures that the axial force in an intersecting member does not produce additional forces in the connection. Axial loads eccentric from fastener group centroids can significantly increase local stresses or individual fastener loads due to additional shear and bending imposed by the eccentricity. To eliminate this increase in stress, concentric connection details should be provided for all connections.

B.6.4. Commentary to paragraph 6.1.1.4, Splices. Field splices may be required due to shipping and handling limitations.

B.6.5. Commentary to paragraph 6.1.2.1, Welding Codes. AWS D1.5 is intended for the fabrication of highway bridges and AWS D1.1 is intended for the fabrication of buildings and other similar type structures. Although HSS are not bridges or buildings, these standards can apply for design and fabrication of HSS in most cases. AWS D1.5 is the preferred welding code as it offers a higher degree of fabrication controls and fabrication quality when compared to AWS D1.1.

Advantages of using AWS D1.5 over AWS D1.1 include:

- Provides for a Fracture Control Plan or FCP (required for fabrication of FCM);
- Provides mandatory nondestructive testing requirements;
- Requires specific welding procedures to maintain material toughness;
- Provides additional requirements for welder certification; and
- Requires material tracking.

For those HSS where fatigue or fracture are not design considerations, cracking can lead to serviceability issues and reduced service life; therefore, AWS D1.5 is still preferred, because welding to this code minimizes the potential for cracking.

EM 1110-2-6054 provides additional information on alternative acceptance criteria based on fracture mechanics and fitness for service concepts. The AWS welding codes are an industry standard considering what level of quality and workmanship is achievable in a fabrication shop. To the greatest extent possible and reasonably practical, new fabrication should be performed according to the AWS codes. Alternatively, under only special circumstance, acceptability can be assessed on new fabrication using FFS methodology in accordance to EM 1110-2-6054. Existing fabrication (i.e., older structures when industry standards were less then today) may use EM 1110-2-6054 to assess FFS.

B.6.6. Commentary to paragraph 6.1.2.2, Weld Metal. An undermatched (lower) weld strength will result in lower residual stresses and higher resistance to weld cracking than using a high-strength weld metal because of the lack of ductility and toughness of higher strength weld materials. When undermatched weld metal is used, the connection must be designed to ensure proper capacity using the lower strength weld material.

B.6.7. Commentary to paragraph 6.1.2.4, Fracture Critical Welds (FCW). While the Engineer is responsible for identifying FCM on contract drawings, the labeling of FCW is often left to the fabricator. The Engineer is responsible for reviewing the shop drawings and ensuring that all FCWs have been properly identified.

B.6.8. Commentary to paragraph 6.1.3, Bolting. Bolted connections have an advantage over welded connections in that the bolted connection interface provides a natural crack arrestor should a crack develop in one of the members being connected. According to AISC fully pretensioned bolts are required in a connection subjected to cyclic loads, for bolts in oversize holes, and when it is necessary to improve water tightness or if corrosion of the joint is a concern. Therefore, for all HSS structural applications, fully tensioned high-strength bolts must be used.

Bolted connections are much less common on HSS than on buildings or bridges. Typically, bolted connections for HSS are limited to machinery and appurtenances, splices, sill plates, thick plates or jumbo sections (over 1.5 in. thick), steel members embedded in or supported by concrete, locations where future adjustments may be required, or elements that are subjected to damage or may need replacing sometime during the life of the structure.

Hybrid bolted HSS structures composed of welded primary members have been used on HSS subjected to vessel impact and to facilitate shipping. For fatigue design, bolts that are not installed to slip critical requirements are classified as fatigue detail category D. If bolts are installed to slip critical requirements the connection is classified as category B. Testing of pretensioned connections with vinyl primed faying surfaces showed the loss of 30% or more of the bolt tension within three weeks due to paint creep with little loss between three and six weeks.

The use of bolted connections in vinyl coated HSS requires additional detailing. One possible solution to ensure bolts remain fully tensioned after installation is to re-tension the connection three weeks after the initial pretensioning by turning the nuts an additional 1/6 turn.

B.6.9. Commentary to paragraph 6.1.3.4, High-Strength Bolts. Only ASTM F3125 Grade A325, ASTM F3125 Grade A490, and ASTM F3148 bolts are permitted by code to be used in structural connections. ASTM A307 bolts are common steel bolts fabricated from similar material to that of ASTM A36 steel. ASTM A307 bolts are not permitted in tensioned connections or connections subjected to fatigue or dynamic load. ASTM F3125 Grade A325 and ASTM F3125 Grade A490 bolts are designed for steel-to-steel connections incorporating heavy hex dimensions and shortened thread lengths to ensure that threads are excluded from the shear plane.

The use of washers in structural connections is not required by code (AISC) for bolts in standard sized holes. For bolts in oversized or long slotted holes, AISC requires hardened washers under the nut side of the connection. If the Engineer would like washers to be used on both sides of the connection, then this requirement must be specified on the drawings or in the specifications. The presence of washers on one or both sides of the connection will affect the length of bolt required and must be specified clearly to ensure that the contractor orders the proper bolt for the grip length required. Bolts that are too short or too long cannot be properly tensioned due to bolt run-out (bolt contacts the threads before elongation) or due to inability to achieve firm contact between the faying surfaces in a connection if the bolt body length is too long. Additional washers to correct for this issue are not permitted by code.

References that should be consulted for design of bolted connections include: the RCSC *Specification for Structural Joints Using High-Strength Bolts*, the AISC 350 *Steel Construction Manual,* the AISC *Guide to Design Criteria for Bolted and Riveted Joints,* and the AISC Steel Design Guide 17*, High-Strength Bolts, A Primer for Structural Engineers*.

B.6.10. Commentary to paragraph 6.1.3.6, Bearing Connections. Bearing connections are often required on HSS for bearing blocks, seals, miscellaneous attachments, etc. The Engineer must ensure that threads are excluded from the shear plane where required by the strength calculations. The drawings must specify A325-X if threads are to be excluded in bearing connections, or if connections are not required to be fully pretensioned. Bearing connections are not allowed under cyclic loading since bearing connections will permit movement and unequal bolt loading to occur in the joint.

B.6.11. Commentary to paragraph 6.1.3.7, Pretensioned Connections. Pretensioned connections are required for all joints where movement of the joint is not desired, for all connections where water tightness is required, and for all connections with oversized bolt holes. Pretensioned connections are defined in the RCSC *Specification for Structural Joints Using High-Strength Bolts*. Pretensioned connections must be verified with testing and verification. For the design of tension connections, the Engineer must ensure the pretension forces are not exceeded because of the following.

As pretensioning is applied to a connection, the bolt elongates, the plates compress (plates become thinner) and equilibrium exists; the bolt tension equals the compressive stress between the plates times the area compressed. When a tension load is applied to the connection, the plates decompress decreasing the stress in the plates and the bolt elongates, increasing the stress in the bolt. The amount of elongation in the bolt and decompression in the plates is a function of the stiffness of the connection, bolt and plates. Since the modulus of elasticity of structural bolts and plates are about the same, the stiffness is the function of bolt area and area of plate in compression.

As the applied tensile load increases, the compression between the plates decreases until the plates are completely decompressed at which time all of the applied load goes into the bolt. Assuming a compressed area of plate equal to 3 times the diameter of the bolt, the tension in the bolt will increase about 10% before all precompression is lost and all of the load is taken by the bolt. Under compressive loading conditions, if the additional compression in the plates is greater than the pretension load, the tension in the bolt will be lost. For example, if the weight of the structure is applied to a bearing block that has been pretensioned, the connection may deflect sufficiently such that the pretensioned bolts become loose. For this reason, it is critical that the Engineer identify when in the assembly sequence pretensioned connections must be verified.

B.6.12. Commentary to paragraph 6.1.3.8, Slip-Critical Connections. Slip-critical connections require additional testing and verification to ensure that the faying surfaces will develop the assumed coefficient of friction. Additional considerations include installation sequence, tensioning sequence, and verification of tension. The use of slip-critical connections is not common in typical building fabrication with the exception of some bracing systems. As a result, the Engineer should take special care to ensure that the connections that are intended to be installed as slip-critical connections are specifically identified on the drawings.

B.6.13. Commentary to paragraph 6.1.3.8.2, Faying Surface Preparation. Common USACE paint systems such as vinyl and coal tar epoxy do not provide adequate faying surfaces for slip-critical connections. In addition to not providing a sufficient coefficient of friction, vinyl paint systems permit creep in assembled connections. The Engineer should be aware of these limitations and should be prepared to evaluate the use of alternative paint systems or faying surface preparation in these areas. Verification of faying preparation and resultant coefficient of friction requires testing following Appendix A of the RCSC S348.

B.6.14. Commentary to paragraph 6.1.3.9, Fitted Bolts and Fitted Connections. There are no standard procedures for the design or installation of fitted bolts. As a result, the use of fitted bolts should be evaluated before selecting fitted bolts over other structural bolted connections.

B.6.15. Commentary to paragraph 6.1.3.10, Other Bolts. ASTM A307 bolts cannot be pretensioned. As a result, ASTM A307 bolts may only be used to carry shear loads in nonstructural connections.

B.6.16. Commentary to paragraph 6.2, Detailing for Performance. Detailing for Performance refers to the practice of selecting details that will perform during the service life for a given set of in-service conditions. This includes selecting details that are resistant to fatigue and fracture as well as detailing to minimize corrosion and other factors contributing to long-term degradation of HSS. Detailing for performance relates to the functionality or use of the HSS, with the aim of ensuring a long service life and extending the time between periodic maintenance inspections.

B.6.17. Commentary to paragraph 6.2.1, Detailing to Minimize Residual Stress. The toughness of the heat-affected zone can be lowered if the heating and cooling produced during welding are not properly controlled. Thick plates and jumbo rolled shapes often exhibit low toughness and high residual stress away from air exposed rolled surfaces due to slow cooling during the manufacturing process, and lamellar discontinuities tend to be more prevalent when compared to thinner plates or sections. Thermal effects due to welding can also decrease material toughness and produce high residual stresses. The residual stresses can act on these low toughness areas in the base metal that increases the potential for cracking.

The potential is further increased where lamellar discontinuities exist. The adverse thermal effects are reduced with gradual heating and cooling of the weldment as it is welded, and through proper selection of weld process and procedures. The residual stresses create a significant stress profile in the materials and consequently can cause issues with fatigue and fracture. These highly localized stresses may commonly reach the yield point of the material. As weld metal is built up, the residual stresses will increase, and should be a consideration in weld sizing (i.e., minimize weld size). Along with controlling residual stresses, proper weld detailing and sequencing the placement of welds will minimize unnecessary constraint. Similar residual stress effects can occur at flame cutting surfaces. See Modern Welding Technology by Howard B. Cary, and Design of Welded Structures by Omar Blodgett.

Multiple welds in an isolated area can produce residual stresses that can exceed the yield strength of the parent material due to constraint. Intersecting welds, intersecting members, and thick weldments can result in highly restrained and highly constrained conditions and increases the potential for constraint-induced fracture.

Residual stresses can increase with an increase in volume of weld material; thus, a complete joint penetration weld (CJP) may have a higher degree of residual stress compared to a fillet weld in the same application. Welds placed on a member directly opposite each other have the potential to create time delayed through-thickness laminar tearing in the parent material between the welds due to residual stress resulting from weld shrinkage during the cooling process. Ways to manage residual stress include proper selection of weld joint geometry, weld process, weld sequencing, weld peening, and post-weld stress relieving.

B.6.18. Commentary to paragraph 6.2.2, Detailing for Fatigue Resistance. For HSS where vibration may produce significant cycles of stress, details with high fatigue resistance will help to reduce the potential for cracking.

B.6.19. Commentary to paragraph 6.2.3, Detailing for Fracture Resistance. See Chapter 5 for a discussion of fracture resistance. Details which perform weld for fatigue typically perform well for fracture.

B.6.20. Commentary to paragraph 6.2.4, Corrosion Control. Consistent weld size should be maintained throughout wrapped welds to ensure weld quality. Detailing should ensure that the weld wrap lies in one plane to maintain good quality. EM 1110-2-2704, Cathodic Protection System for Civil Works Structures, provides additional detailed guidance regarding corrosion control for HSS.

B.6.21. Commentary to paragraph 6.3, Detailing for Fabrication. Detailing for Fabrication refers to the practice of providing those details that promote fabrication quality, facilitate fabrication, ensure good fit-up, and provide sufficient working access, while accommodating standard fabrication processes. It is recommended to provide a minimum of 8 in. of working room at a 45-degree angle to perpendicular members at a weld joint. If tighter working room is required by other constraints, potential fabricators should be consulted for requirements. Fabricators, erectors, and material suppliers should be contacted during the design process if there is concern about constructability. Detailing for Fabrication will help reduce fabrication cost.

B.6.22. Commentary to paragraph 6.3.1, Distortion Control. For long continuous welds, control of the heat input (i.e., voltage, current and travel speed for a given process) is important to controlling the buckling of the member due to restraint. A backstepping procedure may be used to control the localized warping of a plate. Backstepping includes breaking up a long continuous fillet weld into sections and welding back into the previous weld.

It is recommended that a sequencing plan be required as a prefabrication submittal. For example, on a plate girder, the fabrication should begin from the inside and work toward the outside. This is accomplished if the web plates are butt welded before welding the flange plates to ensure that components will fit without having to introduce unneeded restraint and residual stresses. It can also be beneficial in reducing warpage of a flange welded to a girder web by balancing placement of the longitudinal welds connecting the flange to the web. Consideration should be given to the entire member being heat soaked before welding to prevent a large temperature gradient near the weld location that could lead to warping of the member. The cooling rates should also be controlled to provide for uniform cooling of the member. This requirement should be included in the design specifications to avoid any issues if the fabricator will not allow the input of the Engineer in the fabrication procedures.

Heat straightening guidance has been developed by Federal Highway Administration (FHWA) that can be used to help shape steel both in the fabrication shop or in the field during erection. See FHWA document Guide for Heat-Straightening of Damaged Steel Bridge Members for guidance on heat straightening.

B.6.23. Commentary to paragraph 6.3.2, Support and Restraint. Although the Engineer is responsible to detail the connection, the actual fabrication procedures are developed by the fabricator. The fabrication procedures need to be reviewed and accepted by the Engineer before fabrication is initiated. Quality fabrication is a result of accurate design drawings, specifications, clear and concise shop drawings, and an internal shop quality control program. The fabrication shop is responsible for providing specified materials, proper weld joint preparation and processes, and an organized quality control plan. These fabrication procedures and submittals need to be reviewed by the Engineer to ensure that the weld will have acceptable quality. The Engineer should be knowledgeable about the AISC Quality Certification Program and other similar industry quality programs for fabricators, and should consider the applicability of these certifications requirements in the specifications.

B.6.24. Commentary to paragraph 6.3.3, Access for Nondestructive Testing (NDT). Magnetic particle testing is not generally used for overhead testing, but can be performed on almost any ferrous contoured surface. Probes for testing are spaced approximately 6 in. apart and can be adjusted to almost any configuration. Welds with normal weld access holes, snipes, or clips generally allow adequate room to perform testing.

Ultrasonic testing. The area adjacent to the weld needs to be smooth with no weld spatter. The distance from the centerline of weld required for testing on each side of the weld area varies proportional to thickness of the material. The minimum distance will be approximately 2 in. on each side of the centerline of the weld, and this will increase as the thickness increases.

Radiographic testing requires access to both sides of the weld. The weld must be in straight alignment so the radiographic film can be in complete contact with the weld. Any type of kink or bend will not allow for full contact. Radiographic testing may be used on a large radius, but the image will be distorted. The weld area must be clean with no weld spatter.

B.6.25. Commentary to paragraph 6.3.4, Bolting Access. Bolted connections should be detailed with sufficient access to allow pretensioning of the connection. The Engineer should not rely on the table for entering and tightening clearances presented in AISC 360. This table only provides clearances for the socket body itself. The use of hydraulic torque wrenches will affect the required access to bolts. The Engineer should consider the use of 3D Computer-Aided Design And Drafting (CADD) software to model both the connection and the hydraulic torque wrench to ensure that sufficient access has been provided.

B.7. Commentary to Chapter 7.

B.7.1. Commentary to paragraph 7.1, Operability.

Sidesway and binding can be limited by incorporating bumpers, guides, rollers, and other devices.

Excessive deflections are the result of inadequate stiffness and can result in poor seal performance and excessive gate vibration.

Some USACE Divisions design HSS and HSS components to a common regional standard as a way to benefit from interchangeability and to improve system-wide operability.

Having the Engineer actively participating in the initial development of the HSS Operation and Maintenance (O&M) Manual during the design process better assures integration between the design and operation phases. Important design information includes, for example: dead weight assumptions, lifting design and process, location of details more vulnerable to cracking, location of FCM, potential failure modes, estimated projection of scope, and cost anticipated for future budgeting of HSS rehab and repair.

Instrumentation and remote sensing are useful tools in monitoring performance and identifying operational and structural problems that can eventually lead to operational or structural failures.

B.7.2. Commentary to paragraph 7.2, Maintenance. Maintenance is accommodated in the design by providing protective systems that minimize maintenance activities and that provide sufficient access to conduct maintenance activities.

Connections for components that require removal for routine maintenance (e.g., seals, contact blocks, timber bumpers, mechanical equipment, removable walkways, cathodic protection sacrificial components, and bushings, should not be welded).

The Engineer should discuss with O&M Product Development Team (PDT) if there are any unique O&M components, details, and/or features that they prefer to be included with the design of the HSS.

B.7.3. Commentary to paragraph 7.2.1. For a more detailed discussion and guidance on corrosion control methods, see EM 1110-2-3400 and EM 1110-2-2704. A minimum and increased thickness of steel members implies some corrosion and section loss will occur over the design life of the HSS.

B.7.4. Commentary to paragraph 7.2.1.1, Corrosion Mechanisms. Corrosion mechanisms include localized corrosion (e.g., crevice corrosion or pitting corrosion), general atmospheric corrosion, mechanically assisted corrosion, or galvanic corrosion.

B.7.5. Commentary to paragraph 7.2.1.2, Material Selection. Corrosion-resistant steel should only be used in locations where corrosion is expected to be severe or where it is expected to impair the normal efficiency of gate operation. Flame spraying of corrosion-resistant steel particles on surfaces subject to corrosion may be desirable where the use of solid stainless or clad steel is not practical or economical. Weathering steel in submerged conditions has been shown to deteriorate rapidly.

B.7.6. Commentary to paragraph 7.2.1.4, Cathodic Protection. Brief theoretical discussions on corrosion are presented in EM 1110-2-3400 and CASE (1993).

B.7.7. Commentary to paragraph 7.2.1.5, Galvanic Corrosion Considerations. In general, the difference in the anodic index for dissimilar metals provides an indicator of material compatibility and the likelihood of galvanic corrosion. Metallic contamination of the metal surface can cause galvanic corrosion. Nonmetallic contamination on stainless steel can result in loss of passivity at the contamination sites or can create oxygen concentration cells, which can cause pitting. Such components as stainless steel rollers, wheels, axles, track plates, seal plates, and guides should be passivated after fabrication with a nitric acid solution according to ASTM A380. During manufacturing, metals may acquire contamination from metal forming and machining operations. Avoidance of contamination, or the discovery and removal of prior contamination on metals, is critical at the construction site during erection or installation of the structure or equipment.

B.7.8. Commentary to paragraph 7.2.1.6, Detailing. The designer should consider all locations where water can be trapped for all gate positions. Long-term standing water should be avoided because it contributes to degradation of the coating system and eventually to corrosion. The drain hole should have a minimum diameter of 2 in. and should be placed in low stress regions where possible. Cope holes should be used to avoid pockets of water between stiffeners. Holes in flanges should generally be avoided. For designs with enclosed spaces, it may be possible to fill and seal the space with a noncorrosive liquid or solid. This technique has been used on some HSS.

B.7.9. Commentary to paragraph 7.2.2, Access for Maintenance and Inspection. Sufficient access for cleaning, painting, repair, and inspection equipment should be allowed. Provisions (such as access hatches and safety railing) should be made to provide inspectors access to frequently inspected areas. Close up access should be provided for inspection of FCM where the cost of permanent access is less than the cost of temporary access for each inspection. For very large members, access manholes may be necessary. Access is accommodated by providing ladders, walkways, catwalks, access holes, platforms, tie-off points, lighting, and accommodations to ease removal and handling of HSS components.

B.7.10. Commentary to paragraph 7.2.2.1, Dewatering. Bulkheads and stoplogs are typically used to dewater. HSS that cannot be dewatered should include provisions for removal for maintenance and inspection.

B.7.11. Commentary to paragraph 7.2.2.2, Lifting Attachments. Lifting attachments should include considerations for lift load, impact requirements, minimum thickness of the lug pin, lug projection, and clearance as well as tensile and shearing modes of failure and limits on combined stresses.

B.8. Commentary to Chapter 8.

B.8.1. Commentary to paragraph 8.1, Fabrication Responsibilities. Fabrication of HSS according to ER 1110-2-8157 requires extensive designer involvement in the review and approval of fabrication and erection details. Unlike many USACE construction contracts where the Government Quality Assurance Representative or Contracting Officer's Representative is authorized to review and approve drawings, submittals, test results, etc. on behalf of USACE, the responsibilities for HSS differ from this standard practice. According to ER 1110-2-8157 *Responsibility for Hydraulic Steel Structures*, the Engineer, as designated by the District's Chief of the Engineering Division, must be a licensed professional Engineer, be a structural Engineer, have continuing education that includes structural steel design, experience in the design, inspection, and evaluation of HSS. An Engineer conducting or leading the team conducting quality control or Quality Assurance must have at least the same qualifications as the Engineer.

According to ER 1110-2-8157, the Engineer must participate as part of the District's Quality Assurance team. In addition, fabrication according to AWS D1.5 requires Engineer approval for welding repairs and resolution of other fabrication deficiencies. The level of participation required is extensive for HSS and will require additional funding resources and additional communication between the design staff and the construction office. The Engineer should document important communication and decisions in writing, and share that documentation appropriately with fellow PDT members to assure effective communication.

B.8.2. Commentary to paragraph 8.2, Use of Guide Specifications. Guide specifications for fabrication of HSS structures have been provided to ensure that structures are fabricated according to ER 1110-2-8157. Each HSS should begin with the appropriate guide specification. However, each HSS is unique requiring the guide specification to be edited. Ensure guide specifications are edited by the designated engineer to ensure proper materials, welds, and required testing are addressed.

B.8.3. Commentary to paragraph 8.2.8, Bolting Requirements. According to AASHTO fatigue categories, bolted connections are categorized as a Category B Fatigue Detail provided they are fully pretensioned and installed as slip-critical connections. The guide specification UFGS 055920 incorporates the necessary QA testing required to verify fully pretensioned bolted connections and slip-critical connections. Review AISC for requirements for specifying and testing bolted connections for additional guidance.

B.8.4. Commentary to paragraph 8.3, Fabrication Shop Certification. Fabrication shop certification is required to ensure that the fabricator has sufficient capabilities to perform the quality control requirements associated with an FCP including material handling and consumable storage. In addition, shop certification ensures that the fabricator is familiar with developing and qualifying Procedure Qualification Records and Welding Procedure Specifications, and that the fabricator has the necessary skills to perform repairs to FCMs according to AWS D1.5.

The HYD certification was created in a joint effort between AISC and USACE to ensure a sufficient pool of certified fabricators with the quality procedures required to fabricate hydraulic steel structures with fracture critical components was available. In the future only the HYD certification will be permitted as the industry transitions to a single mandatory certification program to ensure fabricators with the skills and quality control procedures are available for fabrication of HSS.

B.8.5. Commentary to paragraph 8.4.1, Welding Codes. Whereas AWS D1.5 is intended for the fabrication of highway bridges and AWS D1.1 is intended for the fabrication of buildings and other similar type structures, there are no fabrication specifications specific to HSS. AWS D1.5 is the preferred welding code as it offers an appropriate higher degree of fabrication controls and fabrication quality when compared to AWS D1.1. There are a number of advantages in using AWS D1.5 over AWS D1.1 in that AWS D1.5:

- Provides for an FCP (required for fabrication of FCM);
- Provides more stringent inspection requirements;
- Requires that all detected discontinuities and defects be reported on FCMs;

- Requires a fabricator implement a quality control plan;
- Requires specific material and welding material toughness properties;
- Includes additional requirements for welder certification, weld procedure qualification, and quality control within the fabrication shop;
- Specifies controls on welding processes, heat treatment, and repair welding;
- Restricts the selection of base materials (ASTM A 709) and weld materials;
- Places greater restrictions on material tracking and handling;
- Requires qualification of weld procedures for certain processes and welding to other steels or castings; and
- Requires engineer approval of weld repairs.

Requiring the use of both AWS D1.1 and AWS D1.5 on a structure requires careful consideration and drawing preparation to ensure that the fabricator is capable of differentiating components of the structure. Fabricators who possess AISC certification to perform work to AWS D1.5 do not typically have AWS D1.1 work in the same shop simply to avoid the complications associated with quality control including consumable storage, material handling requirements, and testing requirements. As the quality control and quality assurance requirements are different between AWS D1.1 and AWS D1.5, combining these two codes into one structure increases the necessary quality control and quality assurance oversight in order to ensure applicable code requirements are properly enforced.

B.8.6. Commentary to paragraph 8.4.2, Acceptance Criteria. The use of existing or original contract drawings when fabricating a new HSS should be avoided. Existing drawings are usually not labeled to current acceptance criteria and often contain welds that do not meet current code requirements. That is to say, the welds shown may not be permissible by code. For example, they may lack weld access holes in CJP welds. A fabricator who is unfamiliar with these requirements may have difficulty meeting these requirements.

It should be noted that the AWS codes are minimum standards for workmanship. The codes have proven through time to produce quality welds that perform adequately for the applications associated with each code (Buildings and Bridges). Additional requirements for testing are at the discretion of the Engineer and should be considered where applicable for fracture critical structures or structures that have shown performance issues. The Engineer must be familiar with the advantages and disadvantages of the various testing methods.

B.8.7. Commentary to paragraph 8.4.2, Welding Procedure Specifications. Generating a WPS requires that key welding parameters be defined for each weld produced. These key welding parameters are known as "Essential Variables" in the AWS code. Essential variables include preheat, interpass temperature, changes in amperage or voltage, shielding gas, groove angle, etc. Changes to essential variables require a new PQR for all WPSs. As a result of this, the guide specification UFGS 055920 requires that all WPS be qualified by testing. This ensures that all essential variables are established for all new WPS and that any variation in essential variables can be monitored during fabrication. The Engineer must be familiar with the process of WPS generation to review and approve WPS submitted.

B.8.8. Commentary to paragraph 8.4.5, Fracture Control Plan. An FCP addresses the fabricators quality control requirements associated with FCMs. The FCP addresses how the fabricator and erector will handle, cut, weld, bolt, assemble, and finish fracture critical components of the HSS according to AWS D1.5 Clause 12 Fracture Critical Requirements.

The FCP addresses consumable requirements including storage and handling requirements, diffusible hydrogen control, control of electrode exposure, and shielding requirements. The FCP additionally addresses Welding Procedure Specification requirements, fabricator certification requirements, thermal cutting and joint preparation requirements, repair of base metal, straightening, repair welding, record keeping, and general handling and storage requirements for fracture critical material including the use of protective slings to prevent nicks and scratches, which are prohibited by AWS D1.5 to minimize stress concentrations, which can lead to fracture and fatigue.

B.8.9. Commentary to paragraph 8.5, Installation of Bolted Structural Connections. The Engineer should note that previous structural guide specifications and those incorporated into previous designs primarily focused on bolting requirements associated with building industry standards. Buildings are not typically designed with either fully pretensioned or slip-critical connections. In addition, the numbers of connections in a typical building are extensive. As a result, the independent testing and verification of bolt installation was previously considered cost prohibitive. As a result, previous guidance and specifications did not address the need for adequate testing and verification.

HSS are more appropriately compared to bridges in terms of the criticality of connections. HSS are typically designed to minimize weight and size. As a result, the redundancy of bolted connections is often limited by space requirements. The critical nature of bolted connections and therefore the need to verify the proper installation and performance of these connections is increased over that of a building. Therefore, the Engineer must ensure that structural bolted connections in HSS are adequately tested and verified before placing the structure in service.

B.8.10. Commentary to paragraph 8.6, Fabrication Shop Quality Assurance. Fabrication shop Quality Assurance is a team effort between USACE Engineers and Construction Staff. Effective lines of communications and roles/responsibilities should be established before award of any HSS contract. While the specifications require a certain level of fabricator experience, there are still few fabricators who are familiar with the requirements of HSS fabrication. As each HSS is unique and differs from the typical fabrication that the construction office and the fabricator is used to, it is critical that the Engineer be present to explain these requirements, both in writing in the form of instructions to construction personnel and in person in shop and site visits. It is critical that the Engineer work closely with the fabricator and erector to understand fabrication and erection capabilities and requirements.

B.8.11. Commentary to paragraph 8.6.3, Transport. The guide specification requires that the contractor's picking plan and design of any picking points be submitted for approval by the Engineer. The Engineer should consider transport capabilities when designing and detailing the structure. Transport capabilities of the fabricator and erector will dictate the splicing and field installation of the structure. If transport will require field splicing of the structure due to shipping considerations, the Engineer should detail and locate these splices in areas of low stress where possible.

B.8.12. Commentary to paragraph 8.6.4, Field Fabrication Inspection Requirements. Inspection Requirements. Quality assurance procedures for field fabrication are identical to shop fabrication requirements. In addition to inspection requirements, the Engineer should be present for installation and operational testing of HSS. The Engineer should be aware that field modifications may be required due to varying site conditions. Ensuring the Engineer is on-site and available to participate as part of the QA staff is essential to maintain effective communication and to review and respond to critical request for information and submittals in a timely manner.

Appendix C
Miter Gate Diagonal Design

C.1. Introduction.

USACE miter gate diagonal design has been essentially the same as that presented in "Torsional Deflection of Miter-Type Lock Gates and Design of the Diagonals" (USAED, Chicago, 1960) with only minor modifications. This was based on a PhD dissertation by Shermer (1951), a Master's thesis by Hoffman (1944), and a USACE technical report by Shermer (1942) that all are similar. Recently, investigation by Eick and Smith (2020) at USACE ERDC has provided further insight and understanding into miter gate diagonal behavior using numerical modeling and field tensioning data. The main change is using an empirical α factor of 250 when calculating Q_0 instead of the value of 4.0 previously used. In addition, errors and typos in the previous ETL 1110-584 have been corrected. Overall, the method provided here is not exact, but experience has shown that the results obtained from this method have been very close to the values needed in the field to achieve gate plumbness and minimum diagonal pretension.

This appendix is organized into in three parts. The first is Diagonal Design, which is a simplified, user-friendly approach to diagonal design using the revised Q_0 value. The second is a Diagonal Design Example using the approach presented in the first part. Finally, the third part is the Historical and Theoretical Basis provided for reference. It is a corrected version of the previous diagonal design guidance.

C.2. Diagonal Design.

C.2.1. Introduction. There are several objectives when designing miter gate diagonals. The first is determining the required pretension in the diagonals to achieve plumb and maintain a minimum pretension during gate operation. This is performed using unfactored service loads and is outlined in Steps 1–6 below. The second objective is to design the steel components of the diagonals according to Chapter 9 using factored loads as discussed in Step 7 below. The third objective is to design the diagonals for fatigue using unfactored service loads. While designing the diagonals for a 100-year fatigue life is ideal, it may not be practically feasible to size the components as such due to structural clearance limitations. If 100-year fatigue life is not feasible, the diagonals may be designed for a minimum 50-year fatigue life with the realization that maintenance and/or replacement of the diagonals will be needed during the miter gate's 100-year service life.

C.2.2. Definitions. The geometric variables of a miter gate used in diagonal design are shown in Figure C.1. Here, a miter gate with arbitrary placement of diagonal panels is shown to highlight the difference in the geometric properties used.

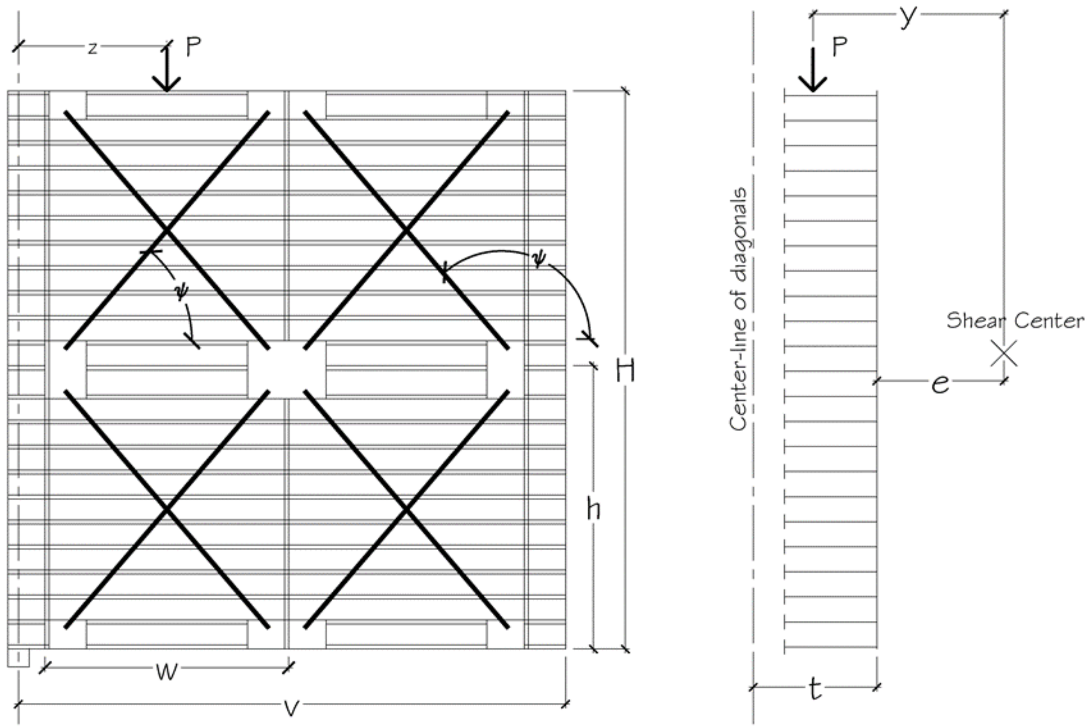

Figure C.1. Diagram of geometric properties of a miter gate necessary for diagonal design. Diagram arbitrarily shows a hypothetical miter gate with four separate diagonal panels to highlight the meaning of different values

Table C.1
Variable Descriptions

Variable designation	Description
e	Distance from skin plate to shear center
h	Height of the diagonal panel
H	Height of the miter gate (centerline to centerline of girders)
L	Length of the diagonal (pin to pin)
P	Load that creates torsion
t	Depth of miter gate (distance from center of skin plate to centerline of diagonals)
v	Width of miter gate (taken as the distance from the center of the pintle to the miter block)
w	Width of the diagonal panel
y	Moment arm of a torsional load
z	Horizontal distance from centerline of pintle to torsional load
ψ	Angle of a diagonal with respect to the horizon. For diagonals on the upstream side of the gate, this angle is measured from the bottom girder towards the miter. For diagonals on the downstream side of a gate, this angle is measured from the bottom girder toward the quoin. See Figure C.2.

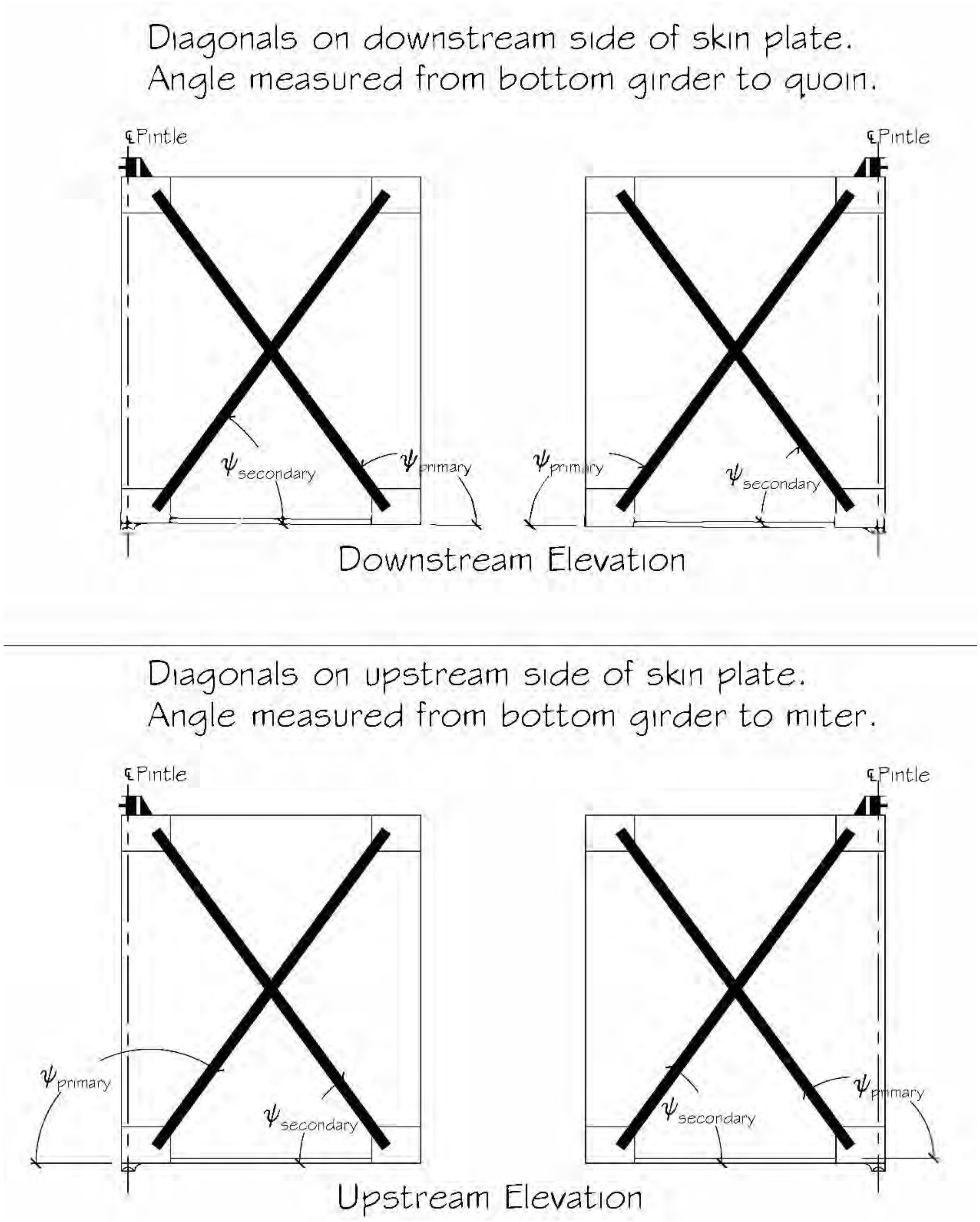

Figure C.2. Definition of Diagonal Angles

C.2.2.1 Positive Deflection. The top of the miter block deflects upstream with respect to the bottom of the miter block.

C.2.2.2 Negative Deflection. The top of the miter block deflects downstream with respect to the bottom of the miter block.

C.2.2.3 Primary Diagonal. A diagonal which, when pretensioned, will cause the gate to deflect in the positive direction. Note, primary diagonals generally have "negative stiffness." That is, a torsional load that causes a positive deflection of the gate will lead to a loss of tension in the primary diagonals. The cosine of the angle of the diagonal will ultimately define the sign of the stiffness.

C.2.2.4 Secondary Diagonal. A diagonal which, when pretensioned, will cause the gate to deflect in the negative direction. Note, secondary diagonals will generally have a positive stiffness. That is, a torsional load that causes a positive deflection of the gate will lead to an increase in tension in the secondary diagonal. The cosine of the angle of the diagonal will define the sign of the stiffness.

C.2.3. **Step 1**: Calculate the Necessary Geometric Properties.

To determine the torsional behavior of a miter gate, the necessary geometric properties must first be calculated. This includes the moment of inertia (I_{xx}), the torsion constant (J), the location of the center of gravity ($\overline{x}$), the location of the shear center (e), the geometric constant unique to diagonal design (γ), and the torsional stiffness of the miter gate (excluding the effects of diagonals) (Q_0). I_{xx}, J, $\overline{x}$, and e are found from engineering first principles (see example at the end of this section). Q_0 should be found as follows:

$$\mathrm{Q_0} = \frac{\alpha \mathrm{GJ}}{\mathrm{H}} \qquad \text{(Equation C.1)}$$

Where G is the shear modulus of the material (taken as 11,200 ksi for steel) of the miter gate and α is an empirical factor that can be conservatively taken as 250 for modern welded miter gates. For older, riveted miter gates, the historically used value of 4.0 should be used.

γ, a geometric constant unique to diagonal design, is found as:

$$\gamma = \frac{2\mathrm{ht}}{\mathrm{Hv}} \qquad \text{(Equation C.2)}$$

In the unlikely event that a miter gate has diagonal panels with different dimensions, γ must be calculated for each panel individually.

C.2.4. **Step 2**: Calculate Applied Torsion from All Loads.

$$\mathrm{T_z} = \mathrm{Pzy} \qquad \text{(Equation C.3)}$$

Where P is a load that creates torsion in the miter gate, y is the torsional moment arm, and z is the horizontal distance from the point of load application to an axis passing through the center of the pintle.

C.2.5. **Step 3**: Calculate the Required Area of Diagonal.

$$\sum A = \frac{\sum T_z}{\gamma h v \, \phi \sigma_y \cos\psi} \qquad \text{(Equation C.4)}$$

Where σ_y is the yield stress of the material used in the diagonals and ϕ is the appropriate LRFD capacity reduction factor. Utilizing σ_y will effectively establish a lower bound estimate for the required area of the diagonals. T_z are any torsional loads considered in a load combination. The minimum negative value of the sum of T_z should be used to size the primary diagonals, and the maximum positive value of the sum of T_z should be used to size the secondary diagonals.

C.2.6. **Step 4**: Calculate the Stiffness Added to a Gate by Including a Tensioned Diagonal.

$$Q_i = 2htK_{diag_i}\cos(\psi_i) \qquad \text{(Equation C.5)}$$

$$K_{diag_i} = \frac{A'}{A'+A_i}\gamma\frac{EA_i}{L_i}\cos(\psi_i) \qquad \text{(Equation C.6)}$$

$$A' = \frac{1}{8}A_p \qquad \text{(Equation C.7)}$$

This step is performed for each of the *i* diagonals on the gate. For definition of miter gate angles, see Figure C.2. Primary diagonals will generally have an angle between 90 and 180 degrees, while secondary diagonals will generally have an angle between 0 and 90 degrees. Therefore, K_{diag} will be negative for primary diagonals and positive for secondary diagonals. Note that $\cos(\psi)$ is present in both the definition of Q and K_{diag}, and so it is effectively squared in the evaluation of Q; therefore, Q will always be positive.

A_p is defined as the sum of the average cross-sectional area of the girders bounding a diagonal panel. On a horizontally framed miter gate with one diagonal panel, this would be the sum of the cross-sectional area of the top and bottom girders, and the cross-sectional area of the vertical diaphragms closest to the quoin and miter (4 total girders).

C.2.7. **Step 5**: Calculate Change in Stress in Diagonals Due to All Expected Torsional Loads.

Calculate the change in stress in each of the *i* diagonals due to each unfactored combination of torsional loads other than the gate's self-weight. The self-weight of the gate will be taken into consideration during the calculation to make the gate hang plumb:

$$(\boldsymbol{\Delta\sigma_i})_{LL} = \frac{\boldsymbol{K_{diag_i}}}{\boldsymbol{A_i}}\left(\frac{(\sum \boldsymbol{T_z})_{LL}}{\boldsymbol{Q_0}+\sum_{j=1}^{n}\boldsymbol{Q_j}}\right) \qquad \text{(Equation C.8)}$$

This change in stress is calculated for each *i*th diagonal. In the denominator of this equation, the sum of *Q* over *j* is performed for all *n* diagonals on the gate. The important note in this equation is that the torsional load combinations must neglect the service-level self-weight (if that is included in the load combination). Explicitly,

$$(\sum T_z)_{\mathrm{LL}} = LC - 1.0SW \qquad \text{(Equation C.9)}$$

Where:

LC refers to any “load combination” of *unfactored service loads* that is considered for diagonal design and SW is self-weight. For example, consider the following typical load combination:

$$LC = D + (C + M) + Q \qquad \text{(Equation C.10)}$$

Calculating the change in stress in the diagonals using Equation C.8 based on the above load combination, the following should be used for the applied torsional loads:

$$(\sum \boldsymbol{T_z})_{\mathbf{LL}} = (D + (C + M) + Q) - 1.0D \qquad \text{(Equation C.11)}$$

Simplifying:

$$(\sum \boldsymbol{T_z})_{\mathbf{LL}} = (C + M) + Q \qquad \text{(Equation C.12)}$$

C.2.8. **Step 6**: Calculate the Required Tension.

The initial post-tension in each diagonal, σ_0, must be selected to satisfy two criteria. The post-tension should be applied such that the gate will hang plumb under its own weight and the diagonals should not go slack when subjected to the expected loads during operation of the gate. Formally, for each diagonal, σ_0 should be selected such that:

$$\mathbf{1}\ \boldsymbol{ksi} < \boldsymbol{\sigma_0} + (\Delta\boldsymbol{\sigma})_{\boldsymbol{LL}} < \mathbf{0.6}(\boldsymbol{F_y}) \qquad \text{(Equation C.13)}$$

Where:

Both positive and negative changes in stress need to be considered. Equation C.13 must be satisfied for all appropriate combinations of $\Delta\sigma$ found in Equation C.8. Then, for the gate to hang plumb, the following equation must be satisfied:

$$\sum \boldsymbol{T_{DL}} + \sum_{\boldsymbol{i=1}}^{\boldsymbol{n}} 2ht{\sigma_0}_{\boldsymbol{i}} A_i \boldsymbol{cos}(\boldsymbol{\psi})_{\boldsymbol{i}}) = \mathbf{0} \qquad \text{(Equation C.14)}$$

Where:

T_{DL} refers to the torsion induced by dead load of the gate. T_{DL} is taken as the weight of the gate times the horizontal distance between the center of gravity of the gate and the shear center, times the horizontal distance from the pintle to the center of gravity. Note that, for plumbing the gate, self-weight should be taken as unfactored, service-level loads. That is:

$\mathrm{T_{DL} = P_{DL} z_{DL} y_{DL}}$ (Equation C.15)

The exact location of z_{DL} and y_{DL} would require an extensive numerical model of the individual gate. In as much as this method of calculating required stress is already imperfect, simply utilizing the distance from the shear center to the centerline of the girders is sufficiently accurate for the location y_{DL}, and the distance from the pintle to the centerline of the gate is sufficiently accurate to use for z_{DL}. The diagonals meet their intended function when Equation C.13 and Equation C.14 are satisfied simultaneously.

The designer should take note that there are infinite combinations of diagonal stresses that will satisfy these equations. Accordingly, the designer should choose an appropriate value for σ_0 for one set of diagonals that satisfies Equation C.13, then solve for the other σ_0 using Equation C.14, and iterate until both equations are simultaneously satisfied. To minimize fatigue issues, the designer should target a minimum value of required prestress.

C.2.9. **Step 7**: Calculate Factored Force in Diagonals for Chapter 9 LRFD Steel Design.

For LRFD steel design of miter gate components, it is advantageous to know the maximum expected forced that will develop in a diagonal.

Calculating the force for steel design is done by summing the required initial prestress, ${\sigma_0}_i$, found considering the *unfactored service-level* self-weight of the gate with the *load factored* change in stress due to a load combination considering only live loads. The force in a diagonal due to a live load (LL) load combination (LC) is thus:

$$\left(P_{diagonal_i}\right)_{LC} = A_i\left({\sigma_0}_i + ((\Delta\sigma_i)_{LL})_{LC}\right)$$ (Equation C.16)

The $P_{diagonal}$ must be calculated for each diagonal and each *load factored* load combination considered in design, and the critical values should be used.

The *load factored* change in stress due to a load combination can be calculated as follows:

$$(\boldsymbol{\Delta\sigma_i})_{\boldsymbol{LL}} = \frac{\boldsymbol{K_{diag}}}{\boldsymbol{A_i}}\left(\frac{(\sum \boldsymbol{T_z})_{\mathbf{LL}}}{\boldsymbol{Q_0} + \sum_{\boldsymbol{j=1}}^{\boldsymbol{n}} \boldsymbol{Q_j}}\right)$$ (Equation C.8, Repeated)

This change in stress is calculated for each *i*th diagonal. In the denominator of this equation, the sum of *Q* over *j* is performed for all *n* diagonals on the gate. The important note in this equation is that the torsional load combinations of factored loads must neglect the service-level self-weight (if that is included in the load combination). Explicitly,

$$\sum \boldsymbol{T_z} = \boldsymbol{LC} - \mathbf{1.0}\boldsymbol{SW}$$ (Equation C.17)

Where:

LC refers to any "load combination" of *factored loads* that is considered for diagonal design and SW is self-weight. For example, consider the following typical load combination:

$$LC = 1.2D + 1.6(C + M) + 1.3Q \qquad \text{(Equation C.18)}$$

Calculating the change in stress in the diagonals using Equation C.8 based on the above load combination, the following should be used for the applied torsional loads:

$$\sum T_z = (1.2D + 1.6(C + M) + 1.3Q) - 1.0D \qquad \text{(Equation C.19)}$$

Simplifying:

$$\sum T_z = 0.2D + 1.6(C + M) + 1.3Q \qquad \text{(Equation C.20)}$$

C.2.10. Derivation of New Equations from Existing Equations. The work by Shermer (1951), subsequently adopted by the USACE, for miter gate diagonal design was derived in terms of deflections of the gate. It is presumed that this is the case because a prominent method of tensioning miter gate diagonals was to physically twist the gate using the calculated deflection to induce slack in the diagonals, then a turn-buckle was turned to eliminate the slack. Modern practice tends to utilize tension values directly by means of strain gages. Thus, in this updated guidance, the equations have been rewritten to be streamlined and to be in terms of stress. Here, the derivation of the equations shown throughout section C.2 is provided and shown with regards to the historical calculations used by the USACE and provided in section C.4 for historical context.

C.2.10.1 Equation C.1.

Equation C.1 is taken directly from Equation C.96 in section C.4, which is in turn taken directly from Appendix B in Shermer's thesis. Design engineers have known for some time that this equation greatly underestimates the torsional stiffness of modern miter gates, and this was definitively shown by Eick and Smith in their investigation into the torsional behavior of modern miter gates. Equation C.96 in section C.4 is corrected by an empirical factor K of 4.0, derived from dated experiments that may not adequately represent modern fabrication practices.

Using data from modern miter gate deflections, Eick and Smith found that a conservative value of $K = 250$ can be used to more accurately capture the torsional stiffness of modern miter gates. Equation C.1 abandons the use of J_v, because there is no physical reasoning to include this. J_v would be a quantity used for twist of the gate if it is (for example) supported continuously about the bottom girder and twisting about a vertical axis that passes through the center of the gate.

C.2.10.2 Equation C.2.

Equation C.2 is derived from Equation 8 in Shermer's thesis, which is similar to Equation C.81 in section C.4. In Shermer's thesis, Equation 3.11 states:

$$\mathrm{R_0} = \pm\frac{\mathrm{2wht}}{\mathrm{Hvm}} \qquad \text{(Equation C.21)}$$

Where *m* (as defined by Shermer) is the length of the diagonal dimension of the diagonal panel. Equation C.81 in Section C.4 states:

$$\mathrm{R_0} = \pm\frac{\mathrm{2wt}}{\mathrm{vm}} \qquad \text{(Equation C.22)}$$

As seen, Shermer has an additional ratio of *h/H*, where *h* is the height of the diagonal panel and *H* is the height of the entire miter gate. For the vast majority of miter gates, this ratio is basically equal to 1.0, and so it is assumed that the historical calculations provided in section C.4 simply remove that ratio. However, for very tall miter gates (e.g., the downstream miter gate at The Dalles) there may be several diagonal panels oriented vertically, and so *h/H* cannot be assumed to be 1.0. This is addressed later in section C.4 Equation C.81' (section C.4.7).

There is no reason to have two separate equations, since including the ratio *h/H* is appropriate for all miter gates. A further adjustment is made in this update in order to define the positive and negative sign of R_0. The sign of R_0 was defined based on the definition of the diagonal. This leads to potential confusion of which diagonal should have a positive and a negative R_0. To eliminate potential confusion, positive and negative signs are determined based on the cosine of the angle, and so the term *w/m* simply becomes $\cos\psi$, so that:

$$\mathrm{R_0} = \gamma\cos\psi \qquad \text{(Equation C.23)}$$

$$\gamma = \frac{\mathrm{2ht}}{\mathrm{Hv}} \qquad \text{(Equation C.2, Repeated)}$$

Using this updated definition, along with the definition of the miter gate angles, will form a mathematical basis for which constants are positive and which are negative. In this update, the constants *R* and R_0 are abandoned because an *R* already has a well-established scientific use (i.e., the universal gas constant), and explicitly including the $\cos\psi$, coupled with studious definition of the angle of miter gate diagonals, will provide the appropriate sign (positive or negative) for calculated values.

C.2.10.3 Equation C.4.

Equation C.4 is taken from section C.4 Equation C.95. Section C.4 states that Equation C.95 is found by equating Equation C.85a and Equation C.87, substituting *sA* for *S*, and summing for diagonals in a set. Taking the stated steps, the following equation is found:

$$\sum\frac{1}{2}\mathrm{sAR_0}(\Delta - \mathrm{D}) = \sum\frac{1}{\mathrm{2hv}}\mathrm{T_z}(\Delta - \mathrm{D}) \qquad \text{(Equation C.24)}$$

$$\sum\mathrm{A} = \frac{\sum\mathrm{T_z}}{\mathrm{R_0hvs}} \qquad \text{(Equation C.25)}$$

In this update, continue with substituting R_0 with $\boldsymbol{\gamma}\cos\psi$, and substitute the variable *s* with the standard variable used for stress: σ. Thus,

$$\sum\mathrm{A} = \frac{\sum\mathrm{T_z}}{\gamma\mathrm{hv}\,\sigma_\mathrm{y}\cos\psi} \qquad \text{(Equation C.26)}$$

C.2.10.4 Equation C.5.

Equation C.5 is taken directly from Equation C.88 in section C.4. Note that, in the derivation of Equation C.88 provided in section C.4, *h* (the height of the diagonal panel) is incorrectly used, where it should be *H* (the height of the entire gate). This equation is correctly written in Equation 25 in Shermer's thesis. For most miter gates, *h* is either equal to or very close to *H* so this is not an issue. However, for very tall miter gates with panels separated vertically (e.g., The Dalles), the difference in value is consequential. Substituting *H* into Equation C.88, it is found:

$$Q = \frac{RR_0 EAHv}{L} \qquad \text{(Equation C.27)}$$

Explicitly writing out all the values in this equation, it is found that:

$$Q = \frac{A'}{A'+A}\left(\frac{2wht}{Hvm}\right)^2 Hv\,\frac{EA}{L} \qquad \text{(Equation C.28)}$$

From previous definitions of γ, it is seen that:

$$\left(\frac{2wht}{Hvm}\right)^2 = \gamma^2 \cos^2 \psi \qquad \text{(Equation C.29)}$$

Substituting it is seen that:

$$Q = \frac{A'}{A'+A}\gamma^2 Hv\frac{EA}{L}\cos^2 \psi \qquad \text{(Equation C.30)}$$

Here, it is noted that:

$$Hv\gamma = 2ht \qquad \text{(Equation C.31)}$$

Substituting this, it is seen that:

$$Q = 2ht\frac{A'}{A'+A}\gamma\frac{EA}{L}\cos^2 \psi \qquad \text{(Equation C.32)}$$

Here, some variables directly related to the stiffness of the diagonals are consolidated, as they will be used for additional calculations. The grouping of these variables together assists in intuitive understanding of the behavior of the diagonals, as the value has units of stiffness. Thus, let the stiffness of the diagonals, K_{diag}, be defined as:

$$K_{diag} = \gamma\frac{A'}{A'+A}\frac{EA}{L}\cos \psi \qquad \text{(Equation C.33)}$$

Substituting this value, arrives at Equation C.5:

$$Q = 2htK_{diag}\cos(\psi) \qquad \text{(Equation C.5, Repeated)}$$

C.2.10.5 Equation C.8.

Equation C.8 is taken from Equation C.85 and Equation C.93 in section C.4, which are both implementations of Hooke's Law relating force *F* of a linearly elastic system to the displacement of the system by a proportionality constant *k* (referred to as stiffness). Formally:

$$F = ku \qquad \text{(Equation C.34)}$$

or equivalently,

$$u = \frac{F}{k} \qquad \text{(Equation C.35)}$$

Equation C.93 is a manipulation of Hooke's law to relate the torsional deflection of the miter end of a miter gate to the applied torsion on the gate. Using the notation of Hooke's law shown above for Equation C.93, the force *F* is the applied live load torsions from a particular load combination:

$$F = \sum(T_z)_{LL} \qquad \text{(Equation C.36)}$$

The displacement *u* is the torsional deflection of the miter block,

$$u = \Delta \qquad \text{(Equation C.37)}$$

and the stiffness, *k*, is the torsional stiffness of the gate itself, plus the stiffness added to the gate by each of the *n* pretensioned diagonals:

$$k = Q_0 + \sum_{i=1}^{n} Q_i \qquad \text{(Equation C.38)}$$

Plugging Equations (3), (4), and (5) into Equation (2), Equation C.93 is found as:

$$\Delta = \frac{(\sum T_z)_{LL}}{Q_0 + \sum Q} \qquad \text{(Equation C.93, Shown Early Here)}$$

Equation C.85 is also an implementation of Hooke's law for the axial extension of a beam, where the beam in this case is the miter gate diagonal. It is important to note that this equation relates the force in a particular diagonal to the torsional deflection of the entire gate, and so it must be calculated diagonal-by-diagonal. Using the notation for Hooke's law, the force *F* is the axial force in the diagonal:

$$F = S \qquad \text{(Equation C.39)}$$

The stiffness, *k*, is the axial stiffness of the diagonal:

$$k = \frac{EA}{L} \qquad \text{(Equation C.40)}$$

The displacement, *u*, is the axial displacement of the diagonal. While not explicitly true, it is helpful to consider the unique quantity of the diagonal *R* as a conversion factor relating the torsional deflection of the gate delta to the axial extension of the diagonal, such that *u* in Hooke's Law takes the form:

$u = R(\Delta - D)$ (Equation C.41)

Where:

D is the deflection of the gate required to cause a diagonal to go slack and lose its pretension and is unique to each diagonal. Plugging the relevant values into Hooke's Law, Equation C.85 is found (adding in subscript *i* as appropriate, to highlight that this equation is for an individual diagonal):

$$S_i = \frac{R_i E A_i}{L_i}(\Delta - D_i) \quad \text{(Equation C.42)}$$

The value of *D* for each diagonal was a historically important quantity to determine, as this was the deflection required to impart on the gate to obtain the necessary pretension in the diagonals using the so-called "twist of the leaf" method of tensioning. This method of tensioning is rarely, if ever, used to tension diagonals anymore. Instead, a force or stress quantity is specified, and tension is applied to the diagonals via direct mechanical means such as turnbuckles or jack-bolt-style tensioning nuts. Thus, it is more advantageous to design engineers to eliminate the need to calculate *D* and keep everything in force or stress units. This is performed by finding the change in force in the diagonals due to a change in torsional deflection by simply differencing Equation C.42:

$$S_{i_2} - S_{i_1} = \frac{R_i E A_i}{L_i}((\Delta_2 - D_i) - (\Delta_1 - D_i)) \quad \text{(Equation C.43)}$$

Thus, the change in force in a diagonal due to a change in torsional deflection of the gate is:

$$\Delta S_i = \frac{R_i E A_i}{L_i}(\Delta_2 - \Delta_1) \quad \text{(Equation C.44)}$$

Note, this equation is valid for any change in deflection on the gate such that the behavior of the material of the gate and diagonals remains in the linear elastic regime. In Step 5 of the design phase, it is of particular interest to find the change in force (or stress) in a diagonal due deflections of the gate caused by live loads applied to the gate after the pretensioning procedure when the gate is hanging plumb. In this case, when the gate is hanging plumb, $\Delta_1 = 0$ (that is, there is no torsional deflection on the gate). And so, Δ_2 is found via Equation C.93 above, and plugged into Equation C.44 such that:

$$\Delta S_i = \frac{R_i E A_i}{L_i}\left(\frac{(\sum T_z)_{\boldsymbol{LL}}}{Q_0 + \sum Q}\right) \quad \text{(Equation C.45)}$$

In this update, in order to highlight the relationship to Hooke's law, the stiffness terms are grouped together and designated as a variable using the standard notation for stiffness, *K*:

$$K_{diag_i} = \frac{R_i E A_i}{L_i} \quad \text{(Equation C.46)}$$

Plugging Equation C.46 into Equation C.45:

$$\Delta S_i = K_{diag_i}\left(\frac{(\sum T_z)_{LL}}{Q_0+\sum Q}\right) \quad \text{(Equation C.47)}$$

Often time, it is more advantageous for a design engineer to work in terms of stress, and so, Equation C.47 is divided by the area of the diagonal to obtain the change in stress due to the application of live loads to the gate:

$$(\Delta\sigma_\mathrm{i})_{LL} = \frac{K_{diag_i}}{A_i}\left(\frac{(\sum T_z)_{LL}}{Q_0+\sum Q}\right) \quad \text{(Equation C.8, Repeated)}$$

C.2.10.6 Equation C.14.

Equation C.14 is taken from Equation C.85 and Equation C.92 in Section C.4, which are the same as Equation 23 and Equation 30 in Shermer's thesis. Equation C.92 states:

$$\sum \mathrm{T_{DL}} + \sum(\mathrm{QD}) = 0 \quad \text{(Equation C.48)}$$

Equation C.85 is repeated again (making the same substitutions as above):

$$\mathrm{S} = \frac{\mathrm{REA}}{\mathrm{L}}(\Delta - \mathrm{D}) = \mathrm{K_{diag}}(\Delta - \mathrm{D}) \quad \text{(Equation C.49)}$$

For the gate to hang plumb, Δ = 0. Thus, when the gate hangs plumb, the force in the diagonal is found as:

$$\mathrm{S} = \mathrm{K_{diag}}(-\mathrm{D}) \quad \text{(Equation C.50)}$$

or, rewriting:

$$\frac{-\mathrm{S}}{\mathrm{K_{diag}}} = \mathrm{D} \quad \text{(Equation C.51)}$$

Substituting into Equation C.92:

$$\sum \mathrm{T_{DL}} - \sum \mathrm{Q}\frac{\mathrm{S}}{\mathrm{K_{diag}}} = 0 \quad \text{(Equation C.52)}$$

Recall the equation for Q:

$$\mathrm{Q} = 2\mathrm{htK_{diag}}\cos(\psi) \quad \text{(Equation C.53)}$$

Substituting this into C.52, and rewriting *S* in terms of stress (i.e., S = σ*A*):

$$\sum \mathrm{T_{DL}} - \sum 2\mathrm{htK_{diag}}\cos(\psi)\frac{\sigma\mathrm{A}}{\mathrm{K_{diag}}} = 0 \quad \text{(Equation C.54)}$$

Simplifying:

$$\sum \mathrm{T_{DL}} + \sum 2\mathrm{ht}\sigma\mathrm{A}\cos(\psi) = 0 \quad \text{(Equation C.14, Repeated)}$$

C.3. Design Example: Horizontally Framed Gate Design Example.

Horizontally Framed Miter Gate at Lock and Dam 27, Mississippi River.

The miter gate at Lock and Dam 27 is a horizontally framed gate with 13 girders. Each girder has 2 longitudinal stiffeners welded to the web. The cross section used for determining geometric properties of the gate is shown in Figure C.3.

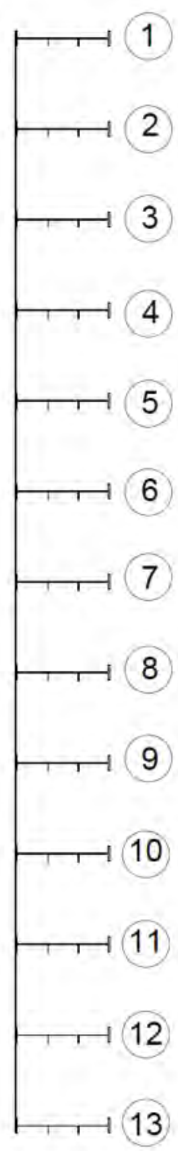

Figure C.3. Cross section of miter gate at Lock 27

For this miter gate, the following values are used in diagonal design:

Table C.2.
Diagonal Variables for Lock 27 Miter Gate

Variable* designation	Value
h	834 in
H	834 in
L	911.625 in
t	60.25 in
v	716 in
w	603 in
$\boldsymbol{\psi}$ **primary diagonal**	125.868 degrees
$\boldsymbol{\psi}$ **secondary diagonal**	54.132 degrees
Weight	460 kips

*See Figure C.1 for variable definitions

C.3.1. Distance to Center of Gravity from Skin Plate.

To find an estimate of the distance from the centerline of the skin plate to the center of gravity, a typical horizontal girder cross-section is used (including the longitudinal stiffeners and the cross section of the diagonals in their appropriate place) and is shown in Figure C.4. Where dimensions differ across horizontal girders (e.g., the flange widths and thicknesses), the dimensions are averaged across all girders. X-bar is found using C.55 and the values shown in Table C.1 and Figure C.1. Note, the values for the area of the diagonals are taken as a first guess based on other similar miter gates.

$$\mathbf{\bar{x}} = \frac{\sum_{\mathbf{i}}^{\mathbf{n}} \mathbf{A_i x_i}}{\sum_{\mathbf{i=1}}^{\mathbf{n}} \mathbf{A_i}} \qquad \text{(Equation C.55)}$$

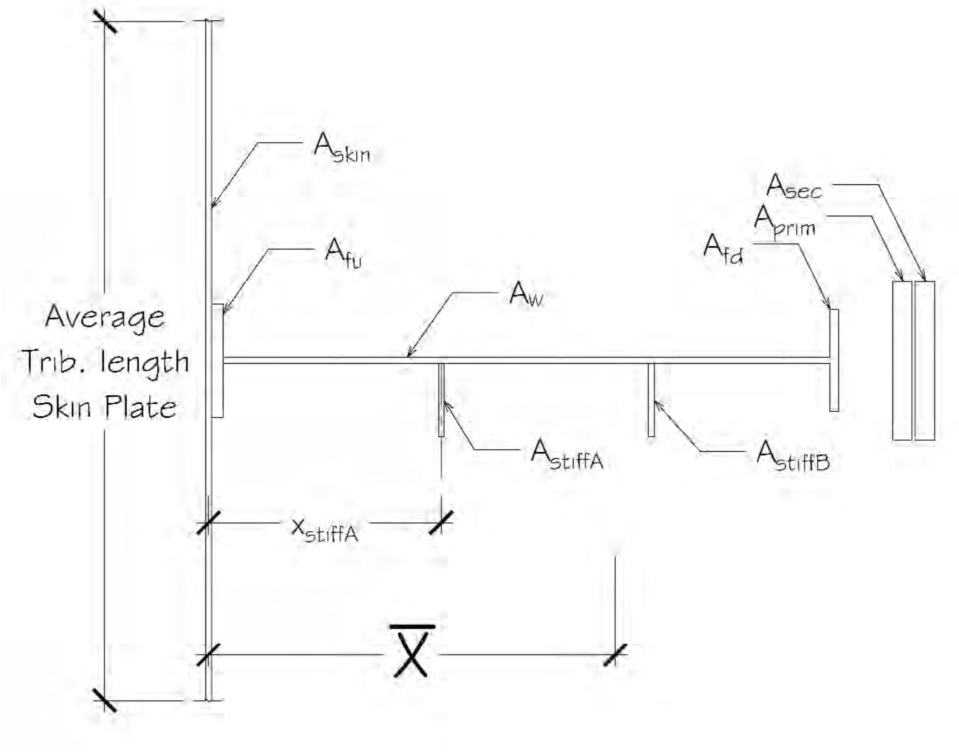

Figure C.4. Representative cross section used to find x-bar

Table C.3.
Values Used to Find X-Bar

Component	Average area	Average x
Skin Plate (skin)	32.1	0
Upstream Flange (*fu*)	10.69	0.5
Web (*w*)	27.5	28.5
Downstream Flange (*fd*)	9	56.375
Stiffener A (*stiffA*)	4.0625	20.5
Stiffener B (*stiffB*)	4.0625	39
Primary Diagonal (*prim*)	24.5	60.25
Secondary Diagonal (*Sec*)	24.5	60.25
x-bar (Equation C.39)	32.9 in. from skin plate	

C.3.2. Moment of Inertia (I_{xx}).

The moment of inertia about the horizontal axis in Figure C.3 is found by first principles using the dimensions of the components of the cross section, as listed in Table C.4.

Table C.4.
Relevant Dimensions for Lock 27 Miter Gate, All Values in Inches

Girder No	b_{fu}	t_{fu}	b_{fd}	t_{fd}	t_w	h_w	t_{stiffA}	h_{stiffA}	t_{stiffB}	h_{stiffB}
1	10	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
2	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
3	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
4	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
5	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
6	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
7	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
8	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
9	9	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
10	15	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
11	15	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
12	15	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
13	12	1	12	0.75	0.5	54.5	0.625	6.5	0.625	6.5
Skin	834	0.5								

First the vertical distance to the geometric centroid is found, using the centerline of girder 13 as the datum. Equation C.55 is again utilized with the values in:

Table C.5.
Dimensions Used to Find Vertical Position of Centroid

Girder no	A	d_{cg}
1	54.375	834
2	53.375	762
3	53.375	690
4	53.375	618
5	53.375	546
6	53.375	474
7	53.375	402
8	53.375	330
9	53.375	258
10	59.375	189
11	59.375	126
12	59.375	63
13	56.375	0
Skin	417	417
Centroid	405.6 inches from Girder 13	

To find I_{xx}, the weak axis moment of inertia is found for each girder. Then, the parallel axis theorem is used measuring the distance from the centerline of each girder to the centroid.

Table C.6.
Calculating the Moment of Inertia for Lock 27

Girder no	I_{weak}	A	d	Ad^2
1	306.33	54.625	428.43	9981008
2	283.75	53.625	356.43	6781174
3	283.75	53.625	284.43	4318291
4	283.75	53.625	212.43	2408799
5	283.75	53.625	140.43	1052700
6	283.75	53.625	68.43	249992
7	283.75	53.625	-3.566	677
8	283.75	53.625	-75.56	304753
9	283.75	53.625	-147.56	1162222
10	504.25	59.625	-216.56	2784644
11	504.25	59.625	-279.56	4640461
12	504.25	59.625	-342.56	6967596
13	367	56.625	-405.56	9272608
Skin	24170571	417	-11.43	54510
Sum	24175027.08			49979482
Ixx	74154509.32 in^4			

C.3.3. Find Shear Center (e).

The location of the shear center is also found using first principles. For horizontally framed miter gates, an acceptable estimation of the location of the shear center from the centerline of the skin plate can be found by utilizing the following:

$$e = \frac{1}{I_{xx}} \sum_{i=1}^{n} \left(\frac{1}{2} A_{w_i} + \frac{1}{2} A_{stiffA_i} + \frac{1}{2} A_{stiffB_i} + \frac{1}{2} A_{stiffM_i} + A_{fd_i}\right) h_{w_i} d_i^2 \qquad \text{(Equation C.56)}$$

Where *n* in the summation refers to the number of girders on the gate, h_w is the depth of the girder web (inside-to-inside distance between girder flanges). *M* longitudinal stiffeners are shown in Equation C.56 to accommodate the possibility of any number of stiffeners, but for Lock 27, there are only two. Note, Equation C.56 employs a great deal of simplifications but provides an adequate representation of the location of the shear center for initial hand calculations. Employing the dimensions shown in Table C.2, the shear center for Lock 27 is found to be 17.56 in. upstream from the skin plate.

Table C.7.
Locating the Shear Center for Lock 27

Girder	$\frac{1}{2}A_w$	$\frac{1}{2}A_{stiffA}$	$\frac{1}{2}A_{stiffB}$	A_{f_d}	h_w	d
1	13.625	2.03	2.03	9.0	54.5	428.43
2	13.625	2.03	2.03	9.0	54.5	356.43
3	13.625	2.03	2.03	9.0	54.5	284.43
4	13.625	2.03	2.03	9.0	54.5	212.43
5	13.625	2.03	2.03	9.0	54.5	140.43
6	13.625	2.03	2.03	9.0	54.5	68.43
7	13.625	2.03	2.03	9.0	54.5	-3.56
8	13.625	2.03	2.03	9.0	54.5	-75.56
9	13.625	2.03	2.03	9.0	54.5	-147.56
10	13.625	2.03	2.03	9.0	54.5	-216.56
11	13.625	2.03	2.03	9.0	54.5	-279.56
12	13.625	2.03	2.03	9.0	54.5	-342.56
13	13.625	2.03	2.03	9.0	54.5	-405.56
I_{xx}						74154509.32
e *(Equation C.56)*						17.56

C.3.4. Calculate J.

J, the torsion constant of the cross-section, is found by first principles. The cross-section of the miter gate is comprised of narrow rectangles, and so, *J* can be found by the following equation:

$$\boldsymbol{J = \frac{1}{3}\sum_{i=1}^{n} b_i t_i^3} \qquad \text{(Equation C.57)}$$

Where *n* here refers to every rectangular member of the miter gate cross section (e.g., the cross section of the skin plate, all girder flanges, webs, stiffeners, etc.), *b* is always the longer dimension of the rectangle, and *t* is always the shorter dimension.

Table C.8.
Calculating J for Lock 27

Girder	$b_i t_i^3$ Upstream Flange	Downstream Flange	Web	Stiffener A	Stiffener B
1	10	5.0625	6.8125	1.586914	1.586914
2	9	5.0625	6.8125	1.586914	1.586914
3	9	5.0625	6.8125	1.586914	1.586914
4	9	5.0625	6.8125	1.586914	1.586914
5	9	5.0625	6.8125	1.586914	1.586914
6	9	5.0625	6.8125	1.586914	1.586914
7	9	5.0625	6.8125	1.586914	1.586914
8	9	5.0625	6.8125	1.586914	1.586914
9	9	5.0625	6.8125	1.586914	1.586914
10	15	5.0625	6.8125	1.586914	1.586914
11	15	5.0625	6.8125	1.586914	1.586914
12	15	5.0625	6.8125	1.586914	1.586914
13	12	5.0625	6.8125	1.586914	1.586914
Skin					104.25
J (Equation C.57)					146.2949

C.3.5. Calculate Q_0.

Using Equation C.1:

$$Q_0 = \frac{\alpha GJ}{H} = \frac{250 \times 11200 \frac{kips}{in^2} \times 146.29 in^4}{834 in} = 491{,}158\ kips \cdot in \quad \text{(Equation C.58)}$$

Calculate γ using Equation C.2.

$$\gamma = \frac{2ht}{Hv} = \frac{2 \times 834 in \times 60.25 in}{834 in \times 716 in} = 0.1683 \quad \text{(Equation C.59)}$$

C.3.6. Calculate Torsion for All Considered Loads.

This is best performed in tabular form. For self-weight, the moment arm is equal to the distance from the center of gravity to the shear center. For *z* of the dead load, it is sufficiently accurate to simplify the calculation by assuming the center of gravity is ½ *v*. For strut arm loads, the critical moment arm occurs when an obstruction is considered at the bottom corner at the miter end of the gate, and so, the moment arm is the vertical distance from the bottom of the gate to the strut arm. To find *z*, the structural drawings must be consulted. The loads below are shown for demonstration purposes only.

Table C.9.
Loads Considered for Diagonal Design

Description	P	y	z	Pyz
Self-weight (D)	-490 kips	50.46 in	358 in	-8,851,693 kip-in^2
Ice (C)	-100 kips	50.46 in	358 in	-1,806,468 kip-in^2
Mud (M)	-100 kips	50.46 in	358 in	-1,806,468 kip-in^2
Gravity (G = C + M)	-200 kips	50.46 in	358 in	-3,612,936 kip-in^2
Machinery Opening (Q)	150 kips	846 in	177 in	22,461,300 kip-in^2
Machinery Closing (Q)	-150 kips	846 in	177 in	-22,461,300 kip-in^2

Combine loads using load combinations in Chapter 9 for the gate in the open position with the load factors removed:

Load Combination 2a: D = -8,851,693 kip-in^2

Load Combination 2b: D + G = -12,464,629 kip-in^2

Load Combination 2c: D + G + Q (closing) = -34,925,929 kip-in^2

Load Combination 2c: D + G + Q (opening) = 9,996,670 kip-in^2

Load Combination 2c: D + 0.0G + Q (closing) = -31,312,993 kip-in^2

Load Combination 2c: D + 0.0G + Q (opening) = 13,609,607 kip-in^2

$Max\ Pos = 13{,}609{,}607$ kip-in^2

$Max\ Neg = -34{,}925{,}929$ kip-in^2

C.3.7. Calculate Required Area.

From Equation C.4, using the max negative torsion for the primary diagonal design:

$$\sum A = \frac{\sum T_z}{\gamma h v\, \sigma_y \cos\psi} \qquad \text{(Equation C.60)}$$

$$\sum A = \frac{-34{,}925{,}929\text{kip}*\text{in}^2}{0.1683*834\text{in}*716\text{in}*\left(0.6*50\frac{\text{kip}}{\text{in}^2}\right)*\cos 125.87} = 19.77\text{in}^2 \qquad \text{(Equation C.61)}$$

Using the max positive torsion for the secondary diagonal:

$$\sum A = \frac{13{,}609{,}607\text{kip}*\text{in}^2}{0.1683*834\text{in}*716\text{in}*\left(0.6*50\frac{\text{kip}}{\text{in}^2}\right)*\cos 54.132} = 7.70\text{in}^2 \qquad \text{(Equation C.62)}$$

For this example, we chose to use the same area for the primary and secondary diagonal (19.8 in^2).

C.3.8. Calculate Q for Each Diagonal.

Calculate A', for which A_p is required:

$$A' = \frac{1}{8} A_p \quad \text{(Equation C.63)}$$

Table C.10.
Calculation of A' for Lock 27

	A_{fu}	A_{fd}	A_w	A_{stiff_A}	A_{stiff_B}	$\sum A$
Girder 1	10	9	27.25	4.0625	4.0625	54.375
Girder 13	12	9	27.25	4.0625	4.0625	56.375
Diaphragm Right	0	6	34.6875	0	0	40.6875
Diaphragm Left	0	6	34.6875	0	0	40.6875
Sum, A_p						192.125
A'						24.01

From Equation C.6:

$$\mathrm{K_{diag}} = \frac{\mathrm{A'}}{\mathrm{A'} + \mathrm{A}} \gamma \frac{\mathrm{EA}}{\mathrm{L}} \cos(\psi)$$

For primary diagonals:

$$\mathrm{K_{diag}} = \frac{24}{24 + 19.8} 0.1682 \frac{29000 * 19.8}{911.625} \cos(125.868)$$

$$\mathrm{K_{diag}} = -34.016 \frac{\mathrm{kips}}{\mathrm{in}}$$

For secondary diagonals:

$$\mathrm{K_{diag}} = \frac{24}{24 + 19.8} 0.1682 \frac{29000 * 19.8}{911.625} \cos(54.132)$$

$$\mathrm{K_{diag}} = 34.016 \frac{\mathrm{kips}}{\mathrm{in}}$$

Then, from Equation C.5:

$$\mathrm{Q} = 2\mathrm{htK_{diag}} \cos(\psi)$$

$$\mathrm{Q} = 2 \times 834\mathrm{in} \times 60.25\mathrm{in} \times 34.016 \frac{\mathrm{kips}}{\mathrm{in}} \cos(54.132)$$

$$Q = 2{,}002{,}950 \; kips \cdot in$$

This value is the same for both primary and secondary diagonals, as the same area and length are chosen, and the cosine of the angles are simply opposite each other.

C.3.9. Calculate the Change in Stress for Each Applied Torsion.

From Equation C.8, the change in stress due to the *unfactored* loads in the diagonals is found as:

$$\Delta\sigma_i = \frac{K_{diag}}{A}\left(\frac{\Delta Pxy}{Q_0 + \sum Q}\right) \quad \text{(Equation C.64)}$$

Here, only the loads other than the self-weight of the gate are considered. The maximum negative load occurs with the following combinations:

$$G + Q\text{-close} = -26{,}074{,}236 \text{ kip-in}^2 \quad \text{(Equation C.65)}$$

The maximum positive load occurs when ice and mud are not considered as:

$$Q\text{-open} = 22{,}461{,}300 \text{ kip-in}^2 \quad \text{(Equation C.66)}$$

As an example, the increase in stress in the secondary diagonals due to the maximum positive load is found as:

$$\Delta\sigma_i = \frac{34.02}{19.8}\left(\frac{22{,}461{,}300}{491158 + 2(2{,}002{,}950)}\right)$$

$$\Delta\sigma_i = 8.59\ ksi \quad \text{(Equation C.67)}$$

Calculating the max increase and decrease in stress for all diagonals and all loads, the maximum expected change in stress is listed in Table C.11.

Table C.11.
Change in Stress for Both Diagonals

	Max Increase in Stress	Max Decrease in Stress
Primary Diagonal	9.98 ksi	8.59 ksi
Secondary Diagonal	8.59 ksi	9.98 ksi

C.3.10. Calculate Required Tension

For the gate to hang plumb, Equation C.13 and Equation C.14 must be satisfied simultaneously for all diagonals. This is performed by arbitrarily selecting an appropriate value for σ_0 for one set of diagonals, and then solving for σ_0 for the other set of diagonals. This process is iterated, changing the initially selected σ_0 until the design requirements for the diagonals are satisfied.

This iteration was performed many times in the design example until it was ultimately found that considering an initial stress of 19 ksi in the primary diagonal is sufficient to satisfy the design requirements for both sets of diagonals. For illustrative purposes, and for brevity, only this final iteration is shown, where a stress of 19 ksi is assumed in the primary diagonals. If, for example, a stress of 15 ksi is assumed in the primary diagonal, the stress in the secondary diagonal required to plumb the gate would be too low to satisfy Equation C.13 (the decrease in stress in the secondary diagonal shown in table C.11 would cause the diagonal to go slack), and so it would be determined that the initially assumed stress of 15 ksi is insufficient and should be increased.

$$\sum \boldsymbol{T_{DL}} - \sum \boldsymbol{2ht\sigma A}\mathbf{cos}\,(\boldsymbol{\psi}) = \mathbf{0} \qquad \text{(Equation C.14)}$$

When the diagonals are initially tensioned for the gate to hang plumb, no mud or ice will be present on the structure. Accordingly, only the self-weight of the gate is considered. Here, unfactored service-loads are used. The torsion induced by the self-weight of the gate is found as:

$$T_{DL} = -490kips \times 50.46in \times 358in = -8851693kips \cdot in^2$$

The torsion induced by 19 ksi of prestress in the primary diagonal is:

$$\boldsymbol{2h\sigma A}\,\mathbf{cos}(\boldsymbol{\psi}) = 2(834in)(60.25in)(19ksi)(19.8in^2)\cos(125.88)$$

$$\boldsymbol{2ht\sigma A}\,\mathbf{cos}(\boldsymbol{\psi}) = -22{,}151{,}855\ kips \cdot in^2$$

Then, Equation (C.14) is solved for the required tension in the secondary diagonal:

$$\sigma_{0\_secondary} = \frac{T_{DL} - T_{prim}}{2htAcos(\psi)}$$

$$\sigma_{0\_secondary} = \frac{(-8851693 - (-22{,}151{,}855))kips \cdot in^2}{2(834in)(60.25in)(19.8in^2)cos(54.132)}$$

$$\sigma_{0\_secondary} = 10.98\ ksi$$

Now, Equation C.13 is checked to ensure that the diagonals never go slack and the stress never exceeds yield. The initial stress values are combined with the maximum expected change in stress from Table C.12 such that:

$$\boldsymbol{1ksi} < \boldsymbol{\sigma_0} + (\Delta\boldsymbol{\sigma})_{\boldsymbol{LL}} < 0.6(\boldsymbol{F_y}) \qquad \text{(Equation C.13)}$$

Table C.12.
Stress Configuration for Diagonals of Lock 27

Diagonal	Initial stress	Initial + max decrease	Initial + max increase
Primary	19.0 ksi	10.4 ksi	29.0 ksi
Secondary	11.0 ksi	1.0 ksi	19.6 ksi

As seen, the selected values of pretension are sufficient to satisfy the design requirements for diagonals: The gate will hang plumb, the stress in the diagonals will not exceed the allowable yield stress, and the diagonals will not go slack during operation. There are infinitely many combinations of stress in the positive and negative diagonals that can be selected to satisfy the design requirements for diagonals. For the sake of keeping stresses as low as possible (for fatigue considerations) the designer should iterate results to obtain initial stress values as low as is reasonable. This iteration was already performed in this example, and only the final iteration is shown.

Note, if (from a fatigue perspective) the stresses in the diagonal are deemed too high for life-cycle considerations of the gate, the cross-sectional area of the diagonals can be increased. For example, it may be studious for the designer to keep stresses in the diagonals below a theoretical endurance limit of steel. If the cross-sectional area of the diagonals is increased, the design should be iterated over again to ensure appropriate results are maintained. For comparison, using section C.4, the as-built miter gate at Lock 27 was designed to have diagonals with a cross-sectional area of 24.5 in^2, with 17.8 ksi stress in the primary diagonal and 12.1 ksi stress in the negative diagonal. Converting stresses in the diagonal to forces, the values calculated in this example are of similar magnitude to the values on the as-built miter gate.

For miter gate fabrication or installation contracts, the designer may want to allow a range of initial pretension values such that the two main contract acceptance criteria can be achieved in the field: plumbness and keeping pretension during gate operation above 1.0 ksi.

C.3.11. Calculate Factored Force in Diagonals for Chapter 9 LRFD Steel Design.

Table C.13.
Unfactored Loads Considered for Diagonal Design

Description	P	y	z	Pyz
Self-weight (D)	-490 kips	50.46 in	358 in	-8,851,693 kip*in2
Ice (C)	-100 kips	50.46 in	358 in	-1,806,468 kip*in2
Mud (M)	-100 kips	50.46 in	358 in	-1,806,468 kip*in2
Gravity (G = C + M)	-200 kips	50.46 in	358 in	-3,612,936 kip*in2
Machinery Opening (Q)	150 kips	846 in	177 in	22,461,300 kip*in2
Machinery Closing (Q)	-150 kips	846 in	177 in	-22,461,300 kip*in2

Calculate the live load factored torsions (i.e., self-weight subtracted) using factored load combinations in Chapter 9 for the gate in the open position:

Load Combination 2a: 1.4D – 1.0D = 0.4D = -3,540,677 kip-in^2

Load Combination 2b: (1.2D – 1.0D) + 1.6G = -7,551,036 kip-in^2

Load Combination 2b: (0.9D – 1.0D) + 0.0G = 885,169 kip-in^2

Load Combination 2c: (1.2D – 1.0D) + 1.6G + 1.3Q (closing) = -36,750,726 kip-in^2

Load Combination 2c: (1.2D – 1.0D) + 1.6G + 1.3Q (opening) = 21,648,654 kip-in^2

Load Combination 2c: (0.9D – 1.0D) + 0.0G + 1.3Q (closing) = -28,314,521 kip-in^2

Load Combination 2c: (0.9D – 1.0D) + 0.0G + 1.3Q (opening) = 30,084,859 kip-in^2

$Max\ (\sum T_z)_{\mathrm{LL}}\ Pos =\ 30{,}084{,}859$ kip-in^2

$Max\ (\sum T_z)_{\mathrm{LL}}\ Neg =\ -36{,}750{,}726$ kip-in^2

Load factored increase in stress due to the governing load combinations:

$$(\Delta\sigma_i)_{LL}\ = \frac{K_{diag}}{A_i}\left(\frac{(\sum T_z)_{\mathrm{LL}}}{Q_0+\sum_{j=1}^{n} Q_j}\right) \qquad \text{(Equation C.68)}$$

Primary diagonal:

$$\left(\Delta\sigma_{primary}\right)_{LL}\ = \frac{-34.016\ kip/in}{19.8\ in^2}\left(\frac{-36{,}750{,}726\ \text{kip} * in^2}{491{,}158\ kips \cdot in + 2(2{,}002{,}950\ kips \cdot in)}\right)$$

$$\left(\Delta\sigma_{primary}\right)_{LL}\ = 14.1\ ksi$$

Secondary diagonal:

$$\left(\Delta\sigma_{secondary}\right)_{LL}\ = \frac{34.016\ kip/in}{19.8\ in^2}\left(\frac{30{,}084{,}859\ \text{kip} * in^2}{491{,}158\ kips \cdot in + 2(2{,}002{,}950\ kips \cdot in)}\right)$$

$$\left(\Delta\sigma_{secondary}\right)_{LL}\ = 11.5\ ksi$$

Factored load in diagonals:

$$\left(P_{diagonal_i}\right)_{LC} = A_i\left(\sigma_{0_i} + ((\Delta\sigma_i)_{LL})_{LC}\right)$$

Primary diagonal:

$$\left(P_{diagonal_{primary}}\right)_{LC}\ = 19.8 in^2(19.0\ \text{ksi} + 14.1\ \text{ksi}) = 655\ kip$$

Secondary diagonal:

$$\left(P_{diagonal_{secondary}}\right)_{LC}\ = 19.8 in^2(11.0\ \text{ksi} + 11.5\ \text{ksi}) = 445\ kip$$

Perform Steel Design Checks for the Various Cross-Sections of the Diagonal. Example of checking gross cross-section of the diagonal:

P_u = 655 kip (Equation C.69)

$\phi_t P_n$ = 0.9 (50 ksi) (19.8 in^2) = 891 kips (Equation C.70)

Therefore, capacity is sufficient.

Perform fatigue design of diagonal and diagonal components using unfactored service loads/stresses from Table C.12 and following Chapters 5 and 9. While designing the diagonals for a 100-year fatigue life is ideal, it may not be practically feasible to size the components as such due to structural clearance limitations. If 100-year fatigue life is not feasible, the diagonals may be designed for a minimum 50-year fatigue life with the realization that maintenance and/or replacement of the diagonals will be needed during the miter gate's 100-year service life.

C.4. Diagonal Design Historical and Theoretical Basis.

C.4.1. Diagonal Design. The following information is applicable to open frame gates and is essentially the same as that presented in "Torsional Deflection of Miter-Type Lock Gates and Design of the Diagonals" (USAED, Chicago, 1960) with only minor modifications.

C.4.2. Definitions of Terms and Symbols.

Deviations from these symbols are noted at the places of exception:

Δ = Total torsional deflection of the leaf measured, at the miter end, by the movement of the top girder relative to the bottom girder (see Figure C.5). The deflection is positive if the top of the miter end is moved upstream relative to the bottom.

Positive diagonal: A diagonal that decreased in length with a positive deflection of the leaf (see Figure C.8).

a = The cross-sectional area of that part of a horizontal girder that lies outside the midpoint between the skin and the flange (see Figure C.10).

A = Cross-sectional area of diagonal.

A' = Stiffness of the leaf in deforming the diagonal. Until more test data are available, it is suggested that A' be taken as the sum of the average cross-sectional areas of the two vertical and two horizontal girders that bound a panel times:

1/8 for welded horizontally framed leaves with skin of flat plates, and 1/20 for riveted vertically framed leaves with skin of buckle plate (see paragraph C.4.4.9.1).

b = Distance from the centerline of the skin plate to the flange of a horizontal girder (see Figure C.10).

c = The smaller dimension of a rectangular cross section.

d = Pitch diameter of the threaded portion of the diagonals.

D = Prestress deflection for a diagonal D is the deflection of the leaf required to reduce the stress in a diagonal to zero. D is always positive for positive diagonals and negative for negative diagonals.

E = Bending modulus of elasticity.

E_s = Shearing modulus of elasticity.

h = Height of panel enclosing diagonal.

H = Vertical height over which H is measured, usually the distance between top and bottom girders.

I = Moment of inertia about the vertical axis of any horizontal girder.

I_x = Moment of inertia, about the horizontal centroidal axis, of a vertical section through a leaf (see Figure C.9)

J = Modified polar moment of inertia of the horizontal and vertical members of the leaf.

K = A constant, taken equal to 4 (see paragraph C.4.4.9.2).

l = The larger dimension of a rectangular cross section.

L = Length of a diagonal, center to center of pins.

n = Number of threads per inch in sleeve nut of diagonal.

Q_0 = Elasticity constant of a leaf without diagonals (see paragraph C.4.4.9.2).

Q = Elasticity constant of diagonal defined by Equation C.88.

R_0 = Ratio of change in length of diagonal to deflection of leaf when diagonal offers no resistance (refer to Equation C.81). R_0 is positive for positive diagonals and negative for negative diagonals.

R = Ratio of actual change in length of diagonal to deflection of leaf (refer to Equation C.83). R is positive for positive diagonals and negative for negative diagonals.

s = Unit stress in diagonal.

S = Total force in diagonal.

t = Distance from centerline of skin plate to centerline of diagonal (for curved skin plate, see paragraph C.4.4.8).

T_z = Torque-area. Product of the torque T of an applied load and the distance z to the load from the pintle. z is measured horizontally along the leaf. T is positive if the load produces a positive deflection.

v = Distance from centerline of pintle to extreme miter end of leaf.

w = Width of panel (refer to Figure C.5).

X = Distance from centerline of skin plate to vertical shear center axis of leaf (refer to Equation C.100).

y = Distance to any horizontal girder from the horizontal centroidal axis of a vertical section through a leaf.

y_n = Distance to any horizontal girder from the horizontal shear center axis of a vertical section through a leaf.

Y = Distance to horizontal shear center axis from the horizontal centroidal axis of a vertical section through a leaf (refer to Equation C.97).

z = Distance longitudinally along the gate leaf between the load and pintle.

C.4.3. Introduction.

A lock gate leaf is a very deep cantilever girder with a relatively short span. The skin plate is the web of this girder. If the ordinary equations for the deflection of a cantilever under shearing and bending stresses are applied, the vertical deflection of the average leaf will be found to be only a few hundredths of an inch. Because the skin plate imparts such a great vertical stiffness to the leaf, the stresses in the diagonals are a function of only the torsional (twisting) forces acting on the leaf. These forces produce a considerable torsional deflection when the gate is being opened or closed. It is this torsional deflection and the accompanying stresses in the diagonals with which this chapter is concerned.

The shape of the twisted leaf is determined geometrically. Then the work done by the loads is equated to the internal work of the structure. From this, the resistance that each diagonal offers to twisting of the leaf is computed as a function of the torsional deflection of the leaf and the dimensions of the structure. Equations for torsional deflection of the leaf and stresses in the diagonals are derived.

Experiments were made on a model of the proposed gates for the MacArthur Lock at Sault Ste. Marie, MI. Tests were also conducted in the field on the lower gates of the auxiliary lock at Louisville, KY. Both experiments indicate that the behavior of a gate leaf is accurately described by the torsional deflection theory.

Examples of the application of the theory are presented together with alternate methods for prestressing the diagonals of a leaf.

C.4.4. Geometry. To make a torsional analysis of a lock gate, the geometry of the deflected structure must be known. The change in length of the diagonal members will be determined as a function of the torsional deflection of the leaf. For the present, the restraint offered by the diagonals will not be considered.

C.4.4.1 Diagonal Deformation. In Figures C.6 and C.7, the panel ak of Figure C.5 is considered separately. As the leaf twists the panel ak twists as indicated by the dotted lines. In Figure C.7, movements of all points are computed relative to the three reference axes gf, gb, and gk shown in Figure C.6. The girders and skin plate are free to twist, but they remain rectangles, except for second-order displacements. Therefore, the three reference axes are always mutually perpendicular. Let δ_0 equal the change in length of either diagonal of Figure C.7.

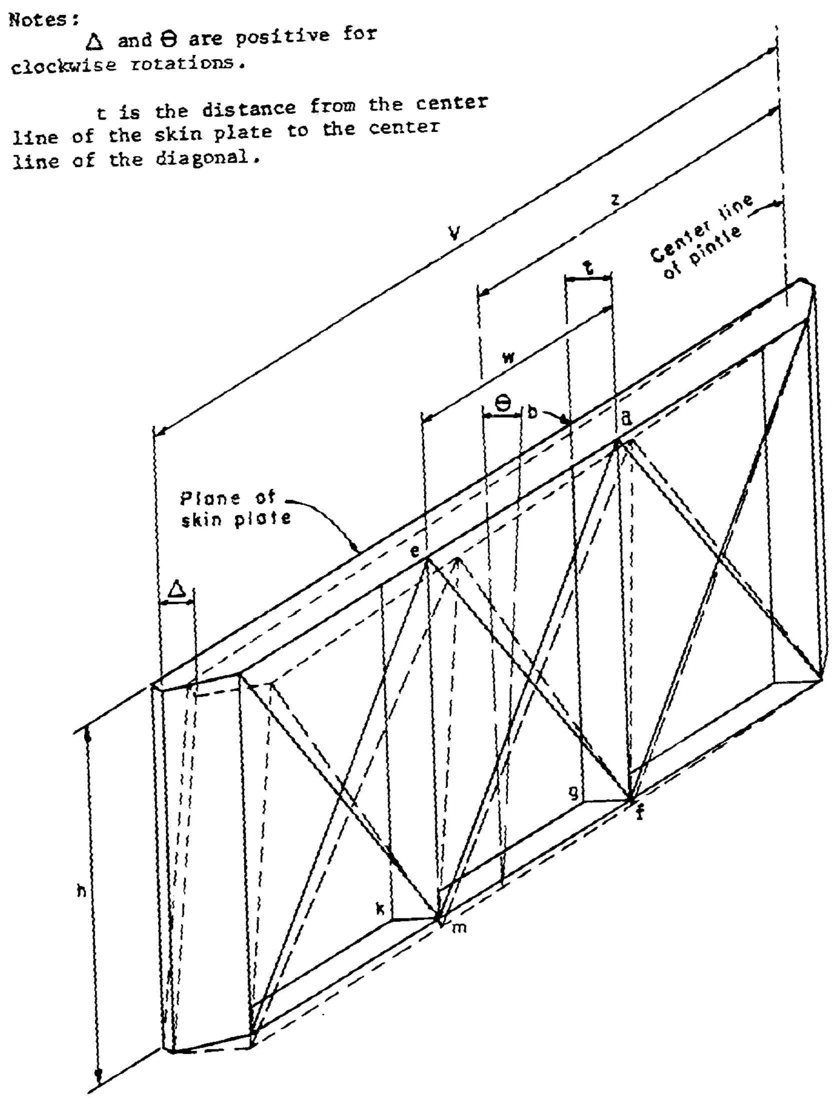

Figure C.5. Schematic Drawing of a Typical Miter-Type Lock Gate Leaf

$$\delta_o = \frac{d}{w} t \cos\alpha + \left(\frac{d}{h} t \sin\alpha \right)$$
$$= \frac{dt}{w} \frac{w}{(w^2+h^2)^{1/2}} + \frac{dt}{h} \frac{w}{(w^2+h^2)^{1/2}}$$
$$= \frac{2dt}{\dfrac{w}{(w^2+h^2)^{1/2}}}$$

(Equation C.71)

C.4.4.2 Sign Convention. For the necessary sign convention, let the deflection *d* be positive when the top of the leaf moves upstream in relation to the bottom. With a positive deflection, those diagonals that decrease in length are considered positive diagonals. With negative deflection, where the top of the gate moves downstream in relation to the bottom, those diagonals that decrease in length are considered negative diagonals.

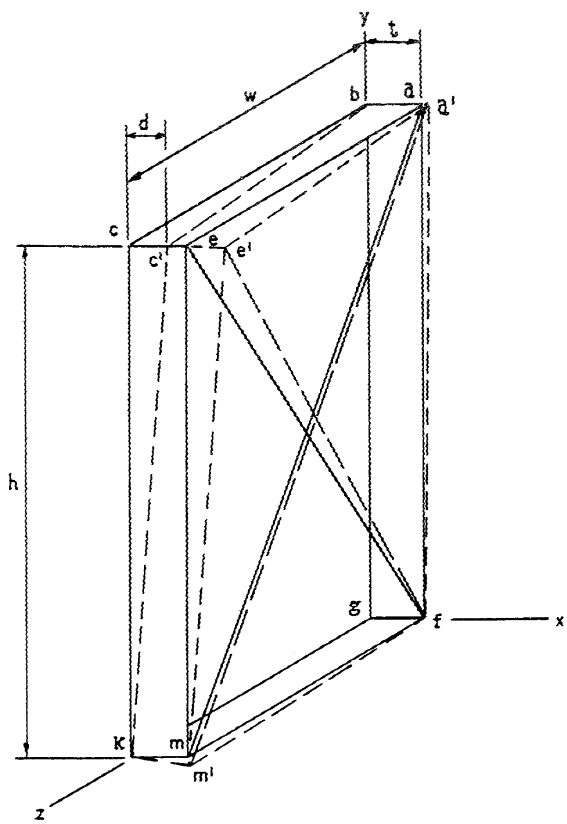

Figure C.6. Schematic Drawing of Panel ak

C.4.4.3 Ratio of Diagonal Deformation to Panel Deflection.

In the following information, a decrease in any diagonal length, either positive or negative diagonal, is designated as a positive change in length. Let r_0 be defined as follows:

$$r_o = \frac{\delta_o}{d} \qquad \text{(Equation C.72)}$$

Which, from Equation 71, becomes:

$$r_o = \pm \frac{2t}{(w^2 + h^2)^{1/2}} \qquad \text{(Equation C.73)}$$

r_0 is positive for positive diagonals and negative for negative diagonals. Figure C.8 illustrates the positive and negative diagonals of a typical leaf.

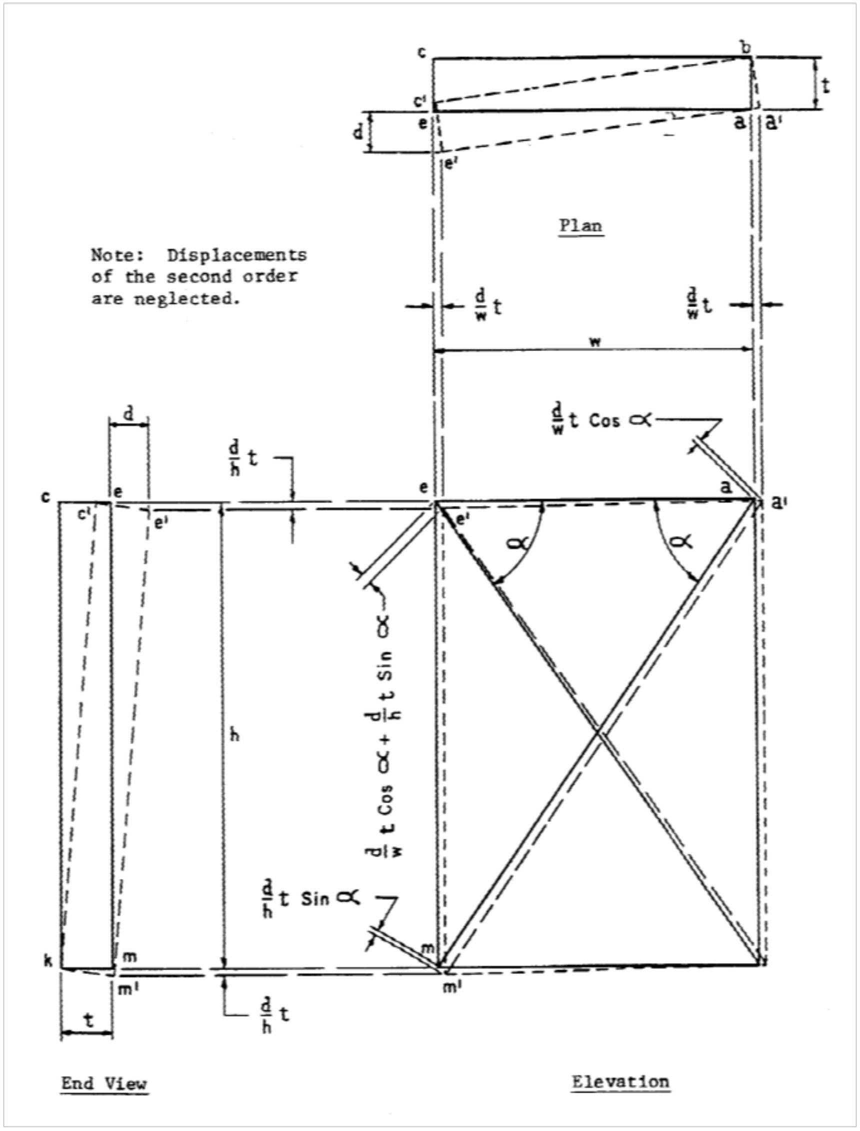

Figure C.7. Displacements of Points of Panel ak

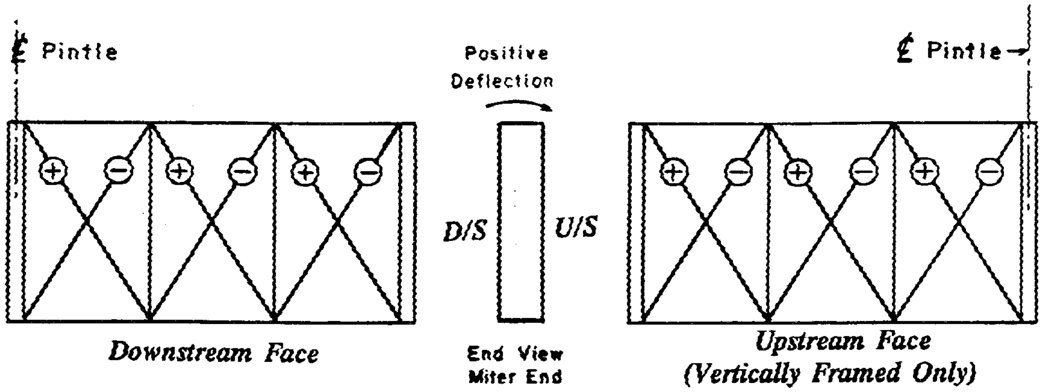

Figure C.8. Positive and Negative Diagonals of a Typical Leaf

C.4.4.4 Diagonal Restraint.

Up to this point, the restraint offered by the diagonal members has not been considered. Equation C.71 gives the change in length of a diagonal if the diagonal offers no resistance. However, unless a diagonal is slack, it does offer resistance to change in length. Therefore, when a deflection *d* is imposed on the panel, the length of the diagonal does not change an amount δ_0. The actual deformation is δ, which is less than δ_0 by some amount δ':

$$\delta = \delta_o - \delta'$$

(Equation C.74)

It is evident that δ is inversely proportional to the resistance of the diagonal and that δ' is inversely proportional to the ability of the panel to elongate the diagonal. Let the resistance of the diagonal be measured by its cross-sectional area *A*. Then:

$$\frac{\delta}{\delta_1} = \frac{A'}{A}$$

(Equation C.75)

In which A' is a measure of the stiffness of the panel in deforming the diagonal. The significance of A' and the method of determining its magnitude will be discussed later. Let it be assumed for the present, however, that A' is known.

Solving Equation C.74 for δ' and substituting its value in Equation C.75:

$$\frac{\delta}{\delta_o - \delta} = \frac{A'}{A}$$

(Equation C.76)

Let r be defined as the ratio of the actual deformation of the diagonal to the deflection of the panel:

$$r = \frac{\delta}{d} \qquad \text{(Equation C.77)}$$

Using Equations C.72 and C.77, Equation C.76 can be written:

$$\frac{rd}{r_o d - rd} = \frac{A'}{A}$$

and solving for r:

$$r = \frac{A'}{A + A'} r_o \qquad \text{(Equation C.78)}$$

Note, when the diagonal offers no restraint (that is to say that $A = o$), $r = r_0$.

Let Δ be defined as the torsional deflection of the whole leaf (see Figure C.5). It is evident that the relative deflection d from one end of a panel to the other is proportional to the width of the panel.

$$d = \frac{w\Delta}{v} \qquad \text{(Equation C.79)}$$

Let R_0 be defined as follows:

$$R_o = \frac{\delta_o}{\Delta} \qquad \text{(Equation C.80)}$$

Substituting the values of δ_0 and Δ from Equations C.72 and C.79, respectively:

$$R_o = \frac{r_o d}{(^v/_w)d} = \frac{w r_o}{v}$$

Which, from Equation C.73 becomes:

$$R_o = \pm\left(\frac{2wt}{v(w^2 + h^2)^{1/2}}\right) \qquad \text{(Equation C.81)}$$

Let R be defined by:

$$R = \frac{\delta}{\Delta} \qquad \text{(Equation C.82)}$$

Substituting in Equation C.82 the values of δ and Δ obtained from Equations C.77 and C.79, respectively:

$$R = \frac{rd}{(v/w)d} = \frac{w}{v}r$$

Which, from Equation C.78 becomes:

$$R = \frac{w}{v}r_o\frac{A'}{A+A'} = R_o\frac{A'}{A+A'}$$

(Equation C.83)

C.4.4.5 Deflection of Leaf and Stresses in Diagonals.

In general, the diagonals of any lock gate leaf will have, as a result of adjustments, an initial tension that is here called a prestress. The prestress in all diagonals is not the same. However, for any diagonal the leaf can be deflected by some amount Δ, such that the stress in that diagonal is reduced to zero. The magnitude of this deflection is a measure of the initial tension in the diagonal and will be called the prestress deflection D for that diagonal. By selecting the value of D, the designer can establish a definite prestress in any diagonal (see examples in sections C.4.5 and C.4.6). It can be seen from the definition of a positive diagonal that D is positive for positive diagonals and negative for negative diagonals.

C.4.4.5.1 Deflection of Leaf.

Referring to Equation C.82, it is seen that the prestress in any diagonal results from a change in length equal to $R(-D)$. If an additional deflection Δ is imposed on the leaf, the total change in length will be:

$$\delta = R(-D) + R(\Delta) = R(\Delta - D)$$

(Equation C.84)

And similarly:

$$\delta_o = R_o(\Delta - D)$$

(Equation C.84a)

Since a positive value of δ represents a decrease in length, the elongation of a diagonal is $(-\delta)$ and the total force is:

$$S = \frac{(-\delta)EA}{L}$$

Which, from Equation C.84, becomes:

$$S = \frac{-REA}{L}(\Delta - D)$$

(Equation C.85)

If the diagonal offered no resistance to change in length, its deformation would be, from Equation C.74, $\delta_o = \delta + \delta'$. Therefore, the force in the diagonal does not only elongate the diagonal an amount δ'. The total work done by the force *S* in the diagonal is, therefore:

$$W_D = \frac{1}{2}(\delta - \delta') = \frac{1}{2} S\delta_o$$

which, by adapting Equation C.84a, becomes:

$$W_D = \frac{1}{2} SR_o(\Delta - D)$$

(Equation C.85a)

Substituting the value of *S* from Equation 85:

$$W_D = \frac{-RR_oEA}{2L}(\Delta - D)^2$$

(Equation C.86)

The force *S* in the diagonal is produced by some external torque *T*. The work done is:

$$W_T = \frac{1}{2} T\theta$$

It is evident from Figure C.5 that the angle of rotation θ of any section of the leaf is proportional to the distance *z* from the pintle. If the leaf is twisted an amount (Δ – *D*), the angle of rotation at the end is (Δ – *D*)/*h*. Therefore, at any section:

$$\theta = \frac{(\Delta - D)}{h}\frac{z}{v}$$

Making this substitution for 0 in the equation for W_T:

$$W_T = \frac{(\Delta - D)}{2hv} T_z$$

(Equation C.87)

The term T_z is the area of the torque diagram for the torque *T*. T_z will hereinafter be called "torque-area" (see definitions).

Equating the sum of W_D and W_T, as given by Equations 86 and 87, respectively, to zero and simplifying:

$$T_z - \frac{RR_oEAhv}{L}(\Delta - D) = 0$$

let:

$$Q = \frac{RR_oEAhv}{L}$$ (Equation C.88)

then:

$$T_z + Q(D - \Delta) = 0$$ (Equation C.89)

Since T_z is the torque-area of the external load, the quantity $Q(D - \Delta)$ may be called the resisting torque-area of the diagonal. All factors of Q are constant for any diagonal. Q, therefore, is an elasticity constant of the diagonal. Even if there were no diagonals on a leaf, the structure would have some resistance to twisting. Let the resisting torque-area of the leaf without diagonals be defined as $Q_0(\Delta)$. A prestress deflection D is not included in this definition since the leaf does not exert any torsional resistance when it is plumb. In other words, D for the leaf is zero. Q_o will be evaluated later. For the present, let it be assumed that Q_o is known.

The total torque-area of all external loads plus the torque-area of all resisting members must equal zero. Therefore, Equation C.89 may be written as follows:

$$\sum(T_z) - Q_o\Delta + \sum[Q(D - \Delta)] = 0$$ (Equation C.90)

Where:

$\Sigma[Q(D - \Delta)]$ includes all diagonals of the leaf.

Since Δ is a constant for any condition of loading, Equation C.90 may be solved for Δ:

$$\Delta = \frac{\sum(T_z) + \sum Q(D)}{Q_o + \sum Q}$$ (Equation C.91)

Where:

This equation is the fundamental equation for deflection.

If the leaf is to hang plumb ($\Delta = 0$) under dead load, the numerator of the right-hand member of Equation C.91 must equal zero:

$$\sum(T_z)_{D.L.} + \sum(QD) = 0$$ (Equation C.92)

Where:

Equation C.92 represents the necessary and sufficient condition that a leaf hang plumb under dead load.

If the live load and dead load torque-areas are separated, Equation C.91 may be written as:

$$\Delta = \frac{\Sigma(T_z)_{L.L.} + \Sigma(T_z)_{D.L.} + \Sigma(QD)}{Q_o + \Sigma Q}$$

but if Equation C.92 is satisfied, $\Sigma(T_Z)_{D.L.} + \Sigma\,(QD) = 0$. Therefore:

$$\Delta = \frac{\Sigma(T_z)_{L.L.}}{Q_o + \Sigma Q} \qquad \text{(Equation C.93)}$$

which is the fundamental equation for deflection of a leaf with all diagonals prestressed. Equation C.93 shows that the live load deflection of a leaf is independent of the prestress deflection *D* for any diagonal.

Stress in Diagonals. The unit stress in a diagonal is obtained by dividing Equation C.85 by A:

$$s = \frac{RE}{L}(D - \Delta) \qquad \text{(Equation C.94)}$$

Where:

This equation is the fundamental equation for unit stress in a diagonal.

C.4.4.5.2 Maximum Numerical Value of *D*.

If the maximum allowable unit stress is substituted for *s* in Equation C.94, the maximum allowable numerical value of $(D - \Delta)$ will be obtained. Since the maximum values of Δ are known from Equation C.93, the maximum numerical value of D for any diagonal can be determined.

C.4.4.5.3 Minimum Numerical Value of *D*.

The diagonals of a gate leaf should be prestressed so that all of them are always in tension. If this is to be so, the quantity $(D - \Delta)$ must always represent an elongation of the diagonal. Therefore, for positive diagonals, *D* must be positive and greater than the maximum positive value of Δ. For negative diagonals, *D* must be negative and numerically greater than the maximum negative deflection. These then are the minimum numerical values of *D*.

C.4.4.5.4 Values of *D*.

Values of *D* must be selected such that they satisfy Equation C.92 and lie within the limits specified above. If this is done, the leaf will hang plumb under dead load, and none of the diagonals will ever become overstressed or slack. In addition, the deflection of the leaf will be held to a minimum since a prestressed tension diagonal is in effect a compression diagonal as well.

C.4.4.6 Preliminary Area of Diagonals.

In the design of diagonals, it is desirable to have a direct means of determining their approximate required areas. With these areas, the deflection and stresses can then be found and, if considered unacceptable, the areas could be revised and the process repeated. A close approximation to the required area can be found by equating Equation C.85a and Equation C.87:

$$\frac{1}{2}SR_o(\Delta - D) = -\frac{(\Delta - D)}{2hv}T_z$$

Treating R_o as equal for all diagonals, substituting sA for S, and taking Σ for all diagonals in a set:

$$\sum A = -\frac{\sum T_z}{sR_o hv} \qquad \text{(Equation C.95)}$$

With the above, the maximum positive ΣT_z will give the total area required in the set of negative diagonals and the maximum negative ΣT_z, the area for the positive diagonals.

C.4.4.7 Vertical Paneling of Leaf.

By differentiating Q with respect to h, it has been found that the most effective slope for a diagonal exists with $h = w(2)^{1/2}$. If h approaches 2.5 w, it will be desirable to subdivide the panel vertically to reduce the area of the diagonals or, possibly, to reduce their total cost. The example in paragraph C.4.7 shows the slight modification necessary to apply this method of design to panels subdivided vertically. In general, diagonals are most effective in panels having the ratio of:

$$\frac{\text{Greater dimension}}{\text{Lesser dimension}} = (2)^{1/2}$$

C.4.4.8 Curved Skin Plate.

The geometric relationships derived herein apply equally well to a leaf with curved or stepped skin plating and the more general value of t is the plan view divided by the width. The plan-view area is the area bounded by the skin plate, the centerline of the diagonals, and the side boundaries of the panel.

C.4.4.9 Discussion.

C.4.4.9.1 The Constant A'.

Except for the constants A' and Q_o, all properties of the gate leaf are known, and the deflection of the leaf and the stresses in the diagonals can be determined. A' appears in the equations for both R and Q as follows:

$$R = \frac{A'}{A + A'} R_o$$

(from Equation C.83)

$$Q = \frac{RR_0 EAhv}{L} = \frac{R_0^2 EAhv}{L} \times \frac{A'}{A + A'}$$

(from Equation C.88)

Measurements were made on the 1/32-size celluloid model of the gates for the MacArthur Lock at Sault Ste. Marie (Soo). Field measurements were also made on the lower gate at Louisville, KY, and 29 gate leaves in the Rock Island District on the Mississippi River. The Soo and Louisville gates are horizontally framed and have flat skin plates, and the Mississippi gates are vertically framed and have buckle skin plates. In all cases, δ was determined from strain gage readings on the diagonal and Δ was measured directly as the leaf was twisted. Equation C.82 gave the value of R. A' was then calculated from Equation C.82 in which the theoretical value of R_o obtained from Equation C.81 was substituted.[2] Values of A' obtained are:

Sault Ste. Marie $A' = 0.025$ in^2 (model)
$= 0.025 \times (32)^2 = 26$ in^2 (prototype)
Louisville = 13 in^2
Mississippi River Gates = 10 in^2

It seems reasonable to suppose that the size of the horizontal and vertical girders to which the diagonal is attached can be used as a measure of A'. At Sault Ste. Marie, A' is 0.14 of the sum of the cross-sectional areas of the girders that bound the diagonal. At Louisville, the factor is 0.07 and for the Mississippi River gates, 0.045. Additional experiments are desirable. However, until more data are obtained, it is believed that a conservative value of A' for the average diagonal is the sum of the average cross-sectional areas of the girders that bound the diagonals times 1/8 for the heavier, welded, horizontally framed leaves with flat skin plate and 1/20 for the lighter, riveted, vertically framed leaves with buckle plates.

It is believed that for any gate leaf diagonal, A' will usually be as large or larger than A. Therefore, a large error in A' will result in a much smaller error in the fraction $A'/(A+A')$. Hence, it is necessary to know the approximate value of A' in order to apply the foregoing theory. This is especially true of the diagonal stress, as can be seen from Equation C.94 where an error in A' produces an error R, which is opposite to that produced in $(D - \Delta)$. Thus, stress is nearly independent of A'.

C.4.4.9.2 The Constant Q_o.

Q_o is an elasticity constant that is a measure of the torsional stiffness of a leaf without diagonals. Q_o is a function of many properties of the leaf. However, it seems reasonable that the torsional work done on the typical main members of the leaf, as the leaf twists, might be used as a measure of Q_o.

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[2] In the model test, the experimental value of R_o was also determined and was found to agree with the theoretical value within 1 percent.

When a leaf twists, the horizontal and vertical members rotate through angles of Δ/h and Δ/v, respectively. The work done in any member is:

W = $\frac{1}{2}\frac{E_s J}{v}\frac{(\Delta)^2}{h^2}$ for horizontal members,

W = $\frac{1}{2}\frac{E_s J}{h}\frac{(\Delta)^2}{v^2}$ for vertical members,

E_s = shearing modulus of elasticity, and

J = modified polar moment of inertia. This is the gate's cross-sectional area's resistance to torsional deformation when twisted about an axis in the direction of the gate's length.

The work done by an external torque is, from Equation C.87:

$$W_T = \frac{\Delta}{2hv} T_z$$

In this case, the value of D in Equation C.87 is zero since the members are not supplying a resisting torque when the deflection is zero. Equating W_T to W and solving T_z:

Tz = $\frac{E_S J}{H}\Delta$ for horizontal members, and

Tz = $\frac{E_S J}{v}\Delta$ for vertical members.

The quantities E_sJ/h and E_sJ/v might be called the values of Q_o for horizontal and vertical members, respectively, hence:

$$Q_o = K\ E_s\ \Sigma\,(J/h + J/V) \qquad \text{(Equation C.96)}$$

where the value of K as determined experimentally for the Sault Ste. Marie model and the Louisville prototype is approximately 4. Until additional measurements can be made, this value should be used.

Nearly all members of a leaf subject to torsion are made up of narrow rectangles. For these, the value of J is:

$$J \approx \sum \frac{1}{3} b t^3$$

where b is the long dimension and t is the thin dimension of rectangular elements.

Where plates are riveted or welded together, with their surfaces in contact, they are considered to act as a unit with c equal to their combined thickness.

Using Equation C.96, Q_o can be evaluated very easily, as will be demonstrated in the examples. However, in many cases Q_o can be neglected entirely without being overly conservative. In neglecting Q_o, the stiffness of the leaf itself, without diagonals, is neglected. An experiment has shown this stiffness to be small. Furthermore, anyone who has seen structural steel shapes handled knows how easily they twist. Unless closed sections are formed, the total stiffness of a leaf is just the arithmetic sum of the stiffness of all members taken individually and this sum can be shown to be small. The lack of torsional stiffness is also illustrated by a known case in which a leaf erected without diagonals twisted several feet out of plumb under its own deadweight. Q_o is included in examples 1 and 2, but its values are only 5% and 3%, respectively, of the total stiffnesses, Q, contributed by the diagonals.

C.4.4.9.3 Load Torque-Areas.

By definition, a load applied through the shear center of a section will cause no twisting of the section. In computing dead load torque-area, the moment arm of the dead load is, therefore, the distance from the vertical plane through the shear center to the center of gravity of the leaf. The method of locating the shear center of a lock gate leaf is given in paragraph C.4.4.9.5. The water offers resistance against the submerged portion of the leaf as it is swung. There is also an inertial resistance to stopping and starting. Since the resultant of these resistances is located near or below the center height of leaf and the operating force is near the top of the leaf, a live load torsion results. From tests performed to determine operating machinery design loads, the maximum value of the above-mentioned resistances was found to be equivalent to a resistance of 30 psf on the submerged portion of the leaf.

Until additional data become available, it is recommended that this value be generally used in computing the live load torque-area. However, in the case of locks accommodating deep-draft vessels, water surges are created during lockages that appear to exceed the above-mentioned equivalent load. Until more data are obtained, it is recommended that for these cases, 45 psf or higher be used.[3] The diagonals will also be checked for obstruction loads and temporal hydraulic loads and the governing loading condition will be used for diagonal design. For definition of obstruction and temporal hydraulic loads, refer to paragraph C.4.8.

C.4.4.9.4 Skin Plate Consisting of Buckle Plates.

The theoretical basis for diagonal design assumes that the skin plate remains rectangular at all times. If the skin consists entirely of buckle plates and if the shear in the skin is large, this assumption may be in error. However, if the diagonals extending downward toward the miter end are made larger or prestressed higher than the others, the prestress in them can be made to carry a large part, if not all, of the dead load shear. Although the action of buckle plates in shear is not understood, it is recommended that they be treated as flat plates. As a precaution, however, the diagonals should be prestressed to carry as much of the dead load as possible

[3] The operating strut mechanism should also then be designed for these larger forces.

within the restrictions imposed on D (see paragraph C.4.4.5). The reader is referred to paragraph C.4.6, Example 2.

C.4.4.9.5 General Method for Locating Shear Center of a Lock Gate Leaf.

The shear center of a gate leaf is the point through which loads must be applied if the leaf is not to twist.

- Horizontal Shear Center Axis.

Consider the leaf restrained against rotation about the hinge. To prevent twisting of the leaf due to horizontal forces, the resultant of these forces must be located so that the load to each horizontal girder is proportional to their relative stiffnesses. This is equivalent to saying that the resultant must be located along the horizontal gravity axis of the girder stiffnesses. This gravity axis is then the horizontal shear center axis and is located a distance from the centroidal axis equal to:

$$Y = \frac{\Sigma(I_n Y)}{\Sigma I_n} \qquad \text{(Equation C.97)}$$

in which I_n is the moment of inertia of any horizontal girder about its vertical centroidal axis.

- Vertical Shear Center Axis

A lock gate leaf is a cantilever beam supported by the pintle gudgeon. A vertical load on the leaf causes tension above and compression below the centroidal axis. Therefore, longitudinal shearing stresses exist in the structure and shearing stresses of equal magnitude and at right angles to the longitudinal shearing stresses exist in the plane of any vertical cross section.

A shear diagram with arrows to indicate the direction of the shear is shown in Figure C.9. Since the shears of the flanges of the top and bottom girders are small and since the shear on one side of a flange is usually equal and opposite to the shear on the other side of the same flange, these shears will be neglected. The horizontal shears in the webs of the top and bottom girders produce a torsional moment on the section, which must be balanced by the torsional moment VX of the vertical forces if the leaf is not to twist.

The shear diagram for the web of the right-hand part of the top girder is redrawn to a larger scale in Figure C.10. The trapezoidal shape of this diagram is based on the assumption that the thickness of the web is constant within the limits of the diagram. The ordinate of the diagram at any point is VQ/I. The area of the shear diagram is the total horizontal shear S on this part of the girder. This area is $(VQ/I)b$ in which VQ/I is the ordinate at the center of the diagram. Therefore, Q is the statical moment, about the centroidal axis of the whole section, of that part of the section lying within the circle of Figure C.10. If a is the area of this part of the section, then $Q = ay$, and:

$$S = \frac{Vay}{I} b$$

The torsional moment of all these horizontal shearing forces about the horizontal shear center axis is:

$$T = \Sigma \frac{Vay}{I} by_n = \frac{V}{I} \Sigma(ayby_n)$$

(Equation C.98)

If the leaf is not to twist, the sum of the moments of the vertical and horizontal forces must equal zero:

$$VX + \frac{V}{I} \Sigma(ayby_n) = 0$$

(Equation C.99)

and solving:

$$X = -\left[\frac{\Sigma(ayby_n)}{I}\right]$$

(Equation C.100)

which is the horizontal distance from the centerline of the skin to the shear center of the section. In this equation, *a* is always positive and *b* and *X* are positive when measured to the right of the skin and negative when measured to the left.

Equation C.97 and Equation C.100 are general expressions, independent of the number of horizontal girders, and as such apply equally well to horizontally framed gates.

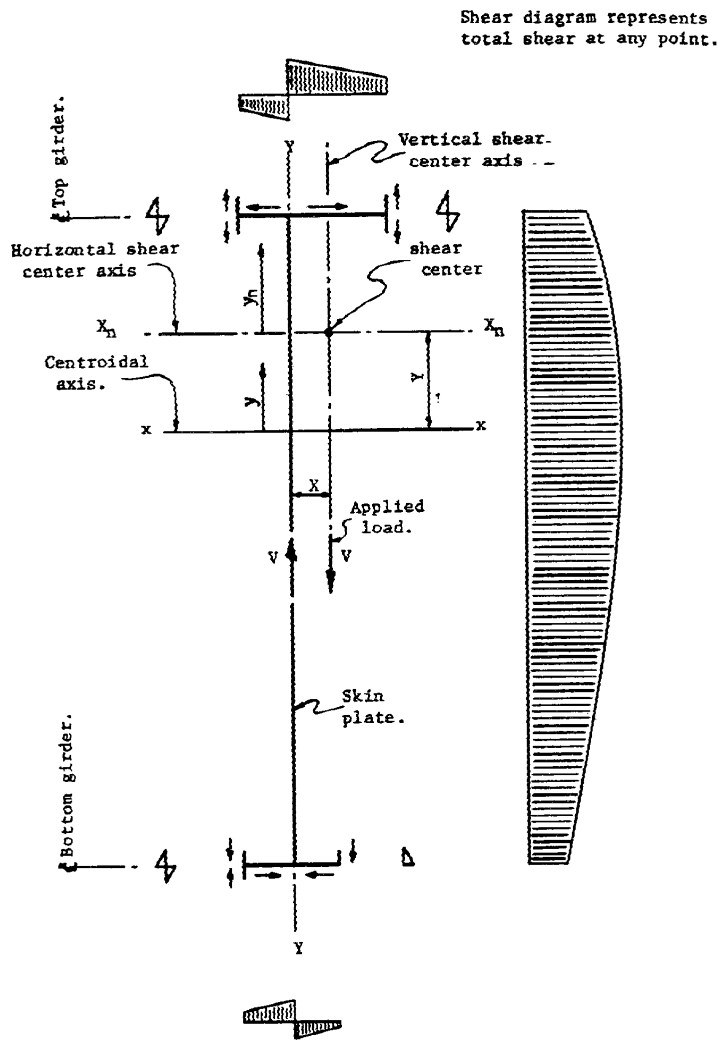


Figure C.9. Shear Diagram for Typical Vertically Framed Lock Gate Leaf

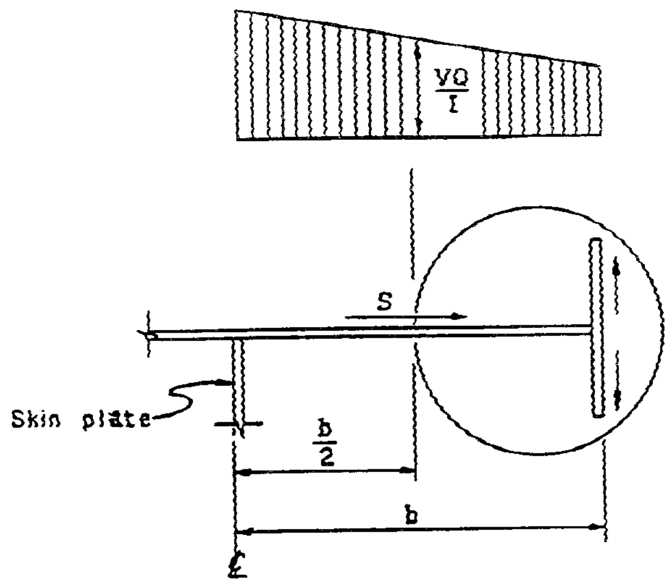


Figure C.10. Shear Diagram for Web of the Right-Hand Part of the Top Girder

C.4.5. Example 1: Horizontally Framed Gate.

Lower operating gates, MacArthur Lock, Sault Ste. Marie (see Figure C.11).

C.4.5.1 Evaluation of A'.

The bottom and top girders and the vertical end girders are W36X150 with a cross-sectional area of 44.16 in^2. Therefore, *A'* is (see paragraph C.4.4.9.1):

$A' = 1/8\ (4 \times 44.16) = 22\ in^2$

C.4.5.2 Evaluation of Q_o.

(See paragraph C.4.4.9.2 and Table C.14.)

$$Q_o = K\ E_s\ \Sigma(j/h + j/v)$$

$$Q_o = 4 \times 12 \times 10^6\ \frac{4320.0}{3 \times 684.0} + \frac{590}{3 \times 529}$$

$$= 120.0 \times 10^6\ \text{in.lb.}$$

(from Equation C.96)

C.4.5.3 Location of Shear Center (see Figure C.9).

Computations for the centroidal axis and moment of inertia of the vertical section through the leaf (see Figure C.11) are not given. Tables C.15 and C.16, respectively, list computations of distances *x* and *y*:

$Y = 310$ in

$I = 42.6\ x\ 10^6$ in^4

Horizontal shear center axis:

$$Y = \frac{\Sigma(I_n y)}{\Sigma I_n} = \frac{-1.61\times 10^6}{162{,}000} = -10.0 \text{ in}$$

(From Equation C.97)

Vertical shear center axis:

The value of b for all girders is -36.1 inches.

$$X = \frac{b}{I}\Sigma(ayy_n) = -\left(\frac{-36.1}{42.6\times 10^6}\right)\times 13.54\times 10^6 = 11.4 \text{ in}$$

(From Equation C.100)

C.4.5.4 Load Torque-Areas (see paragraph C.4.4.9.3).

The forces that produce twisting of the leaf are shown in Figure C.12. Table C.17 lists computation of the torque-area. Computations for the location of the center of gravity and deadweight of the leaf are not given. Because this lock handles deep-draft vessels, a water resistance of 45 psf is used.

C.4.5.5 Evaluation of R_o, R, and Q.

$$R_o = \pm\frac{2wt}{v(w^2+h^2)^{1/2}} = \pm\frac{2\times 483\times 37.8}{529(483^2+684^2)^{1/2}} = \pm 0.0822$$

(Equation C.101)

Required size of diagonals:

For diagonal U_oL_1:

$$A = -\Sigma\frac{T_z}{sR_o hv} = -\left(\frac{-11{,}570\times 10^6}{18{,}000\times 0.0822\times 684\times 529}\right) = 21.5 \text{ in}^2$$

(from Equation C.95)

For diagonal L_oU_1:

$$A = -\left(\frac{9{,}200 \times 10^6}{18{,}000 \times 0.0822 \times 684 \times 529}\right) = 17.1 \text{ in}^2$$

For diagonal L_oU_1, the dead load torque is no longer included since diagonal U_oL_1 will be prestressed to support this load. The following diagonal sizes will be used throughout the remainder of the design and revised later, if necessary:

U_0L_1 = 24 in^2 (2 @ 12 in^2)

L_0U_1 = 18 in^2 (2 @ 9 in^2)

$$R = \frac{A'}{A + A'} R_o = \pm \frac{22}{A + 22} \times 0.822$$

(from Equation C.83)

$$Q = \frac{RR_oEAhv}{L} = \frac{R \times 0.0822 \times 29 \times 10^6 \times A \times 684 \times 529}{771} = 112 \times 10^7 \times RA$$

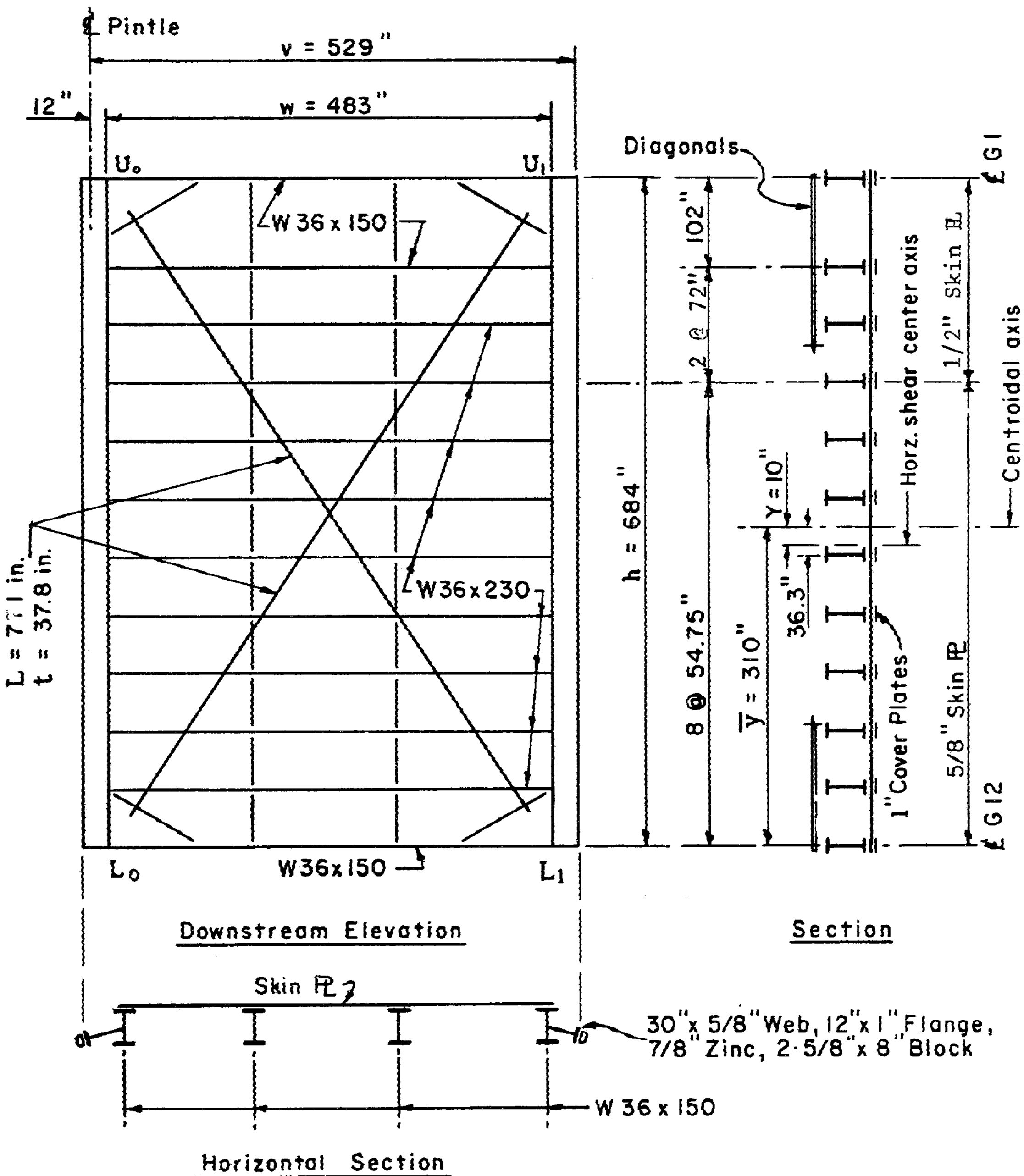


Figure C.11. Lower Gate Leaf, MacArthur Lock, Sault Ste. Marie

Table C.14
Computation of Modified Polar Moment of Inertia J

Elements	n (No. of Elements)	1 (in)	c (in)	nlc^3 Horizontal Members	Vertical Members
Horizontal Girders					
US flange	3	12.0	2.44	520.0	—
Web	3	34.0	0.63	30.0	—
DS flange (G1, 2, and 12)	3	12.0	0.94	30.0	—
US flange	9	16.5	2.78	3190.0	—
Web	9	33.5	0.77	140.0	—
DS flange, G3 through G11	9	16.5	1.26	300.0	—
Skin (between flanges)					
1/2-in plate	1	203.0	0.50	30.0	—
5/8-in plate	1	308.0	0.63	80.0	—
Vertical Girders					
US flange	4	12.0	1.57	—	190.0
Web	4	34.0	0.62	—	30.0
DS flange	4	12.0	0.94	—	40.0
Quoin & Miter Posts					
Web	2	30.0	0.63	—	20.0
Flange	2	12.0	1.00	—	20.0
Block	2	8.0	2.63	—	290.0
Total =				4320.0	590.0

Table C.15
Computation of Distance Y

Girder	I_n (in^4)	y (in)	$I_{n.y}$ (in^5 x 10^6)
G−1	9,000	+374.0	+3.37
G−2	9,000	+272.0	+2.44
G−3	15,000	+200.0	+3.00
G−4	15,000	+128.0	+1.92
G−5	15,000	+73.3	+1.10
G−6	15,000	+18.5	+0.28
G−7	15,000	+36.3	-0.55
G−8	15,000	-91.0	-1.36
G−9	15,000	-145.8	-2.18
G−10	15,000	-200.5	-3.00
G−11	15,000	-255.3	3.84
G−12	9,000	-310.0	-2.79
	Σ 162,000	Σ	-1.61

Table C.16
Computation of Distance X

Girder	a(in^2)	y(in)	y_n (in)	ayy_n (in^4 x 10^6)
G−1	22.1	+374.0	+384.0	3.17
G−2	22.1	+272.0	+282.0	1.69
G−3	33.9	+200.0	+210.0	1.42
G−4	33.9	+128.0	+138.0	0.60
G−5	33.9	+73.3	+83.3	0.21
G−6	33.9	+18.5	+28.5	0.02
G−7	33.9	-36.3	-26.3	0.03
G−8	33.9	-91.0	-81.0	0.25
G−9	33.9	-145.8	-135.8	0.67
G−10	33.9	-200.5	-190.5	1.29
G−11	33.9	-255.3	-245.3	2.13
G−12	22.1	-310.0	-300.0	2.06
			Σ	13.54

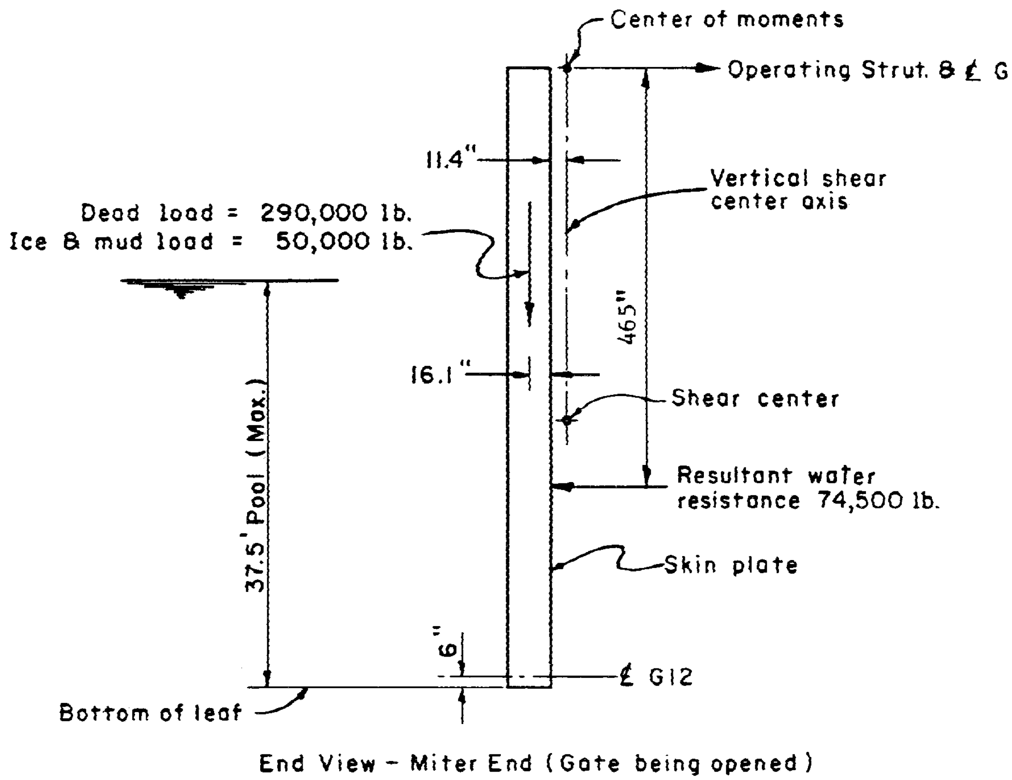


Figure C.12. Forces Acting on Leaf Being Opened

Table C.17
Computation of Torque-Area

Load	Force (lb)	Moment Arm (in)	z (in)	T_z(in.^{2}lb x 10^6)
Dead load	290,000	27.5[a]	253	-2,020
Ice & mud	50,000	27.5	253	-350
Water	74,500	465.0	265	±9,200[b]

[a] From determinations of shear center and center of gravity for various horizontally framed gates, this arm is approximately 3/4t

[b] Plus value for gate opening

Table C.18
Computation of Constant Q

Diagonal	A (in.^{2})	R	Q (in-lb x 10^6)
U_oL_1	24.0	+0.0393	1,050
L_oU_1	18.0	-0.0452	910
		ΣQ =	1,960

C.4.5.6 Deflection of Leaf.

Gate opening $\Delta = \frac{\sum(T_Z)_{L.L.}}{Q_0+\sum Q} = \frac{9{,}200\times10^6}{(120+1{,}960)\times10^6} = 4.4$ (from Equation C.93)

Gate closing $\Delta = \frac{(-9{,}200-350)\times10^6}{(120+1{,}960)\times10^6} = -4.6$

C.4.5.7 Prestressed Deflections and Stresses in Diagonals.

Table C.19 lists prestress deflections. The minimum numerical values of *D* (line 3) are the maximum deflections of the leaf. Maximum numerical values of ($D - \Delta$) are found by solving Equation C.94.

$$(D-\Delta) = \frac{sL}{RE} = \frac{18{,}000\times771}{R\times29\times10^6} = \frac{0.478}{R}$$ (Equation C.102)

Having the maximum numerical values of ($D - \Delta$), the maximum values of *D* are determined and placed in line 5. Values of *D* (line 6) are then selected between the above limits such that Equation C.92 is satisfied, that is $\Sigma(QD)$ must equal +2,030 x 10^6 in^2-lb. Further, to ensure that the diagonals will always be in tension, *D* should be such that the minimum stress is more than 1 kip per in^2. Stresses that occur during normal operation of the gate are computed from:

$$s = \frac{RE}{L}(D-\Delta)$$ (from Equation C.94)

and are placed in lines 8, 9, and 10.

From Table C.19, it is seen that the diagonal sizes chosen are quite satisfactory.

Table C.19
Stresses in Diagonals During Normal Operation

Line	Parameter	Positive Diagonal U_oL_1	Negative Diagonal L_oU_1
1	R	+0.0393	-0.0452
2	Q (in-lb x 106)	1,050	910
3	Minimum numerical value of D (in)	+4.4	-4.6
4	Maximum numerical value of D – Δ) (in)	+12.1	-10.6
5	Maximum numerical value of D (in)	+7.5	-6.2
6	D (selected value) (in)	+6.7	-5.5
7	QD (in^2-lb x 10^6)	+7,030	-5,000
		$\Sigma(QD) = 2{,}030 \times 10^6$ in^2-lb	
	Operation	**Stress**	**ksi**
8	Gates stationary $\Delta = 0$	9.9	9.4
9	Gates being opened $\Delta = +4.4$	3.4	16.8
10	Gates being closed $\Delta = +4.6$	16.7	1.5

C.4.5.7.1 Plumb/Out of Plumb.

With the completion of this operation, the leaf will nearly always hang plumb. If it does not, the corrected prestress deflection for this diagonal can be found from Equation C.91 with Δ equal and opposite to the out-of-plumb deflection. This prestress deflection can then be substituted in Equation C.91 to obtain the corrected number of turns required to make the leaf hang plumb. For instance, for a final out-of-plumb deflection of +1/2 inch, the corrected prestress deflections would be found from $\Sigma QD = (\Delta Q_o + \Sigma Q) - (T_Z)D.L.$ to be 980 in^2-lb x 10^6. With D for diagonal L_oU_1 maintained at -5.5 inches, the D then required for diagonal U_oL_1 would be +5.7 inches. The remainder of the computations would be repeated.

C.4.6. Example 2: Vertically Framed Gate.

See Figures C.13 and C.14.

C.4.6.1 Evaluation of A'.

The cross-sectional area of the bottom girder (see Figure C.14) is 36.7 in^2, the cross-sectional area of any vertical girder is 37.0 in^2 (see Figure C.13), and the cross-sectional area of the top girder is 112.5 in^2. Therefore, the value of A' (see definition) for all diagonals is:

A' = (1/20) (36.7 + 74.0 + 112.5) = 11.0 in^2

C.4.6.2 Evaluation of R_o, R, and Q.

Since this is an existing lock, the diagonal sizes are fixed.

$$R_o = \pm\frac{2wt}{v(w^2+h^2)^{1/2}} = \pm\frac{2\times 232t}{723(232^2+535^2)^{1/2}} = \pm 0.00110t$$
(from Equation C.81)

$$R = \frac{A'}{A+A'}R_o = \pm 0.0121\frac{t}{(A+11)} = \frac{11}{(A+11)}R_o$$
(from Equation C.83)

$$Q = \frac{RR_oEAhv}{L} = \frac{RR_o\times 29\times 10^6\times A\times 535\times 723}{471} = 238\times 10^8\times RR_oA$$
(from Equation C.88)

Table C.20 lists computation of the elasticity constant Q.

Table C.20
Computation of Elasticity Constant Q

Diagonal	*A* (in^2)	*t* (in)	R_o	*R*	*Q* (in-lb x 10^6)
[a] D'stream U_0L_1	10.00	31.5	+0.0347	+0.0182	150.0
[a] D'stream U_1L_2	8.00	35.2	+0.0388	+0.0224	165.0
[a] D'stream U_2L_3	4.50	31.3	+0.0345	+0.0244	90.0
[a] Upstream L_0U_1	4.50	18.3	+0.0202	+0.0143	31.0
[a] Upstream L_1U_2	4.50	14.4	+0.0159	+0.0112	19.0
[a] Upstream L_2U_3	4.50	17.9	+0.0197	+0.0140	30.0
[b] Upstream U_0L_1	10.00	17.2	-0.0189	-0.0099	45.0
[b] Upstream U_1L_2	8.00	13.3	-0.0146	-0.0085	24.0
[b] Upstream U_2L_3	4.50	17.0	-0.0187	-0.0133	27.0
[b] D'stream L_0U_1	4.50	32.6	-0.0359	-0.0255	98.0
[b] D'stream L_1U_2	4.50	36.2	-0.0399	-0.0282	120.0
[b] D'stream L_2U_3	4.50	32.2	-0.0355	-0.0252	96.0
				$\Sigma Q =$	895
[a] Positive diagonals [b] Negative diagonals					

Because all the skin in the end panels is not in the same plane, *t* (in the end panels) is measured from the mean skin shown in Figure C.13 (see paragraph C.4.4.8 for the determination of *t* for skin not in a plane).

This example provides a good illustration of the inefficiency of past designs. The upstream diagonals are quite ineffective because they are so close to the skin plate. If all the upstream diagonals were omitted (in other words, the number of diagonals cut in hail) and the skin plate placed in their location instead, the leaf would be stiffer and the stresses in the remaining diagonals would be lower. Further, with a flat skin plate, all positive diagonals could have been made the same size and all negative diagonals another size (for simplification of details and reduction in cost).

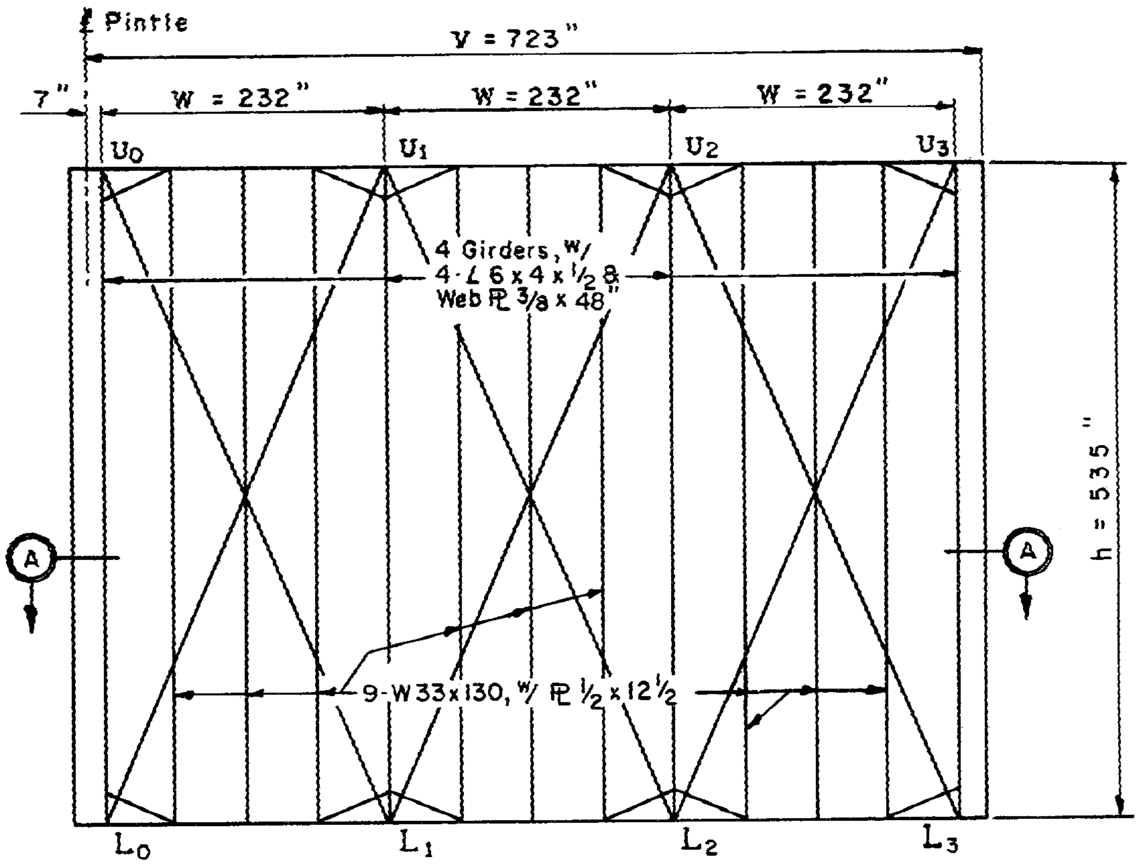
C.4.6.3 Evaluation of Q_0 (see paragraph C.4.4.9.2 and Table C.18).

$$Q_o = K \times E_s \times \Sigma(J/h + J/v)$$
$$= 4 \times 12 \times 10^6 \left(\frac{310}{3 \times 535} + \frac{700}{3 \times 723} \right) = 25 \times 10^6 \text{ in.-lb}$$

(from Equation C.96)

Table C.21
Computation of Modified Polar Moment of Inertia J

Elements	No. of Elements	1 (in)	c (in)	Horizontal Elements	Vertical Members
Top Horizontal Girder					
U/S flange	1	18.0	2.38	240	
Web	1	72.0	0.50	10	
D/S Flange	2	14.0	0.88	20	
Bottom Horizontal Girder					
U/S flange	1	12.0	0.50	0	
Web	1	48.0	0.38	0	
D/S flange	1	8.0	1.13	10	
Skin plate	1	535.0	0.38	30	
Vertical Girders					
U/S flange	8	10.0	0.50		10
Intermed. Flange	6	7.0	0.38		0
Web	4	48.0	0.38		10
U/S flange	8	10.0	0.50		10
Vertical Beams					
U/S flange	9	11.5	1.73		540
Web	9	31.4	0.58		60
D/S flange	9	11.5	0.86		70
			Total =	310	700



Diagonals on both US and DS faces.
Pin-to-pin length of all diagonals is 471 in.

Downstream Elevation

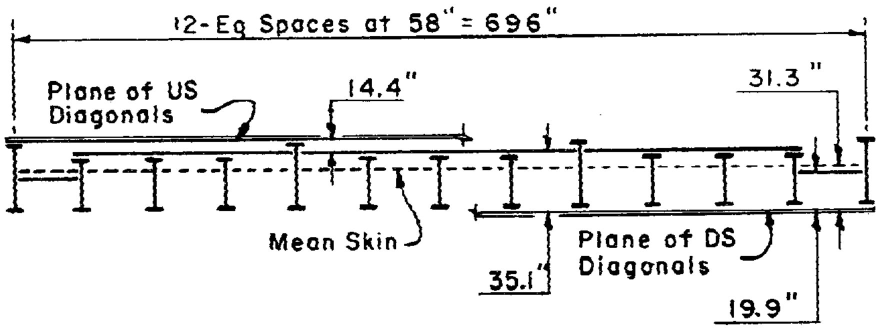


Figure C.13. Schematic Drawing of a Vertically Framed Leaf

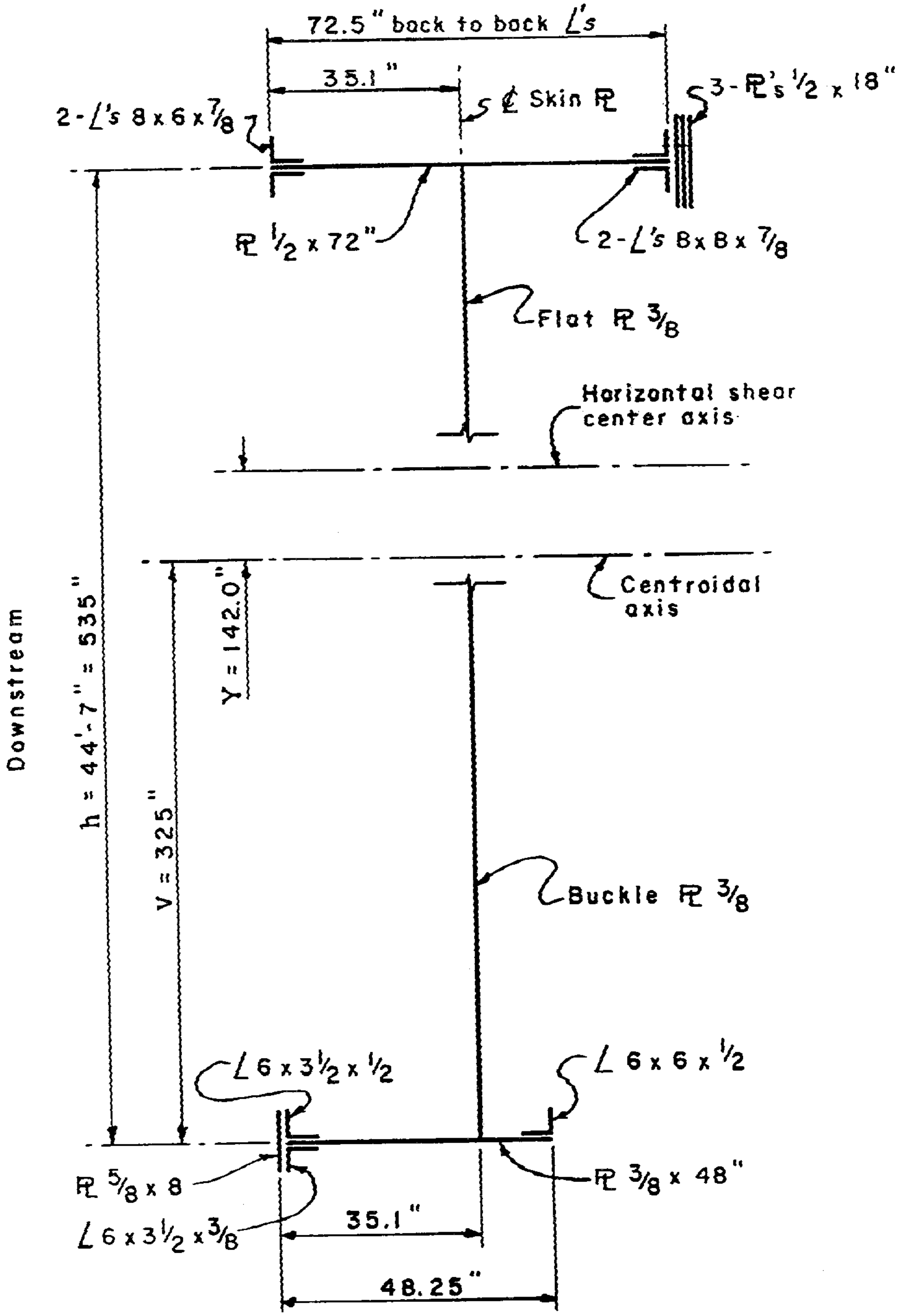


Figure C.14. Average Vertical Section Through Leaf

C.4.6.4 Location of Shear Center (see paragraph C.4.4.9.5).

Computations for the centroidal axis and the moment of inertia of the vertical section through the leaf are not shown (see Figure C.13).

y = 325 in

$I = 14.3 \times 10^6 \text{ in}^4$

Horizontal shear center axis:

Moment of inertia of top girder = 84,100 in^4

$$Y = \frac{\Sigma(I_n \times y)}{\Sigma I_n} = \frac{84{,}100 \times 210 - 12{,}100 \times 325}{96{,}200} = +142 \qquad \text{(from Equation C.97)}$$

Vertical shear center axis:

Table C.22 lists computation of the distance X.

$$X = -\left[\frac{\Sigma(aybyn_n)}{I}\right] = -\left(\frac{-69.9 \times 10^6}{14.3 \times 10^6}\right) = +4.9 \text{ in} \qquad \text{(from Equation C.100)}$$

Table C.22
Computation of Distance X for Vertically Framed Gate

Girder	a (in^2)	b (in)	y (in)	y_n (in)	$ayby_n$ (in^5 x 10^6)
Top girder – U/S	62.8	+37.4	+210	+68	+33.5
Top girder – D/S	31.8	-35.1	+210	+68	-15.9
Bottom girder – U/S	8.2	+13.1	-325	-467	+16.3
Bottom girder – D/S	19.5	-35.1	-325	-467	-103.8
				Σ =	-69.9

C.4.6.5 Load Torque-Areas (see discussion in paragraph C.4.4.9.3).

The forces that produce twisting of the leaf are shown in Figure C.15. Again, computations for locating the center of gravity and deadweight of the leaf are not shown. Since this is a 9-ft channel handling only shallow-draft vessels, a water resistance of 30 psf is used.

For dead load:

T_z = -235,000 (10.7 + 4.9) x 355

= -1,300 x 10^6 in^2-lb

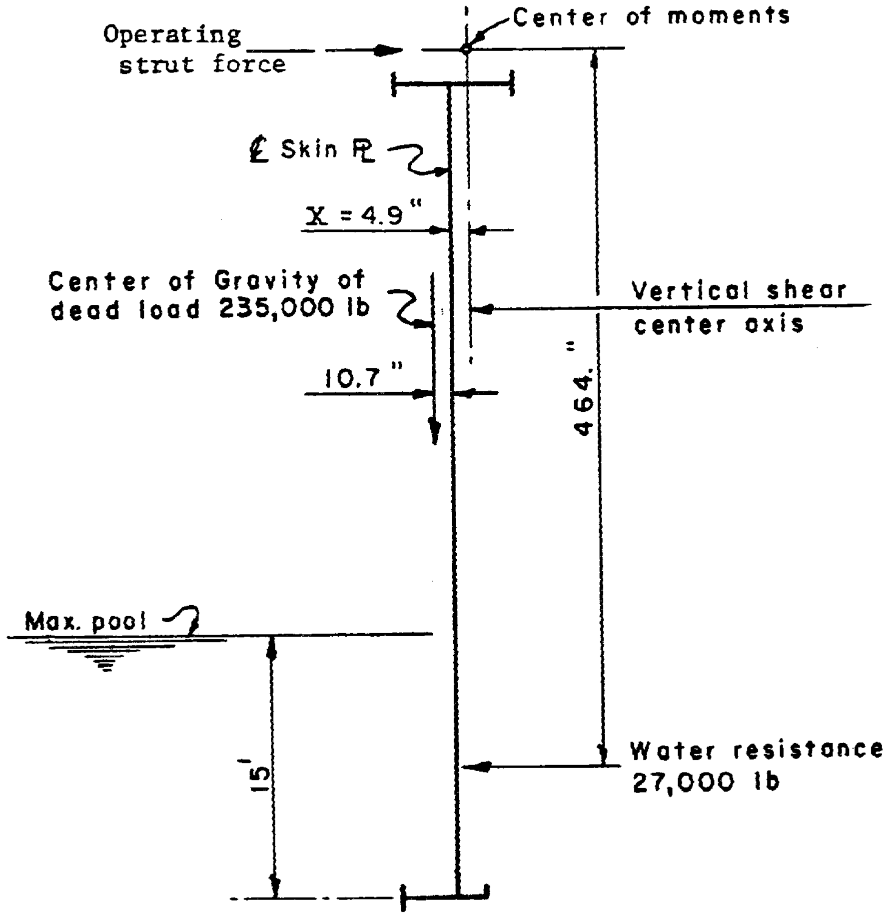


Figure C.15. Torsional Forces on Leaf

For live load:

T_z = ±27,000 x 464 x 362

= ±4,350 x 10^6 in^2-lb

(positive value for gate opening)

Deflection of leaf:

$$\Delta = \frac{\sum T_z}{Q_o + \sum Q} = \frac{\pm 4{,}530 \times 10^6}{(25+895)\times 10^6} = \pm 4.9 \text{ in} \qquad \text{(from Equation C.93)}$$

Where:

Positive value is for gate opening.

C.4.6.6 Prestress Deflections and Stresses in Diagonals.

Table C.23 lists the prestress deflections. The minimum numerical values of *D* (column 4) are the maximum deflections of the leaf. Maximum numerical values of ($D-\Delta$) are found by solving Equation C.94:

$$(D-\Delta)\max = \frac{sL}{RE} = \frac{18{,}000\times 471}{R\times 29\times 10^6} = \frac{0.292}{R}$$

Having the maximum numerical values of ($D-\Delta$), the maximum numerical values of *D* are determined and placed in column 6. Values of *D* (column 7) are then selected such that Equation C.92 is satisfied; that is, ΣQD must equal +1,300 x 10^6 in^2-lb. Because all but the top 10 ft of the skin consists of buckle plates (see paragraph C.4.4.9.4), an attempt is made to have the diagonals carry as much of the vertical dead load shear as possible. Therefore, values of *D* are made as large as possible for the diagonals extending downward toward the miter end, and as small as possible for the other diagonals. Further, to ensure that the diagonals are always in tension, *D* should also be such that the minimum stress is more than 1,000 psi. The unit stresses in the diagonals are found from:

$$s = \frac{RE}{L}(D-\Delta)$$

(from Equation C.94)

Before computing normal stresses (columns 10, 11, and 12), the stresses that occur during the prestressing operation are computed (column 9) as a check on the value of *D*. The twist-of-the-leaf method for prestressing is used. Because of the large value of *D* for some of the negative diagonals, it is best to prestress all negative diagonals first.

C.4.6.7 Dead Load Shear in Skin (Buckle Plates).

Prestressing of many gates in the Rock Island District has proved that buckle plates can support the shear imposed on them during and after the prestressing operation without any apparent distress. However, it is still considered desirable to have the diagonals carry as much of the vertical dead load shear as possible. If the skin had been flat plate, this consideration would have been omitted. Table C.24 lists the dead load shear remaining in the skin (buckle plates).

C.4.6.8 Method of Prestressing.

The maximum force will be required when the leaf is deflected +10.0 inches against the action of the negative diagonals (which are prestressed, in this case, first):

$$P = \frac{\Delta(Q_o+\Sigma Q)-\Sigma QD-(\Sigma T_z)DL}{hv}$$

$$= \frac{[+10.0(25+410)-(2{,}620)-(-1{,}300)]\times 10^6}{535\times 723} = 21{,}000\text{ lb}$$

On completion of this prestressing operation, the leaf is very rarely out of plumb. Should it be, however, the corrected prestress deflections can be found from Equation C.91 with Δ equal and opposite to the out-of-plumb deflection, as:

$$\Sigma QD = \Delta\,(Q_0 + \Sigma Q) - (\Sigma T_Z)_{DL}$$

In this example, for a final out-of-plumb deflection of +1/2 inch, revised values of *D* would be selected to make ΣQD equal to +840 x 10^6 in^2-lb. The leaf would then hang plumb. Repeat computations, if necessary.

Table C.23
Computation of Diagonal Stresses

		POS. DIAGONALS						NEG. DIAGONALS					
1	DIAGONAL	D. S. U_0L_1	D. S. U_1L_2	D. S. U_2L_3	U. S. L_0U_1	U. S. L_1U_2	U. S. L_2U_3	U. S. U_0L_1	U. S. U_1L_2	U. S. U_2L_3	D. S. L_0U_1	D. S. L_1U_2	D. S. L_2U_3
2	R	+0.0182	+0.0224	+0.0244	+0.0143	+0.0112	+0.0140	-0.0099	-0.0085	-0.0133	-0.0255	-0.0282	-0.0252
3	Q IN. - LB X 10^6	150	165	90	31	19	30	45	24	27	98	120	96
4	MINIMUM NUMERICAL VALUE OF D	+4.9	+4.9	+4.9	+4.9	+4.9	+4.9	-4.9	-4.9	-4.9	-4.9	-4.9	-4.9
5	MAXIMUM NUMERICAL VALUE OF (D - Δ)	+16.1	+13.0	+12.0	+20.4	+26.1	+20.8	-29.5	-34.3	-22.0	-11.4	-10.3	-11.6
6	MAXIMUM NUMERICAL VALUE OF D	+11.2	+8.1	+7.1	+15.5	+21.2	+15.9	-24.6	-29.4	-17.1	-6.5	-5.4	-6.7
7	D	+10.0	+7.5	+6.5	+7.5	+7.5	+7.5	-12.0	-12.0	-5.25	-6.25	-5.25	-5.25
8	QD IN.^{2} - LB. X 10^6	+1500	+1240	+590		+600		-830		-140		-1,650	
													$\sum QD = +1{,}310$
9	STRESS, LB / SQ. IN. — DURING PRESTRESSING Δ = + 10.0 IN.	PRESTRESSED LAST						13,400	11,500	12,500	23,900	26,400 a	23,600
10	STRESS, LB / SQ. IN. — GATE STATIONARY Δ = 0	11,200	10,300	9,800	6,600	5,200	6,500	7,300	6,300	4,300	8,300	9,100	3,100
11	STRESS, LB / SQ. IN. — GATE BEING OPENED Δ = + 4.9 IN.	5,700	3,600	2,400	2,300	1,800	2,200	10,300	8,800	8,300	15,900	17,600	15,700
12	STRESS, LB / SQ. IN. — GATE BEING CLOSED Δ = -4.9 IN.	16,700	17,100	17,100	10,900	8,500	10,700	4,300	3,700	300	600	600	600

a TOO HIGH, BUT INHERENT IN THIS EXISTING GATE.

Table C.24
Computation of Dead Load Shear in Buckle Plates

Panel	Diagonal	A (in^2)	s (lb/in^2)	As (lb)	Σ(As(h/L) (lb)	Panel	Skin
0-1	DSU_0L_1	10.0	11,200	+112,000			
	USU_0L_1	10.0	7,300	+73,000			
	USL_0U_1	4.5	6,600	-29,000	+119,000 lb	-196,000 lb	+77,000 lb
	DSL_0U_1	4.5	8,300	-37,000			
1-2	DSU_1L_2	8.0	10,300	+82,000			
	USU_1L_2	8.0	6,300	+50,000			
	USL_1U_2	4.5	5,200	-23,000	+68,000	+117,000 lb	+49,000 lb
	DSL_1U_2	4.5	9,100	-41,000			
2-3	DSU_2L_3	4.5	9,800	+44,000			
	USU_2L_3	4.5	4,300	+19,000			+41,000 lb
	USL_2U_3	4.5	6,500	-29,000	-2,000	-39,000 lb	
	DSL_2U_3	4.5	8,100	-36,000			

C.4.7. Vertical Paneling of Leaf.

The previous design applies to miter gate leaves that are divided into panels (not necessarily equal) longitudinally. With a slight modification of the term R_0, the design is extended to apply to leaves that are divided into panels vertically as well as longitudinally. Figure C.16 shows the most general arrangement of paneling. In practice, an effort would be made to make the panel heights and widths the same. To design the diagonals use:

$$R_o = \pm\left(\frac{2w \cdot h \cdot t}{H \cdot v \cdot (w^2+h^2)^{\frac{1}{2}}}\right) \quad \text{(from Equation C.81')}$$

This value of R_0 replaces that given in Equation C.81, being a more general expression. It is seen that for a value of $h = H$ (no vertical paneling) the derivative from Equation C.81' reverts to Equation C.81. With the above value of R_0, all the other expressions and the method of analysis remain identical to that previously outlined.

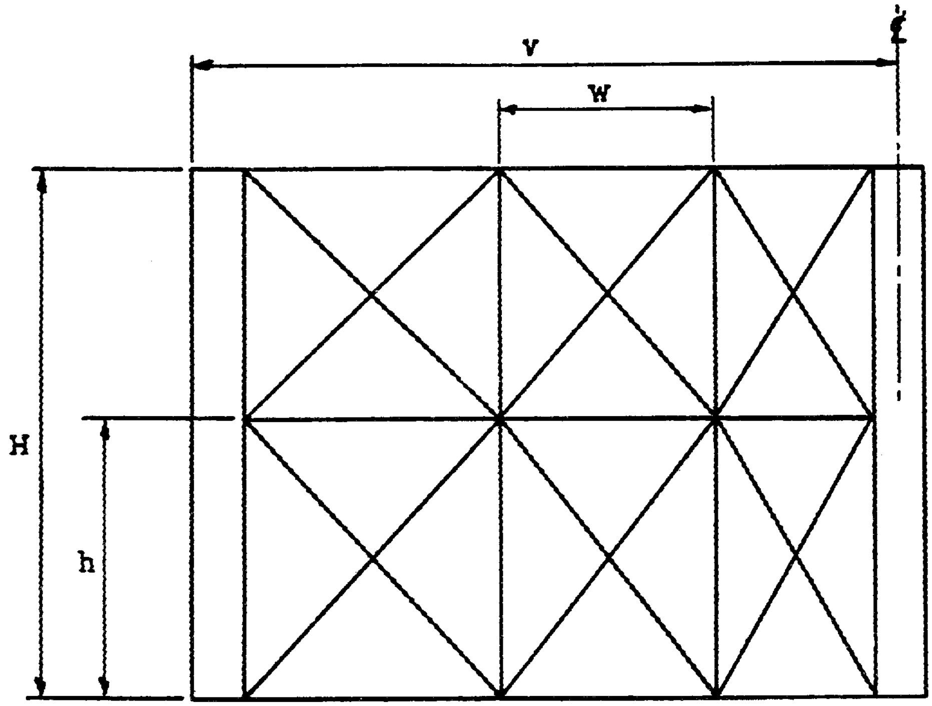


Figure C.16. Vertical and Longitudinal Arrangement of Leaf Panels

C.4.8. Derivation of Equation C.81'.

The general value of R_0 can be found as follows (refer to paragraph C.4.4.4). Let d = deflection of panel; other symbols are as defined previously. Figure C.17 illustrates the displacements of points of a vertical divided panel.

Let δ_0 = change in length of any diagonal:

$$\delta_o = \left(\frac{d}{w}t\cos\alpha\right) + \left(\frac{d}{h}t\sin\alpha\right) = \frac{dt}{w}\left[\frac{w}{(w^2+h^2)^{1/2}}\right] + \frac{dt}{w}\left[\frac{h}{(w^2+h^2)^{1/2}}\right]$$

$$\delta_o = \left[\frac{2dt}{\frac{w}{(w^2+h^2)^{1/2}}}\right]$$

(See Figure C.17.)

Where:

h and *d* are the height and deflection of one panel, then:

$$r_o = \frac{\delta_o}{d} = \pm\left[\frac{2t}{(w^2+h^2)^{1/2}}\right]$$

The relation between the deflection of the panel and the leaf becomes:

$$d = \left(\frac{w}{v}\right)\left(\frac{h}{H}\right)\Delta \text{ or } \Delta = \left(\frac{v}{w}\right)\left(\frac{H}{h}\right)d$$

$$R_o = \frac{\delta_o}{\Delta} = \left[\frac{2dt}{(w^2+h^2)^{1/2}}\right]\left[\frac{1}{\left(\frac{v}{w}\right)\left(\frac{H}{h}\right)d}\right]$$

$$R_o = \left[\frac{2w\,h\,t}{H\ v\,(w^2+h^2)^{1/2}}\right]$$

(from Equation C.81)

The remainders of the expressions are the same as before, for distance:

$$R_o = \frac{\delta}{\Delta} = \frac{r\,d}{\left(\frac{v}{w}\right)\left(\frac{H}{h}\right)d} = \left(\frac{w}{v}\right)\left(\frac{h}{H}\right)r = \left(\frac{w}{v}\right)\left(\frac{h}{H}\right)\left(\frac{A'}{A+A'}\right)$$

$$R_o = \left(\frac{w}{v}\right)\left(\frac{h}{H}\right)\left(\frac{A'}{A+A'}\right)\pm\left(\frac{2t}{(w^2+h^2)^{1/2}}\right)$$

Therefore:

$$R_o = \pm\left[\frac{2w\,h\,t}{H\ v(w^2+h^2)^{1/2}}\right]\left(\frac{A'}{A+A'}\right) = R_o\left(\frac{A'}{A+A'}\right)$$

In similar manner, it can be shown that the expressions for Q and Q_0 (Equation C.88 and Equation C.96 respectively) still apply with H substituted for h.

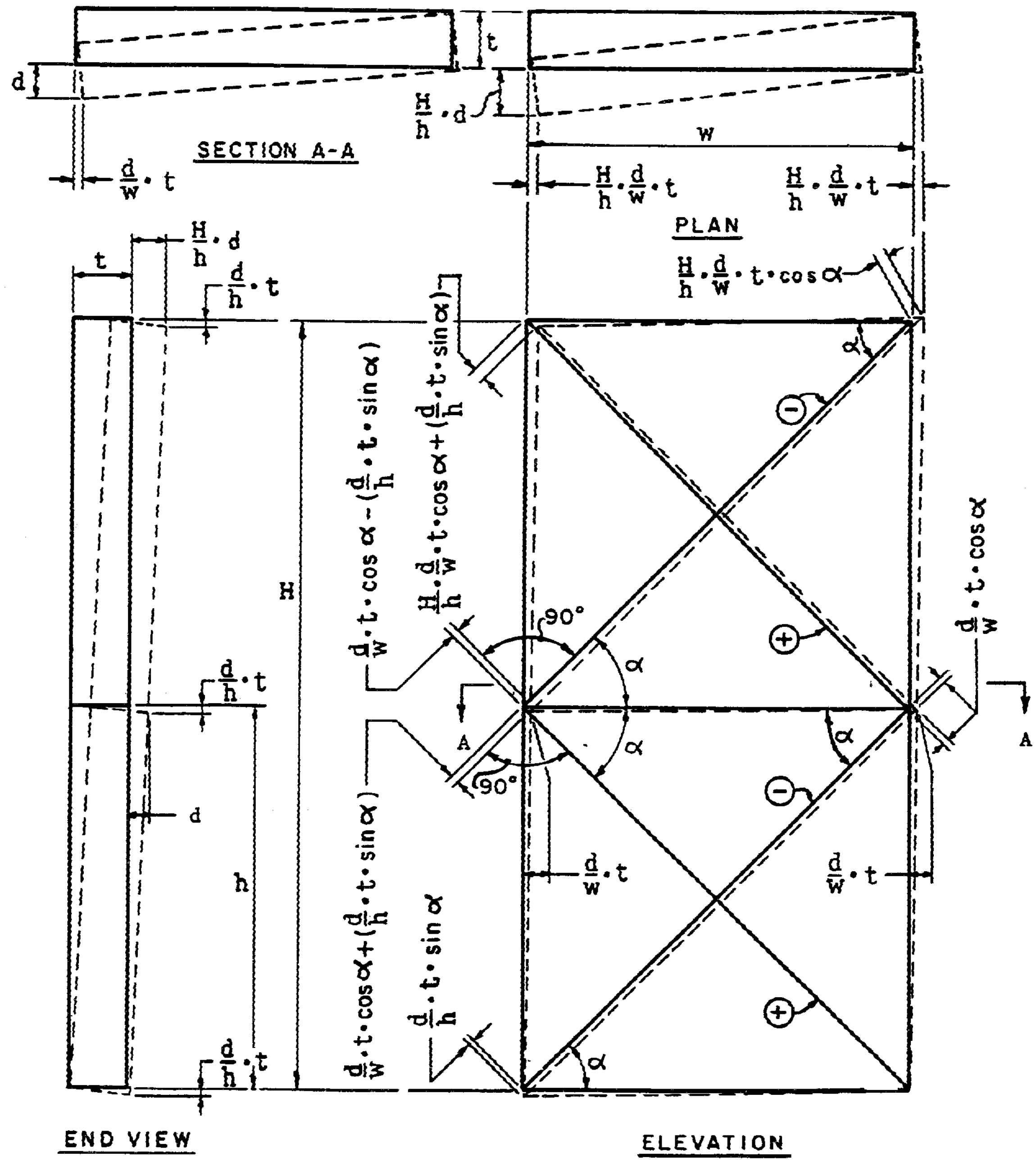


Figure C.17. Displacement of Points of a Vertical Divided Panel

Appendix D
Simplified Ground Motion Amplification Estimate for Concrete Gravity Dams

D.1. Background.

The original EM 1110-2-2702 and EM 1110-2-2105 documents specify that hydrodynamic pressures due to seismic ground motions are to be calculated using the acceleration at the location of the gate. However, no information was given as to how to determine this acceleration. It is well known that the accelerations recorded within a structure are different than the accelerations measured at the base of the dam or in the free-field. This is due to the structural response, and depending on the vibration properties of the structure, can result in significantly higher accelerations within the structure. This effect has been measured at numerous concrete gravity dams where accelerometers have been placed at the base and at the top of the structure.

A brief list of amplification factors measured at various gravity dams is given in Table D.1. Though a comparison to free-field motion cannot be made, Table D.1 demonstrates that there can be large increases in the peak acceleration for concrete gravity dams. When designing a structure located at some height of a concrete gravity dam, such as spillway gates, a reasonable estimate of this amplification must be made in order to determine the actual loading on the structure. One method to accomplish this would be to build a finite element model. This may be the best course of action in many cases where the seismic loads govern the design. However, in cases where it is unlikely that the seismic loads govern or the designer is unsure, a simple method can be used to determine if the seismic loads will control the design or not. This paper proposes a simplified method to estimate the amplification factor for the initial evaluation of spillway gates located near the top of concrete gravity dams.

Table D.1
Measured Amplification Factors

Dam	Height (ft)	Event	Maximum Acceleration (g)		Amplification Factor
			Base	Crest	
Dworshak	717	Lincoln, MT (2017)	0.00186	0.0168	9.06
Chief Joseph	236	Nisqually (2001)	0.0023	0.011	4.69
Wynoochee	175	Nisqually, WA (2001)	0.010	0.0361	3.58
		Satsop, WA (1999)	0.012	0.0343	2.97
Detroit	463	Scotts Mills (1993)	0.021	0.164	7.72
Hakkagawa	171	Honshu (2007)	0.17	0.87	5.12
Gin-Mian	115	Meinong (2016)	0.25	0.31	1.24
Takou	252	Tohoku Aftershock (2011)	0.38	1.79	4.71
Kasho	152	Western Tattori (2000)	0.54	2.09	3.87

D.2. Introduction.

The best method for the simplified analysis of concrete gravity dams is referred to here as the pseudo-dynamic method (also commonly called Chopra's method). This method was developed by Chopra and Tan in 1989. This 1989 report was commissioned by USACE for the purposes of extending previous work by the authors on non-overflow sections to gated spillways of concrete gravity dams and is also discussed in EM 1110-2-6053. The method was primarily developed to evaluate peak seismic stresses in the dam. This is accomplished using equivalent lateral loads on the structure, which are developed using the dynamic properties of the dam. Various correlations were developed to estimate the period and damping of the dam-foundation-reservoir system without the use of a finite element model. These vibration properties can be used to determine the amplification in the dam for the purposes of determining the loading on appurtenant structures.

The following sections will give an overview of the theoretical basis for the pseudo-dynamic method, compare the results of the method with existing finite element models, and provide two approaches to estimate a conservative amplification factor in lieu of the pseudo-dynamic approach for use in screening seismic load cases.

D.2.1. Pseudo-Dynamic Method Overview.

The pseudo-dynamic method is a form of the generalized single degree of freedom method for evaluating structures with distributed mass and stiffness (Chopra, Dynamics of Structures: Theory and Applications of Earthquake Engineering, 2007). The basic concept for this method is to convert a system with distributed mass and stiffness, which would have an infinite number of vibration modes, to a single degree of freedom system. This is done by determining a generalized mass and stiffness that is representative of the distributed mass and stiffness in the system. Additionally, a generalized excitation function must be defined since the inertial loads are also distributed with the mass. The equation of motion for the single degree of freedom system defined by Figure D.1 is given in Equation D.1.

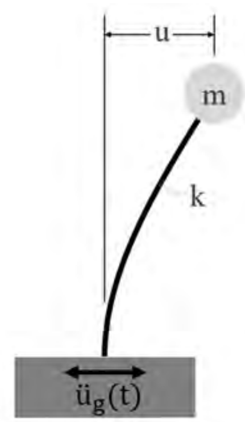


Figure D.1. Single Degree of Freedom System

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g(t) \qquad \text{(Equation D.1)}$$

Where m = lumped mass
k = system stiffness
c = system damping coefficient
$\ddot{u}_g(t)$ = acceleration time history of the ground

Since this is a single degree of freedom system, the mass, damping, and stiffness are defined with discrete values. For a cantilever structure such as a gravity dam where the mass and stiffness are a function of height, z, the generalized mass ($\tilde{m}$) and stiffness ($\tilde{k}$) can be found using Equations D.2 and D.3. Generally, these integrals are evaluated numerically by dividing the dam into some discrete number of horizontal slices. In these equations the mass and stiffness are combined with the mode shape of the structure and integrated along the full height of the structure to arrive at a single value representing the mass and stiffness (Figure D.2). Similarly, the generalized excitation multiplier ($\tilde{L}$) is found with Equation D.5. The damping is determined from the mass and stiffness along with a specified damping ratio (ζ) as shown in Equation D.4.

$$\tilde{m} = \int_0^{H_s} m(z)[\varphi(z)]^2 dz \qquad \text{(Equation D.2)}$$

$$\tilde{k} = \int_0^{H_s} EI(z)[\varphi''(z)]^2 dz \qquad \text{(Equation D.3)}$$

$$\tilde{c} = 2\zeta\sqrt{\tilde{k}\tilde{m}} \qquad \text{(Equation D.4)}$$

$$\tilde{L} = \int_0^{H_s} m(z)\varphi(z) dz \qquad \text{(Equation D.5)}$$

Where H_s = height of structure
z = location in structure above the base
m(z) = mass variation with height
EI(z) = stiffness variation with height
$\varphi(z)$ = Mode shape

With the distributed mass and stiffness system defined, Equation D.1 can be modified to Equation D.6. Dividing through by the generalized mass results in Equation D.7, which is the standard single degree of freedom system equation of motion with the ground motion scaled linearly by the factor $\tilde{\Gamma}$.

$$\tilde{m}\ddot{x} + \tilde{c}\dot{x} + \tilde{k}x = -\tilde{L}\ddot{u}_g(t) \qquad \text{(Equation D.6)}$$

$$\ddot{x} + 2\zeta\omega_n\dot{x} + {\omega_n}^2 x = -\tilde{\Gamma}\ddot{u}_g(t) \qquad \text{(Equation D.7)}$$

Where $\omega_n = \frac{2\pi}{T_n} = \sqrt{\frac{\tilde{k}}{\tilde{m}}}$

$\tilde{\Gamma} = \frac{\tilde{L}}{\tilde{m}}$

This means that if the frequency and damping of the distributed mass and stiffness system can be determined, the peak response of the system can be determined directly from a response spectrum scaled by $\tilde{\Gamma}$. Additionally, if the mode shape is known, the deflection at any point in the structure can be determined at any time. The amplification factor at any height in the structure can therefore be determined from Equation D.8.

$$A(z) = \frac{\left[S_A(T_1, \zeta)\tilde{\Gamma} - PGA\right]\varphi(z) + PGA}{PGA} \qquad \text{(Equation D.8)}$$

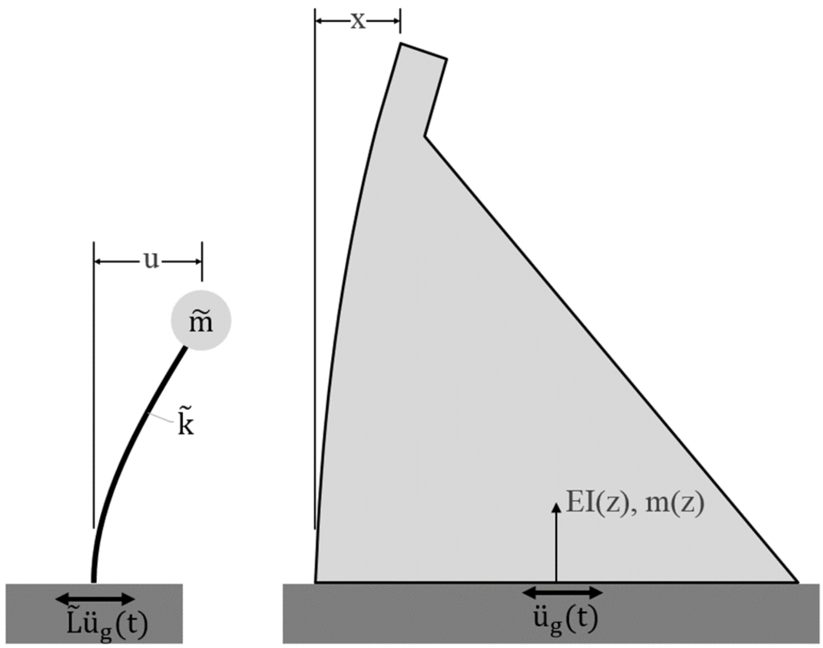


Figure D.2. Generalized Single Degree of Freedom Representation of a Distributed Mass and Stiffness System

This method reduces the continuous structure down to a single degree of freedom system by constraining it to vibrate only in one mode shape. This requires that an appropriate mode shape for the continuous structure is known ahead of time. The shape can be approximated, but the accuracy of the method hinges on the accuracy of this approximation. For simple systems the mode shape, generalized stiffness, and generalized mass can be determined directly from analysis of the structure. For gravity dams, the stiffness is heavily influenced by interaction with the foundation and the mass is influenced by the interaction with the reservoir.

To use the generalized single degree of freedom method for concrete gravity dams, Chopra developed relationships to determine the needed vibration properties for the method. The relationships are published in tables in Chopra and Tan (1989). This was done by performing numerous parametric studies using the frequency domain finite element program EAGD84. Many of the relationships used in the pseudo-dynamic approach are published in tables. These tables can be used to work through the method and develop the amplification factor.

The program EAGD84 is not a general purpose finite element code and has specific assumptions that are important to understand since these assumptions also apply to the pseudo-dynamic method. The foundation, reservoir, and dam are each evaluated as subsystems using their respective governing equations. Once evaluated separately the equations are combined to account for the interaction between each sub-system. Each sub-system is idealized in order to develop the governing equations.

Figure D.3 shows the idealized model of the dam used by EAGD84. This shows that the foundation rock is considered to be an infinite half space. The material for the foundation is idealized as an isotropic visco-elastic material. The reservoir is assumed to extend indefinitely upstream of the dam. Damping in the reservoir is considered by allowing some energy in the water to be absorbed in a silt layer upstream of the dam or by the foundation rock. The dam is modelled using the finite element method. Analyzing the system in this manner also allows for the ground motion to be applied as the free-field ground motion at the base of the dam.

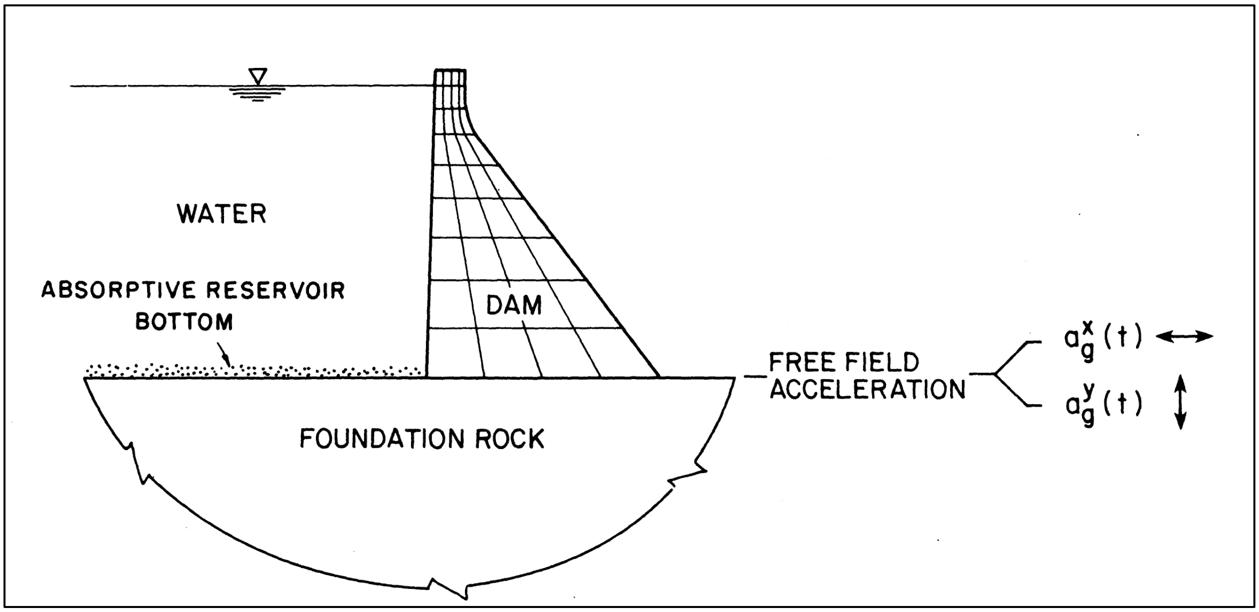


Figure D.3. EAGD84 Idealized Model

As noted previously, the accuracy of this method relies on the ability to approximate the mode shape and other vibration properties accurately. Based on the EAGD84 analyses for standard shapes and real dam geometries, the mode shape was found to be very similar for gravity dams of varying heights since as the height of a gravity dam increases, the base width increases as well. This relative proportionality of gravity dams results in little variation of the mode shape when the geometry of the dam fits the standard dimensions. The standard shape used in the study assumed a constant pier height of 60 ft regardless of the height of the dam. Though Equation D.8 is a general equation, in cases where there is a large variation from the standard shape, the published mode shape, period, and damping cannot be used. However, these properties can be calculated for the structure in question and applied in Equation D.8.

D.2.2. Comparison of Pseudo-Dynamic Method with Finite Element Models.

The period in the upstream/downstream direction of three different height dams was calculated using finite element software and compared to the period calculated using the pseudo-dynamic method using the same material properties and pool elevations. The three finite element models are shown in Figure D.4. The models for Foster and Dworshak are made of shell elements and only contain the concrete dam, piers, and foundation; while the Green Peter model is made of three dimensional elements and has the gates and bridge included in the model.

The comparison of the natural period of the structure calculated with the pseudo-dynamic method and the three finite element models is shown in Table D.2, where H_s is the structure height. The table shows general agreement between the methods. The finite element fundamental period is somewhat longer than the pseudo-dynamic method predicts. However, the pseudo-dynamic period is reasonable since it must account for all modes of vibration with a single period. For comparison, the weighted average period from the finite element model (using the mass participation as the weighting factor) is also shown in the table. The pseudo-dynamic period is found to fall between the finite element fundamental period and the weighted average period.

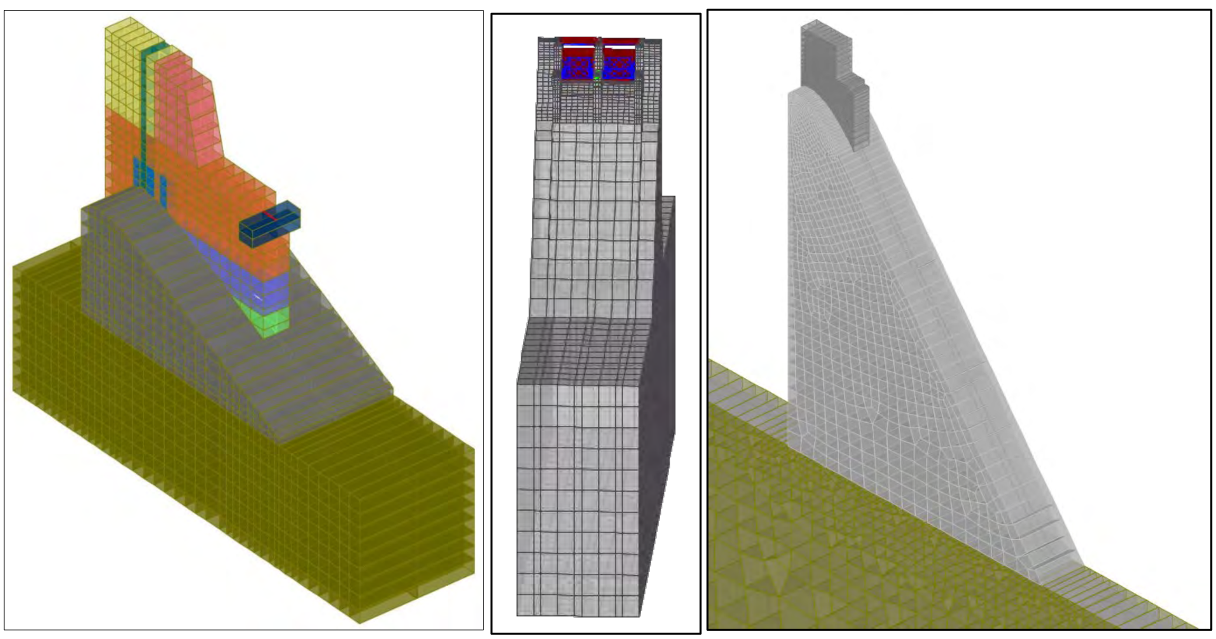


Figure D.4. Finite Element Models: Foster (Left), Green Peter (Middle), and Dworshak (Right)

Table D.2
Comparison of Pseudo-Dynamic and Finite Element Modeling

Dam	H_s	Period		
		Pseudo-Dynamic	Finite Element	
			1st Mode	Weighted
Foster	88	0.082	0.09	0.075
Green Peter	340	0.32	0.37	0.31
Dworshak	503	0.59	0.67	0.54

Only two of the dams (Foster and Green Peter) have time history analysis results for comparison of the amplification factor. For the mid height dam (Green Peter), the amplification factor was calculated using Equation D.8, the pseudo-dynamic method, and the uniform hazard spectrum for a 1/2475 AEP at the site. The effective damping using this method was found to be 15%, which accounts for added damping from the reservoir and foundation. The damping scale factor was estimated from Rezaeian et al. (2012) to convert the uniform hazard spectrum at 5% to that at 15% damping. The amplification factor at the trunnion using this method was found to be 3.3. This can be compared to the response of the finite element model analyzed with a series of time histories that, on average, match the 1/2475 AEP uniform hazard spectrum.

The finite element model was run with a constant modal damping of 5% applied to the model. Figure D.5 shows the mean and extreme values of the response spectrum of a node at the trunnion for the suite of time histories. Figure D.6 shows the response spectra divided by the input motion response spectra to determine the amplification relative to the free-field motion. The vertical lines shown in the figures are the periods of the first six modes of vibration. This illustrates that the frequency content near the fundamental mode and several higher modes is being amplified. However, the peak acceleration that the Tainter gate will experience is represented by the 0.01 second period, which represents the peak acceleration of the trunnion in the time history.

The amplification chart shows that the 3.3 factor predicted by the pseudo-dynamic approach is well within the range of amplification seen in the FE model. If the damping in the pseudo-dynamic method is set to 5%, and time histories are used to assess the variability in the amplification, the pseudo-dynamic method predicts amplification between 3.1 and 4.8, which is very close to the range seen in the finite element model.

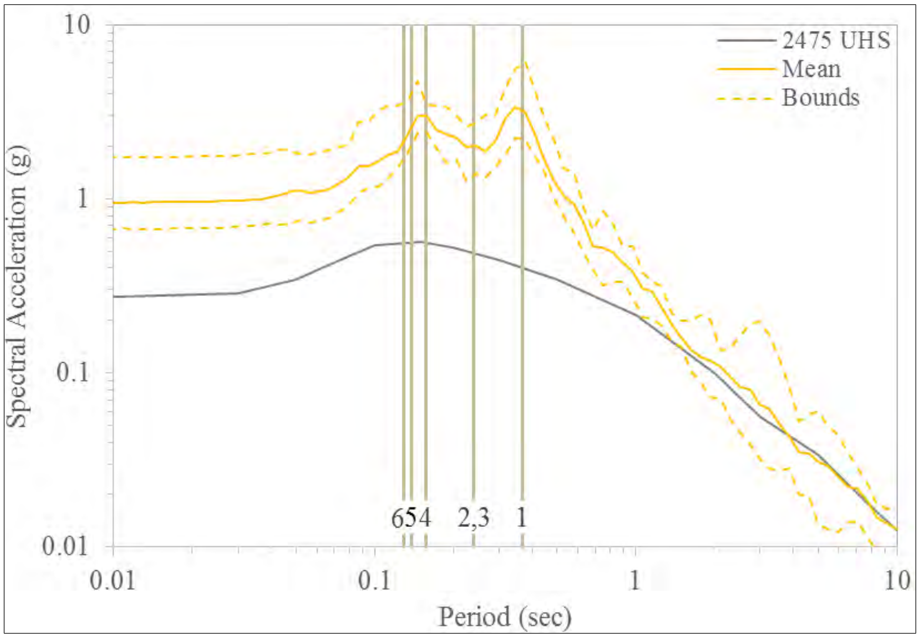


Figure D.5. Response Spectra at the Trunnion for Green Peter Dam

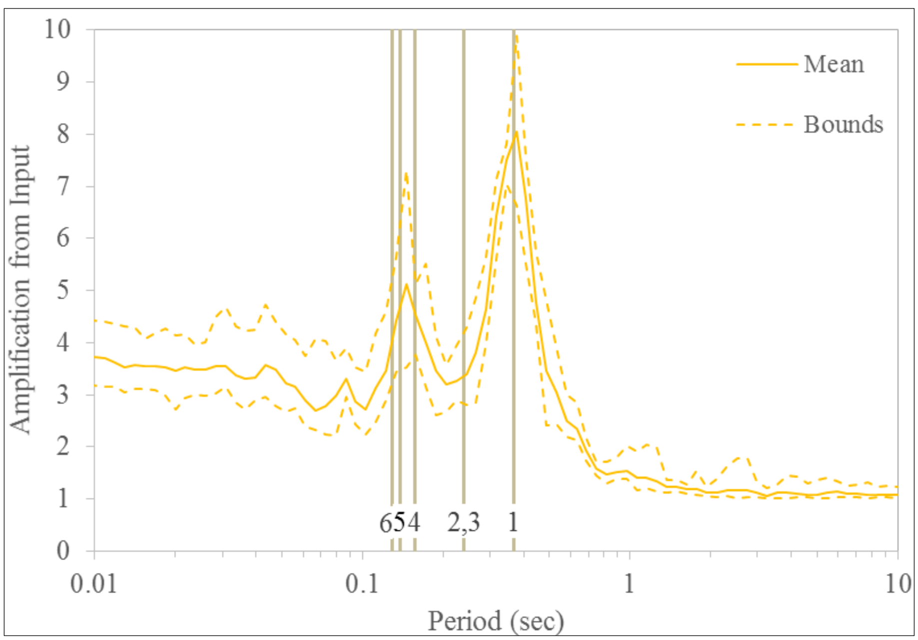


Figure D.6. Amplification at the Trunnion for Green Peter Dam

The short dam (Foster) was modelled using non-linear shell elements in the finite element model. The Foster amplification chart is shown in Figure D.7, which was calculated in the same manner as Figure D.6. The pier is much more significant relative to the height of the dam at Foster as compared to Green Peter, which results in the fundamental mode being weak axis bending of the pier in the cross-canyon direction. Therefore, the fundamental mode in the upstream/downstream direction is the 2nd mode, which shows significant amplification in Figure D.7. The pseudo-dynamic analysis of Foster predicts 10% effective damping, which results in an amplification of 2.4 and is at the upper end of the amplification factors from the non-linear finite element model.

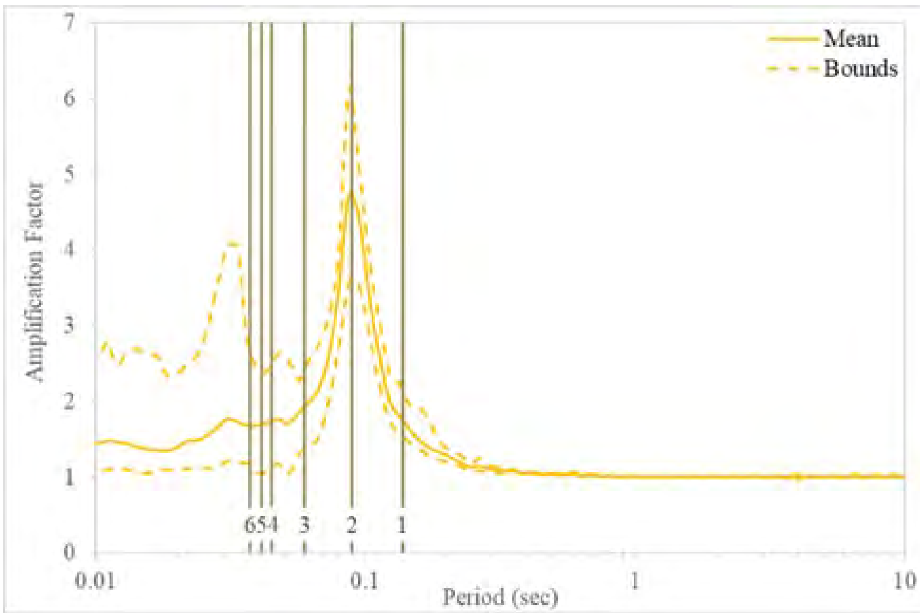


Figure D.7. Amplification at the Trunnion for Foster Dam

Figures D.6 and D.7 and Table D.2 illustrate that the pseudo-dynamic method can be used to estimate reasonable amplification factors for gates located at the top of the dam.

D.2.3. Simplified Screening Method.

In order to use Equation D.8, the period, damping, mode shape, and scale factor must be estimated. While the pseudo-dynamic method is not complicated and can be used to determine these factors, some further simplifications can be made to develop a conservative estimate of the acceleration of the gates at the top of a dam.

The mode shape was evaluated for several structural heights by Chopra and Tan (1989). For short structural heights, the pier becomes more significant relative to the height of the structure and begins to affect the mode shape. Since the mode shape is normalized to 0.0 and 1.0, most of the variation occurs at mid-height. Chopra created two mode shapes: one for short spillways and one for tall spillways. These two mode shapes are not significantly different. For the purposes of determining an amplification factor, there is little gain in using two mode shapes. Therefore, a single mode shape was found to represent all structures. This is shown in Figure D.8 and the equation for the mode shape is given in Equation D.9.

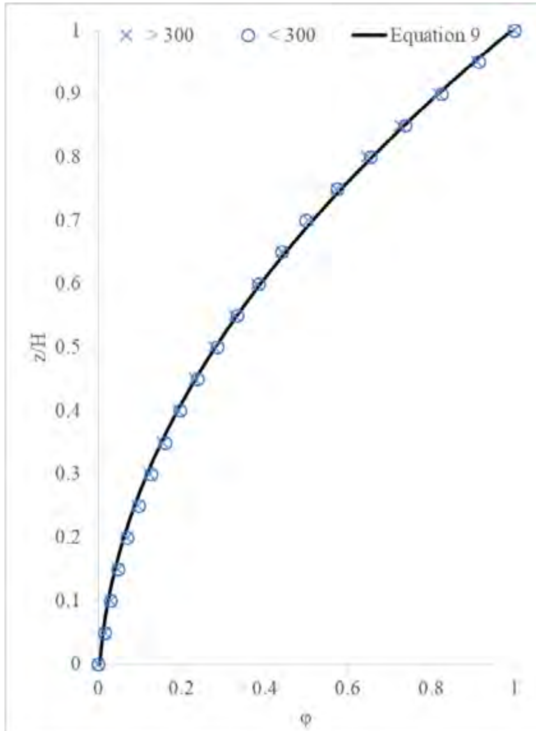


Figure D.8. Fundamental Mode Shape

$$\varphi = 23.41\sin^2\left(\frac{2\pi z}{32.18H_s} + 0.0122\right) \qquad \text{(Equation D.9)}$$

For gravity dams, the scale factor can be assumed to be 2.8, which represents an average across a range of structure heights with a pool high enough to load the spillway gates, as shown in Table D.3.

Table D.3
Scale Factor for Three Dam Heights

Dam	H_s	$\tilde{\Gamma}$
Foster	88	2.75
Green Peter	328	2.97
Dworshak	503	2.77

With the mode shape and scale factor approximated, the spectral acceleration must be determined. The damping can be assumed to be 5%. The period can be calculated for the structure or can be conservatively taken as the period that corresponds to the highest spectral acceleration for the site.

To understand the limits of this method, two extreme cases can be evaluated. Evaluating the amplification at the top of the structure simplifies Equation D.8 to Equation D.10. When evaluating a perfectly rigid structure, S(T, ζ) becomes equal to PGA. This means the amplification will be equal to $\tilde{\Gamma}$. If $\tilde{\Gamma}$ is assumed to be 2.8, then the amplification will never be less than 2.8, when in the case of a rigid structure the amplification should be equal to 1.0. As Equations D.2 and D.5 indicate, the scale factor, $\tilde{\Gamma}$, is dependent on the distribution of mass and stiffness in the structure, and will therefore vary for different height/shape structures and pool levels. The more tapered a structure is, the higher the scale factor will be. This method should therefore give a conservative estimate in most cases when the stiffness is higher than a scale factor of 2.8 would imply.

As the structure becomes wider relative to its height with less of a taper, the scale factor will reduce. For a structure twice as wide as tall, a scale factor of 1.5 is reasonable. For very stiff structures, the amplification could be set to 1.0 without going through any calculation. The scale factor can be calculated for any structure, but will likely require finite element to determine the mode shape.

At the other end of the spectrum, if an infinitely flexible structure is evaluated, S(T, ζ) becomes zero, and the amplification factor also becomes zero. This is correct, illustrating that the equation results in continuously decreasing amplification factors as the period shifts to the right of the peak spectral acceleration.

$$A(z) = \frac{\left[\tilde{\Gamma} S(T, \zeta) - PGA\right] + PGA}{PGA} \qquad \text{(Equation D.10)}$$

The simplified method is illustrated with the example below.

16.3.3.1. Example.

For the example problem, the acceleration at the trunnion of a gated spillway on a concrete gravity dam is needed for the design of the Tainter gates. The top of the dam is at elevation 600, the base of the dam is at elevation 250, the trunnion is at elevation 562, and the pool being evaluated is at elevation 590. The upstream/downstream width of the dam at the base is 400 ft. The modulus of elasticity of the concrete is 4×10^6 psi, which is similar to the modulus of elasticity of the rock foundation. If the Tainter gates breach, there is the potential for downstream life loss. This structure is therefore a critical structure and the site specific MCE is shown in Figure D.9 calculated for 5% damping.

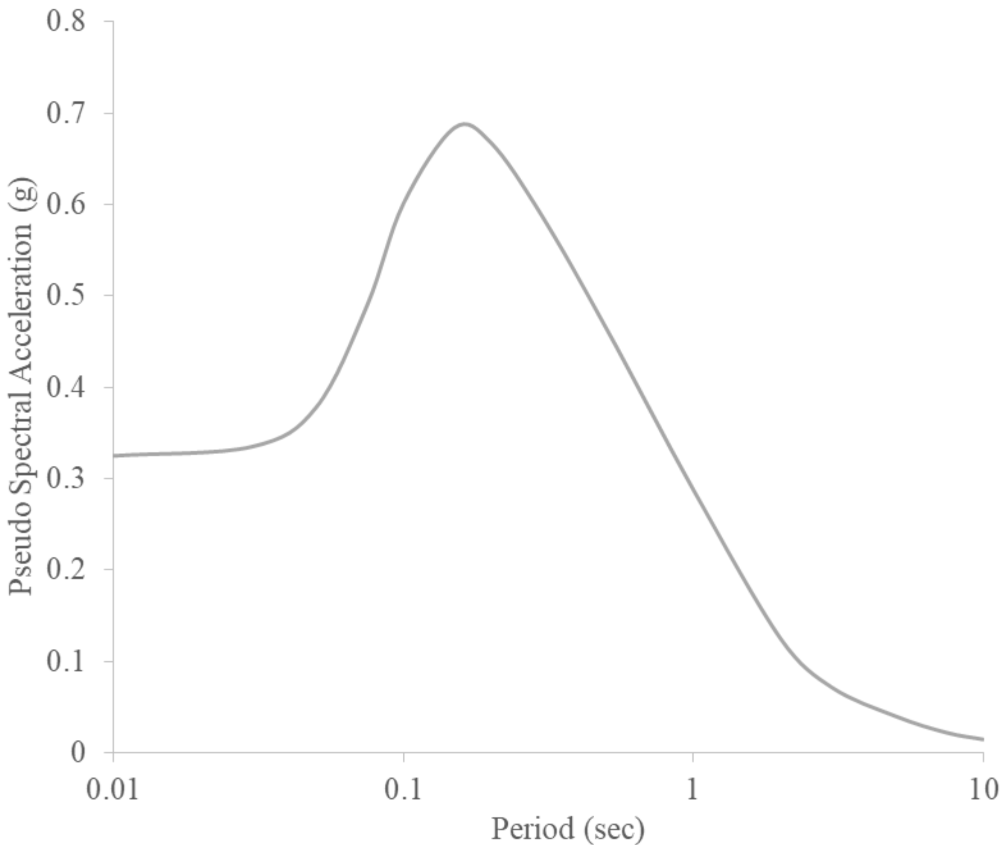


Figure D.9. Example Problem Design Spectrum

16.3.3.2. Solution 1.

- Step 1: Calculate height and gate location.

$$H_s = 600 - 250 = 350$$
$$z = 562 - 250 = 311.5$$
$$\frac{z}{H_s} = \frac{311.5}{350} = 0.89$$

- Step 2: Calculate normalized mode shape.

Since the structure is a gravity dam, the standard gravity dam mode shape can be used.

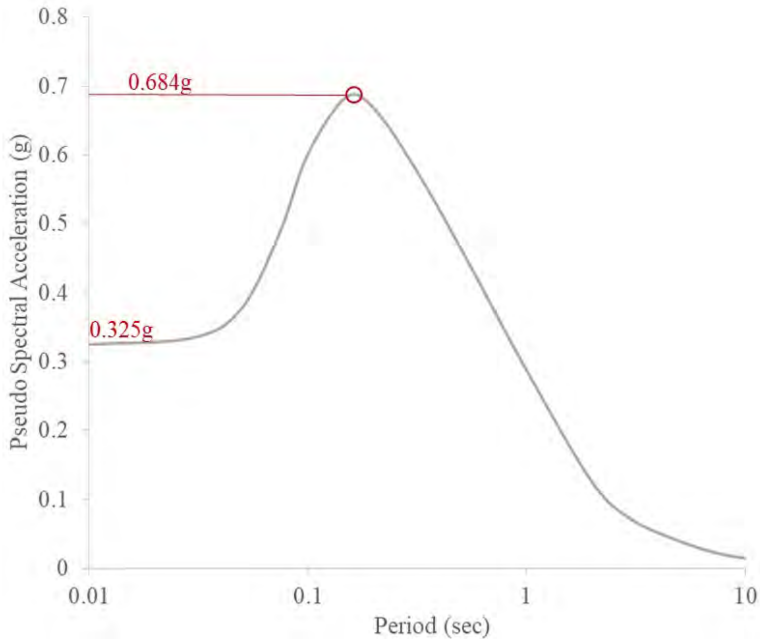
$$\varphi(z) = 23.41\sin^2\left(\frac{2\pi z}{32.18H_s} + 0.0122\right) = 23.41\sin^2\left(\frac{2\pi}{32.18}0.89 + 0.0122\right) = 0.8$$

- Step 3: Estimate the scale factor.

Since the structure is a concrete gravity dam and the base is 1.14 times the height, the scale factor can be assumed equal to 2.8.

- Step 4: Estimate the spectral acceleration and peak ground acceleration.

The period of the dam is not given, so conservatively take the spectral acceleration at the peak of the response spectrum.



- Step 5: Determine the amplification and acceleration at the trunnion.

The acceleration at the trunnion for the MCE is 1.6 g which corresponds to an amplification above the peak ground acceleration of 4.9.

$$A(z) = \frac{[(0.684)(2.8) - 0.325]0.8 + 0.325}{0.325} = \frac{1.6}{0.325} = 4.9$$

16.3.3.3. Solution 2.

- Steps 1–3: Remain the same as above.

- Step 4: Estimate the spectral acceleration and peak ground acceleration.

If the period of the dam is approximated, a better estimate of the spectral acceleration can be obtained. Chopra and Tan recommend Equation D.11 to estimate the period of the dam, where β is a function of the height of the dam as well as the period lengthening effect of the water and foundation. To accurately determine β, the full pseudo-dynamic method must be used. However, based on the results of the three dams above, β can be estimated using Equation D.12 if the pool is near the top of the dam (>80% of the structure height) and the modulus of the foundation is near that of the dam (i.e., a relatively hard rock foundation).

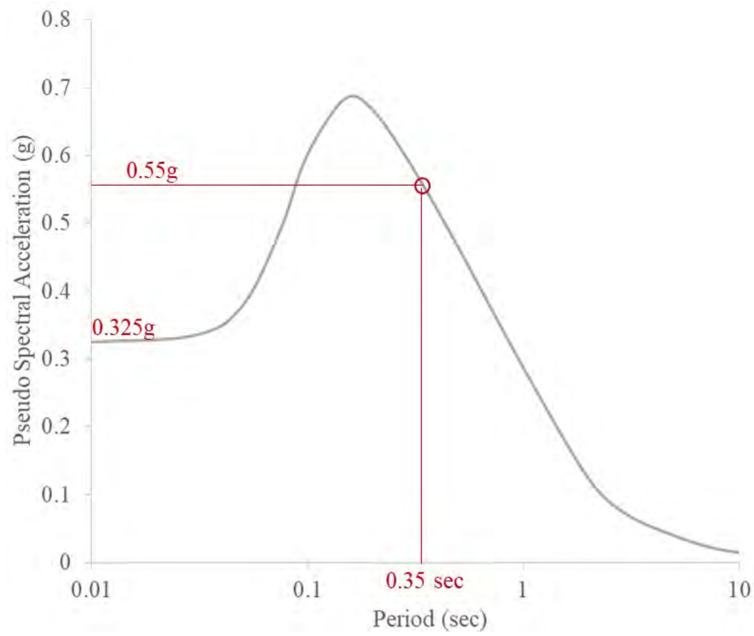
$$T = \beta \frac{H_s}{\sqrt{E_s}} \qquad \text{(Equation D.11)}$$

$$\beta = 2.49 \frac{H_s}{\sqrt{E_s}} + 1.56 \qquad \text{(Equation D.12)}$$

$$\beta = 2.49\frac{350}{\sqrt{4x10^6}} + 1.56 = 2.0$$

$$T = 2.0\frac{350}{\sqrt{4x10^6}} = 0.35 \text{ sec}$$

Based on this period, the spectral acceleration is 0.55g and the PGA is unchanged.



- Step 5: Determine the amplification and acceleration at the trunnion.

The acceleration at the trunnion for the MCE when considering the period of the structure is 1.3 g, which corresponds to an amplification above the peak ground acceleration of 4.0.

$$A(z) = \frac{[(0.55)(2.8) - 0.325]0.8 + 0.325}{0.325} = \frac{1.3}{0.325} = 4.0$$

D.3. Conclusion.

The pseudo-dynamic method (Chopra and Tan, 1989) can be used to determine reasonable accelerations at the elevation of spillway gates for use in the initial seismic evaluation of the gates. Using some simplifying assumptions, a conservative estimate can be made with very little effort to rapidly determine if the seismic load case will be a governing load for the gates. In the event that seismic loading is a governing load case, finite element modeling of the dam will be required to perform the design.

Appendix E
Load Combination Examples

E.1. Introduction. This Appendix provides example project load combinations. It demonstrates how load combinations are determined, but it does not show how loads are calculated. The examples are a lock miter gate and a spillway crest Tainter gate. The examples are based on actual sites. The site names are not identified because some of the data used in the examples were not verified before the examples were published. This does not affect the procedures used to develop the load combinations for the examples.

E.2. Miter Gate.

E.2.1. Introduction. This example is an upstream miter gate on a 110-ft wide lock. The lock is located on the Mississippi River as shown in Figure E.1.

Figure E.1. Miter Gate Example

E.2.2. Geometry. Each vertically framed gate is 61 ft 5 in. wide and 22 ft 7 in. high. The gate is depicted in Figure E.2. The top of the sill that supports the gate is elevation 621 ft msl. The top of the gate is elevation 644 ft msl.

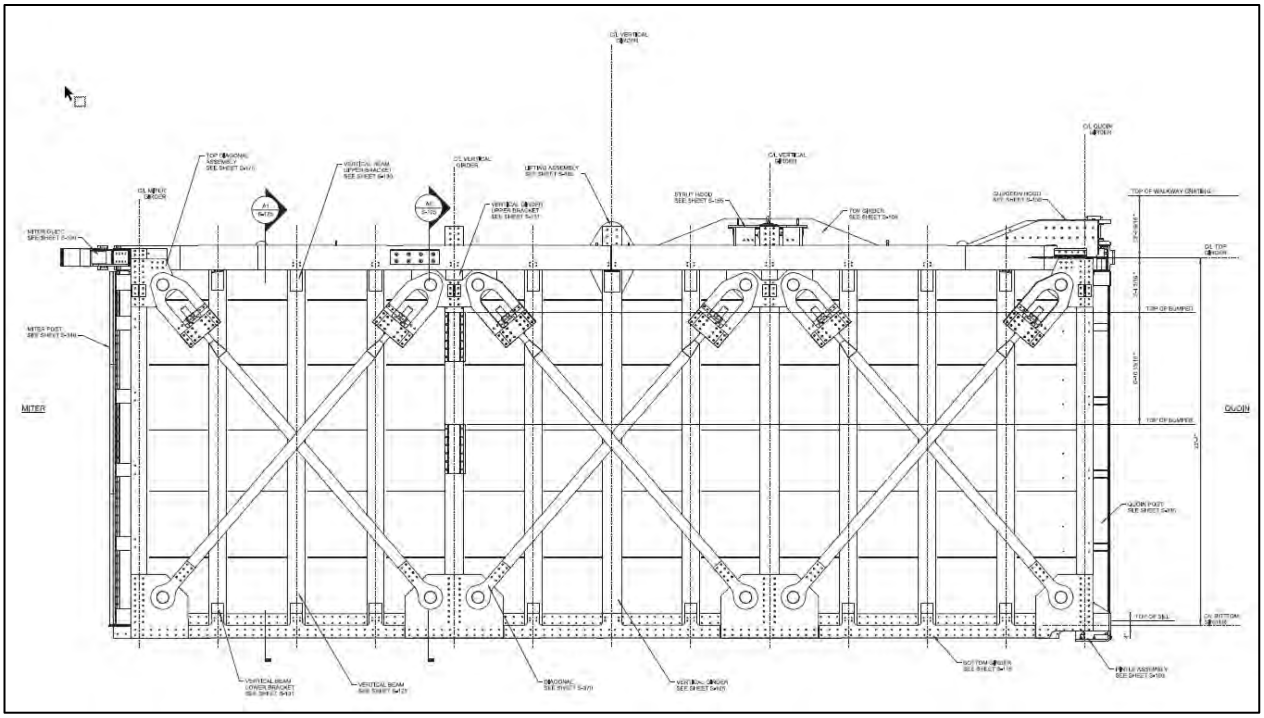


Figure E.2. Miter Gate Geometry

E.2.3. Structure Classification. The lock is on a dam in the Mississippi River. The head on the lock and dam is fairly low. If a miter gate failed, it would not create stages that would result in loss of life due to uncontrolled release of water. There would be large economic impacts that could be considered in the classification; however, for this example, the gate will be classified as normal.

E.2.4. Dead Load (D). The dead load of the gate is calculated from member sizes including a factor for connections. The dead load is a permanent load with a load factor (γ_p) of 1.2 or 0.9, each to be applied for maximum effect according to paragraph 4.3.3.

E.2.5. Gravity Load (G). According to paragraph 4.2.2.1, a 1-in. thick layer of mud is assumed in all areas where silt can accumulate. For this vertically framed gate, this is only on the bottom girder. The gravity load is a permanent load with a load factor (γ_p) of 1.6 or 0, each to be applied for maximum effect according to paragraph 4.3.3. For fatigue load combinations, an expected gravity load should be used for G. For waterways with little or no silt load, G may be zero for fatigue calculations.

E.2.6. Hydrostatic Loadings (Hs).

E.2.6.1 A plot of pool and tailwater vs. discharge is shown in Figure E.3. Figure E.4 shows duration curves for the pool and tailwater. Maximum hydrostatic loading is created at low discharges. Stage frequency relationships were not provided for low discharge for this example. They should be obtained, if possible, from the hydraulic engineers. For the example, design loads can be established from the data that is available.

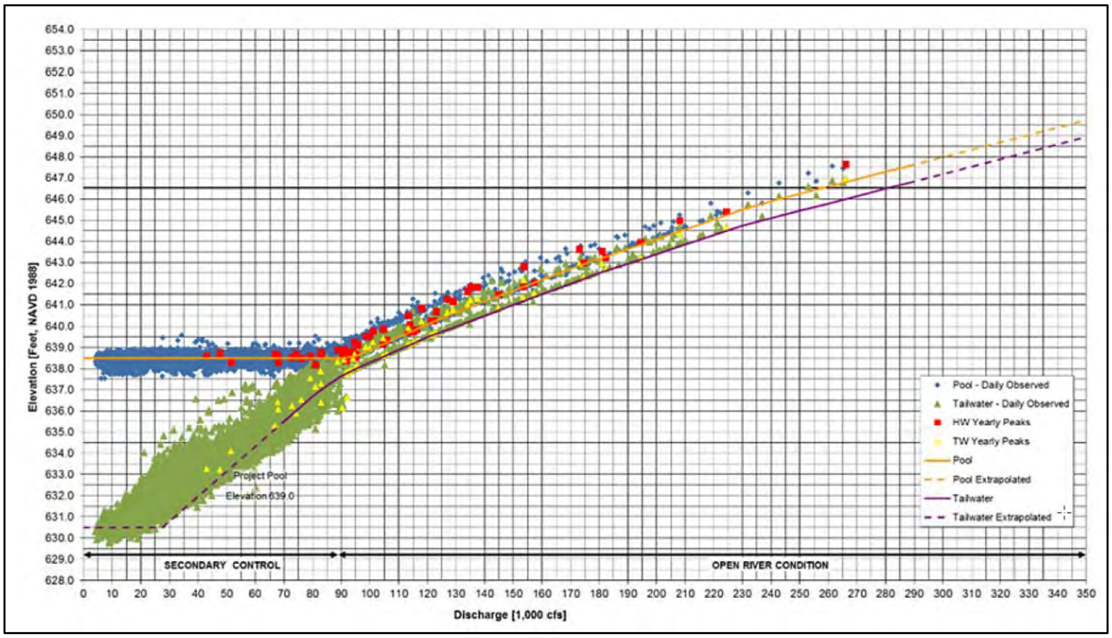


Figure E.3. Pool and Tailwater vs. Discharge

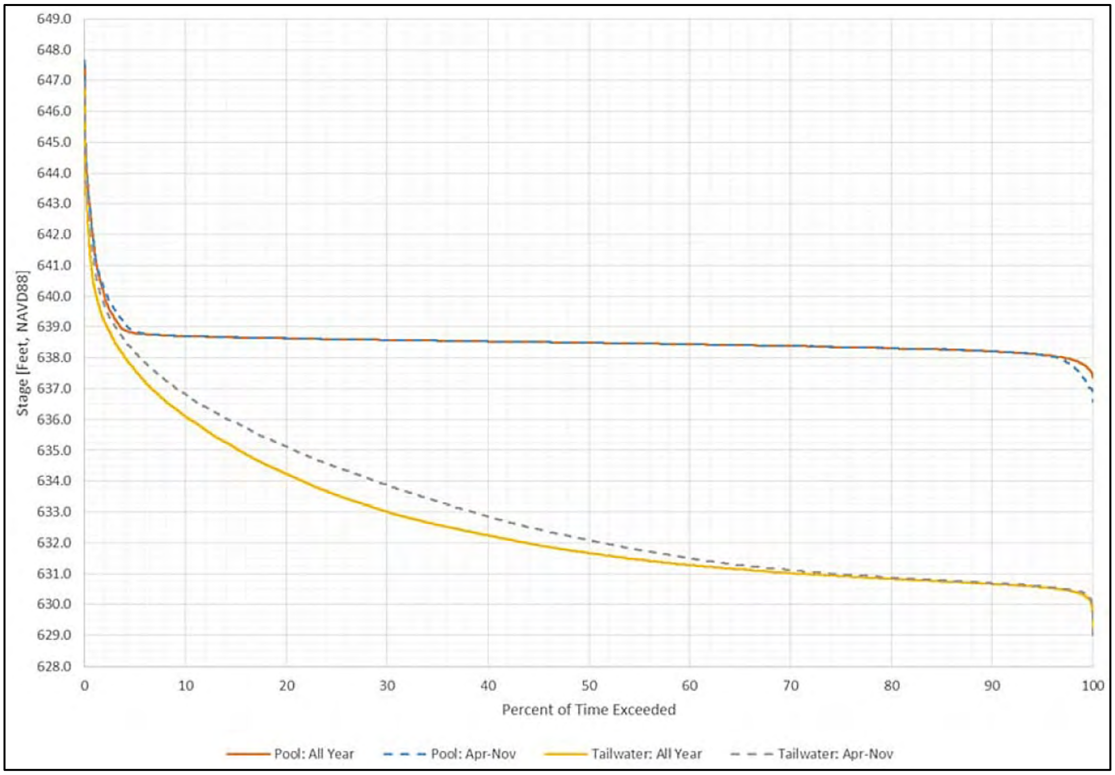


Figure E.4. Pool and Tailwater Duration Curves

E.2.6.2 For hydrostatic loading as a principal load (Hs_{pr}), the maximum loading condition must be identified. The pool varies over about a 1-ft band during low discharges. The maximum pool level for low discharges is elevation 639 ft msl from Figure E.3.

E.2.6.3 Figure E.3 shows a minimum tailwater of about 629.5 ft msl. However, if the pool was lost at the lower lock and dam, a lower tailwater condition could be experienced than shown in Figure E.3. It was determined that the minimum tailwater that could be experienced is elevation 626.3 ft msl. Theses elevations define the maximum $Hs_{pr.}$ Since this load is limited by geometry, it is a principal load condition 2 load according to paragraph 4.3.4.2. The return period for loss of tailwater is not known but has never occurred on the Mississippi and is therefore very rare. Therefore, the load combination will be considered extreme with a principal load factor (γ_{pr}) of 1.3 used from paragraph 4.3.4.2.1.

E.2.6.4 If the tailwater was lowered due to a loss of the dam downstream, there would be no navigation of loaded barges because the channel depth would be insufficient. For the case of hydrostatic principal load during navigation, a tailwater of 629.5 ft msl can be identified from Figures E.3 and E.4 as a low minimum pool. Since this load is limited (without loss of tailwater from a downstream dam break), it is a principal load condition 2 load according to paragraph 4.3.4.2. The return period of the load was determined to be an unusual load. Therefore, the load case will be considered unusual with a principal load factor (γ_{pr}) of 1.4 used from paragraph 4.3.4.2.2.

E.2.6.5 When the water is a companion load (Hs_c), according to paragraph 3.3.3.3, the level with a 10-year return period should be used. This is not known but can be approximated from available data. The pool elevation is 639 ft msl, the upper portion of the operating band for lower discharges. The tailwater level should be low but not extreme. From Figure E.4, the tailwater goes below elevation 630 ft msl less than one percent of the time. This elevation will be used for the tailwater. According to paragraph 4.3.5, a companion load factor (γ_c) of 1.0 is used.

E.2.6.6 The gate will not be designed for dewatering the lock with a full hydrostatic head on it. A separate dewatering system will be provided.

E.2.6.7 For fatigue load combinations, water levels are selected according to paragraph 5.1.3. The stress can be computed using a simplified approach or by a more precise approach. For design, the simplified approach is adequate. For this approach, the magnitude of stress assumed for each stress cycle is the stress in a member caused by the hydrostatic head with a one-year return period. This magnitude of stress is assumed to occur for the number of load cycles expected during the design life. From Figure E.3, this is the operating condition with a pool of 638.5 ft msl and a tailwater of 630.5 ft msl.

E.2.7. Wave Loads (Hw). Wave loads are typically not significant as principal loads for miter gates but may be significant as companion loads, Hw_c. For waves applied as a companion load, a wave using wind speeds from a 10-year return period are used according to paragraph 3.3.3.3. The wind speed probability would need to account for the geometry of the site. According to paragraph 4.3.5, a companion load factor (γ_c) of 1.0 is used.

E.2.8. Hydrodynamic Load (Hd). For strength design of miter gates, Hd is defined as a temporal head of 1.25 ft to account for prop-wash, lock overfill, and seiches (setup and surges) unless there is testing or evidence to support the need for higher design values. Hd is applied as a companion load (Hd_c) to produce maximum load effects on the gate and anchorages, as shown in the load combinations.

E.2.9. Operating Load (Q).

E.2.9.1 The operating load is applied as a principal load (Q_{pr}) as the maximum load that can be exerted by the operating machinery on the gate assuming that it is jammed or stuck. This load is limited and is therefore a principal load condition 2 load according to paragraph 4.3.4.2. The return period of the load is not known but operating to the ultimate capacity of the machinery is not expected. Therefore, a principal load condition 2 (extreme) load factor applies according to paragraph 4.3.4.2.1. A principal load factor (γ_{pr}) of 1.3 is used.

E.2.9.2 For fatigue of anchorage components, operating loads may add to anchor reactions. The expected, normal operating load is used for Q in these load combinations. This can be determined in consultation with the project mechanical engineer.

E.2.10. Barge Impact (BI).

16.3.3.1. For this example, failure of gate would not result in potential loss of life from uncontrolled release of water, and economic consequences from loss of service are considered to be moderate. Other projects in the system were designed using barge impact loads equal to or less than the minimum design values in this manual. A risk assessment was performed and it was determined that the minimum barge impact loads from this manual are to be used.

16.3.3.2. Barge impact load is modeled using point loads as shown in Figure 9.2. Two cases are applied. A 400-kip load is applied in the downstream direction to girders above pool level at the miter point. In addition, a 250-kip load is applied anywhere in the girder span at which a single barge may impact (unsymmetrical loading). This location is anywhere in the span of the gate but at least 35 ft from either lock wall. A principal load factor (γ_{pr}) of 1.3 is used according to paragraph 4.3.4.5.

E.2.11. Thermally Expanding Ice Load (IX). The gates will be kept ice free in the winter and barge impact load on miter gates usually governs over IX. Therefore, IX will not be evaluated as a separate load combination.

E.2.12. Live Loads (L). For design of the walkway on top of the gate, a live load of 100 psf is used according to paragraph 4.2.9. The load factor is 1.6 according to paragraph 4.3.4.4.

E.2.13. Earthquake (EQ). The site is in a low seismicity region. The Operating Base Earthquake (OBE) earthquake is used. As a normal structure, the Maximum Design Earthquake (MDE) is used with a return period of 950 years. The standard earthquake motions from the U.S. Geological Survey (USGS) can be used to determine if earthquake controls for this site. The earthquake would be applied to pool level with the companion hydrostatic loads with a pool of 639 ft msl and a tailwater of 630 ft msl.

E.2.14. Load Combinations.

E.2.14.1 Load Combination 1: Strength Limit State. Gate Closed.

E.2.14.1.1 Load Combination 1a. Upper gate subjected to maximum hydrostatic loading, Hs_{pr}, with tailwater at elevation 626.3 ft msl ($Hs_{(626.3)}$), and applicable companion wave loads, Hw_c.

$$1.2\ D + 1.6\ G + 1.3\ Hs_{(626.3)} + 1.0\ Hw_c$$

$$0.9\ D + 0.0\ G + 1.3\ Hs_{(626.3)} + 1.0\ Hw_c$$

E.2.14.1.2 Load Combination 1b. Lower gate subjected to maximum operational hydrostatic loading, Hs_{pr}, at minimum navigation tailwater of 629.5 ft msl ($Hs_{(629.5)}$) with companion hydrodynamic (temporal head), Hd_c, of 1.25 ft. The minimum navigation tailwater is an unusual load.

$$1.2\ D + 1.6\ G + 1.4\ Hs_{(629.5)} + 1.0\ Hd_c$$

$0.9\ D + 0.0\ G + 1.4\ Hs_{(629.5)} + 1.0\ Hd_c$

E.2.14.2 Load Combination 2: Strength Limit State. Gate Open.

E.2.14.2.1 Load Combination 2a. Dead Load Only.

$1.4\ D$

E.2.14.2.2 Load Combination 2b. Dead Load Plus Mud and Ice. (There is no need to analyze the case using the lower load factors for this combination.)

$1.2\ D + 1.6\ G$

E.2.14.2.3 Load Combination 2c. Gate Operating on an Obstruction. Gate subjected to dead, gravity, and maximum machinery load, Q_{pr}.

$1.2\ D + 1.6\ G + 1.3\ Q_{pr}$

$0.9\ D + 0.0\ G + 1.3\ Q_{pr}$

E.2.14.3 Load Combination 3: Strength Limit State. Gate Closed. Barge Impact. Loads consist of barge impact loads, BI_{pr}, and companion hydrostatic load, Hs_c.

E.2.14.3.1 For impact on the miter:

$1.2\ D + 1.6\ G + 1.3\ (400) + 1.0\ Hs_c$

$0.9\ D + 0.0\ G + 1.3\ (400) + 1.0\ Hs_c$

E.2.14.3.2 For impact 35 ft from the wall:

$1.2\ D + 1.6\ G + 1.3\ (250) + 1.0\ Hs_c$

$0.9\ D + 0.0\ G + 1.3\ (250) + 1.0\ Hs_c$

E.2.14.4 Load Combination 4: Strength Limit State. Gate Closed. Loads consist of extreme thermally expanding ice force, IX_X. Not used for this gate.

E.2.14.5 Load Combination 5: Strength Limit State. Live Load. Gate Closed. Loads consist of live load (L) of 100 psf, as the principal load, plus dead, gravity, and companion hydrostatic, Hs_c. Live load is additive to D and G, and the lower load factors for those loads is not needed.

$1.2\ D + 1.6\ G + 1.6\ L + 1.0\ Hs_c$

E.2.14.6 Load Combination 6: Strength Limit State. Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading (Hs_c), dead load, and gravity loads.

E.2.14.6.1 For standard and site-specific OBE ground motion analysis:

$1.2\ D + 1.6\ G + 1.5\ EQ + 1.0\ Hs_c$

$0.9\ D + 0\ G + 1.5\ EQ + 1.0\ Hs_c$

E.2.14.6.2 For standard MDE ground motion analysis (assuming earthquake does not control for the site and a site-specific analysis is not performed):

$1.2\ D + 1.6\ G + 1.25\ EQ + 1.0\ Hs_c$

$0.9\ D + 0\ G + 1.25\ EQ + 1.0\ Hs_c$

E.2.14.7 Load Combination 7: Fatigue Limit State. Design must satisfy requirements for either Infinite Life or Finite Life:

E.2.14.7.1 Load Combination 7a. Fatigue Limit State I. Infinite Life. Fatigue created by hydrostatic forces on a closed gate and by hydrodynamic forces from opening and closing the gate.

2.0 Hs or 2.0 Hd

E.2.14.7.2 Load Combination 7b. Fatigue Limit State II. Finite Life. Fatigue created by hydrostatic forces on a closed gate and by hydrodynamic forces from opening and closing the gate.

1.0 Hs or 1.0 Hd

E.2.14.7.3 Load Combination 7c. Fatigue Limit State I. Infinite Life. Fatigue in anchorage components.

2.0 D + 2.0 G + 2.0 Q

E.2.14.7.4 Load Combination 7d. Fatigue Limit State II. Finite Life. Fatigue in anchorage components.

1.0 D + 1.0 G + 1.0 Q

E.3. Spillway Tainter Gate.

E.3.1. Introduction. This example is a spillway with three Tainter gates. The spillway shown in Figure E.5 is on a dam in a northern state.

Figure E.5. Spillway Gate Example

E.3.2. Geometry. The gates are 35 ft wide. The sill elevation is elevation 1,251 ft msl, the top of the gate in the closed position is elevation 1,271 ft msl. The gate radius is 20 ft to the inside face of the skin plate. The trunnion is at elevation 1,262.5 ft msl. The gate is shown in Figure E.6.

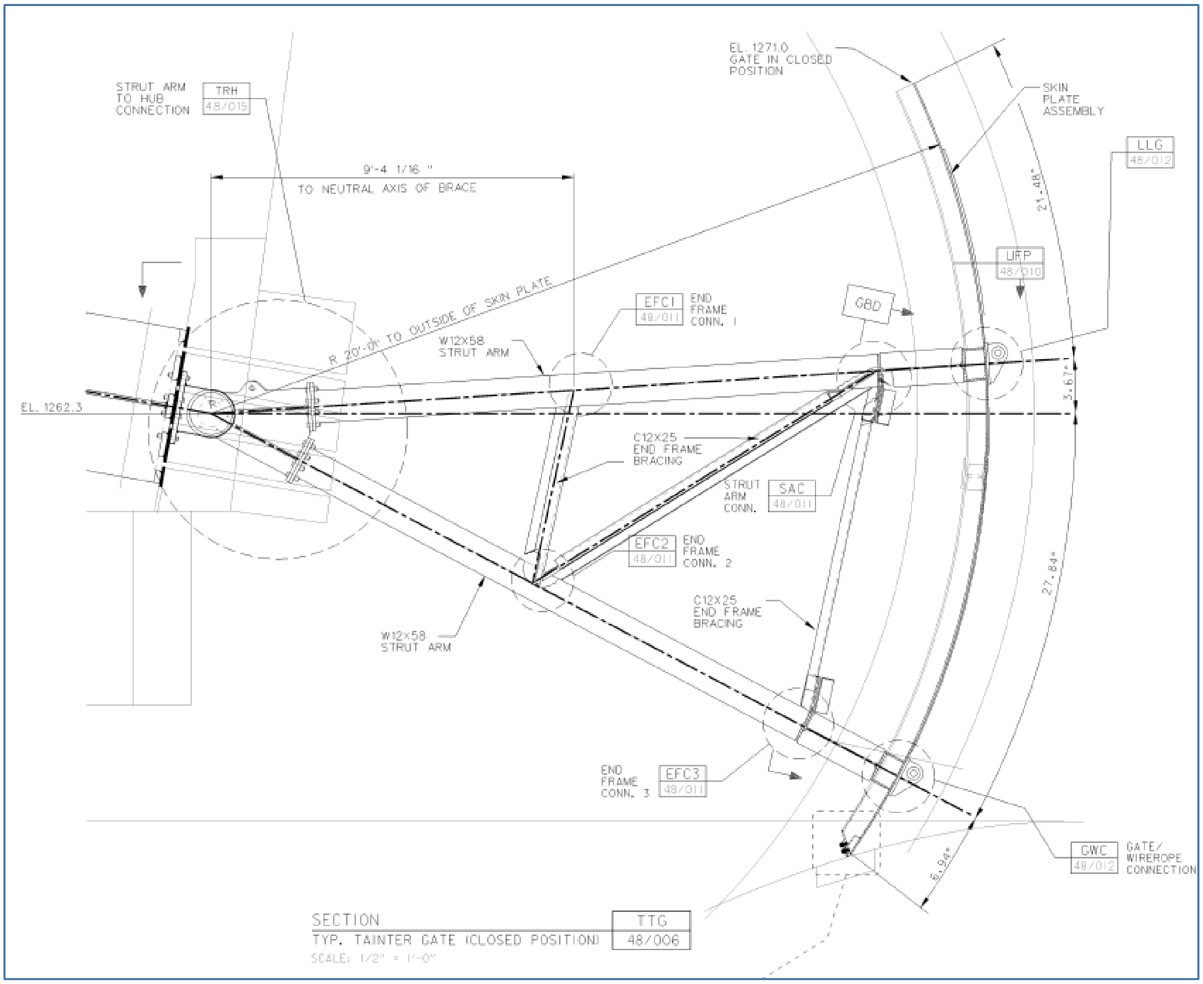


Figure E.6. Tainter Gate Geometry

E.3.3. Structure Classification. There is a town with a population of 6,500 about 12 miles downstream of the dam. A significant portion of the town is in the flood plain. The loss of a single gate will pass flows that exceed the channel capacity through town. There is potential loss of life in this event. Therefore, the gates must be classified as critical according to paragraph 3.2.3.

E.3.4. Dead Load (D). The dead load of the gate is calculated from member sizes including a factor for connections. The estimated gate weight is 46 kips. The dead load is a permanent load with a load factor (γ_p) of 1.2 or 0.9, each to be applied for maximum effect according to paragraph 4.3.3.

E.3.5. Gravity Load (G). This is a spillway crest gate and the back side is above water at all times. There will be no silt load, but an ice load will be applied. Ice load will usually not exist but may be present during spring time gate operation. Gravity load is estimated from ice from spray or leakage that may accumulate on the gate. The gravity load is a permanent load with a load factor (γ_p) of 1.6 or 0, each to be applied for maximum effect according to paragraph 4.3.3.

E.3.6. Hydrostatic Loadings (Hs).

E.3.6.1 The stage verses frequency table for the dam was provide by hydraulic engineers. It is provided in the Table E.1 as a function of average annual return period. The pool level at the top of the gates in the close position (elevation 1,271 ft) has a return period of about 10,000 years. If the pool did reach that stage, all gates would be open to pass flow.

Table E.1.
Return Period Versus Pool Elevation

Return Period (Years)	Pool Elevation (ft msl)
2	1,265.4
5	1,266
10	1,266.1
20	1,266.2
50	1,266.8
100	1,267.8
200	1,268.8
500	1,269.6
1,000	1,270.0
10,000	1,271.0

E.3.6.2 A gate could be closed through misoperation, and this can be set as the maximum hydrostatic load on the gate. Pool levels high enough to overtop the gate are possible. Since this is a 10,000-year event, requires gate misoperation to occur, and the overflow spillway is at this elevation, design to the top of the gate is adequate in this case. If the top of the gate has been at pools with lower return periods, design with overtopping should be considered (with misoperation).

E.3.6.2.1 Since the water elevation is limited by the top of the gates, the load factor is selected from Condition 2 from paragraph 4.3.4.2. The pool at the top of the gate has a return period of about 10,000 years and full load on the gate is only realized through misoperation, making the likelihood of loading even lower. Therefore, the principal load factor (γ_{pr}) of 1.2 from principal load condition 1 can be used, according to paragraph 4.3.4.1.

E.3.6.2.2 When the water is a companion load (Hs_c) according to paragraph 3.3.3.3 the level with a 10-year return period should be used. That is elevation 1,266.1 ft msl. According to paragraph 4.3.5 a companion load factor (γ_c) of 1.0 is used. This elevation could be lowered to investigate effects from ice expansion or impact at lower elevations if they are critical to certain parts of the gate.

E.3.6.2.3 An unusual hydrostatic load is used in load combination 4. Since this gate is critical, a pool level at the top of the unusual load category from Figure 3.1 is selected. This is a 750-year return period and the pool level is 1,270 ft msl.

E.3.7. Thermal Ice Loading (IX).

E.3.7.1 Being in a northern climate, sheet ice may form on the gates and develop forces from thermal expansion of the ice. From paragraph 10.2.8, a 5 kips per foot (k/ft) ice load will be applied where the ice load is possible. This will be a principal load (IX_{pr}) that is in the extreme category (IX_X). Since the return period of the load is not known, a principal load condition 3 (Extreme) load factor applies according to paragraph 4.3.4.3. Therefore, a principal load factor (γ_{pr}) of 1.3 is used.

E.3.7.2 This site is operated with a conservation pool no higher than elevation 1,266 ft msl in the winter. The pool is operated as low as elevation 1,259.5 ft msl in the winter. The gates are closed during the winter with flow through the dam provided by low flow conduits. This load will only be applied to gates in the closed position resting on the sill.

E.3.8. Friction (F). Values of the coefficient of friction are 0.5 for side seals (F_s) and 0.3 for trunnion friction (F_t) according to paragraph 10.2.11. A load factor of 1.4 is applied to the friction forces according to paragraph 4.3.5.

E.3.9. Operating Load (Q). The capacity of the lifting machinery is estimated by mechanical engineers to be 115 kips for this gate. This load is limited and is therefore a principal load condition 2 load according to paragraph 4.3.4.2. The return period of the load is not known, but operating to the ultimate capacity of the machinery is not expected and is a hard upper bound. Therefore, a principal load condition 2 (extreme) load factor applies according to paragraph 4.3.4.2.1. A principal load factor (γ_{pr}) of 1.3 is used.

E.3.10. Wave Loadings (Hw).

E.3.10.1 The dam is aligned in northwest to southeast axis as shown in Figure E.5. The reservoir extends upstream for about 5 miles in a northeast direction as shown in Figure E.7. The statistics for the waves are determined by wind data from a local wind station using winds in a general southerly and easterly direction. The wave heights and pressures are calculated from the wind data by a hydraulic engineer using EM 1110-2-2100. This example will not describe how the wave heights and loads are calculated.

E.3.10.2 For waves as a principal load (Hw_{pr}), the return period can be extrapolated from wind data and this would be a load meeting Condition 1 in paragraph 4.3.4.1. The wave loads would be computed using wind speeds with a 10,000-year return period. The principal load factor (γ_{pr}) will be 1.2. This is an extreme load, Hw_X. Given the narrow range of winds that can create waves from a direction that is not common for high winds, the load combinations are developed assuming that ice loading is likely higher than wave loads that would be computed for this site.

E.3.10.3 For waves applied as a companion load (Hw_c), a wave using wind speeds from a 10-year return period are used according to paragraph 3.3.3.3. According to paragraph 4.3.5, a companion load factor (γ_c) of 1.0 is used.

Figure E.7. Reservoir Upstream of Dam

E.3.11. Impact (IM). According to paragraph 10.2.4.2, for sites where floating ice is present, IM is specified as a uniform distributed load of 5,000 lbs/ft (5 k/ft) applied along the width of the gate at the upper pool elevation. This is considered an extreme load and the probability of loading is unknown. Therefore, the Principal Load Condition 3 load factor (γ_{pr}) of 1.3 is used according to paragraph 4.3.4.3. The maximum load is combined with the companion hydrostatic load. The magnitude of this load coincides with the thermal expansion ice force that is applied to a gate resting on the sill. Therefore, for this case, IM only needs to be applied to the case with the gates hanging by the wire ropes.

E.3.12. Wind (W). Wind loads are small compared to the hydrostatic loads and will be neglected.

E.3.13. Barge Impact (BI). The site is not a navigable waterway and no additional impact loads from barges need to be applied.

E.3.14. Earthquake (EQ). The site is in a low seismicity region. As a critical structure, the MDE earthquake is the maximum considered event (MCE). The standard earthquake motions from USGS can be used to determine earthquake controls for this site. The earthquake would be applied to pool level with a 10-year return period as a companion load. As stated under Hs, that would be a pool elevation of 1,266.1 ft msl.

E.3.15. Load Combinations.

E.3.15.1 Gate Closed and on Sill.

E.3.15.1.1 Load Combination 1a. Maximum Hydrostatic. Loads consist of maximum hydrostatic loading (Hs_{pr}) at a pool of 1,271 ft msl ($Hs_{(1271)}$) with gate subjected to dead load, gravity loads, and wave as a companion load.

$1.2\ D + 1.6\ G + 1.2\ Hs_{(1271)} + 1.0\ Hw_{(10\ yr)}$

$0.9\ D + 0\ G + 1.2\ Hs_{(1271)} + 1.0\ Hw_{10\ yr}$

E.3.15.1.2 Load Combination 1b. Maximum Ice (or impact). Loads consist of extreme thermally expanding ice load of 5 k/ft ($IX_{(5\ k/ft)}$), plus companion hydrostatic loading at 1,266 ft msl ($Hs_{(1266)}$) and 1,259.5 ft msl ($Hs_{(1259.5)}$), with gate subjected to dead load and gravity loads.

$1.2\ D + 1.6\ G + 1.3\ IX_{(5\ k/ft)} + 1.0\ Hs_{(1266)}$

$0.9\ D + 0\ G + 1.3\ IX_{(5\ k/ft)} + 1.0\ Hs_{(1266)}$

$1.2\ D + 1.6\ G + 1.3\ IX_{(5\ k/ft)} + 1.0\ Hs_{(1259.5)}$

$0.9\ D + 0\ G + 1.3\ IX_{(5\ k/ft)} + 1.0\ Hs_{(1259.5)}$

E.3.15.1.3 Load Combination 1c. Maximum Wave. Loads consist of wave load from a 10,000-year wind event, plus companion hydrostatic loading at 1,266 ft msl ($Hs_{(1266)}$), with gate subjected to dead load and gravity loads.

$1.2\ D + 1.6\ G + 1.3\ Hw_{(10{,}000\ yr)} + 1.0\ Hs_{(1266)}$

$0.9\ D + 0\ G + 1.3\ Hw_{(10{,}000\ yr)} + 1.0\ Hs_{(1266)}$

E.3.15.2 Gate Supported by Two Hoists.

E.3.15.2.1 Load Combination 2a. Gate Supported by Two Hoists. Maximum Hydrostatic. Loads consist of maximum hydrostatic loading (Hs_{pr}) at a pool of 1,271 ft msl ($Hs_{(1271)}$) with gate subjected to dead load, gravity loads, and companion wave or impact. Operating machinery forces are a reaction:

$1.2\ D + 1.6\ G + 1.2\ Hs_{(1271)} + 1.0\ Hw_{(10\ yr)}$

$0.9\ D + 0\ G + 1.2\ Hs_{(1271)} + 1.0\ Hw_{10\ yr}$

E.3.15.2.2 Load Combination 2b. Gate Supported by Two Hoists. Maximum Impact. Loads consist of extreme impact of 5 k/ft, plus companion hydrostatic loading at a pool of 1,266 ft msl ($Hs_{(1266)}$), with gate subjected to dead load and gravity loads. Operating machinery forces are a reaction:

$1.2\ D + 1.6\ G + 1.3\ IM_{(5\ k/ft)} + 1.0\ Hs_{(1266)}$

$0.9\ D + 0\ G + 1.3\ IM_{(5\ k/ft)} + 1.0\ Hs_{(1266)}$

E.3.15.3 Load Combination 3: Gate Operated by Two Hoists. Loads consist of maximum hydrostatic loading (Hs_{pr}) at a pool of 1,271 ft msl ($Hs_{(1271)}$) with gate subjected to dead load, gravity loads, and side seal and trunnion friction. Operating machinery forces are a reaction.

$1.2\ D + 1.6\ G + 1.2\ Hs_{(1271)} + 1.4\ Fs + 1.4\ Ft$

$0.9\ D + 0\ G + 1.2\ Hs_{(1271)} + 1.4\ Fs + 1.4\ Ft$

E.3.15.4 Load Combination 4: Gate Operating on One Hoist. Loads consist of unusual hydrostatic loading (Hs_N)—the principal load—at a pool of 1,270 ft msl ($Hs_{(1270)}$) with gate subjected to dead load, gravity loads, side seal and trunnion friction, and side sway friction load (if present).

$1.2\ D + 1.6\ G +1.4\ Hs_{(1270)} + 1.4\ Fs + 1.4\ Fb + 1.4\ Ft$

$0.9\ D + 0\ G + 1.4\ Hs_{(1270)} + 1.4\ Fs + 1.4\ Fb + 1.4\ Ft$

E.3.15.5 Load Combination 5: Gate Jammed. Loads consist of maximum operating equipment forces ($Q_{(115\ kip)}$) plus companion hydrostatic loading at a pool of 1,266 ft msl ($Hs_{(1266)}$), dead load, and gravity loads.

$1.2\ D + 1.6\ G + 1.2\ Q_{(115\ kip)} + 1.0\ Hs_{(1266)}$

$0.9\ D + 0\ G + 1.2\ Q_{(115\ kip)} + 1.0\ Hs_{(1266)}$

E.3.15.6 Load Combination 6: Gate Fully Opened. Supported or Operating on Two Hoists. This case is not considered to control for this gate.

E.3.15.7 Load Combination 7: Earthquake. Loads consist of earthquake, EQ, plus companion hydrostatic loading (Hs_c) at a pool of 1,266 ft msl ($Hs_{(1266)}$), dead load, and gravity loads. The gate may be closed or open.

E.3.15.7.1 For standard and site-specific OBE ground motion analysis:

$1.2\ D + 1.6\ G + 1.5\ EQ + 1.0\ Hs_{(1266)}$

$0.9\ D + 0\ G + 1.5\ EQ + 1.0\ Hs_{(1266)}$

E.3.15.7.2 For standard MDE ground motion analysis (assuming earthquake does not control for the site and a site-specific analysis is not performed):

$1.2\ D + 1.6\ G + 1.25\ EQ + 1.0\ Hs_{(1266)}$

$0.9\ D + 0\ G + 1.25\ EQ + 1.0\ Hs_{(1266)}$

Appendix F
Tainter Gate Load Determination

F.1. Side Seal Friction Load Derivation.

F.1.1. Side Seal Friction Loads. The derivation of the side seal friction force, Equation 10.1, is shown in the following paragraphs. The total friction force is the sum of the force created from the seal preset (first term of the equation) and the force created by the hydrostatic pressure exerted on the seal.

$$F_s = \mu_s S l + \mu_s \gamma_w {}^{d}/_{2} (l_1 {}^{h}/_{2} + h l_2) \qquad \text{(Equation 10.1)}$$

$$S = 3\delta EI/d^3 \qquad \text{(Equation F.1)}$$

Where:

μ_s = Coefficient of side-seal friction

l = Total length of side seal

l_1 = Length of the side seal from the headwater to the tail water elevations or bottom of the seal if there is no tail water

l_2 = Length of the side seal from the tail water elevation to the bottom of the seal (equals zero if there is no tail water on the gate)

S = Preset force, force per unit length induced by presetting the seal

δ = Preset distance

γ_w = Unit weight of water

d = Width of the J-seal exposed to upper pool hydrostatic pressure

h = Vertical distance taken from the headwater surface to the tail water surface or the bottom of the seal if there is no tail water on the gate

F.1.2. Seal Free Body Diagram. The forces acting on the seal are shown in Figure F.1. The resulting free body diagram (FBD) is shown in Figure F.2. The length of seal exposed to hydrostatic pressure, d, will be slightly longer than depicted in Figure F.1 due to compression of the seal. However, the increased length is small for stiff seals and the effects can be neglected.

F.1.3. Preset Seal Force Derivation. The preset force, S, is that required to cause a deflection equal to the preset distance, δ, assuming the seal acts as shown in Figure F.2. This force can be determined using standard beam tables (e.g., AISC Steel Construction Manual) with the result as shown in Equation F.1. The moment of inertia used to calculate the preset force is taken over one foot (12 inches) of length to be compatible with length of seal l. The corresponding force is per unit length (foot) of seal.

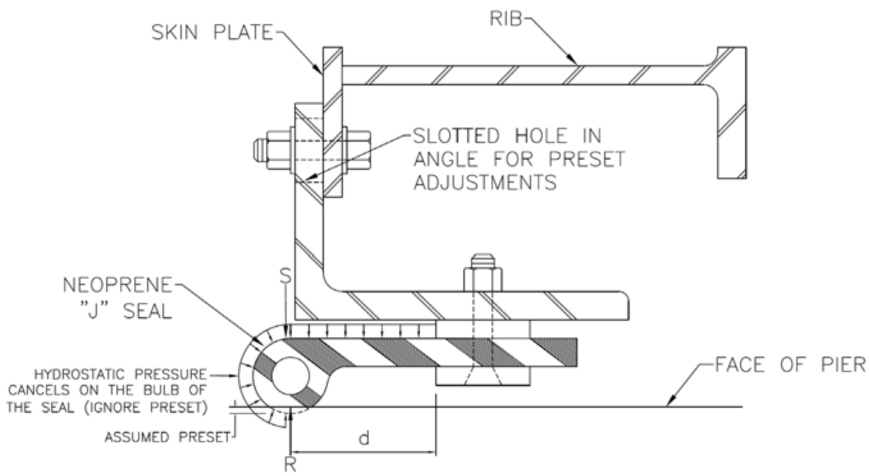


Figure F.1. Side Seal FBD

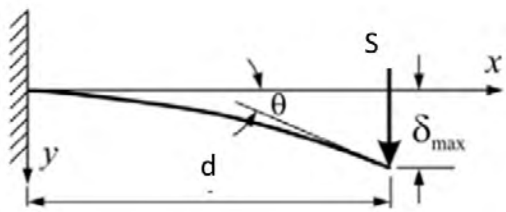


Figure F.2. Side Seal FBD Due to Unit Displacement

F.1.4. Hydrostatic Loading Seal Force. The hydrostatic load exerted on the seal is the pressure due to a projection of a column of water exerted on the gate. The pressure varies linearly from zero at the water surface to a maximum at the bottom of the gate as shown in Figure F.3. The tailwater cancels out any head water below the tailwater elevation with the resulting pressure diagram as shown in Figure F.4. The force due to the headwater on an incremental length of seal is depicted in Figure F.5. The length of seal loaded is slightly longer than the height of water column due to the arc length of the seal. The pressure at an increment of seal length can be determined as follows.

Pressure at a point is: $p_i = \gamma_w h_i$

Thus, the force on the incremental length of seal is: $f_i = \frac{\gamma_w}{2}(\Delta h)\Delta l d$

The total force from headwater to tailwater is: $F_1 = \sum f_i = \frac{\gamma_w}{2} h l_1 d$ (Equation F.2)

The pressure on the side seals below the tailwater is uniform due to the counteracting effects of tailwater on headwater. The resulting force on the side seal in this area is found in a similar manner.

$F_2 = \gamma_w h l_2 d$ (Equation F.3)

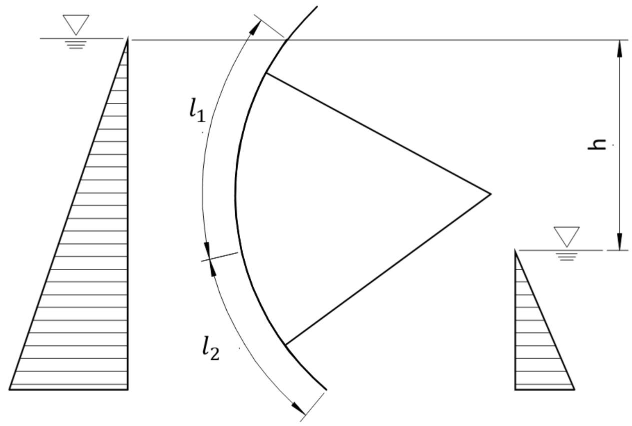


Figure F.3. Hydrostatic Pressure on Gate

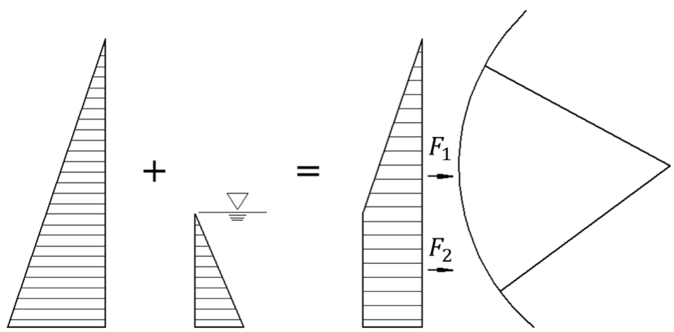


Figure F.4. Hydrostatic Forces on Side Seals

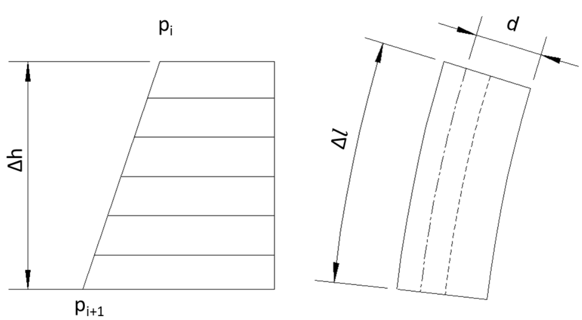


Figure F.5. Incremental Load on Side Seal

The total force on the side seal is the sum of the individual forces above and below the tailwater. The force acting on the pier is the reaction on the pier at the center of the seal contact length. This can be determined assuming the seal acts as uniformly loaded, simply supported beam of length d. The reaction is thus equally divided between the point of seal contact and seal attachment.

$$F = (F_1 + F_2)/2 = \frac{\gamma_w}{4} h l_1 d + \frac{\gamma_w}{2} h l_2 d = \frac{\gamma_w}{2} hd(\frac{l_1}{2} + \ l_2)$$

F.1.5. Side Seal Friction Force. The total force acting on the pier is the sum of Equations F.1 and F.2. During gate movement, the friction between the pier and seal creates a force opposing the direction of gate motion and is determined by applying the appropriate coefficient of friction, μ, between the seal material and pier material. The total side seal force is then:

$$\mu_s\left(\frac{3\delta EI}{d^3}\right)(l_1 + l_2) + \mu_s\left(\frac{\gamma_w}{2}hd(\frac{l_1}{2} + l_2)\right)$$

Rearranging and substituting gives:

$$\mu_s Sl + \mu_s \gamma_w h\frac{d}{2}(\frac{l_1}{2} + l_2)$$

This is equivalent to Equation 10.1.

F.2. Wire Rope Load Derivation.

F.2.1. Load Cases. The three wire rope load cases on a Tainter gate are:

a. Wire rope not tangent to the skin plate (wire rope is pulling directly on the lifting attachment bracket).
b. Wire rope is tangent to the skin plate.
c. Wire rope is more than tangent to the skin plate (wire rope deviates from the arc and is wrapped around the top edge of the skin plate).

F.2.2. Case a. This case is the result of the gate being fully or nearly fully open. The tension, T, is either a reaction due to other loads (dead, gravity, friction forces) or is the pull of the machinery on the gate due to a stuck gate or engaged gate stops. In the former case, force is determined using simple statics. In the latter case, the force is known and may be resolved into horizontal and vertical components as shown in Figure F.6.

F.2.3. Case b. Case b is shown in Figure F.7. The wire rope is tangent along the arc in contact with the Tainter gate skin plate at two locations. The first or lower point is where the wire rope first contacts the skin plate and is a function of the geometry of the lifting attachment bracket. The second point will vary depending on the location of the gate relative to the lifting mechanism and the opening height of the gate. A random location is shown.

F.2.3.1 In deriving the wire rope load for this case, the wire rope to skin plate contact is assumed frictionless and the load is assumed to be uniform. The wire rope load can be determined examining the wire rope tension, T, and the reaction of this tension created on the wire rope by the curvature of the gate.

F.2.3.2 The length of the wire rope contact forms the central angle, θ. The wire rope force can be resolved along or parallel to the bisector of the central angle and summing forces in this direction. Components of the wire rope tension and its reaction at the lifting attachment bracket are T sin (θ/2).

The wire rope load can be broken up into uniform increments, w, along the central angle bisector (see Figure F.8):

$w = W\Delta s = WR\Delta\theta$ where $\Delta s = R\Delta\theta$

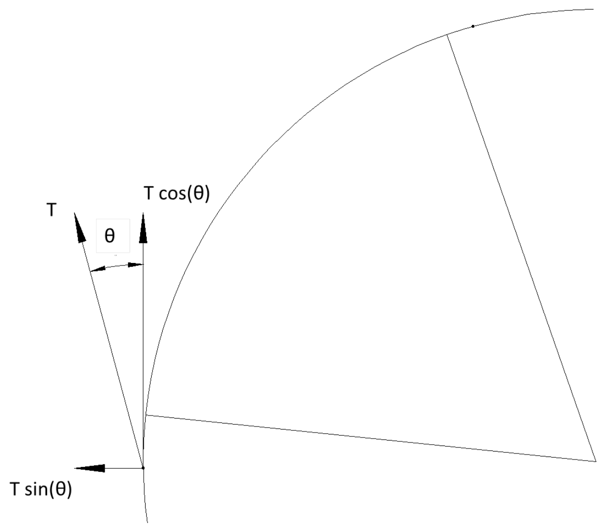


Figure F.6. Wire Rope Case a, Wire Rope Not Tangent to the Skin Plate

The component parallel to the central angle bisector is: $WR\Delta\theta \cos\theta_i$

Summing increments gives: $\sum W \cos\theta_i \, R\Delta\theta$

At the limit, Δθ = dθ and summing forces along the bisector,

$2RW \int_0^{\theta_2} \cos\theta \, d\theta = 2RW \sin\frac{\theta}{2}$

The sum of the forces along the bisector = 0: $2RW \sin\frac{\theta}{2} = 2T \sin\frac{\theta}{2}$

Solving for W:

$W = T/R$ (Equation F.4)

The Total force is:

$Q_T = R \; x \; \theta_W \; x \; W$ (Equation F.5a)

Substituting W with Equation F.4:

$Q_T = \theta_W \; x \; T$ (Equation F.5b)

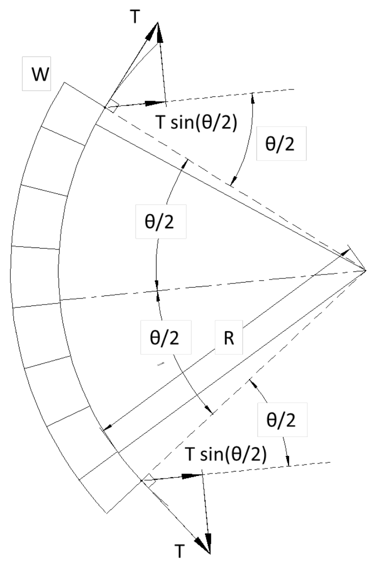


Figure F.7. Wire Rope Case b, Wire Rope Tangent to the Skin Plate

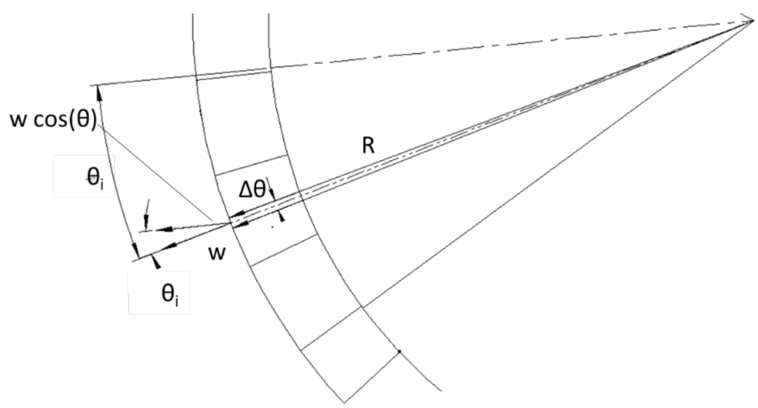


Figure F.8. Incremental Load on the Skin Plate

F.2.4. Case c. The case where the wire rope wraps over the edge of the gate creates a reaction at the edge, shown as E in Figure F.9. The reaction is due to the components of the wire rope tension force. One component lies on a line tangent to the skin plate arc at the top of the arc and is perpendicular to a line, r, formed from the centerline of the trunnion pin to the top of the arc. The other component lies on the line at the angle of the bend, shown as B in Figure F.9 and measured from the tangent line extended. The reaction, E, is oriented along a line that bisects the angle formed by the two vectors, T. The angle between the two vectors is:

$(180°\text{-B})/2 = 90° - \frac{B}{2}$

The angle of the reaction, E, measured from r is: $90° - \left(90° - \frac{B}{2}\right) = \frac{B}{2}$

The two vectors, T, can be resolved along the line of the reaction as (Figure F.10): $T \sin\frac{B}{2}$

The reaction, E, is computed by summing forces along the reaction line:

$$E - 2T\left(\sin\frac{B}{2}\right) = 0$$

$$E = 2T\left(\sin\frac{B}{2}\right) \quad \text{(Equation F.6)}$$

The wire rope load, W, is computed as shown for Case b.

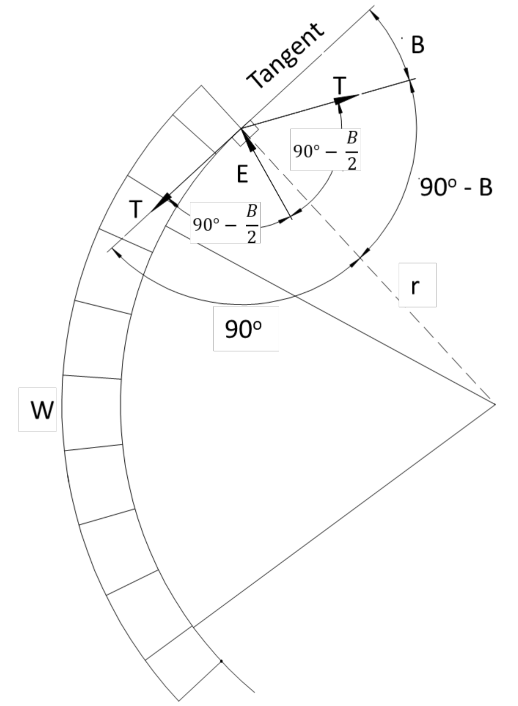


Figure F.9. Wire Rope Case c. Wire Rope More Than Tangent to the Skin Plate

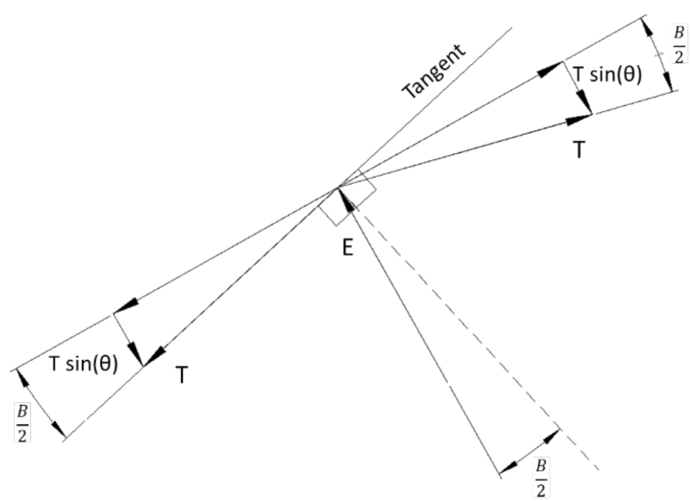


Figure F.10. Wire Rope Case c. Resolved Loads

F.3. Hydrostatic Load Derivation.

F.3.1. Background. Water has the properties of a fluid. An ideal fluid has no shear modulus and thus deforms continuously under an applied shear stress. Because of this, water exerts a pressure perpendicularly to the surface it is applied. When applied to the circular surface of a Tainter gate, the resultant at a point continually changes along the skinplate and radiating to the center of the circle (center of the trunnion pin).

F.3.2. Methods. Three solutions for calculating the magnitude and direction of the force for the hydrostatic load are provided and consist of:

F.3.2.1 Integration of the hydrostatic loads along the surface of the skin plate.

F.3.2.2 Calculation of the hydrostatic loads applied to the vertical and horizontal projections of the skin plate using fluid mechanics.

F.3.2.3 Step-wise calculation of hydrostatic loads acting on segments of the skin plate.

In all cases, it is assumed the water surface is at or below the top of the skin plate assembly (i.e., no overtopping or no submergence).

F.3.3. Integration. The length of skinplate subjected to hydrostatic force is measured as the arc length from the sill elevation to the top of water surface. Angles are referenced from a horizontal line taken at the elevation of the trunnion centerline. Angles are positive below horizontal and negative above horizontal. The variable associated with the hydrostatic forces are show in Figure F.11 and described as follows.

The radial force on the skin plate at each increment, i, is:

$P_i = p_i \times \Delta s$ (Equation F.7)

p_i is the water pressure at each increment

Δs is the length of incremental skin plate

$p_i = \gamma_w(Y + y_i)$

$\Delta s = R \times \Delta\theta$

Y is the depth from the water surface to the centerline of trunnion pin

y_i is the incremental depth (positive below the trunnion centerline and negative above),

$y_i = R \sin\theta_i$

$\Delta\theta$ is the incremental angle

Substituting variables into (F.6):

$P_i = \gamma_w(Y + R\sin\theta_i) \times R \times \Delta\theta$ (Equation F.8)

The radial force is calculated by summing the incremental values:

$P = \gamma_w R \int_{\theta_1}^{\theta_2} Y + R \sin\theta d\theta$

And integrating:

$P = R\gamma_w[R(\cos\theta_1 - \cos\theta_2) - Y\theta_1 + Y\theta_2]$ (Equation F.9)

The horizontal and vertical components of the incremental force are obtained by multiplying the cosine and sine of the incremental angle respectively. See Figure F.12.

Horizontal Component: $P_h = R\gamma_w \int_{\theta_1}^{\theta_2}(Y + R\sin\theta)\cos\theta d\theta$

$P_h = R\gamma_w\left[Y(\sin\theta_1 - \sin\theta_2) - \frac{1}{2}R(\sin{\theta_2}^2 - \sin{\theta_1}^2)\right]$ (Equation F.10)

Vertical Component: $P_v = R\gamma_w \int_{\theta_1}^{\theta_2}(Y + R\sin\theta)\sin\theta\, d\theta$

$P_v = R\gamma_w\left[Y(\cos\theta_1 - \cos\theta_2) + \frac{1}{4}R(2\theta_2 - 2\theta_1 + sin2\theta_1 - sin2\theta_2)\right]$ (Equation F.11)

The angle of the radial force from the horizontal is computed by computing the arctangent of the horizontal component over the vertical component.

$$\theta_p = \tan^{-1}\left({P_h}/{P_v}\right)$$

The location of application of the radial load is determined by summing incremental horizontal or vertical forces (moments) about the centerline of the trunnion pin and dividing by the corresponding component force.

Sum horizontal components: $M_h = R^2\gamma_w \int_{\theta_1}^{\theta_2}(Y + R\sin\theta)\cos\theta\sin\theta\, d\theta$

$M_h = R^2\gamma_w\left[\frac{Y}{2}\left(\sin{\theta_2}^2 - \sin{\theta_1}^2\right) + \frac{1}{3}R(\sin{\theta_2}^3 - \sin{\theta_1}^3)\right]$ (Equation F.12)

Sum vertical components: $M_v = R^2\gamma_w \int_{\theta_1}^{\theta_2}(Y + R\sin\theta)\sin\theta\cos\theta\, d\theta$

$M_v = -R^2\gamma_w\left[\frac{Y}{2}(\sin{\theta_1}^2 - \sin{\theta_2}^2) + \frac{R}{3}(\sin{\theta_1}^3 - \sin{\theta_2}^3)\right]$ (Equation F.13)

Resultant location, horizontal component: $Y_p = \frac{M_h}{P_h}$ (Equation F.14)

Resultant location, vertical component: $X_p = \frac{M_v}{P_v}$ (Equation F.15)

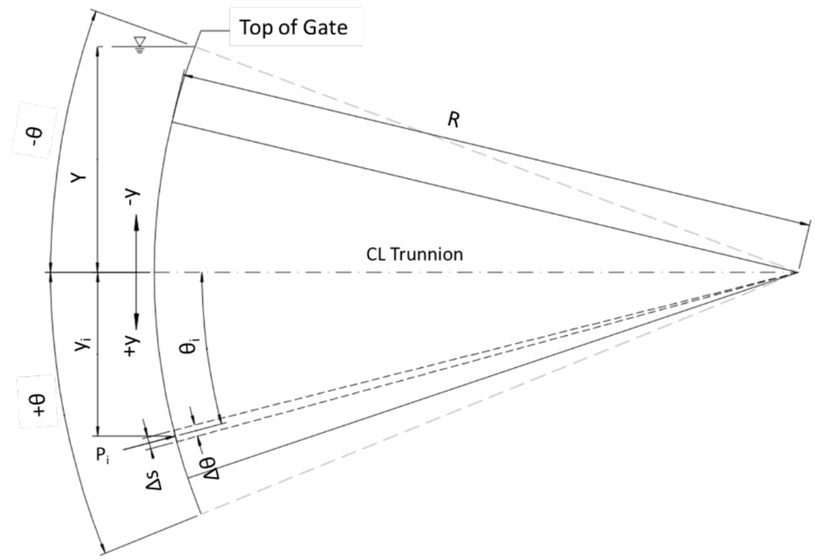


Figure F.11. Hydrostatic Load Determination by Integration

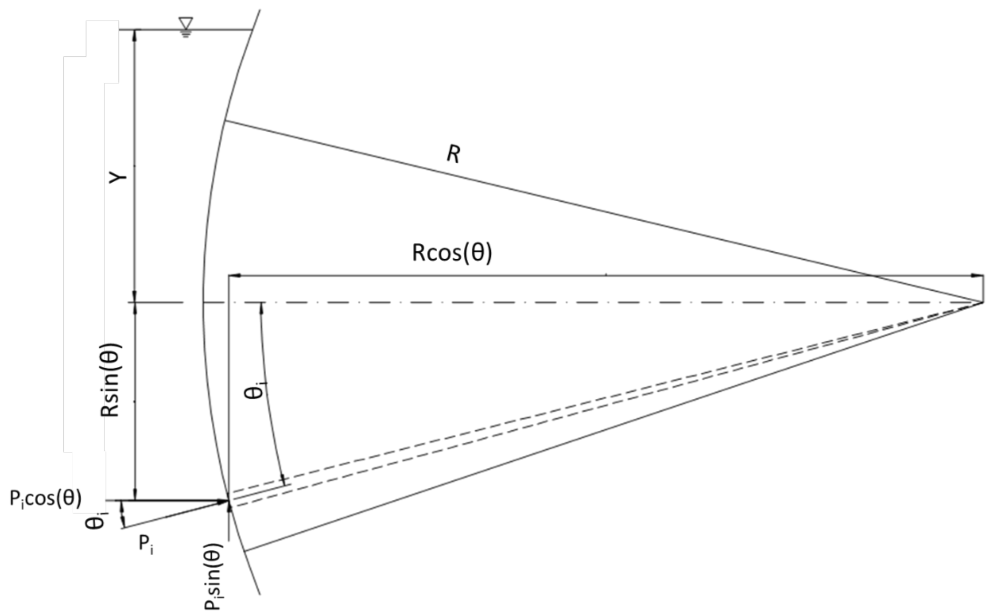


Figure F.12. Horizontal and Vertical Components

F.3.4. Horizontal and Vertical Projections.

F.3.4.1 Horizontal and vertical components of the radial water force can be calculated by projecting the loads in vertical and horizontal directions, computing the areas of load geometries created, and multiplying by the weight of water. The moments are calculated by multiplying the areas by the respective moment arms from the centerline of trunnion pin to the centroids of the geometrical areas.

F.3.4.2 The horizontal component is calculated by projecting the skin plate onto a vertical plane. The force is calculated by multiplying the area of the load distribution by the weight of water. See Figure F.13.

$P_h = \frac{1}{2}H^2\gamma_w$ (Equation F.16)

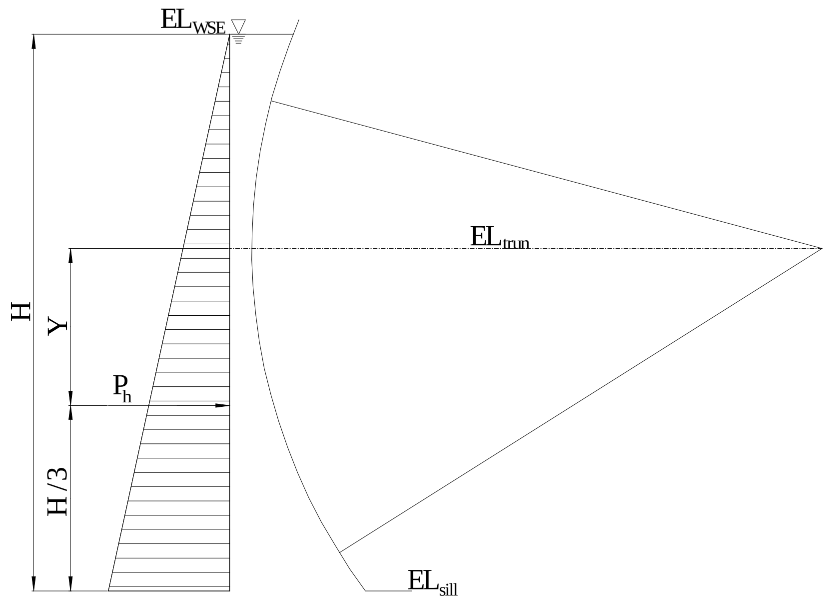


Figure F.13. Hydrostatic Load Projected Horizontally

The centroid of the force is located one-third the height, H, above the sill and the moment arm is the elevation of the trunnion pin less the elevation of the centroid.

$$Y = (EL_{trun} - EL_{sill}) - \frac{H}{3}$$

F.3.4.3 The vertical component is divided into two parts: water above the trunnion pin and water below the trunnion pin.

F.3.4.3.1 The vertical water above the trunnion pin acts downward with a magnitude of the body of water directly above the skin plate. The body of water is a 3-sided area comprised of two straight lines and an arc. The area of the body can be determined by combining positive and negative geometries as shown in Figures F.14 and F.15. The two geometric bodies are a triangle and segment of a circle. The area of the water load is the remaining area of the triangle with the segment removed. In all calculations, θ is the angle from the horizontal to the top of water elevation at the skin plate.

The area of a segment is ${R^2}/{2}\,(\theta_1 - sin\ \theta_1)$

The area of water on the gate above the trunnion elevation is:

$A = a_1 - a_2 = \frac{1}{2}ab - \frac{R^2}{2}(\theta_1 - \sin\theta_1)$

Where, $a = R(1 - \cos\theta_1)$ $b = (EL_{WSE} - EL_{trun})$

The moment arm of body of water is determined in a similar manner with all arms originating from the centerline of the trunnion pin. The resultant moment for the vertical water areas above horizontal is the sum of the individual moments. See Figure F.16.

Centroid of the triangle: $A_1 = R - \frac{a}{3}$

Centroid of a segment, general: $C_s = \frac{4R\sin\left(\frac{\theta_1}{2}\right)^3}{3(\theta_1 - \sin\theta_1)}$

Centroid of the segment, vertical component: $A_2 = C_s \cos\frac{\theta_1}{2}$

Sum of the moment areas: $M_a = a_1 A_1 - a_2 A_2$

The force due to vertical force above horizontal is: $P_{v_1} = A\gamma_w$

The moment due to vertical force above horizontal is: $M_{v_1} = M_a \gamma_w$

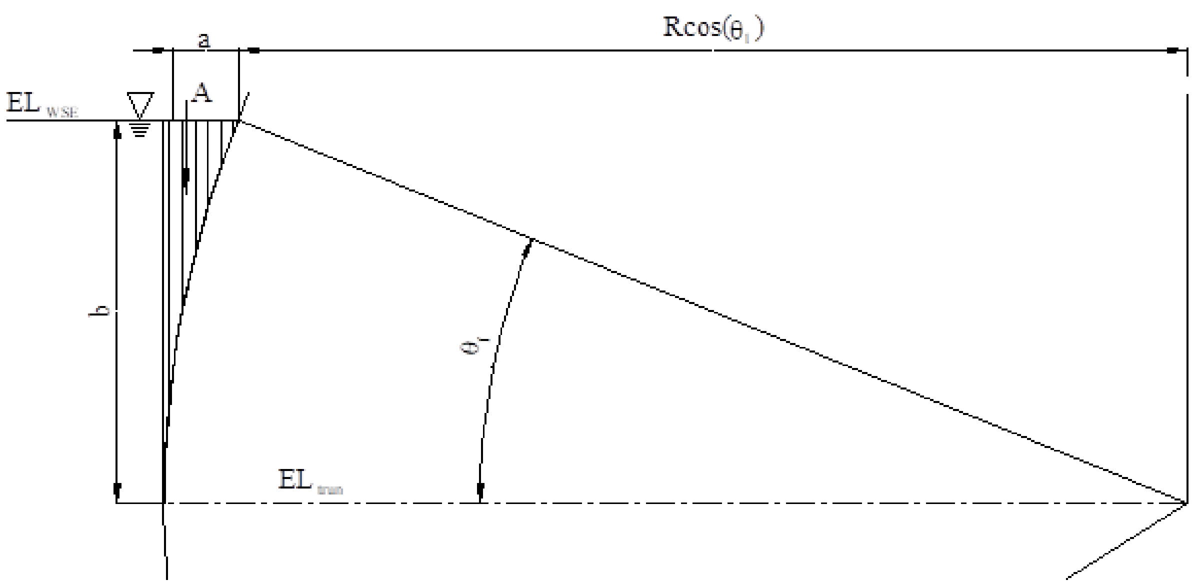


Figure F.14. Column of Water Acting Down on Tainter Gate

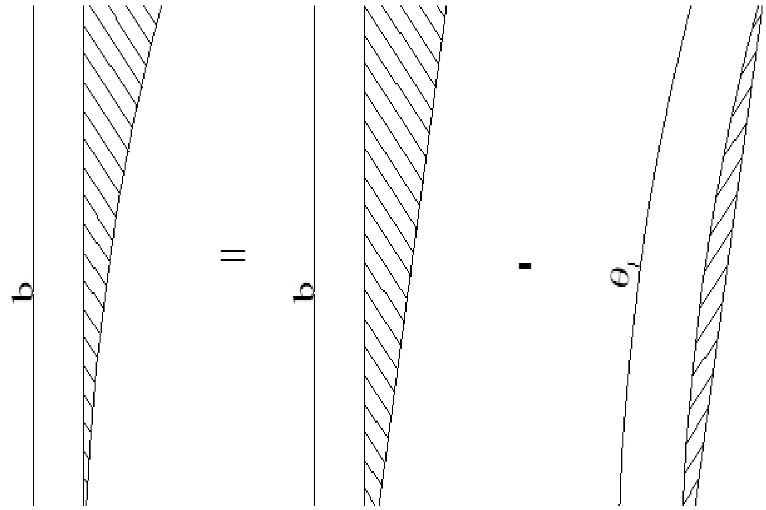


Figure F.15. Geometry Used to Compute Water Acting Down on Tainter Gate

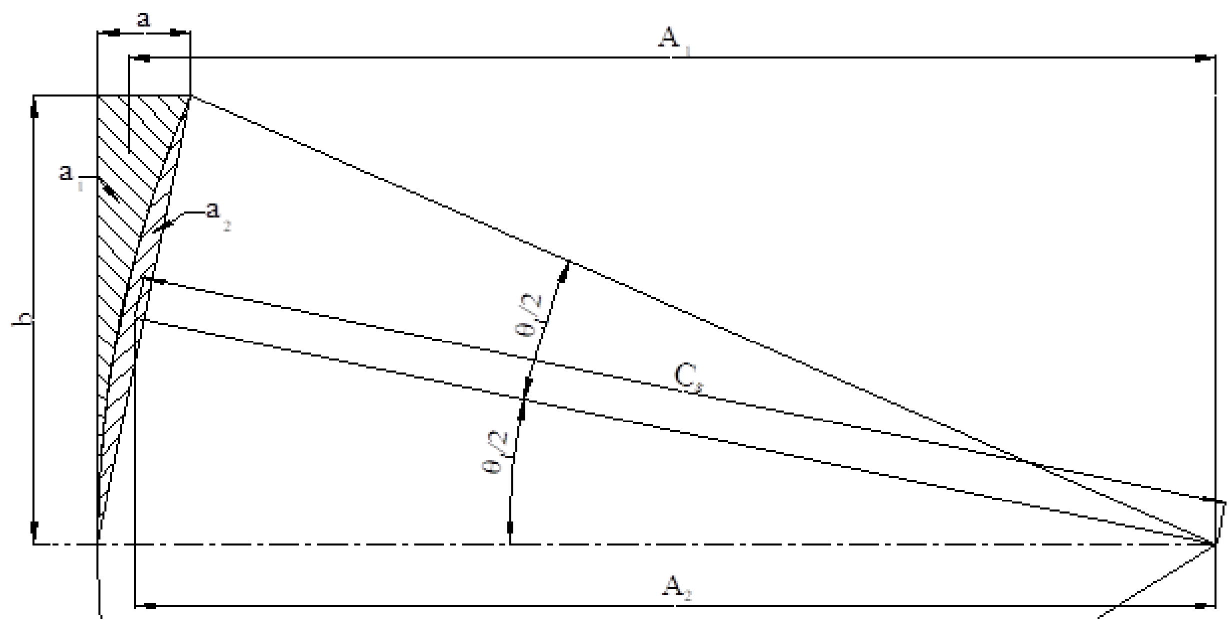


Figure F.16. Moment Arms of Water Acting Down on Tainter Gate

F.3.4.3.2 The vertical water below the trunnion pin is a buoyant force (acting upward) with a magnitude equal to the column of water projecting above the surface of skin plate subjected to water. See Figures F.17, F.18 and F.19. The area of the body can be divided into three separate geometries to simplify area calculations. The three geometric bodies are a rectangle, triangle, and segment of a circle. The area of the water load is the sum of the geometries. In all calculations, θ is the angle from the horizontal to the sill elevation.

$$A = a_1 + a_2 + a_3 = ab + \frac{1}{2}ac + \frac{R^2}{2}(\theta_2 - \sin\theta_2)$$

Where a and b are as defined previously and: $c = (EL_{trun} - EL_{sill})$

The moment arm of body of water is determined in a similar manner with all arms originating from the centerline of the trunnion pin. The resultant moment for the buoyant water force areas is the sum of the individual moments.

$A_1 = R - \frac{a}{2}$ $\quad A_2 = R - \frac{2}{3}a \quad A_3 = C_s \cos\frac{\theta_2}{2}$

$M_a = a_1 A_1 + a_2 A_2 + a_3 A_3$

The Force due to vertical force above horizontal is: $P_{v_2} = A\gamma_w$

The moment due to vertical force above horizontal is: $M_{v_2} = M_a \gamma_w$

F.3.4.4 The net vertical water loads are the sum of the two forces computed in previously.

Net Force: $P_v = P_{v_1} + P_{v_2}$

Net Moment: $M_v = M_{v_1} + M_{v_2}$

The resultant location is: $X = \frac{M_v}{P_v}$

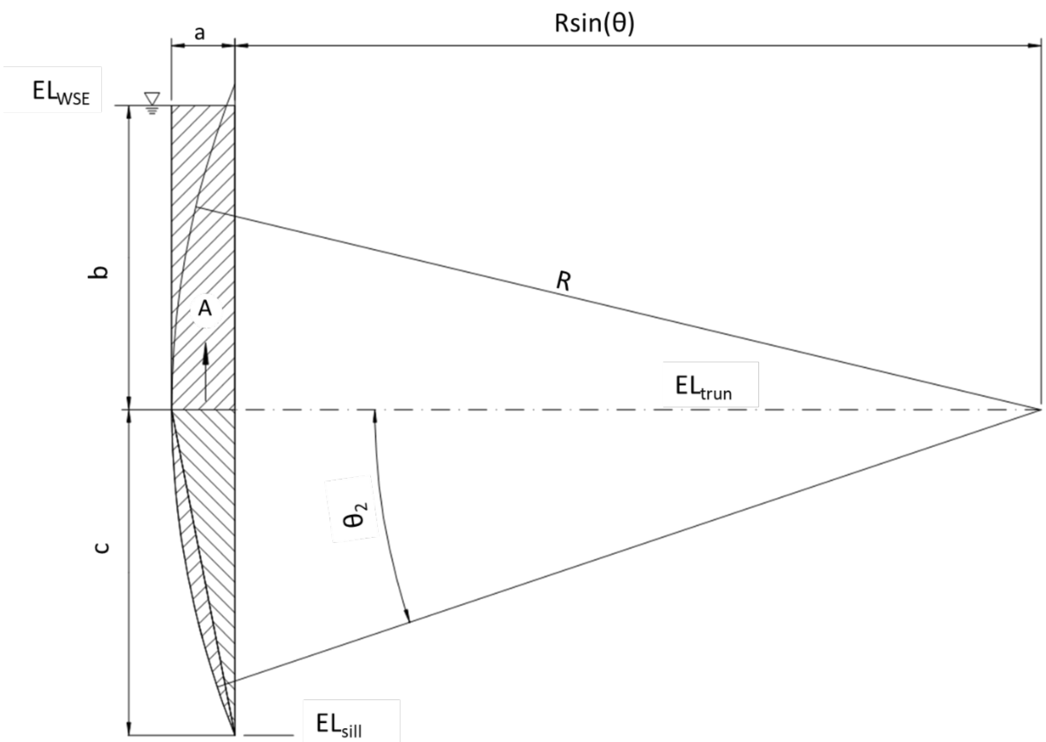


Figure F.17. Column of Water Acting Up on Tainter Gate

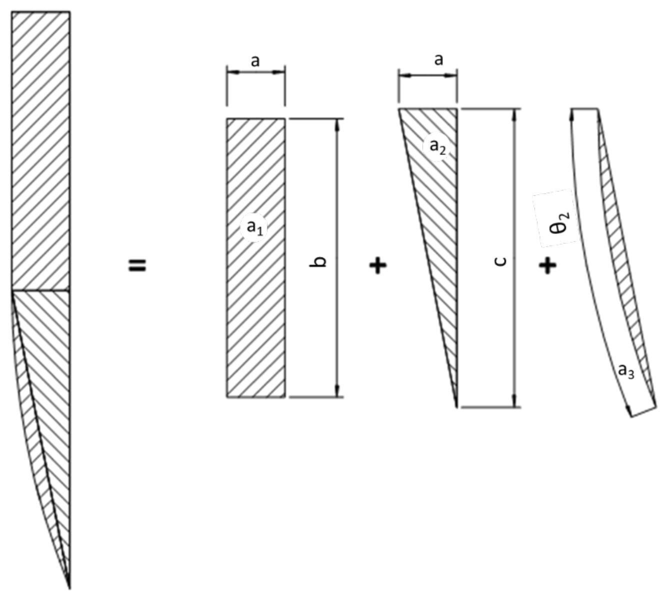


Figure F.18. Geometry Used to Compute Water Acting Up on Tainter Gate

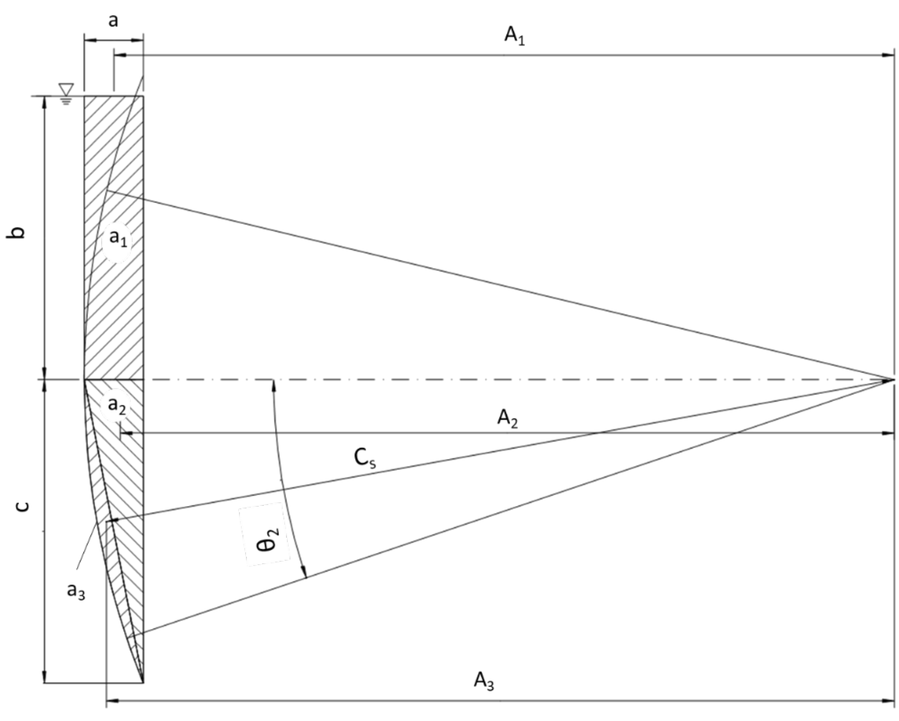


Figure F.19. Moment Arms of Water Acting Up on Tainter Gate

F.3.5. Step-Wise Iterations.

F.3.5.1 Background. Similar to the integrated solutions, discrete areas of water are evaluated and summed to get forces and moments. The more discrete the areas, the more accurate the results will be. Select the number of discrete steps, i, compute the individual forces and moments, and sum to get totals.

F.3.5.2 Vertical Loads. The direction of the load must be accounted for when calculating vertical loads. A convenient method is to separate increments above and below trunnion elevation to avoid one iteration that lies both above and below the trunnion centerline. See Figures F.20 and F.21 for variable references. Vertical loads are represented as a series of water columns acting on the skin plate. The shape of the column can vary depending on the level of accuracy desired. A rectangular distribution, with the column height taken at mid height of a segment provides for simple calculation. A trapezoidal distribution adds complexity of calculation but improves accuracy.

A trapezoidal distribution is demonstrated here. This leads to slightly unconservative results due to the net loss of load contained within the segment not included in the column shape. Including the segment should result in loads identical to that obtained from the previous section. It is convenient to separate the rectangle and triangle for computing moments as shown here.

Compute the force at each location as: $p_{v_{i1}} = \gamma_w \Delta x_i y_i$ $\quad p_{v_{i2}} = \gamma_w \frac{\Delta x_i}{2}(y_{i+1} - y_i)$

Where; $\Delta x_i = R(\cos\theta_{i-1} - \cos\theta_i)$ $\quad y_i = h - R\sin\theta_i$ $\quad y_{i+1} = h - R\sin\theta_{i+1}$

Compute the moment at each location as: $m_{v_{i1}} = p_{v_{i1}} a_{i1}$ $\quad m_{v_{i2}} = p_{v_{i2}} a_{i2}$

Where, $a_{i1} = R\cos\theta_i + \frac{\Delta x_i}{2}$ $\quad a_{i2} = R\cos\theta_i + \frac{2\Delta x_i}{3}$

Total loads: $P_v = \sum(p_{v_{i1}} + p_{v_{i2}})$ $\quad M_v = \sum(m_{v_{i1}} + m_{v_{i2}})$

Resultant location is as computed previously.

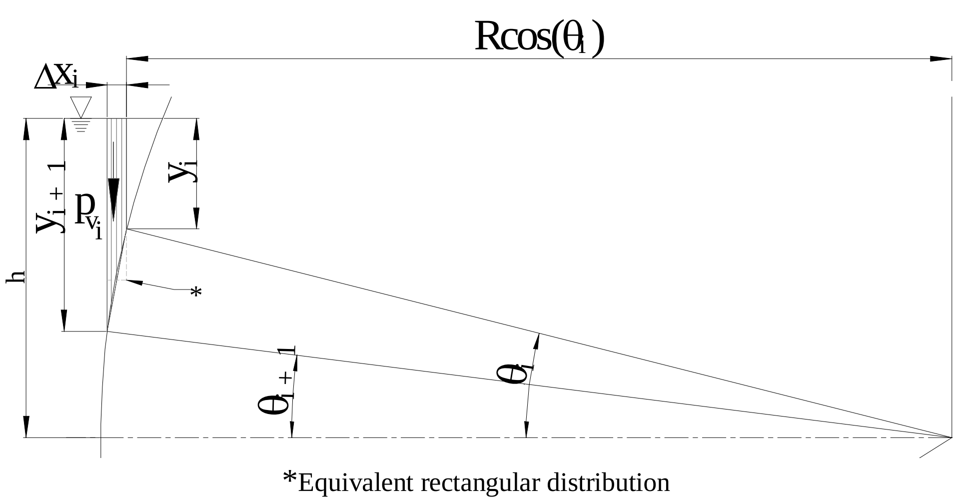


Figure F.20. Iteration of Vertical Water Load Above Trunnion

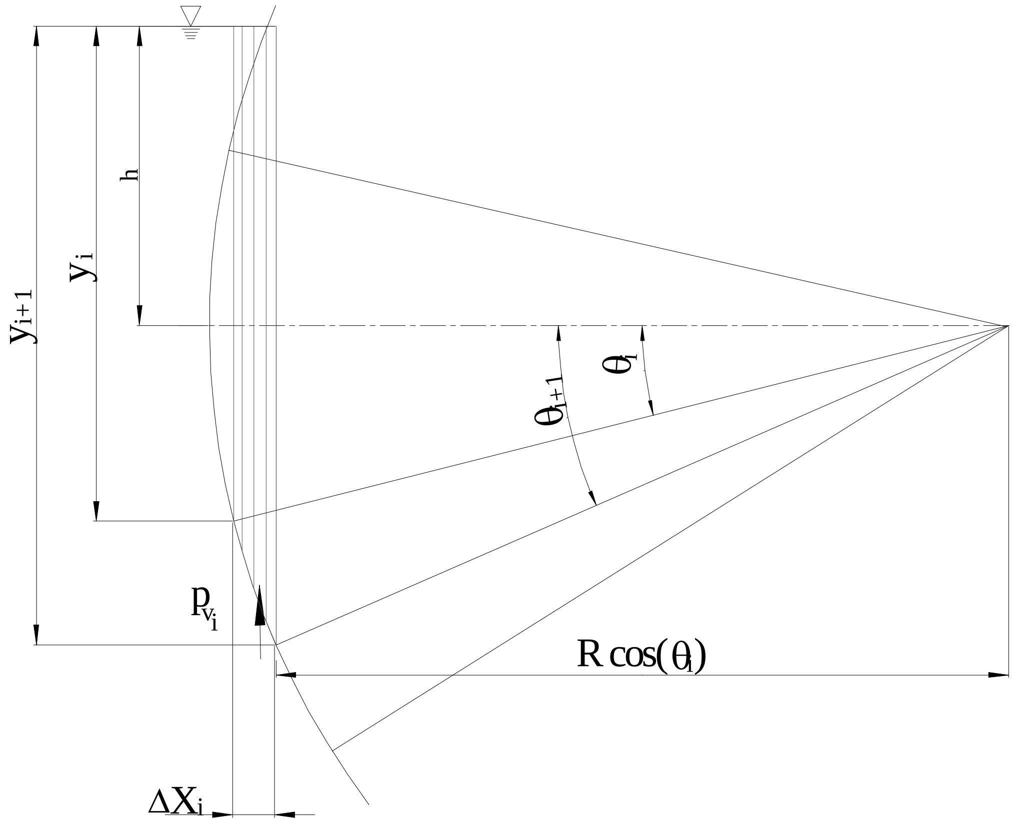


Figure F.21. Iteration of Vertical Water Load Below Trunnion

F.3.5.3 Horizontal Loads. A similar approach can be applied to the horizontal load. However, applying a trapezoidal pressure distribution to each iteration yields identical results to the equation of paragraph F.3.4.2 and thus is not necessary.

F.4. Trunnion Friction Determination.

F.4.1. Procedure. Gate movement incurs an additional resisting force due to friction between the trunnion pin and bushing. The friction force can be computed from the reactions due to all externally applied loads using an iterative process as follows.

a. Compute all externally applied loads.
b. Determine reaction, Q_T, at the lifting attachment bracket by summing moments about the CL of pin.
c. Sum the horizontal and vertical components of each load to determine the horizontal and vertical components of the trunnion reaction.
d. Resolve the horizontal and vertical reactions into the trunnion reaction force.
e. Compute the friction forces using an assumed coefficient of friction.
f. Repeat steps b–c and compare changes in the friction forces.
g. Iterate until the differences are at an acceptable magnitude.

F.4.2. Using the loads and geometries of Figure F.22, trunnion friction loads are computed as follows. Use load combination 3, Equation 10.7 for this demonstration. The equation is modified to include the wire rope load, Q. See Figure F.23 for depiction of trunnion forces.

(1.2 or 0.9) D + (1.6 or 0) G + γ_{pr} Hs_{pr} + 1.4 Fs + 1.4 Ft + Q (10.7 modified)

F.4.2.1 Applied loads are given.

D = Dead Load
G = Gravity Load
Hs = Hydrostatic Load
Fs = Side Seal Friction Load
Ft = Trunnion Friction Load
Q = Wire Rope Reaction Loads
Mt = Moment caused by pin friction
θ_p = Orientation of Trunnion Reaction, Rt
r = Pin Radius
μ = Trunnion Friction Coefficient

F.4.2.2 The wire rope reaction, Q_T, is determined by summing moments about the trunnion pin. The load acts on the gate at the lifting point (see Figure F.22) and in this case, is a reaction resisting load and not an external force (e.g., the machinery is exerting a force on a jammed gate). This value is calculated from factored loads and so needs no other load factor.

$$Q_T = (1.2D \; x \; X_D + \; 1.6G \; x \; X_G \; + \; 1.4Fs \; x \; X_{Fs})/R \qquad \text{(Equation F.17)}$$

F.4.2.3 Horizontal and vertical loads are as follows with signs applied based on directions of loads:

$$Rt_x = \; Hs\cos\theta_H + \; Q\cos\theta_Q + Fs\sin\theta_{Fs} - \; Q_T\sin\theta_T \qquad \text{(Equation F.18)}$$

$$Rt_y = \; Hs\sin\theta_H + \; Q\sin\theta_Q - Fs\cos\theta_{Fs} + \; D + G - \; Q_T\cos\theta_T \qquad \text{(Equation F.19)}$$

F.4.2.4 The total trunnion reaction and orientation are computed as follows:

$$Rt \; = \; \sqrt{{Rt_x}^2 + \; {Rt_y}^2} \qquad \theta_{Rt} = \; \tan^{-1} {Rt_x}/{Rt_y}$$

F.4.2.5 The friction forces are:

$$Ft = \; \mu Rt \qquad Mt = Ft \; x \; r$$

F.4.2.6 Recompute Q_T including the moment due to friction and compare changes in reactions.

$$Q_T = \frac{(1.2D \; x \; X_D + \; 1.6G \; x \; X_G + \; 1.4Fs \; x \; X_{Fs} + 1.4Mt)}{R} \qquad \text{(Equation F.20)}$$

F.4.2.7 An alternative to this procedure is to assume a value for Q_T and change values of Q_T until changes in reaction forces converge.

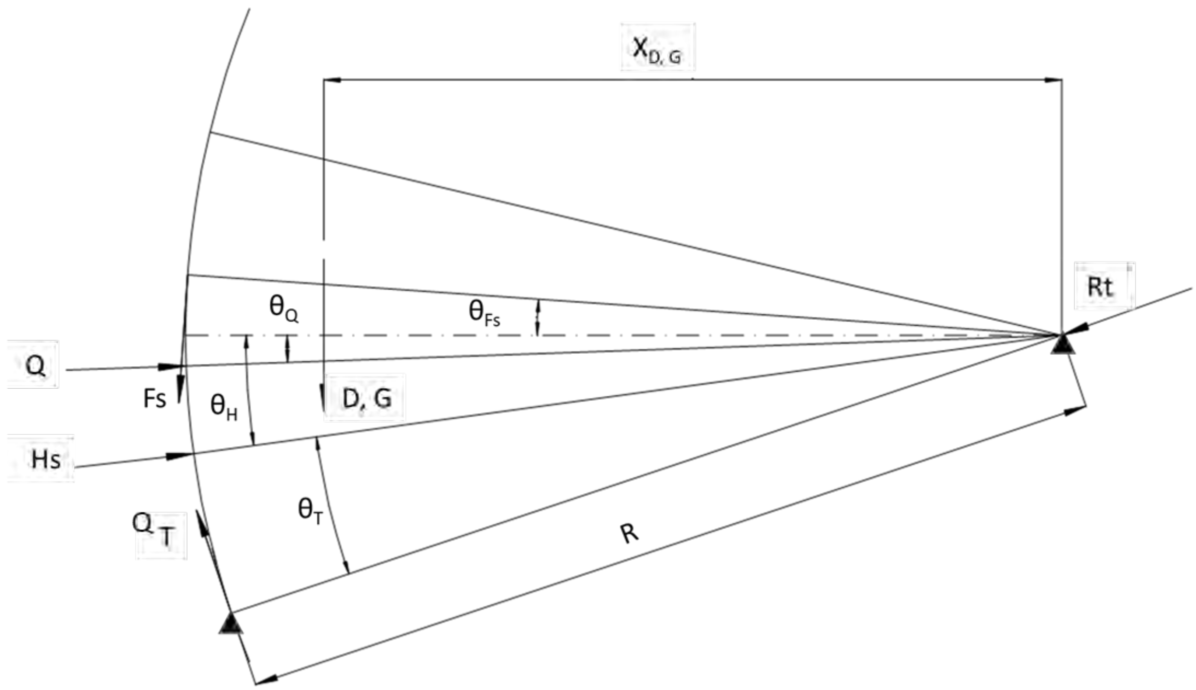


Figure F.22. Example Tainter Gate Loads

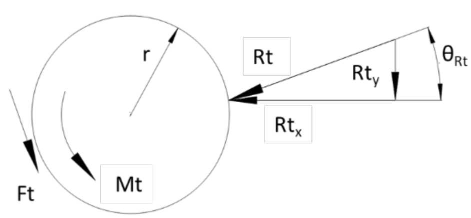


Figure F.23. Trunnion Pin Forces

F.5. <u>Example Calculations</u>. Example calculations are provided for each of the loads described in this appendix.

F.5.1. Tainter Gate Data. Each example will use the same gate size and geometry and same head and tailwater elevations. Data is provided in Table F.1 except where noted.

F.5.2. Side Seal Friction Forces. Side Seal data is provided in Tables F.1 and F.2. The length of seal engaged due to preset will typically be different than the length exposed due to hydrostatic head due to the connection of seal to the gate. The seal modulus of elasticity coefficient of friction can be obtained from seal manufacturers. The moment of inertia calculated is per 12 in. length of seal so that:

$I_j = \frac{12in\ x\ t^3}{12}$ Substituting t, $I_j = 1in^4$ per foot of seal length

F.5.2.1 The preset force, S, is computed using Equation F.1.

$S = 3\delta E I_j / d^3 = 7.03 lbs$ per foot of seal length (Equation F.1)

F.5.2.2 The preset friction force is computed using the first part of Equation 10.1. This force acts on the entire length of seal.

$$F_{s1} = \mu_s S l = \ 0.15\ kips$$

Table F.1
Example Tainter Gate Loads

Elevations					Angles			
Location	Variable	Value	Unit	Location	Variable	Value		Unit
Top of Gate	EL_T	926	ft	Trunnion pin to top of gate	θ_1	ASIN((EL_{TP}-EL_T)/R)	0.412	rad
WSE, headwater	EL_{WSE}	926	ft	Trunnion pin to bottom of gate	θ_2	ASIN((EL_{TP}-EL_B)/R)	0.644	rad
WSE, tailwater	EL_{TW}	886	ft	Tailwater to bottom of gate	θ_{TW}	ASIN((EL_{TW}-EL_B)/R)	0.000	rad
CL Trunnion Pin	EL_{TP}	910	ft	Trunnion pin to WSE	$\theta_{TP\text{-}WSE}$	ASIN((EL_{WSE}-EL_{TW})/R)	0.412	rad
Bottom of Gate	EL_B	886	ft	WSE to tailwater	θ_{WSE}	$\theta_2 + \theta_{TP\text{-}WSE}$	1.055	rad
Lifting Bracket	EL_{LB}	892	ft	Trunnion pin to wire rope attachment	θ_{LB}	ASIN((EL_{TP}-EL_{LB})/R)	0.467	rad
Dimensions				Wire rope attachment to bottom of gate	$\theta_{LB\text{-}B}$	$\theta_2 - \theta_{LB}$	0.177	rad
Radius	R	40	ft	Loads				
Gate Length	L	40	ft	Unit weight of water	Υ_w	Given	62.5	pcf
Gate Height	H_G	40	ft	Gate dead load	D	Given	50	kips
Gate Height, below trunnion	H_2	24	ft	Gate gravity load	G	Given	20	kips
Head	H	40	ft	Centroid of dead/gravity loads	$X_{D,G}$	Given	30	ft

F.5.2.3 Since this is a uniform load, the force resultant acts and the center of the arc or at elevation (926 – 20) ft = 906 ft. This is below the trunnion pin. The angle between the resultant and pin elevation is:

$$\vartheta_{F_{S1}} = \sin^{-1}\frac{EL_{TP} - 906ft}{R} = 0.100\,rad$$

F.5.2.4 The horizontal and vertical components are:

$$F_{s1x} = F_{s1}\sin\vartheta_{F_{S1}} = 0.0148 kips$$

$$F_{s1y} = F_{s1}\cos\vartheta_{F_{S1}} = 0.148 kips$$

F.5.2.5 The friction force due to hydrostatic pressure acts in a horizontal direction, perpendicular to the exposed area of the seal surface. Since there is no tailwater in this example, the pressure varies uniformly by the unit weight of water from the water surface to the bottom of the gate. The force is computed using the second part of Equation 10.1.

$$F_{s2} = \mu_s \gamma_w \, {}^{d}/_{2} \left(l_1 \, {}^{h}/_{2} + hl_2\right) = 0.5(0.0625kcf)(\tfrac{0.5\,ft.}{2})(42.2\,ft.x\,\tfrac{40\,ft.}{2} + 0) = 6.59\text{kips}$$

Table F.2
Data for Computing Side Seal Friction Forces

Description	Variable	Value		Unit
Hydrostatic head				
Depth of headwater	H	EL_{WSE}-EL_B	40	ft
Depth of tailwater	H_T	EL_{TW}-EL_B	0	ft
Headwater to tailwater	h	EL_{WSE}-EL_{TW}	40	ft
Length of Side Seal				
Headwater to tailwater	l_1	R x θ_{WSE}	42.20	ft
Tailwater to bottom of gate	l_2	R x θ_{TW}	0	ft
Total seal length	l	R x (θ_1 + θ_2)	42.20	ft
Coefficient of friction	μ	Given	0.5	
Preset distance	δ	Given	0.25	in
Length of seal exposed to preset	d_1	Given	4.0	in
Length of seal exposed to head	d_2	Given	6.0	in
Modulus of elasticity, seal	E_J	Given	600	psi
Seal thickness	t	Given	1.0	in

F.5.2.6 The force resultant is $^1/_3$ h =13.33 ft above the bottom of the gate or at elevation 899.33. This is 10.67 ft below the trunnion centerline. The corresponding angle and horizontal and vertical components are:

$$\vartheta_{F_{S2}} = \sin^{-1}\frac{10.67ft}{R} = 0.270\,rad$$

$$F_{s2x} = F_{s2}\sin\vartheta_{F_{S2}} = 1.76kips$$

$$F_{s2y} = F_{s2}\cos\vartheta_{F_{S2}} = 6.36kips$$

F.5.2.7 The total side seal force and moments are as follows. Since the resultants of the two components are below the trunnion pin centerline in this case, the direction of the forces are the same. The direction of the forces are vertical down and towards the trunnion when opening the gate and in the opposite direction when closing the gate.

$F_s = F_{s1} + F_{s2} = 6.74\ kips$

$F_{sx} = F_{s1x} + F_{s2x} = 1.77\ kips$

$F_{sy} = F_{s1y} + F_{s2y} = 6.50\ kips$

$M_{F_s} = F_s\ x\ R = 269.7$ kip-ft

F.5.2.8 The centroid of the friction force is assumed to at the radius, R, of the gate. It may be slightly more or less depending on the configuration of the seals.

F.5.3. Hydrostatic Load Calculations Using Integration. The height of water surface, Y, above the trunnion pin elevation is (924 – 910) ft = 14 ft. The angle to the top of WSE, θ_1 in the equations below, is $\theta_{TP\text{-}WSE}$ given in Table F.1 and because of the definition of angle directions in the derivation of the equations, will be negative or – 0.412 radians (clockwise for the orientation shown in Figure F.11). The values for each variable shown in the following equations are provided in Table F.3. Results are per foot of length of gate.

Table F.3
Hydrostatic Loads by Integration Per Foot of Gate Length

Variable	Value	Unit	Function	Value	Vectors (kip, kip-ft, rad)	
R	40	ft	$\cos\theta_1$	0.917	P	53.852
Υ_w	62.5	pcf	$\cos\theta_2$	0.8	Ph	50.0
Y	16	ft	$\sin\theta_1$	-0.4	Pv	15.081
θ_1	-0.412	Rad ↻	$\sin\theta_2$	0.6	θ_p	0.293
θ_2	0.644	Rad ↺	$\sin\theta_1^2$	0.160	Mh	533.333
$Y\theta_1$	-6.584	ft	$\sin\theta_2^2$	0.360	Mv	533.333
$Y\theta_2$	10.296	ft	$\sin\theta_1^3$	-0.064	Yp	10.667
			$\sin\theta_2^3$	0.216	Xp	35.364
			$\sin(2\theta_1)$	-0.733	θ_p	0.293
			$\sin(2\theta_2)$	0.960		

$$P = R\gamma_w[R(\cos\theta_1 - \cos\theta_2) - Y\theta_1 + Y\theta_2] = 53.85\ \frac{kips}{ft} \quad \text{(Equation F.9)}$$

$$P_h = R\gamma_w\left[Y(\sin\theta_2 - \sin\theta_1) - \frac{1}{2}R\left(\sin\theta_2{}^2 - \sin\theta_1{}^2\right)\right] = 50.0\ \frac{kips}{ft} \quad \text{(Equation F.10)}$$

$$P_v = R\gamma_w\left[Y(\cos\theta_1 - \cos\theta_2) + \frac{1}{4}R(2\theta_2 - 2\theta_1 + sin2\theta_1 - sin2\theta_2)\right] = 15.08\frac{kips}{ft}$$
(Equation F.11)

$\theta_p = \tan^{-1}\frac{P_v}{P_h} = 0.293\ rad$

$$M_h = R^2\gamma_w \left[\frac{Y}{2}\left(\sin\theta_2{}^2 - \sin\theta_1{}^2\right) - \frac{1}{3}R(\sin\theta_2{}^3 - \sin\theta_1{}^3) - \right] = 533.33\ \frac{kip-ft}{ft} \quad \text{(Equation F.12)}$$

$$M_v = -R^2\gamma_w \left[\frac{Y}{2}(\sin\theta_1{}^2 - \sin\theta_2{}^2) + \frac{R}{3}(\sin\theta_1{}^3 - \sin\theta_2{}^3)\right] = 533.33\ \frac{kip-ft}{ft} \quad \text{(Equation F.13)}$$

Moments of each component are identical, which they should be because the moment of the hydrostatic head about the trunnion pin is zero (the moments act in opposite directions).

Resultant location below trunnion, horizontal component:

$$Y_p = \frac{M_h}{P_h} = 10.67\ ft$$

Resultant location from trunnion centerline, vertical component:

$$X_p = \frac{M_v}{P_v} = 35.36\ ft$$

F.5.4. Hydrostatic Load Calculations Using Vertical and Horizontal Projections.

F.5.4.1 Horizontal Projection. Compute the horizontal components of the hydrostatic forces, per foot of gate length, and the centroid of the horizontal force.

$$P_h = \frac{1}{2}H^2\gamma_w = \ 50.0\ \frac{kips}{ft} \quad \text{(Equation F.13)}$$

$$y = (EL_{TP} - EL_B) - \frac{H}{3} = 24 - \frac{40}{3} = 10.67ft \quad \text{(Equation F.14)}$$

$$M_h = P_h y = 50.0 * 10.67 = 533.33\frac{kip-ft}{ft}$$

F.5.4.2 Vertical Projection. Compute the vertical components of the hydrostatic forces, per foot of gate length, and the centroid of the vertical forces. The values of Table F.1 are applied to the formulas of Section F.3.4 and the results shown in Table F.4. See also Figure F.23. The negative sign for a_1 accounts for the direction of load (downward is negative for this example).

F.5.5. Hydrostatic Load Calculations Using Iteration.

F.5.5.1 Vertical loads are calculated for the parameters of Table F.1. Twenty four increments are used, 10 above and 14 below the trunnion pin centerline. Increments are defined in terms of angle above and below the trunnion centerline. X and Y locations are calculated for each increment. Water column geometry is simplified assuming the shape of a trapezoid. The vertical loads are iterated and summed to calculate the total forces. See Figure F.23 for layout of increments. Results are presented in Table F.5.

F.5.5.2 Table F.6 shows a comparison for differing distributions of the vertical columns, rectangular, trapezoidal, and actual with per cent difference from actual. The differences are not significant and will likely not have any impacts on design.

Table F.4
Vertical Hydrostatic Loads by Projection Per Foot of Gate Length

Function	Value	Dimensions, ft		Force, kips		Arm, ft		Moment, kip-ft	
Water Acting Downward									
θ_1	0.412	a	3.34	a_1	-1.67↓	A1	38.89	a1xA1	-64.9 ↺
$\sin\theta_1$	0.4	b	16.0	a_2	0.58↑	Cs	39.49		
$\sin(\theta_1/2)$^3	0.0085					A2	38.66	a2xA2	22.26 ↻
$(\theta_1-\sin(\theta_1))$	0.0115			total	-1.094↓			total	-42.67 ↺
Water Acting Upward									
θ_2	0.644	a	8	a1	8↑	A1	36	a1xA1	288 ↻
$\sin\theta_2$	0.6	b	16.0	a2	6↑	A2	34.67	a2xA2	208 ↻
$\sin(\theta_2/2)$^3	0.0316	c	24	a3	2.175↑	Cs	38.77		
$(\theta_1-\sin(\theta_1))$	0.0435					A3	36.78	a3xA3	80 ↻
				total	16.175↑			total	576.0 ↻

TOTALS			
Force	15.08↑	kips	acting up
Moment	533.33	kip-ft ↻	rotating clockwise
Resultant, X	35.36	ft	from CL trunnion

Table F.5
Vertical Hydrostatic Loads by Iteration Per Foot of Gate Length

Increments		
Trunnion CL	No.	Radians
Above	10	-0.0412
Below	14	0.0460

Trapezoidal Distribution					
incr.	θ_i, rad.	y_i, ft	Δx_i, ft	pv_i, kips	mv_i, kip-ft
	-0.4115	0	36.6606		
1	-0.3704	1.5218	37.2878	-0.0298	-1.1059
2	-0.3292	3.0680	37.8519	-0.0809	-3.0422
3	-0.2881	4.6362	38.3519	-0.1204	-4.5885
"	"	"	"	"	"
9	-0.0412	14.3544	39.9661	-0.0859	-3.4280
10	0.00	16	40	-0.0321	-1.2844
11	0.0460	17.8379	39.9578	0.0447	1.7860
12	0.0919	19.6720	39.8311	0.1485	5.9225
13	0.1379	21.4983	39.6203	0.2712	10.7730
"	"	"	"	"	"
23	0.5975	38.5043	33.0690	2.3562	79.0856
24	0.6435	40	32	2.6224	85.3106
			Totals	15.06	532.69

F.5.6. Comparison of Hydrostatic Load Calculation Methods. Table F.7 shows a comparison of the results using the different methods. The comparison shows that any method is acceptable for this example. Values are per foot of gate length.

Table F.6
Vertical Hydrostatic Loads by Iteration Per Foot of Gate Length

Increments		
Trunnion CL	No.	Radians
Above	10	-0.0412
Below	14	0.0460

Trapezoidal Distribution					
incr.	θ_i, rad.	y_i, ft	Δx_i, ft	pv_i, kips	mv_i, kip-ft
	-0.4115	0	36.6606		
1	-0.3704	1.5218	37.2878	-0.0298	-1.1059
2	-0.3292	3.0680	37.8519	-0.0809	-3.0422
3	-0.2881	4.6362	38.3519	-0.1204	-4.5885
"	"	"	"	"	"
9	-0.0412	14.3544	39.9661	-0.0859	-3.4280
10	0.00	16	40	-0.0321	-1.2844
11	0.0460	17.8379	39.9578	0.0447	1.7860
12	0.0919	19.6720	39.8311	0.1485	5.9225
13	0.1379	21.4983	39.6203	0.2712	10.7730
"	"	"	"	"	"
23	0.5975	38.5043	33.0690	2.3562	79.0856
24	0.6435	40	32	2.6224	85.3106
			Totals	15.06	532.69

Table F.7
Comparison of Vertical Hydrostatic Loads by Iteration

			% Difference	
Distribution	P_v	M_v	P_v	M_v
Rectangular	14.940	531.68	99.1%	99.7%
Trapezoidal	15.06	532.69	99.89%	99.88%
Actual	15.081	533.33	--	--

F.5.7. Trunnion Friction Load Reactions

$$(1.2 \text{ or } 0.9)\ D + (1.6 \text{ or } 0)\ G + \gamma_{pr}\ Hs_{pr} + 1.4\ Fs + 1.4\ Ft + Q \qquad (10.7 \text{ modified})$$

F.5.7.1 Using the values of Table F.1 and load factors of modified Equation 10.7, compute the trunnion reactions without pin friction. Refer to Figure F.21 for variable designations and previous examples for load magnitudes. Assume the gate is just lifting off of the sill. Sign convention is positive for upward vertical loads and horizontal loads acting toward the trunnion.

F.5.7.2 Calculate wire rope reaction, Q_T, and pin reactions using Equation F.17. All resisting moments are acting in a counterclockwise motion for the orientation shown in Figure F.21. Thus the reaction Q_T acts clockwise. Compute the vertical and horizontal components. Recall the trunnion pin to wire rope attachment angle, θ_{LB}, is 0.467 rad.

$$Q_T = \frac{(1.2\ x\ 50kips\ x\ 30ft + 1.6\ x\ 20kips\ x\ 30\ ft + 1.4\ x\ 6.74\ kips\ x\ 40ft)}{40ft} = 78.44kips$$

$$Q_{T_x} = Q_T \cos\theta_{LB} = -35.30\ kips\ \ (\leftarrow)$$

$$Q_{T_y} = Q_T \sin\theta_{LB} = 70.05\ kips\ \ (\uparrow)$$

F.5.7.3 Assume for simplicity the wire rope first contacts the gate at the lifting bracket location and ends just before the top of the gate. The total angle of skin plate engaged is, from Table F.1.

$$\theta_W = \theta_1 + \theta_{LB} = 0.878\ rad$$

F.5.7.4 Calculate wire rope reaction force using Equation F.5b.

$$Q = 0.878\ rad\ x\ 78.44kips = 68.89kips \qquad \text{(Equation F.5b)}$$

F.5.7.5 The centroid of the force is located at the mid-point of the arc of wire rope contact at:

$\theta_{W_T} = \frac{1}{2}(\theta_1 + \theta_{LB}) - \theta_1 = \frac{1}{2}(\theta_{LB} - \theta_1) = 0.028\ rad$, where the positive sign indicates the centroid is above the trunnion pin centerline.

F.5.7.6 The horizontal (towards the trunnion) and vertical (down) reactions are:

$$Q_x = Q\cos\theta_{W_T} = 68.87\ kips\ (\rightarrow)$$

$$Q_y = Q\ sin\,\theta_{W_T} = -1.90kips\ (\downarrow)$$

F.5.7.7 Compute horizontal and vertical reactions with no pin friction. The sum of the forces must equal zero to satisfy equilibrium. Use the values computed previously. The load factor for Hs_{pr} is taken from Table 4.1. Assuming a normal structure and extreme event (return period between 300 – 3,000 years), the load factor, γpr, is 1.3. The hydrostatic load is multiplied by 20 ft, half the length of the gate. The variables Hs_x and Hs_y replace P_h and P_v respectively to be consistent with the variables defined in Chapter 4. The net side seal friction force is acting below the trunnion pin centerline; thus, the horizontal component is acting in the positive direction.

$Rt_x = -(Hs_x + Fs_x + Q_x + Q_{T_x} = 1.3(50)(20) + 1.4(1.77) + 68.67 - 35.30) = -1{,}336 kips$
(←)

$Rt_y = -(Hs_y + Fs_y + D + G + Q_y + Q_{T_y} = 1.3(15.08)(20) - 1.4(6.50) - 1.2(50) - 1.6(20) - 1.90 + 70.05) = -363.0 kips$ (↓)

F.5.7.8 The total trunnion reaction and orientation is as follows.

$Rt = \sqrt{(-1336)^2 + (-363.0)^2} = 1{,}385 kips \qquad \theta_{Rt} = \tan^{-1}\left({}^{363}/_{1336}\right) = 0.2653 rad$

F.5.7.9 The friction forces are as follows. Use μ of 0.3 and assume a 12 in. diameter pin.

$Ft = \mu Rt$ = 0.3 x 1385kips = 415.3kips

$Mt = Ft\ x\ r$ = 415.3kips x 0.5ft = 207.7kip-ft ↺

F.5.7.10 Recompute Q_T including the moment due to friction and compare changes in reactions. See Equation F.20.

$$Q_T = \frac{(1.2D\ x\ X_D + 1.6G\ x\ X_G + 1.4Fs\ x\ X_{Fs} + 1.4Mt)}{R}$$ (Equation F.20)

F.5.7.11 Table F.8 shows the results of the iterative process for computing trunnion friction. The trunnion friction forces converge quite quickly. The hydrostatic head, side seal friction, dead, and gravity loads on one trunnion are constant with values as follows.

$F_x = Hs_x + Fs_x = 1{,}300 + 2.48 = 1{,}302 kips$ →

$F_y = Hs_y + Fs_y + D + G = 392) + 1.90 - 9 - 60 - 32 = 291 kips$ ↑

$M = 1.2\ x\ D\ x\ X + 1.6\ x\ G\ x\ X + 1.4\ x\ F_s\ x\ R = 1{,}800 + 960 + 377 = 3{,}128\ kip - ft$ ↺

Table F.8
Trunnion Friction Force Iteration Calculations

	D+G+Fs														
Iteration	M, kip-ft	F_X, kips	F_Y, kips	Q_T, kips	Q_{Tx}, kips	Q_{Ty}, kips	Q, kips	Q_x, kips	Q_y, kips	θ_{Rt}, radians	Rt_X, kips	Rt_Y, kips	Rt, kips	Ft, kips	Mt, kip-ft
1	3,138	1,302	291	78.44	70.05	-35.30	68.89	68.87	1.903	-0.2653	1,336	363.0	1,384	415.3	207.67
2	3,138	1,302	291	83.63	74.68	-37.63	73.45	73.42	2.029	-0.2682	1,338	367.7	1,388	416.4	208.18
3	3,138	1,302	291	83.64	74.70	-37.64	73.46	73.43	2.029	-0.2682	1,338	367.7	1,388	416.4	208.18