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ETL 1110-2-584
30 June 2014

**US Army Corps of Engineers®**
ENGINEERING AND DESIGN

# DESIGN OF HYDRAULIC STEEL STRUCTURES

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

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Engineering and Design

# DESIGN OF HYDRAULIC STEEL STRUCTURES

1. Purpose. This manual prescribes guidance for designing new hydraulic steel structures (HSS) by Load and Resistance Factor Design (LRFD). This guidance is not intended for use in designing repairs to existing HSS. Allowable Stress Design (ASD) guidance is provided for those structure types where LRFD criteria have yet to be developed.

2. Applicability. This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibility for design of civil works projects.

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

4. References. References are listed in Appendix B.

FOR THE COMMANDER:

JAMES C. DALTON, P.E., SES
Chief, Engineering and Construction Division
U.S. Army Corps of Engineers

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Appendix F Closure Gates
Appendix G Bulkheads and Stoplogs

This manual supersedes EM 1110-2-2105, dated 31 March 1993, EM 1110-2- 2701, dated 30 November 1997, EM 1110-2-2702, dated 01 January 2000, EM 1110-2-2703, dated 30 June 1994, and 1110-2-2705, dated 31 March 1994.

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# CHAPTER 1

## Introduction

C1. Introduction

Commentary is provided throughout this manual to provide additional background and understanding of the guidance and is not mandatory.

1.1. Purpose. This manual prescribes guidance for designing new Hydraulic Steel Structures (HSS) and major rehabilitation of existing HSS by load and resistance factor design (LRFD). Repairs to existing HSS shall follow Engineer Manual (EM) 1110-2-6054. Allowable Stress Design (ASD) guidance is provided for those structure types where LRFD criteria have yet to be developed. Mechanical and electrical design considerations are addressed in EM 1110-2-2610.

See ER 1110-2-8157, Responsibility for Hydraulic Steel Structures, for definition of HSS.

C1.1 The LRFD design criteria was initiated within USACE effective 31 March 1993. The load and resistance factors provided for the design of new HSS are calibrated specifically for magnitude and frequency of loading events anticipated during the design life of the structure and material properties and condition associated with new construction. For evaluation and repair of existing HSS, the load and resistance factors provided in this guidance do not take into account existing conditions and evaluation life. 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 ETL should be used for evaluation. Repairs to existing HSS that were originally designed using allowable stress design (ASD) may be designed using AISC Allowable Strength Design (ASD) procedures. EM 1110-2-6054 “Inspection, Evaluation, and Repair of Hydraulic Steel Structures”, is currently under revision to address repair considerations.

1.2. Applicability. This manual applies to HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having responsibility for design of civil works projects.

1.3. References. References are listed in Appendix A.

1.4. Background. Types of HSS. Typical HSS are lock gates, tainter gates, tainter valves,

C1.4 Design of HSS is unique when compared to other types of steel structures because of the

bulkheads and stoplogs, vertical lift gates, components of hydroelectric and pumping plants, and miscellaneous structures such as lock wall accessories, local flood protection gates, and outlet works gates. HSS may be subject to submergence, wave action, hydraulic hammer, cavitation, impact, corrosion, and severe climatic conditions.

unique loadings and environment to which HSS are exposed. This manual explains how design requirements are impacted by this unique environment.

See EM 1110-2-1610, "Hydraulic Design of Lock Culvert Valves" for additional guidance on design of tainter valves.

1.5. Design Policy.

1.5.1. Engineering responsibilities for the design of HSS are given in ER 1110-2-8157. LRFD is the required method of design and shall be used for those structure types for which LRFD guidance is provided.

C1.5.1. Chapter 3 includes ASD criteria that are applied to those HSS where LRFD has not yet been developed.

ASD is combined with LRFD for certain mechanical components, e.g., bearings, trunnion hubs, lifting eyes and connections, etc., as LRFD procedures have not been developed for these components.

1.5.2. This manual provides the minimum requirements necessary to provide for safety, reliability, and required performance. Only those meeting the requirements for Engineer under ER 1110-2-8157 shall perform or oversee the performance of the design and fabrication of HSS.

C1.5.2 Application of this manual alone is insufficient in ensuring all design limit states are adequately addressed, but must be supplemented with proper training, experience, or exercise of judgment by the Engineer.

Lifting beams are considered HSS but are not included in this ETL at this time. Refer to References in Appendix A for guidance covering design of lifting beams.

Both ASME B30 and ASME BTH-1-2011 provides guidance on block shear, pin connection, stiffening requirements. For the mechanical requirements and address design and best detailing and fabrication practice in AISC 14th ed., and AWS D1.5. A lifting beam cannot be treated the same as other HSS and have specific requirements regarding compact vs. non compact sections, shear on bars, pins, plates, and connection design.

1.6. Mandatory Requirements. The term "shall" 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, but other suitably documented, verified, and approved criterion

may also be used so long as it is in a manner consistent with this design manual.

1.7. Reuse of Existing Designs. All newly fabricated HSS shall be designed in accordance with this manual. Reuse of existing designs is not acceptable unless these designs are modified to meet all provisions of this manual to include quality control and Quality Assurance (QA) reviews.

C1.7 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.

1.8. Evaluation of Existing HSS. The provisions of this manual are intended for design of HSS. They can also be used for evaluation. Many of the provisions, particularly as they relate to modern materials and fabrication processes, will not apply. Therefore, judgment and experience must be exercised when evaluating existing structures under these provisions. See EM 1110-2-6054 for guidance on evaluating existing HSS using Fitness for Service (FFS) procedures.

C1.8 See Section C.1.1.

1.9. Non-Carbon Steel Hydraulic Structures.

Designs for aluminum, timber, cast iron, stainless steel and cold form steel, shall conform to the respective industry standards and are not included in this manual but shall conform to the required load cases outlined in this guidance.

Refer to AISC Guide 27: Structural Stainless Steel, in Appendix A for guidance on design of stainless steel. For design of aluminum structures, see the Aluminum Association document, AA ADM-1, “Aluminum Design Manual”

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# CHAPTER 2

## Design Considerations

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

C2. Design Considerations. This chapter, which lays the foundation for the remaining chapters, is intended to give the user the big picture view of the design of HSS.

2.1. Design Philosophy. HSS shall be designed for the specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability and economy. The structural system of an HSS shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure. Multiple-load-path and continuous structures should be used unless there are compelling reasons to not do so.

C2.1. 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 (see this chapter) is a contributing factor to overall ductile behavior 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).

2.1.1. Failure Modes. All possible modes of unsatisfactory performance shall be considered when designing HSS. Possible failure modes are: general yielding, or excessive plastic deformation, buckling or general instability, subcritical crack growth leading to loss of cross-section or unstable crack growth, unstable crack extension leading to failure of a member, and excessive elastic deformation and excessive vibration that lead to serviceability failures.

C2.1.1. 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.

2.1.2. Reliability. HSS shall be designed to achieve a minimum level of reliability for each of the limit states evaluated.

C2.1.2. Reliability is a measure of an HSS to perform or function consistently for the given variability in loads and resistances. Design to achieve the necessary reliability is inherent in the design procedures, since minimum levels of reliability were considered in the development of the design criteria.

2.1.3. Risk. Risk associated with HSS shall be considered in an overall project risk assessment in accordance with ER 1110-2-1156.

C2.1.3. Risk is the expectation of adverse outcome and is commonly defined as the probability of load times the probability of failure or unsatisfactory performance times consequences.

2.1.4. Limit States. HSS shall be designed to satisfy all applicable limit states, a limit state being a "condition in which a structural system or component becomes unfit for its intended purpose, when it is exceeded" (AISC 14th Ed). Limit States applicable to HSS are listed and described below.

C2.1.4. 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 (usually ASD structures) to achieve a desired level of reliability, and;
- 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 accounting primarily for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance.

Load and resistance factors are also related to the statistics of each through the calibration process. Note that instrumentation of HSS is being planned to further develop load and resistant factors for HSS.

2.1.4.1. Strength. The Strength Limit State ensures safety against failure (local or overall) during the intended life of the structure and will accommodate Unusual load events with the potential for some localized damage such as yielding or buckling while maintaining the structural integrity of the member or system.

C2.1.4.1 The Strength Limit State is applied to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that an HSS is expected to experience in its design life. Under the Strength Limit State, some localized damage may be allowed, but the HSS will continue to function as required.

2.1.4.2. Serviceability. The Serviceability Limit State ensures that the HSS will function as required under normal loading conditions.

C2.1.4.2 Under the Serviceability Limit State, damage is avoided under service loads to ensure that performance requirements are met over the life of the structure through imposing restrictions on stress, deformation, and cracking under regular service conditions.

2.1.4.3. Fatigue. The Fatigue Limit State ensures limited growth of cracks due to repeated stress cycles so that serviceability of the HSS is maintained and fracture is prevented.

C2.1.4.3 The Fatigue Limit State imposes restrictions on stress range because of a single design load occurring at the number of expected stress range cycles. This could be considered 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.

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

C2.1.4.4 Tensile rupture is addressed in the Strength Limit State. Brittle fracture is addressed, at a minimum, by imposing a set of material toughness and detailing requirements. This could be considered a Strength Limit State if the strength of a member or system is reduced by an amount sufficient to compromise function or safety.

2.1.4.5. Extreme. The Extreme Limit State ensures survivability after an extreme event.

C2.1.4.5 Extreme events include earthquake, flood, or vessel impact. Under an Extreme load event, the system may maintain function to some defined level and/or life safety is ensured.

2.1.5. Design Life. The design life of HSS is taken as 100 years. Considerations for any other design life should be in accordance with ER 1110-2-8159. In implementing ER 1110-2-8159 procedures, Service Life is synonymous with Design Life in this context.

C2.1.5. The LRFD procedures are calibrated for a 100-year design life. Some damage and related repairs are expected over the life of the structure. HSS should maintain an acceptable level of performance assuming a reasonable amount of maintenance. Service life is intended to be the design life. Service life is assured by

designing for the Service Limit State.

2.1.6. Constructability and Quality Assurance. The Engineer shall consider the effects of construction, including fabrication and erection, on the design. Constructability issues should include, but not be limited to, consideration of deflection, strength of steel and stability during critical stages of construction. HSS should be designed in a manner such 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 particular sequence of construction to prevent certain stresses, that sequence shall be defined in the contract documents.

C2.1.6. In general, the Engineer should not dictate how HSS will be fabricated or erected except where procedure, process, or sequence has a deleterious effect on the design. Because HSS are complex structures that exhibit 3-dimensional (3-D) behavior, improper sequencing can create residual or locked-in stresses that can have negative implications on design performance. Therefore, the Engineer should consider fabrication by specifying or excluding certain sequencing.

The Engineer shall review all fabrication and erection submittals to ensure that the intended quality and design are achieved, and to ensure that fabrication and erection of the HSS are conducted such that the assembled components are not overstressed or unstable during assembly.

The Engineer should conduct sufficient QA visits to the fabrication shop and the construction site to ensure that HSS are fabricated and erected in accordance with the design assumptions. See ER 1110-2-8157 for additional information on the Engineer's role regarding QA on HSS.

2.2. Loads.

2.2.1. Load Types. All loads to which an HSS is subjected shall be considered in the design and in accordance with Chapter 3. Loads that have a negligible impact on the design may be ignored.

C2.2.1. HSS can be exposed to a variety of loads including gravity (weight, ice, mud), static and flowing water, operational and machinery loads, debris, floating and thermal ice, barge impact and/or other environmental loads (e.g., wind, temperature).

2.2.2. Frequency of Loading. Varying frequency of load applications shall be considered in the design with three load categories, in terms of frequencies of occurrence, defined below.

C2.2.2. Design requirements will vary for varying frequencies of load applications to ensure economical and efficient designs. For example, some localized damage is acceptable under infrequent loading events given the limited occurrences of these loads and the ability of steel structures to redistribute loads under plastic deformations. Designing HSS to respond entirely elastically under these loading events would result in increased weights, fabrication costs, and operating costs. Frequency is determined through a statistical evaluation of the load

data and defined as a specific return period.

2.2.2.1. Usual. The Usual loading category represents daily or frequent operational conditions that require highly reliable performance. The design criteria for the Usual loading category apply to load cases with the predominant load (or joint loads) having a mean return period (Tr) between 1 and 10 years.

C2.2.2.1 This load category is typically associated with the Service and Fatigue Limit States. These loads should not cause damage. Steel should remain within the elastic state and fatigue cracking should be avoided or limited to non-critical size. Damage may accumulate due to normal degradation (operational wear-and-tear) over the service life of the project.

2.2.2.2. Unusual. The Unusual loading category represents infrequent operational conditions that require a defined level of performance, and that can be reasonably expected to occur within the service life of the project. The design criteria for the Unusual loading category apply to load cases with the predominant (or joint loads) having a mean return period (Tr) between 10 and 300 years.

C2.2.2.2 This load category is typically associated with the Strength and Fracture Limit States. These loadings should not cause significant damage or disruption of service. Localized yielding is acceptable (i.e., stresses may exceed the yield limit in localized areas). Damage may accumulate due to normal degradation (operational wear-and-tear) over the service life, and may require major rehabilitation or restoration after 50 years of service.

2.2.2.3. Extreme. The Extreme loading category represents possible conditions that are not likely to occur within the service life of the project. The design criteria for Extreme load cases are applicable if the predominant load (or joint loads) has a mean return period (Tr) greater than 300 years.

C2.2.2.3 This load category is associated with the Extreme Limit State. Significant damage may result from these loads in the form of plastification, buckling, tearing, fracture, and/or other damage, but sufficient redundancy exists to redistribute the loads and prevent catastrophic collapse. Limited operational capability may also be required to maintain minimal project function (e.g., maintain pool) until corrective measures can be implemented.

2.2.3. 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 applied to achieve a consistent level of reliability.

C2.2.3. Load combinations are based on probabilistic load modeling and a survey of reliabilities inherent in traditional design practice. Specific load combinations are based on the notion that the maximum combined load effect occurs when one of the time-varying loads is at the maximum lifetime value and the remaining loads are taken at an arbitrary point in time. The likelihood that two or more loads, excluding self-weight, are at their maximums coincidentally is remote. Load factors are calibrated accordingly to reflect the arbitrary nature of loading events.

2.2.4. Cyclic and Non-cyclic Loading. Cyclically-Loaded structures are those that repeatedly undergo significant changes in stresses during operation. Noncyclically, or statically, loaded structures are defined as those that do not repeatedly undergo significant changes in stresses in the process of operating. Noncyclically-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.

C2.2.4. For most HSS, the changes in stress are induced through changes in position, primarily under specific operating cycles (opening and closing), changes in the loading (e.g., changes in hydraulic head), or both. Two components used to define cyclically-loaded structures are: (1) the magnitude of the imposed stress-range cycles and (2) the frequency of the application of those cycles. The implications of cyclic versus non-cyclic loading are largely related to fatigue cracking. There is a practical lower limit of stress cycles to consider whether an HSS is cyclically-loaded or not, e.g., where fatigue cracking is not likely, but is dependent on magnitude of stress range and number of cycles. Design life and material strength are also considerations as are high-frequency vibration and high stress / low cycle conditions. Given an expected number of stress cycles over the life of the HSS, and live load stress range at a detail, the fatigue strength of a detail can easily be determined and the results used to designate a member as cyclically-loaded or not.

Common examples of cyclically-loaded HSS include Lock Miter gates, Lock Lift gates, Spillway Tainter Gates and Tainter Valves. Common examples of noncyclically-loaded HSS include dewatering bulkheads, stop logs, and flood closure gates.

2.3. Materials. Materials are selected to provide the required strength, ductility and other properties such that the associated design meets all applicable limit states and the performance requirements. Material properties specified may include yield and tensile strengths, ductility, fracture toughness, and corrosion resistance. Material certification reports help ensure that the material requirements are met. American Society of Testing and Materials (ASTM) A 709 is the preferred structural steel specification for meeting the appropriate limit states and performance criteria. ASTM A325 is the preferred structural fastener specification. Weld metal is selected

C2.3. ASTM A709 is the preferred structural steel due to the wide range of strengths and desirable welding and toughness characteristics, fine-grained characteristics, and availability. High Performance Steels (HPS), available in 70 and 100 ksi grades, are good alternates when higher strength steels are required due to improved (compared to other steel specifications) toughness properties and welding characteristics. When compared to higher strength (A490) bolts, structural bolts conforming to ASTM A325 are generally the preferred alternative because of higher ductility and easier installation, and because they can be galvanized. See Unified Facilities Guide

based on strength and toughness requirements and material types. Other materials, for attachments, appurtenances, and connections, are selected on a project-by-project basis.

Specifications (UFGS) UFGS 05 50 15 for guidelines on selecting forgings, castings, seals, and other materials related to fabrication of HSS.

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

C.2.4. A distinction is made among the various member types to better apply the design criteria.

2.4.1. Primary and Secondary. Primary members are the main load-carrying members that provide load path from point of load application to the supports. Secondary members provide support or stability (e.g., bracing, intercostals, and diaphragms) to the primary members. Secondary members also include attachments or other appurtenances on the HSS. Failure of a secondary member is generally a localized failure, 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.

C2.4.1. Secondary members may act on or as primary members whether intended or not and under different loading conditions. 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 being lifted; attachments for access ladders and catwalks are secondary members within HSS design, but may impact the fatigue resistance of primary members; and, bracing members can create distortion or out-of-plane bending in primary members.

2.4.2. Redundancy. Redundancy is a structural condition where there are more elements of support than are 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 or judgment.

C2.4.2. There are generally three types of redundancy considered in HSS:

- Load path redundancy, three or more main load-carrying members or load paths are provided between supports;
- Structural redundancy, provided by continuity of load path from support to support, and;
- Internal redundancy, a member configuration that contains three or more elements that are mechanically fastened together.

Redundant or non-FCMs are members that do not qualify as FCMs or Failure Critical Members.

2.4.2.1. Failure Critical Members. All Failure Critical Members in an HSS should be identified by the Engineer. A determination should be made in the design phase whether to require additional material testing and NDT requirements Additional scrutiny should include

C2.4.2.1 A Failure Critical Member is any member for which failure would cause collapse, partial collapse, or loss of functionality of the structure. Because this category does not have current guidelines for the level of scrutiny needed, the Engineer must determine the added

sufficient testing to assure the required performance is achieved and improved reliability of connections is obtained. By designating a member as Failure Critical, the Engineer may designate a defined level of additional scrutiny through the design and fabrication due to the possibility that failure of the member would result in collapse or partial collapse of the structure or a loss of operational functionality.

requirements for each specific structure. Although the requirements for FCMs may be used to develop the requirements of Failure Critical Members, specifying these requirements in all instances (i.e., non-Failure Critical Members) may result in uneconomical and overly conservative connections. Failure Critical Members can include components loaded in compression, shear, or torsion. Failure critical behavior must be considered under all load cases. Examples of additional scrutiny include additional NDT of welds, additional bolt testing, and excluding bolt threads from the shear plane.

2.4.2.2. FCMs. All FCMs in an HSS shall be identified by the Engineer and labeled in the project plans, and materials and fabrication requirements shall be included in the project specifications.

C2.4.2.2 The definition of FCMs is contained within ER 1110-2-8157. FCMs are subsets of Failure Critical Members. Some examples of FCMs where failure may result in collapse of the structure are truss members and girders in non-redundant structures. Examples of FCMs where failure may result in a significant loss of functionality or collapse of the structure is the gudgeon and diagonals on miter gates, tension chord of a truss-formed stoplog, lifting eyes, and tension component of a simple span or cantilevered trunnion girder.

2.5. 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.

C2.5. HSS have traditionally been analyzed on a member-by-member or component-by-component 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 may be warranted.

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

C2.5.1. While HSS are generally complex structures with complex interaction between members, connections, and moving parts, simplified analysis techniques are often employed in their design and design is based more on individual member loading rather than overall system response. In addition, 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 functional designs.

2.5.2. Simplified Analysis. Simplified analysis models are provided for each gate type. These models are typically 2D and conducive to hand analysis or simplified truss or frame models.

C2.5.2. 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.

2.5.3. Advanced Analysis. Advanced analysis techniques, including finite element methods and higher order numerical analyses, may be employed where complex loading conditions and/or member configurations exist or where verification of simplified analyses is desired. When advanced analysis techniques are used, they shall be sufficiently documented and verified. Documentation shall include a complete description of the analysis model, including element types and descriptions, meshing techniques, boundary conditions and nodal connectivity, loads and load application, and any other information necessary to adequately describe the analysis and results. The documentation should include all assumptions used in the analysis, how they differ from reality and why the differences are acceptable. Verification should include hand calculations and/or independent finite element analysis. See ER 1110-2-1150 for requirements on use of software for analysis.

C2.5.3. The simplified models identified previously may not represent 3D behavior adequately and lack of consideration for this interaction can lead to poor gate performance over time. Advanced analysis techniques provide a useful tool to better predict realistic behavior and more fully addressing member interaction in the design. However, the designer must fully understand the tool that is being used and have sufficient experience to understand behavior of HSS for these analyses to be meaningful.

2.6. Corrosion Control. Corrosion is controlled through proper material selection, coating specifications, detailing, cathodic protection systems. Corrosion control extends the life of the HSS by minimizing deterioration due to corrosion and impacting function of moving parts. Corrosion control on HSS is a life cycle challenge that can be effectively managed through the designer active participation in the development and implementation of a Corrosion Mitigation Plan (CMP).

C2.6. 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.

2.7. 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, and employing good detailing practices, and designing components for easy replacement.

C2.7. Inspection access includes access platforms, ladders and other means for physical access to HSS members and access for visual inspection including inspection tools to all members and connections as well as non-destructive testing (NDT) where applicable. Maintenance access includes physical access for greasing, painting, and other maintenance activities. Good detailing practices include detailing to provide access for painting, fastener and connection repairs, etc. The designer should seek input from operations personnel during the design process to help assure all operation and maintenance needs are considered in the design.

2.8. Plans and Specifications. The HSS design is communicated to the fabricator through the plans and specifications. All information necessary to fabricate the HSS as intended in the design is depicted in these documents.

C2.8. 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. The UFGS, which are intended to be used with Specs-Intact software, and the A/E/C CAD Standards (ERDC/ITL TR-09-2 and TR-12-1), should be used to the greatest extent possible when developing plans and specifications for HSS. Specifications typically supersede plan documents. Review plans and specs for conflicts.

2.9. Fabrication and Erection. Fabrication and erection processes are considered in the design to include considerations for additional stresses imposed from these processes, proper detailing to accommodate the processes, and handling and shipping during fabrication and erection.

C2.9. 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 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.

2.10. Deviations from Prescribed Design. Where special conditions exist, proposed modifications to the design requirements shall be submitted to CECW-CE.

# CHAPTER 3

## Design

Design requirements, including load combinations, load and resistance factors, and limit states to be considered in design of HSS, are addressed in this chapter. The design of members and connections shall be consistent with the intended behavior of the structural system and the assumptions made in the structural analysis. Capacities of HSS members shall be in accordance with the current edition of American Institute of Steel Construction (AISC) Steel Construction Manual, 14th ed. (hence referred to as AISC) as modified within this manual and unless otherwise specified.

3.1. Design Basis. All HSS members and connections shall satisfy Eq. 3.1-1 for each limit state, unless otherwise specified. The basic safety check in LRFD may be expressed mathematically as:

$$\Sigma\gamma_i Q_{ni} \leq \alpha\phi R_n \quad (3\text{-}1)$$

where:

$\gamma$i = load factors that account for variability in loads to which they are assigned

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

$\alpha$ = performance factor (see paragraph 3.1.1)

$\phi$ = 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

Rn = nominal resistance

Load factors and load combinations are defined in paragraph 3.3.

C3.1. 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. Transient loads are less known and, hence, they usually have a higher load factor.

3.1.1. Performance Factors for HSS. For LRFD of HSS, resistance factors of AISC are

C3.1.1. A performance factor is applied to AISC resistance factors for HSS design. This is to

multiplied by a performance factor α. The performance factor α shall be 0.9 except for the following structures where α shall be 0.85:

- For those HSS where inspection and maintenance are difficult because the HSS is normally submerged and removal of the HSS 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.

- For those HSS in brackish water or seawater.

reflect a higher level of reliability (compared to building design) due to the exposure conditions to which HSS are subjected, consequences of failures, and to be consistent with past designs. The variables that require additional consideration for HSS include:

- Facility of inspection;

- Maintenance and repair or replacement (may require dewatering or submerged work by divers);

- Possibility of corrosion (water may be fresh, polluted, brackish, or saline);

- Economic considerations (loss of benefits due to shutdown of a larger project if replacement becomes necessary);

- Possibility of severe vibrations or repeated stress reversals (hydraulic flow may cause vibrations and operating procedures may cause stress reversals);

- Relative importance (HSS may be critical in the project operation), and;

- Design life of the structure in severe environments (100 years).

For these reasons, reliability factors are applied to the resistance factors specified by AISC to effectively increase structural reliability.

3.1.2. Strength Limit State. Structures shall have design strengths at all sections at least equal to the required strengths calculated for all combinations of factored loads and forces. The required strength of structural components shall be determined by structural analysis using appropriate factored load combinations. Each relevant strength limit shall be considered. Elastic analysis is permitted unconditionally by this manual. Plastic analysis is permitted only with the approval of CECW-CE, and is subject to restrictions similar to those provided by AISC.

C3.1.2. 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.

3.1.3. Service Limit State. The overall structure and the individual members, connections, and connectors shall be checked for serviceability. The following limit states shall be considered in design for serviceability:

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

- Vibrations of the seals, equipment, or movable supports shall not impair the operability of the HSS.

- Loss of section or impacts to connections and moving parts that may impair serviceability or operability of the structure during its design life shall be limited by designing to minimize corrosion potential. Closure provisions shall be provided for maintenance access.

C3.1.3. Serviceability is a state of acceptable performance in which the function of an HSS, its maintainability, durability, and operability are preserved under normal service or operating conditions. Serviceability should be maintained for the expected life of the HSS.

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, etc.) shall be chosen to ensure serviceability with due regard to the intended function of the structure.

3.1.4. Fatigue Limit State. The Fatigue Limit State shall be satisfied by selecting fatigue resistant details that satisfy and the load combinations of Section 3.3. Nominal resistance is defined in AASHTO/AISC design specifications. The applied stress range due to fatigue loading shall be the maximum stress range produced by the Usual load condition for Fatigue I and for Fatigue II load combination (Infinite Life). The number of stress cycles shall be determined based on the number of projected operating cycles expected over the design life.

C3.1.4. The Fatigue Limit State is applied to cyclically-loaded structures. See Chapter 4 for further discussion on evaluation of fatigue life.

3.1.5. Fracture Limit State. The Fracture Limit State shall be satisfied by limiting the potential for fracture as prescribed in Chapter 4. Additional requirements shall be imposed on FCMs as defined within Chapters 3, 4, and 7. Where load effects are computed to satisfy the Fracture Limit State, the load and resistance factors for the Strength Limit State shall be used.

C3.1.5. See Chapter 4 for further discussion on fracture and fracture control. Where Fracture Mechanics is used to compute a critical cracks size, members will be sized using Equation 3-2 (p 3-7).

3.1.6. Extreme Limit State. The Extreme Limit State is based on the Extreme load category. HSS shall be designed to survive

C3.1.6. See Chapter 2 for further discussion on the Extreme Limit State.

extreme events and function as needed.

3.2. Loads. Loads shall be applied as specified in this section and in combination with other loads as prescribed in Section 3.3.

3.2.1. Self-weight. Self-weight or dead load includes the total weight of an HSS and its components. Self-weight shall be computed based on the nominal cross-section of the members plus all attachments and appurtenances, all fasteners and welds, and any coating system. Weight of the coating system shall be based on the specified thickness of the coating system. Self-weight shall be treated as a Usual load category.

3.2.1. Self-weight. 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 self-weight, including connections and coatings, is necessary for designing moving parts (bearings), machinery design, various support members (diagonals, anchor bars, etc.), and pick points. Because the self-weight is constant throughout the life of the structure, excluding effects from corrosion, it is treated as a Usual load.

3.2.2. Gravity Loads. Gravity loads consist of silt, debris, and atmospheric ice. Silt and debris loads shall be based on site conditions and past experience except that a minimum of 1-in. thick layer of silt shall be assumed acting in all areas where silt can accumulate without regard to drainage features. The unit weight of silt shall be taken as 90 $lbs/ft^3$ unless site-specific data is available. Atmospheric ice loads shall be determined using guidance of ASCE 7. Other ice loads are determined based on site conditions. Gravity loads are considered Usual

C3.2.2. Silt and debris can accumulate on any horizontal or nearly horizontal surface that is partially enclosed. 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. A minimum thickness of 1 in. accumulation on all non-draining surfaces (excluding presence of drain holes) should be considered for design. Other types of ice accumulation include that which occurs due to fluctuation of pool, spray from leaking seals, etc.

3.2.3. Hydraulic Loads. Hydraulic loads consist of all loads due to water including weight of water, wave action, and flowing water including flow induced vibrations, constricted flow, and water hammer.

C3.2.3. Hydraulic loads are typically the predominant loads, in terms of magnitude and frequency, on HSS.

3.2.3.1. Hydrostatic Loads. Loads caused by hydrostatic water shall consist of the hydraulic head differential across the HSS, with due consideration from any tailwater effects. The magnitude of load is a function of load category, Usual, Unusual, and Extreme, as defined in Chapter 2.

C3.2.3.1 While considered a hydrostatic load, the hydraulic head can be a function of hydrodynamic conditions, particularly the tailwater under flowing conditions. A Hydraulic Engineer should be consulted to quantify tailwater effects.

3.2.3.2. Hydrodynamic Loads. The magnitude of hydrodynamic loads is a function of load category, Usual, Unusual, and Extreme, as defined in Chapter 2.

C3.2.3.2 For consistency in application of LRFD, the coincident effects of hydrostatic and hydrodynamic conditions that produce the maximum load effects for each load category should be used. However, for convenience and conservatively, the individual maximum events, under each load category, can be determined separately and applied together to produce the maximum effects.

3.2.3.2.1. Wave Loads. Wave loads shall be considered for all HSS subject to significant wind and fetch and will be additive to the coincident hydrostatic load. When determining wave-loading effects, the load category will reflect the coincident water surface elevation and wind conditions that produce the maximum effects.

Flow induced vibrations can be controlled or minimized through proper seal detailing and through control of gate operation, e.g., select gate openings where vibration is acceptable. Where high heads exist or geometry promotes disruption of flow (e.g., tainter valves), modeling may be required. Consult with the Hydraulic Engineer for modeling requirements.

3.2.3.2.2. Flow Induced Vibrations. Vibrations shall be minimized through proper detailing and operation considerations. Otherwise, hydrodynamic loading shall be determined through modeling.

3.2.3.2.3. Gate Overtopping. Gate overtopping shall be considered for gates where overtopping is required for the gate operation and shall be considered usual or unusual as applicable. Overtopping may be considered for gates where overtopping is not considered part of the operation, but is possible under Extreme loading conditions.

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 (spillway tainter gates) and may need to be designed to survive this event, e.g., to continue to maintain pool until the event has past.

3.2.3.2.4. Other Hydrodynamic Effects. All other hydrodynamic effects shall be addressed as applicable using accepted fluid mechanics theory or as required for specific gate types.

Water hammer is another form of hydrodynamic load.

3.2.3.3. Operational Loads. Operational loads shall be defined as follows: 3.2.3

C3.2.3.3 Operational loads include all actions resulting from operating the HSS including machinery loads applied directly to the HSS and any loads resisting operation of the gate. 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

3.2.3.3.1. Usual. Usual loads are those caused by normal HSS operation. Normal gate operation is that operation encountered under Usual design loads. Usual operational loads

include machinery loads (acting through hydraulic cylinders, wire ropes, gate arms, or other mechanisms) directly applied to the HSS, resistance to motion of the HSS through friction between moving parts such as bearings, bushings, or seals, and resistance to the operation from externally applied loads.

3.2.3.3.2. Unusual. Loads caused by abnormal gate operation, where abnormal gate operation includes unbalanced hoists, unbalanced operation, or jammed gate condition where machinery is operated up to its safe limit. The safe limit is that imposed within the machinery design through use of load-limiting devices.

3.2.3.3.3. Extreme. Loads caused by a jammed gate subject to the full capacity of the machinery. The HSS shall be designed to function as needed when subjected to this load. Gate machinery loads shall be coordinated with the project Mechanical Engineer.

3.2.3.3.4. Gate machinery loads shall be coordinated with the project Mechanical Engineer.

3.2.3.4. Environmental Loads. Environmental loads include wind acting on HSS and loads induced through thermal movement, which are applied as Unusual loads only.

3.2.3.5. Impact Loads. Impact loads include floating debris and ice, thermally expanding ice, and barge impacts. Floating debris and ice and thermally expanding ice shall be taken as 5,000 lbs/ft applied uniformly over a 2-ft depth across the HSS members exposed to ice. The ice and debris loads shall be applied at or above the low water elevation for the Usual or Unusual load condition to produce maximum effects in each member. Barge impact loads and application are defined within the gate specific guidance. These loads need not be applied to the design of skin plates.

load.

A normal machinery load is that expected under normal operating condition to include design hydrostatic and hydrodynamic load conditions (Usual or Unusual depending on the limit state evaluated) and design friction forces without the need for additional machinery load due to debris, jammed gate, etc. See EM 1110-2-2610 for more detail on machinery design loads.

The machinery safe limit is one that is determined through design, in consultation with the project Mechanical and Electrical Engineer, with the intent to limit the machinery load to some reasonable limit after which the machinery will stop operating due to a load-limiting device.

The full capacity of the machinery can be significant and can cause failures. The extreme machinery load will be considered in the design if the HSS must function under or after this loading.

C3.2.3.4 Environmental loads will generally not govern the design of HSS except for possibly localized members. Consider thermal loading for determining impacts.

C3.2.3.5 Thermally expanding ice is included here because as defined, it is assumed to have the same magnitude as floating debris and ice and is applied in the same manner. 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 overtopping of an HSS, these loads will be considered in member design or provisions shall be

made to protect members from these loads.

3.2.3.6. Earthquake. Earthquake loads shall be based on the operational basis earthquake (OBE). The Earthquake load, E, shall be based on the inertial dynamic forces due to water. The inertial dynamic force due to water shall be determined using Westergaard's equation:

C3.2.3.6 The OBE is defined as that earthquake having a 50% chance of being exceeded in 100 years. This translates into a probability of annual exceedance of 0.0069, or approximately a 145-year mean recurrence interval.

$$p = \frac{7}{8} \cdot \gamma_w \cdot a_c \cdot \sqrt{Hy} \qquad (3\text{-}2)$$

where:

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

$\gamma_w$ = unit weight of water

$a_c$ = max. acceleration of pier or lock wall (a fraction of gravitational acceleration g)

$H$ = pool depth to dam foundation

$y$ = distance below the pool surface

The earthquake inertial forces due to the self-weight and gravity loads generally are insignificant when compared to the force due to the water and need not be considered except for large mass HSS.

3.3. Load Factors and Load Combinations. Load factors for each load and loading condition are defined for each HSS type in the appendices. The load combinations defined in the appendices shall be used to satisfy Equation 3-2 (p 3-7).

C3.3. Load factors provide 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.

3.4. Allowable Strength Design (ASD). The ASD shall only be used on those structures where the LRFD method has not been developed. HSS designed by the ASD method shall conform to specifications contained in AISC, except as specified herein, and to the gate specific guidance.

3.4.1. Design Basis. ASD is a method of proportioning structures such that allowable stresses are not exceeded when the structure is subjected to specified working loads. An elastically computed stress is compared to an allowable stress as represented by:

$$f(\Sigma Q_i) \leq R_n/\alpha\Omega \quad (3\text{-}3)$$

where:

$f(\Sigma Q_i)$ = elastically computed stress arising from the appropriately combined nominal loads

$\alpha$ = allowable stress modifier (see 3.4.c.(2))

$\Omega$ = safety factor specified in AISC

$R_n$ = nominal resistance

3.4.2. *Loads.* Loads are divided into Group I and Group II loadings as follows:

3.4.2.1. Group I

| | |
|---|---|
| Dead load | Buoyancy load |
| Live load (serviceway) | Hydrostatic load |
| Thermal stress load | Operating equipment load |
| Ice loads (static) | |

3.4.2.2. Group II

| | |
|---|---|
| Impact (vessel, debris, ice) | Water hammer |
| Wind loads | Ice loads (transient) |
| Wave loads | |

3.4.2.3. Operational Basis Earthquake (OBE)

Ice loads may be considered as Group I (static load) or Group II (impact, short duration load) loads depending on circumstances..

C3.4.2. ASD guidance for HSS considers Groups I and II loading, and Types A, B, and C stresses.,. Because of the environment in which HSS are placed, modifications to AISC allowable stresses for HSS types are applied to increase the factor of safety above that used in building design.

Group I loads include those loads that are relatively constant for a significant time period, and Group II loads are those that vary with time. Ice loads may be considered either Group I or Group II depending on the circumstances. If ice hanging on the structure is being considered as additional dead load or it is applying a lateral force due to expansion from thermal effects, it is considered a Group I load. Ice is considered a Group II load when it acts dynamically on the structure (waves, flowing water, etc.).

The 1/3 allowable stress increase used in previous versions of this guidance has been rescinded and is no longer permitted for group I and II loads combinations. The reason for this is that code-specified ASD load combinations provide for the proper amplification of loads. This accomplishes what the one-third stress

increase used to accomplish on the material-strength side of the equation. Consequently, the one-third stress increase is inappropriate for use with current ASD load combinations. Engineers that "double-dip," i.e., use both the one-third stress increase on the material side and reduced ASD load combinations are violating the code in an unconservative way.

3.4.3. Strength Limit State. The Strength Limit State is addressed in ASD by limiting member stresses as described in the following paragraphs.

3.4.3.1. Stresses. It is considered necessary to reduce the allowable stresses given in AISC for HSS design. Allowable stresses for three main types of HSS are specified in paragraph 3.4.3.2. If a structure has characteristics of more than one type, the lesser allowable stress is required.

3.4.3.2. Modifications for Allowable Stresses. The allowable stress modifier, $\alpha$, applied in Equation 3-3 (p 3-8) shall be taken as follows for the particular HSS Type:

C3.4.3.2 In general, it is considered that HSS are subjected to more extreme environments and are subject to less predictable loads than are buildings. Therefore, an increase in the design factor of safety over that used for building design is considered necessary for HSS design.

The grouping by HSS type is a means to distinguish characteristics of different HSS. Type A is considered to be the most extreme case, and Type C the least extreme case.

Type A includes those structures that are subject to unpredictable dynamic loading, or those that are normally submerged where maintenance is difficult. Unpredictable dynamic loading may occur as a result of hydraulic fluctuations in velocity and pressure due to abrupt changes in structure geometry or gate position as it is operated. Severe, unpredictable vibrations may also occur on structures subject to significant amounts of passing ice. Type A HSS include emergency gates, regulating gates where the structure passes through moving water under full pressure and flow conditions (unpredictable dynamic loading may occur), and lock valves

3.4.3.2.1. *Type A.* HSS, which are used for emergency closures and which are subject to severe dynamic (hydraulic) loading or are normally submerged where maintenance is difficult, and removal of the HSS causes disruption of the project. For Type A HSS, the allowable stress modifier, $\alpha$, shall be 1.33.

3.4.3.2.2. *Type B.* HSS, which are normally hydraulically loaded and are not subjected to unknown dynamic loading. For Type B HSS, the allowable stress modifier, $\alpha$, shall be 1.2.

3.4.3.2.3. *Type C.* HSS, which are used for maintenance and are not considered emergency closures. For Type C HSS, the allowable stress modifier, $\alpha$, shall be 0.9. The allowable stresses resulting from the application of modifying factors are the maximum allowable values and may not be further increased due to Group II

loading.

(normally submerged and difficult to maintain).

Type B includes structures for which dynamic loading is not significant and maintenance and inspection can be performed on a regular basis. HSS that may be classified as Type B include power intake gates designed for top of power pool, lock gates (miter gates, lift gates, and sector gates), and floodwall closures.

Type C structures include temporary closure items that are used to dewater for maintenance or inspection of gates, gate slots, and draft tubes. Stoplogs, bulkheads, draft tube gates, and bulkhead gates are included in this type. Such structures are not considered emergency closures and are usually opened and closed under balanced head conditions. The 1.1 factor reflects a 1/3 increase of the Type B allowable stresses. This increase is considered appropriate because such structures are used on a temporary basis under essentially constant loading.

3.4.4. Serviceability and Fatigue Limit States. Serviceability and fatigue shall be evaluated in accordance with LRFD procedures of Sections 3.1.3 and 3.1.4.

3.4.5. Extreme Limit State. The Extreme Limit State is not addressed through ASD.

3.4.6. Design for Operations and Maintenance. See Chapter 6 for guidance on designing for operations and maintenance.

# CHAPTER 4

## Fatigue and Fracture

4.1. Design for Fatigue. All cyclically-loaded HSS shall be designed for the Fatigue Limit State.

C4.1. HSS that are routinely operated are subject to cyclic loading. HSS that are susceptible to fatigue cracking will depend 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.

4.1.1. Stress Life. The Stress Life procedures, as defined in AISC and AASHTO, shall be used to design for fatigue. Either reference is acceptable for design. Two design conditions will be considered, Finite Life and Infinite Life.

C4.1.1. The Stress Life approach evaluates stress ranges, stress cycles, and detail category to determine the potential for fatigue. Generally, due to high stress ranges typically encountered in HSS, the Finite Life condition will control except that Infinite Life will govern where high cycles and low stresses exist, e.g., gate vibration due to hydrodynamic effects. The AASHTO/AISC procedures are intended for load induced fatigue, i.e., members subjected to tension from the primary loading direction (axial and flexural tension or the principal tension component from shear).

4.1.2. Selecting Fatigue Details. Details shall be selected to ensure that fatigue cracking will not occur over the life of the structure. Details with higher fatigue resistance, Fatigue Detail Categories A through C, are generally preferred. Detail Categories D through E′ should be avoided. The detail category that best matches those provided in AISC and AASHTO, in terms of geometry, configurations, and degree of stress concentration, should be used for design.

C4.1.2. Detail categories A through C have been shown to provide sufficient fatigue resistance except under Unusual loading conditions. Detail categories D through E′ may prove sufficient in fatigue strength under low stresses, but may perform poorly due to unanticipated or unpredicted stresses. Therefore, these detail categories should be avoided. Note that bolted details are Category B details only if installed as a slip-critical connection.

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 is necessary in selecting the proper detail category.

4.1.3. Selecting Stress Ranges. Stress ranges are calculated for live load stresses only and are the peak-to-peak values over one stress cycle. Where compression and tension exist within one cycle, the compression and tension stresses are combined to compute the total stress cycle. Fatigue should not be considered when the entire stress range is in compression or when the tension component of the maximum live load stress range will never exceed compression stresses due to dead load. In the latter case, fatigue can be ignored where the compression stresses due to dead load exceeds the tension component of the design live load stress range by a factor of two.

Stresses shall be determined through analysis with proper consideration of load path, load distribution, and boundary conditions. Stresses may be determined using the simplified load models provided in the appendices. Alternatively, stresses may be determined using instrumentation of HSS with similar configurations and operation to the HSS designed in conjunction with more refined analyses.

The loads selected for computing stress ranges shall be the normally occurring load for each loading cycle. Alternatively, the statistical variation in loading and resulting stresses can be determined and an equivalent fatigue loading computed.

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

C4.1.3. 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.

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 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 Fatigue Loading (CAFL) to better facilitate laboratory testing, where in reality, stresses are variable. To apply the fatigue provisions, a CAFL 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 one that occurs normally (the mean value). A conservatively assumed value is less than normal, say one or two standard deviations above the mean value. Variable amplitude stress ranges can be converted to constant amplitude through use of stress histograms and summing techniques.

C4.1.4. The number of stress cycles can be calculated as follows:

$$N = 365 * Y * n * ADOC \qquad (4\text{-}1)$$

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.

4.1.5. Distortion Induced Fatigue. Distortion induced fatigue shall be addressed through proper detailing and/or through fatigue design. Where fatigue design is used, stresses shall be determined through refined analysis and the Fatigue Detail Category shall be based on the behavior of the connection.

C4.1.5. Distortion induced fatigue, more commonly seen in bridges, 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 References for reference on Hot Spot Analyses.

4.2. Design for Fracture. The Fracture Limit State shall be considered in design of HSS to minimize crack growth potential and prevent brittle fracture. Design for fracture shall include minimizing stress concentrations, constraint, and specifying proper material properties. Stress concentrations shall be controlled through proper detailing and fabrication specifications. Detailing practices for fracture control shall follow practices used in fatigue design.

C4.2. 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 (see below). 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.

4.2.1. FCM. FCMs and/or components shall be designed and fabricated in accordance with the AASHTO/ AWS (American Welding Society) Fracture Control Plan (FCP). See Chapter 7 for guidance on FCP development.

C4.2.1. The FCP is contained within Chapter 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 toughness requirements, acceptance criteria, etc. The fabricator is responsible for selecting weld procedures, weld procedure specifications, etc.

4.2.2. Constraint. Constraint should be avoided at all connections through proper detailing and fabrication.

C4.2.2. Constraint is induced through existence of intersecting welds, intersecting members, thick weldments, and any other situation that does not allow members to deform freely, through Poisson's effect, and that can lead to sudden and brittle failures. Brittle fractures generally occur with little plastic deformation and cannot be predicted using modern inspection and analysis techniques and therefore should be avoided. Where constraint cannot be avoided, increased toughens, proper weld sequence, joint type, proper joint geometry, and heat treatment should be considered.

4.2.3. Fracture Mechanics. In lieu of the guidance for the design of fracture, the Fracture Limit State can be addressed directly through Fracture Mechanics.

C4.2.3. Fracture mechanics can be used to determine acceptable discontinuity sizes for a given set of design conditions including member stress and material properties. The results from this analysis can be applied directly to the FCP. This is more common in evaluation than in design.

# CHAPTER 5

## Connections and Detailing

5.1. Design of Connections. Connection design shall conform to the specifications contained in AISC and AASHTO/AWS except as specified herein. All connections shall be detailed by the Engineer. Any deviation from details originally specified by the Engineer shall be reviewed and approved by the Engineer. Details that result in safe economical fabrication, erection, and maintenance methods shall be used. Connection details shall be consistent with the assumptions used in the design analysis of the structure and shall be capable of transferring the required forces between connected members.

C5.1. Design of Connections. Connections consist of connecting elements (e.g., stiffeners, gusset plates, angles, brackets) and connectors (bolts, welds, or for older HSS, rivets). Connections for HSS are usually in a more severe environment than connections for buildings. AISC can be used as guidance on design of connections, but should be supplemented with AASHTO/AWS since many HSS members have more in common with bridges (sizes, types of connections, 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. See the HSS Standard Detailing Manual posted on the TEN site at the following URL under Reference Documents: https://ten.usace.army.mil/TechExNet.aspx?p=s&a=COPS;127

5.1.1. Design Considerations. Connections shall be designed to transfer the required forces obtained from the structural analysis, and shall maintain sufficient ductility and rotation capacity to satisfy the particular design assumption. Connection designs shall consider stress concentrations, eccentricities, field splices, imposed restraints (fixity), and fatigue resistance. Constraint shall be avoided in connections by using copes to avoid intersecting welds, limiting weld sizes, and avoiding stiff and constrained connections at intersecting members.

C5.1.1. Design Considerations.

5.1.1.1. Stress Concentrations. The design shall avoid abrupt transitions in thickness or width, sharp corners, notches, and other stress raising conditions where such stress concentrations have adverse effects on performance.

a. Stress Concentrations. Stress concentrations in connections are often ignored in design with no decrease in load-carrying capacity. This is because ductility of the steel redistributes localized high stresses. However, this does not mean details that cause stress concentrations can be ignored. Attention should be given to areas of large change in cross-section such as termination

of cover plates, welds where backing bars have not been removed, and at sharp discontinuities.

5.1.1.2. Eccentricities. The design of connections shall account for effects of eccentricity of fastener groups and intersecting members.

b. Eccentricities. Concentric connection details should be provided for all connections. 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. While eccentricities in statically loaded single-angle, double-angle, and similar members may be of minor consequence, connections for members subject to cyclic loading should be balanced about their center of gravity axes. However, the use of seal welds will preclude this consideration from design, and in these cases, provisions must be made for the effects of eccentricity. Eccentric connections can often be avoided through good detailing. In some cases, a concentric connection will result in undesirable details, such as intersecting welds, intersecting members, or limited weld access. In these cases, eccentric connections may not be avoidable.

5.1.1.1. Splices. Splices should be located in uncongested areas of low or moderate stress. When splices are necessary, they should be shown on the drawings with accompanying splice details.

c. Splices. Shipping restrictions, and operation and maintenance requirements, may require large HSS to be delivered in sections, which makes field splicing necessary to form the completed structure.

5.1.1.2. Fatigue. Connections shall be designed to minimize the possibility of fatigue damage by using proper detailing practices and designing for fatigue. Corrosion-induced fatigue shall be 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.

5.1.2. Welded Connections. Most HSS are constructed using welded connections. Welding requirements of AISC and AWS shall be followed. Thick plate weldments shall be designed considering heat requirements, toughness requirements, and geometric requirements (see AASHTO/AWS). Intersecting, overlapping, and intermittent welds shall be avoided. Through-thickness welds should have backing bars removed, where used, and should be ground smooth. The Engineer shall review and approve the contractor's proposed welding procedures and shop drawings.

C5.1.2. Welded Connections. AISC and AWS are useful aids to selecting the connection details. Many HSS contain thick (greater than 1.5 in. thick) plate weldments. Critical connections on HSS often consist of full penetration or large fillet welds to develop the full strength of a part. Heavy welding is labor intensive and may result in member distortion and large residual stresses. Thick plates and jumbo rolled shapes often exhibit low toughness away from rolled surfaces, and lamellar discontinuities are more prevalent than in thinner plates. Thermal effects due to welding further decrease material toughness and produce high residual stresses that act on these low toughness areas and lamellar discontinuities creating high potential for cracking. The adverse thermal effects are reduced with gradual heating and cooling of the weldment as it is welded, and with proper selection of weld process and procedures. Residual stresses in weldments are increased with increasing external constraint so the designer should detail connections to minimize constraint. While post-weld stress relieving can be used to reduce residual stress its application is typically limited based on the size of the weldment.

5.1.2.1. Welding Codes. AWS D1.5 should be used for fabrication of all HSS. AWS D1.1 may be used on redundant, noncyclically-loaded HSS where fatigue and fracture are not design considerations. Alternatively, weld acceptance criteria may be determined through application of fracture mechanics principles.

C5.1.2.1 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 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 more stringent inspection requirements, requires that all detected flaws be reported on fracture critical members, requires a fabricator have a quality control plan, requires specific material and welding material toughness properties, has 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, greater restrictions on material tracking and handling, requires qualification of weld procedures for certain processes and welding to other steels or castings, and provides for stricter controls on hydrogen diffusion. 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. See EM 1110-2-6054 for additional information on alternative acceptance criteria based on fracture mechanics and fitness for service concepts

5.1.2.2. Weld Metal. Weld metal shall be selected to suit the connection design including considerations for toughness, strength, and base metal specification. When welding to high-strength steel, use of undermatching weld metal strength should be considered to limit residual stresses.

C5.1.2.2. Weld Metal. An undermatched 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.

5.1.2.3. Fracture Control Plan (FCP). The Engineer shall be responsible for developing a Fracture Control Plan for HSS with FCM and for reviewing and approving the fabricator's FCP submittal. An FCP may be developed for members not designated FCM where control of fracture is desired. However, these members shall not be designated FCM for the purposes of implementing the FCP, but will be subject to the specific requirements of an FCP. Toughness limits shall be based on ASTM A709.

C5.1.2.3. Fracture Control Plan (FCP). The FCP is a joint effort between the Engineer and the fabricator. The Engineer is responsible for design and quality requirements including selection of material toughness, reduction of stress concentrations through proper detailing and acceptance criteria, imposition of welding and inspection requirements, and by requirements for quality certifications of the fabrication shop, welders, and testers, and to some extent, fabrication processes and sequencing, at least as they affect the design. The fabricator is responsible for weld procedures and processes including sequencing and control of distortion and quality control procedures.

5.1.2.4. Fracture Critical Welds (FCW). All welds to FCMs shall be considered fracture critical. The Engineer shall ensure that all FCW are identified in the shop drawings.

C5.1.2.4 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.

5.1.3. Bolting. Fully tensioned high-strength bolts shall be used for all HSS structural applications. Slip-critical connections should be used for cyclically-loaded HSS where higher fatigue resistance is required or where slip of the connection may result in inoperability of the structure. Creep of the coating system due to bolt tension shall be accommodated. Bolts shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. Bolting procedures shall be specified that ensure that proper tensioning is demonstrated through calibration testing.

C5.1.3. Bolting. In the past many HSS have used riveted connections; however, the use of rivets has been replaced by use of high strength bolts. 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. Per AISC, full tightening is required for 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 shall 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 may need replacing sometime during the life of the structure. Bolts that are not installed to slip critical requirements are classified as fatigue detail category D; therefore, where a higher fatigue resistance is required, the bolts will need to be installed to slip critical requirements.

5.1.3.1. High-Strength Bolts. High-strength bolts used in the fabrication and erection of structural connections in HSS shall be ASTM A325 or ASTM A490 bolts with compatible nuts and washers. ASTM A307 bolts or graded bolts (SAE J429 Grade 5 or Grade 8) shall not be used for structural connections in accordance with AISC and Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using High-Strength Bolts*.

C5.1.3.1 High-Strength Bolts. Only ASTM A325 and A490 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. A325 and 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, hardened washers are required under the nut side of the connection only by code. If the Engineer would like washers to be used on both sides of the connection, or in all instances, 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 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*.

5.1.3.2. Bearing Connections. Bearing connections are connections in which a snug-tightened joint or pretensioned joint with bolts transmits shear loads and for which the design criteria are based on 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. 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 connec-

C5.1.3.2 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 in the 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.

tion, 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 ensure that threads are excluded from the shear plane where appropriate. Bearing connections should not be used where fatigue or dynamic loads are applied to the connection as bearing connections will permit movement in the joint.

5.1.3.3. Pretensioned Connections. Fully pretensioned connections shall 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 loosening a connection when it is subjected to loads, particularly vibration loading. Pretensioned joints transmit shear and or tensile loads through the bolts and through bearing of the bolts against the connected material. As a result, the preparation of faying surfaces does not require special preparation. The surfaces being joined in a pretensioned connection may be uncoated, coated, or galvanized in any manner since the slip resistance is not being considered as resisting applied loads. Pretensioned installation involves the inelastic elongation of the portion of the threaded length between the nut and the thread run-out. Pretensioned connections are required for all joints where movement of the joint is not desired. Pretensioned connections are required for all connections in oversized bolt holes and required for all connections where water tightness is required.

C5.1.3.3 Pretensioned Connections. Pretensioned connections are defined in the RCSC *Specification for Structural Joints Using High-Strength Bolts*. Pretensioned connections must be verified with testing and verification. Pretensioned connections elongate the bolt so that all subsequent loads imposed on the bolt do not result in additional elongation. It is critical that external loads that would decrease the load on bolted connections be considered before fully pretensioning connections. 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 shall be verified.

5.1.3.4. Slip-Critical Connections. Slip-critical connections shall be used for all cyclically-loaded connections and for all connections where movement of the connection under applied load is considered detrimental to structure performance (i.e., lifting lugs, dogging brackets, gudgeon anchorages etc.). Slip-critical connections are also required for all connections that are to be designed in accordance with AISC/AASHTO as a fatigue category B connec-

C5.1.3.4 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

tion. Slip-critical connections should be used in the following instances for connections involving shear or combined shear and tension:

1. Connections that are subject to fatigue ,load with reversal of the loading direction.

2. Connections that use oversized holes,

3. 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,

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

5.1.3.4.1. 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. Slip-critical connections require both proper faying surface preparation and proper installation of pretensioned bolts. Slip-critical connections are those connections that have specified faying surface conditions that, in the presence of the clamping force provided by pretensioned fasteners, resist a design load solely by friction and without displacement at the faying surfaces. Consequently, it is necessary to prepare the faying surfaces in a manner so that the desired slip performance is achieved.

5.1.3.4.2. Faying Surface Preparation. In accordance with the RCSC *Specification for Structural Joints Using High-Strength Bolts* and AISC 350 *Steel Construction Manual*, faying surfaces of slip-critical connections must be properly prepared to provide the frictional slip resistance. 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 ensure that the connections that are intended to be installed as slip-critical connections are specifically identified on the drawings.

C5.1.3.4.2 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

structures exposure/ submergence and should specify the use of protective coatings or sealing methods as required to prevent corrosion or pack rust from forming resulting in reduced slip resistance and potential failed connections.

5.1.3.5. Fitted Bolts/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. 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/RCSC guidance using clamping force and friction, fitted connections are designed simply to ensure that shear is occurring in all bolts equally on the application of load.

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 A449 bar stock, which is equivalent in material properties to that of A325 bolts. Consultation with an experienced bolting Engineer, typically a mechanical Engineer, is recommended.

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

in accordance with Appendix A of the RCSC Specification for *Structural Joints Using High-Strength Bolts*.

C5.1.3.5 Fitted Bolts/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.

C5.1.3.6 Other Bolts. A307 bolts cannot be pretensioned. As a result, A307 bolts may only be used to carry shear loads in nonstructural connections.

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

C5.2. Detailing for Performance. Detailing for Performance refers to the practice of selecting details that will perform during the design 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.

5.2.1. Detailing for 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 shall 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.

C5.2.1. Detailing for Minimize Residual Stress. Poor weld quality can lower the toughness of the heat-affected zone if the heating and cooling are not 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 and/or strength). 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.

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.

5.2.2. Detailing for Fatigue Resistance. The Engineer shall select fatigue resistant details for all cyclically-loaded HSS, including those subject to vibration and high cycle fatigue, to minimize the potential for crack initiation and propagation in accordance with Chapters 3 and 4.

C5.1.2. 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.

5.2.3. Detailing for Fracture Resistance. The Engineer shall select fracture resistant details for all HSS to avoid restraint and constraint to reduce the potential for cracking.

C5.1.3. Detailing for Fracture Resistance. See Chapters 3 and 4 for a discussion of fracture resistance.

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

C5.1.4. 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. Weld wraps should be inspected for undercut, although small undercuts may not affect the fatigue resistance of the detail significantly (Spadea and Frank 2002). EM 1110-2-2704, *Cathodic Protection System*

welds. In non-submerged conditions or where the detail will not be exposed to a significant amount of moisture, welds may be terminated ½-in. 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.

*for CW Structures*, provides additional detailed guidance regarding corrosion control for HSS.

5.3. Detailing for Fabrication. All HSS shall be detailed to accommodate fabrication processes. Individual members should be fabricated before final assembly of the components into the global structure. Provide 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.

C5.3. 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.

5.3.1. Distortion Control. Provide for proper fit-up, weld sequencing, and avoid warping. Weld sequencing is critical to controlling residual stress during fabrication. Simply stated, individual pieces (e.g., butt 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.

C5.3.1. 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. Con-

sideration 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) (see reference in Appendix A) that can be used to help shape steel both in the fabrication shop our in the field during erection.

5.3.2. Support and Restraint. An adequate amount of support and restraint should be provided during fabrication to maintain that the member remains straight during fabrication. 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.

C5.3.2. 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.

5.3.3. Access for nondestructive testing (NDT). This should be taken into consideration during development of design drawings and shop drawings.

C5.3.3. 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.

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

C5.3.4. Bolting Access. Bolted connections should be detailed with sufficient access to allow pretensioning of the connection. The Engineer should not rely on the table entering and tightening clearances presented in AISC. 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 CADD software to model both the connection and the hydraulic torque wrench to ensure that sufficient access has been provided.

# CHAPTER 6

## Designing for Operations and Maintenance

This chapter contains guidelines for designing to facilitate operation and maintenance and to ensure long-term performance. See the gate specific appendices (C-F) for additional guidelines for specific gate types.

6.1. Operability. Gates shall be designed for a high degree of operational reliability. Considerations include:

- Limiting sidesway and binding.
- Incorporate deicing systems, such as heating systems and air bubbler systems, where ice may accumulate and inhibit gate operation.
- Limiting deflections under service loads so that serviceability or operability is not impaired.
- Providing debris protection, debris removal, and/or debris passage.
- Considering a systematic waterway design that improves interoperability of parts, components, and HSS at other projects within the system.
- The Engineer's responsibility for participating in the development of the HSS Operation and Maintenance Manual at the time of design.
- Minimizing FCM to the greatest extent possible.
- Considering the implementation of technology such as Structural Monitoring and Analysis in Real Time (SMART) on HSS as a way to improve operational reliability.
- Lifting connections should be designed for safe and quick connect and disconnect operability.

C6.1 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 operability systematically.

Having the Engineer actively participating in the initial development of the HSS O&M Manual during the design process better assures integration between the design and operation phases. Important design information includes, for example: self and 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.

SMART technology for HSS has been initiated by ERDC-ITL and is planned for implementation at several locations. The technology provides real time information regarding the structural performance on HSS in service. See ERDC web site URL:

http://itl.erdc.usace.army.mil/

6.2. Maintenance. Structural systems for which maintenance is expected to be difficult should be avoided. Inaccessible cavities and corners should be avoided. Provisions shall be made for lubrication of all moving parts as required by mechanical design.

Design decisions that have a relatively significant maintenance cost should consider life cycle cost in the decision process.

HSS components that have a short life expectancy, or that, by the inherent nature of their functional requirements, involve abrasion should have connections other than welds to more readily enhance efficient replacement and maintenance.

C6.2 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 such as seals, contact blocks, timber bumpers, mechanical equipment, removable walkways, cathodic protection sacrificial components, bushings, etc. 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.

6.2.1. Corrosion Control. Structural steel shall have long life coating systems or cathodic protection. Member thickness shall 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 shall be 3/8-in.

C6.2.1. Corrosion Control. 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.

6.2.1.1. Mechanisms. Design for corrosion control shall account for the mechanisms that cause corrosion.

C6.2.1.1 Mechanisms. Corrosion mechanisms include localized corrosion (e.g., crevice corrosion or pitting corrosion), general atmospheric corrosion, mechanically assisted corrosion, or galvanic corrosion.

6.2.1.2. Material Selection. Materials shall be selected for corrosion control based on exposure conditions and based on the provided corrosion protection system while satisfying all design Limit States. Considerations shall be given for compatibility with connecting materials and cost.

Weathering steel shall not be used for submerged conditions.

C6.2.1.2 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.

6.2.1.3. Coatings. HSS shall be protected from corrosion by applying a protective coating

system or using cathodic protection. Members shall be proportioned to accommodate access for future coatings, coating repairs, and maintenance of cathodic protection systems.

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

C6.2.1.4 Cathodic Protection. Brief theoretical discussions on corrosion are presented in EM 1110-2-3400 and CASE (1993).

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

C6.2.1.5 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.

6.2.1.6. Detailing. Detailing considerations to ensure satisfactory operation and to minimize maintenance are:

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

- Configure seals to minimize gate vibrations.

- Detail members so that all exposed portions of the structure can be properly painted or coated.

C6.2.1.6 Detailing. The designer should consider all locations where water can be trapped for all gate positions. Long-term standing water should be avoided since it contributes to corrosion and becomes stagnant ponds of scum. The drain hole should have a minimum diameter of 2 in. Cope holes can 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 tanks for floating fish entrance gates.

In some cases, specifying a uniform increase in member component thickness provides a

- Seal weld so that water cannot be trapped between the connected plates. Consideration shall be given to minimum weld sizes per AISC and AWS.

- Grind slag, weld splatter, or any other deposits off the steel. These are areas that form crevices that can trap water.

- Break or grind sharp corners or edges to a minimum radius to allow paint or coating to properly cover the surface.

- 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.

- Consider using corrosion-resistant metal for areas that will be inaccessible for replacement.

- Avoid crevices and areas where water may pond.

- Avoid using dissimilar metals that result in galvanic corrosion.

structure with increased resistance to corrosion damage. However, because specifying such an increase is not effective for localized corrosion and because it increases weight and cost, this strategy is not recommended as a general practice.

6.2.2. Access for maintenance and inspection. Members should be located and proportioned to provide workers and required equipment with sufficient access to perform maintenance and repairs. Where practical, access to permit manual or visual inspection and maintenance shall be provided.

C6.2.2. Access for maintenance and inspection. Sufficient access for cleaning, painting, 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. 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 parts.

6.2.2.1. Dewatering. Provisions shall be made for maintenance and inspection dewatering.

C6.2.2.1 Dewatering. Bulkheads and stoplogs are typically used to dewater. HSS that cannot be dewatered should include provisions for removal for maintenance and inspection.

6.2.2.2. Lifting attachments. Lifting attachments, including dogging devices, etc. are used to

Lifting attachments should include considerations for lift load, impact requirements, mini-

accommodate fabrication, shipping moving, maintenance and repairs. If possible, they should be designed by the Engineer during the design phase. If designed during construction or operation phases, the Engineer shall review and approve the design.

mum thickness of the lug pin, lug projection and clearance as well as tensile and shearing modes of failure and limits on combined stresses.

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# CHAPTER 7

## Fabrication

7.1. Fabrication Responsibilities. The fabrication of HSS is unique in that the structures are not frequently fabricated similar to buildings or bridges. While a fabricator may be familiar with applicable building or bridge code requirements, welding procedures, and QA techniques, the requirements necessary for the fabrication of HSS are specialized and unique. As a result, the responsibility for fabrication oversight is more involved and detailed for the fabrication of HSS than many other Corps' projects. The designer of each HSS must be familiar with the fabrication responsibilities associated with each structure in accordance with ER 1110-2-8157, *Responsibility for Hydraulic Steel Structures*. In accordance with ER 1110-2-8157, the Engineer is responsible for the following tasks during the fabrication phase of HSS:

1. Shop Drawing Review and Approval

2. Construction Site Visits

3. Review of VE Proposals/Contract Modifications

4. Consultation on Plans and Specifications Interpretation

5. Final Inspection Of Completed Structures

6. Maintenance of Permanent Records of Construction, Including Testing Reports to be used for Future Evaluations.

C7.1 Fabrication Responsibilities. Fabrication of HSS in accordance with ER 1110-2-8157 requires extensive designer involvement in the review and approval of fabrication and erection details. Unlike many Corps' 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. In accordance with ER 1110-2-8157 *Responsibility for Hydraulic Steel Structures*, the Engineer, as designated by the District's Chief of the Engineering Division,

> shall 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 shall have at least the same qualifications as the Engineer.

In accordance with ER 1110-2-8157, the Engineer shall 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.

7.2. Use of Guide Specifications. New HSS shall be fabricated in accordance with UFGS 055913, *Fabrication of Hydraulic Steel Structures*. The guide specification is available as a Specs-Intact MasterFormat File and shall be amended as appropriate. The guide specification incorporates several key requirements in accordance with ER 1110-2-8157, *Responsibility for Hydraulic Steel Structures*. These requirements include:

C7.2 Use of Guide Specifications. Guide specifications for fabrication of HSS structures have been provided to ensure that structures are fabricated in accordance with 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.

*1. Responsibility of the Engineer* including submittal review, shop drawing review, etc.

1. Responsibility of the Engineer. In accordance with ER 1110-2-8157 and AWS D1.5, the Engineer is responsible for reviewing and approving all submittals related to fabrication of the HSS. The guide specification addresses this issue by incorporating an extensive list of government-approved submittals that are required to be submitted for Engineer approval. These submittals include Shop Drawings, FCP, Procedure Qualification Records, Weld Procedure Specifications, Weld Tracking Log, Welding Repairs, Delivery and Shipping Plan, and Control Dimensions. Many of these submittals are not required in general construction fabrication including structural steel fabrication for buildings. As a result, previous guide specifications for structural steel fabrication may not have required many of these submittals. The guide specification UFGS 055913 should be used to ensure that the proper submittals are being requested and reviewed by the Engineer.

2. *Fracture and Fatigue Requirements.*

2. Fracture and Fatigue Requirements. In accordance with ER 1110-2-8157, new HSS are to be detailed with fracture and fatigue requirements in mind. To fabricate structures with fracture and fatigue requirements, a fabricator must be familiar with the concepts of fracture and fatigue and must be familiar with the requirements to eliminate fracture and fatigue prone details. Much of the work that has been done to minimize the effects of fracture and fatigue has been done in the bridge fabrication industry. AWS D1.5, *Bridge Welding Code*,

incorporates good fabrication practices to minimize fracture and fatigue including proper welding procedures, removal of temporary and or tack welds, prohibited welded joint configurations, removal of weld backing bars and other details that have been shown to reduce the fracture and fatigue resistance of welded structures. FCMs require special welding procedures in accordance with AWS D1.5. All welds to FCMs are considered FCW in accordance with AWS D1.5. In addition, all FCMs identified on HSS must be fabricated from material that possesses toughness to minimize the initiation of fracture.

3. *Incorporation of AWS D1.5 relevant components including* addressing the fabrication of FCMs.

3. FCM Fabrication. The fabrication of FCMs requires the use of techniques and testing in accordance with AWS D1.5. AWS D1.1 does not address the fabrication of FCMs or the requirements of a FCP. The relevant components of AWS D1.5 have been incorporated into the guide specification to ensure that FCMs are properly fabricated.

4. *Incorporation of the Fracture Control Plan* in *accordance* with ER 1110-2-8157 and AWS D1.5.

4. Fracture Control Plan. In accordance with 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. The FCP addresses how the fabricator and steel erector will handle, cut, weld, bolt, assemble, and finish fracture critical components of the HSS in accordance with AWS D1.5 Chapter 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 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

prevent nicks and scratches, which are prohibited by AWS D1.5 to minimize stress concentrations, which can lead to fracture and fatigue.

5. *Special welding provisions* for HSS including the prohibition of tack and temporary *welds* and the incorporation of seal welds.

5. Special Welding Provisions for HSS. HSS are unique structures that are primarily submerged in water. To prevent corrosion of these structures, “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. AWS D1.5 prohibits the use of seal welds on many bridge connections as discussed above to minimize the initiation of fracture and fatigue. The guide specification UFGS 055913 specifically addresses seal weld requirements ensuring that all welds are made with the same level of quality and receive the same level of inspection and testing.

6. *Bolting requirements* including specifying and testing slip-critical connections to meet AASHTO fatigue category B. In accordance with AWS D1.5 and UFGS 055913, 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 that those performing QA on HSS understand the requirements associated with FCMs.

6. Bolting Requirements. In accordance with 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 UF`GS 055913 incorporates the necessary QA testing required to verify fully pretensioned bolted connections and slip-critical connections.

7.3. Fabrication Shop Certification. In accordance with AWS D1.5 Chapter 12, all fabricators performing fracture critical work

> … shall be certified under the AISC Quality Certification Program, Category III, Major Steel Bridges with Fracture Critical Rating, or an equivalent program acceptable to the Engineer.

Alternate fabricator certification criteria (American Society of Mechanical Engineers [ASME], API, AISC Simple Steel Bridge, Standard for Steel Buildings etc.) may be specified by the

C7.3 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 in accordance with AWS D1.5.

Engineer if deemed appropriate. Note that the FCP and fracture critical requirements specified in ER 1110-2-8157 will not be met with an alternate certification.

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 that the 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 weight and size restrictions along the shipping route to the project site, must be considered when designing HSS components.

7.4. Welding.

7.4.1. Welding Codes. AWS D1.5 should be used for fabrication of all HSS. AWS D1.1 may be used on redundant, noncyclically-loaded HSS where fatigue and fracture are not design considerations and where no FCMs are present in the structure. Alternatively, weld acceptance criteria may be determined through application of fracture mechanics principles.

C7.4.1 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;

- Provides for stricter controls on hydrogen diffusion.

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.

It should be additionally noted that AWS D1.5 commentary does not recommend the combination of use of AWS D1.1 and AWS D1.5 without careful consideration. The foreword of AWS D1.5 2010 state that:

> While the D1.1 and D1.5 codes do share a number of common provisions, it should not be assumed that revisions to one document provision automatically revises its analogous provision in the other. Therefore, users are encouraged to treat each code as an independent document.

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.

7.4.2. Acceptance Criteria. AWS D1.5 provides visual weld inspection acceptance criteria and ultrasonic inspection acceptance criteria for both tension and compression welds in Clause 6. Contract drawings must identify all fracture critical members as well as all tension members. The labeling of fracture critical members 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.

C7.4.2 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. Alternate acceptance criteria for weld inspection (ASME Boiler and Pressure Vessel Code, API, or Engineer generated requirements) should be considered carefully before being required. 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.

7.4.3. Welding Procedure Specifications. Welding procedure specifications shall be submitted and reviewed for all welds on an HSS regardless of prequalification. AWS D1.5 only permits pre-qualified welds for A709 steel joined with approved low hydrogen 7018 SMAW electrodes. To maintain records and compliance with ER 1110-2-8157, guide specification 055913 requires that all welds performed on HSS be qualified by testing. WPS qualified by testing shall be qualified in accordance with AWS D1.5. To qualify a WPS by testing, a Pre-Qualification Record (PQR) test shall be performed to test the proposed Welding Procedure Specification. In accordance with

C7.4.3. 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 055913 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 shall be

AWS D1.5 the Engineer or his/her designated representative shall be present for PQR testing. The welding procedure specification shall be reviewed and approved by the Engineer. The use of materials other than A709 requires Engineer approval and a PQR and WPS to be generated for each combination of base metals (A709 welded to A572, A709 welded to A35, etc.). There are often components of HSS that are manufactured from material other than A709 e.g., steel rub blocks or bearing blocks thicker than 4 in. (the maximum thickness of A709). In these instances, it is necessary that the Engineer define the materials that may be welded per the AWS D1.5 code and ensure that the proper PQR and WPS for joining these materials is requested and submitted.

In addition to WPS generated and submitted for new fabrication, a WPS is additionally required for all repair welding performed on an HSS. The WPS for repair welding in accordance with AWS D1.5 Chapter 12 will require separate essential variables including increased preheat and post-weld heat treatment or cooling times. Repair welding should be expected by the Engineer. In accordance with AWS D1.5 requirements, approval of all weld repairs shall be by the Engineer.

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

familiar with the process of WPS generation to review and approve WPS submitted.

C7.4.4 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 in accordance with AWS D1.5 Chapter 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 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 prevent nicks and scratches, which are prohibited by AWS D1.5 to minimize stress concentrations, which can lead to fracture and fatigue.

7.5. Installation of Bolted Structural Connections. Structural bolted connections shall be installed per RCSC specifications. The Engineer should ensure that adequate clearance is available for installation of all bolted connections. The Engineer should verify the dimensions of hydraulic torque wrenches and adequate clearance on both the nut and bolt head side of the connection.

C7.5 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. As a result, 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. In accordance with this, the Engineer shall ensure that structural bolted connections in HSS are adequately tested and verified before placing the structure in service.

7.5.1. Installation Sequence. The installation and tensioning sequence for complex bolted connections should be identified on the drawings. Connections should be installed from the interior of the connection working outward. Drawings and specifications shall ensure that all connections are snug-tightened. Connections that are fully pretensioned shall be snug-tightened before being fully pretensioning. This ensures that all plies of the connection are in contact before tensioning.

7.5.2. Installation procedures. The RCSC specification should be consulted for a description of the pre-installation verification procedures, installation procedures, and inspection procedures. UFGS 055913 includes notes that must be added to design specifications for structures with bolted connections to ensure that proper installation and testing procedures are used.

7.5.3. Testing of Bolted Structural Connections. The testing and verification of bolted structural connections shall be defined in the specifications. Testing and verification requirements are highlighted in the Notes contained in UFGS 055913. 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.

7.6. 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 above, The designer of each HSS should be familiar with the fabrication responsibilities associated with each structure in accordance with ER 1110-2-8157, *Responsibility for Hydraulic Steel Structures*.

In executing the requirements listed above, it is critical that the Engineer establish roles and responsibilities closely with the construction staff. It is critical that the Engineer becomes part of the Quality Assurance team for the fabrication and erection of an HSS.

C 7.6 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.

7.6.1. Responsibility for Shop Inspection. In accordance with ER 1110-2-8157 and UFGS 055913, *Fabrication of Hydraulic Steel Structures*, the Engineer will be performing 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. Witness points are critical points in the fabrication and assembly of HSS where assembly details, dimensional tolerances, and fabrication details are verified. These witness points are established as hold points that ensure that fabrication does not proceed until the Engineer and the USACE Construction Personnel are satisfied that the fabrication is proceeding in accordance with the plans and specifications. At a minimum, witness points shall be established for the Initial Quality Assurance Inspection, Intermediate Quality Assurance Inspection, and Final Inspection. UFGS 055913 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 Quality Assurance staff, additional fabrication inspection should be anticipated.

7.6.2. Transport. The Engineer should review the contractor's transport plan for all components fabricated at the 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 shall address transport and handling of the structure in the FCP as referenced above.

C7.6.2 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.

7.6.3. Field Fabrication Inspection Requirements. The intent of the rules for fabrication and QA in the field are no different than those performed in the fabrication shop. The Engineer should anticipate participating in field fabrication inspections. Field fabrication is typically required for large HSS, particularly

C7.6.3 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.

both welded and bolted splices of primary members due to shipping restrictions. The Engineer should be present to perform similar QA inspections of assembled components including splices, installation of hoisting components (wire ropes, chains etc.), sacrificial anode installation etc. Quality assurance must be maintained in the field to ensure that the FCP is adhered to.

# APPENDIX A

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# APPENDIX B

## Types of Hydraulic Steel Structures

B.1. General. USACE designs, constructs, and operates many types of projects. The primary project purposes are flood control (reservoirs), navigation, and local flood protection. 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 appendix provides a brief overview of the types and uses of various gates. The following Appendixes provide more detailed design information about specific types of gates, including:

- Lock gates (Appendix C);
- Tainter gates (Appendix D);
- Lift gates (Appendix E);
- Flood closure gates (Appendix F);
- Bulkheads and stoplogs (Appendix G).

B.2. Project Types.

B.2.1. Navigation Dams. 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 or less. Along the Mississippi and Ohio Rivers, lifts of 15 to 25 ft are common. In more hilly or mountainous areas, especially in the Pacific Northwest, lift heights can reach 100 ft. Lock gates obviously open and close to pass navigation traffic. There are some unique issues for design of these gates:

- Barge impacts on the gate;
- A need to pass ice and debris accumulating upstream of the gate;
- Subject to many load cycles and frequent operation of machinery;
- Require bulkheads or other closure systems for lock maintenance;

Lock gates, similar to gates for flood control projects, can also affect dam safety. In addition, failure of a lock gate could cause a lengthy halt to navigation. This could result in large economic losses, especially on waterways with large traffic volumes.

B.2.2. Flood Control Dams. Flood control dams provide volume in the reservoir to store incoming flows to prevent downstream flooding. They use gates 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 are usually kept low to provide the flood storage capacity. Flood control reservoirs use gates for spillways, sluices and intake towers. Spillway gates are frequently tainter

gates. Sluice gates are used mostly for low flows through the lower part of the dam, or through intake towers.

The gates can be a critical component affecting dam safety, so reliable performance is essential. Failure to close the gates can result in excess flows downstream, possibly resulting in flooding. Failure to open the gates to release flows can result in a rising pool possibly overtopping the dam. Erosion due to the overflow could cause dam failure. Structural failure of a gate can result in sudden excess flows downstream, with possible flooding. This could also result in foundation erosion and dam failure.

B.2.3. Hydropower. Although USACE projects do not include hydropower as the primary function, it may be included as a secondary function of flood control or navigation dams. Gates for projects with hydropower are similar to other dam gates. There are usually gates to control flows through the turbines, and bulkheads to allow turbine maintenance. One difference at dams with hydropower is that reservoir levels are usually kept as high as possible to maximize power generation. So while a spillway gate at a flood control dam (with the reservoir kept low) might not have any water against it much of the time, hydropower dams often have large heads against the spillway gates most of the time.

B.2.4. Local Flood Protection. Local flood control projects often consist of levees or floodwalls surrounding a populated area. Openings are provided through the levees or walls to permit road, railroad or pedestrian traffic without going over the levees. Gates are provided to close these openings to provide the occasional flood protection. These gates have a variety of hinged and rolling configurations suitable for the different height and width of the openings. These gates are seldom in use, but must operate reliably during floods.

B.2.5. Water Supply. Water supply is generally not the primary function of USACE projects, but may be included as a part of a mixed-use reservoir to include flood control and hydropower. Pool levels, which fluctuate based on need, must be balanced with other uses.

B.3. Gate Types.

B.3.1. Lock Gates. Several types of gates are used for locks. The most common is a miter gate. Miter gates consist of two leaves mounted on the lock walls. These gates are fairly simple in construction and operation and can be opened or closed more rapidly than other types of gates. Maintenance costs generally are low. A disadvantage of this gate is that it cannot be closed during an emergency situation with an appreciable flow through the chamber. When closed and retaining pool, they form a shallow three-hinged arch, with the arch pointing in the upstream direction. When open, they rotate into recesses in the lock walls. The arch action is an efficient way to span between lock walls, and miter gates require a relatively small force to operate. However, to perform safely, it is critical that the two leaves meet properly when closed. Figure B-1 shows a typical miter gate.

A navigation lock requires closure gates at both ends of the lock so that the water level in the lock chamber can be varied to coincide with the upper and lower approach channels. The sequence of "locking" a vessel upstream is: (1) lower the water level in the lock to the downstream water level, (2) open the lower gate and move the vessel into the lock chamber, (3) close the lower gate and fill the lock chamber to the level of the upper pool, and (4) open the upstream gate and move the vessel out of the lock. Lockage of a vessel downstream involves a similar sequence in reverse order. Lock

gates may also serve as guard gates, valves for filling and emptying the lock chamber, for passing ice and debris, to unwater the lock chamber, and to provide access from one lock wall to the other by means of walkways installed on top of the gates.

Two types of lift gates are used for locks, submersible and overhead. Submersible lift gates are sometimes used as the upstream gate. In the raised position, they resist pool. To pass navigation, they lower behind the upstream sill of the lock. These gates usually consist of two or three leaves that can move independently. Sometimes the upper leaf is lowered slightly below the water surface to allow accumulated floating ice or debris to pass through the lock. Figure B-2 shows a typical submersible lift gate.

An overhead lift gate is sometimes used as the downstream gate for high-head locks. The closed gate retains high pool levels, and the gate is raised to allow barges to pass beneath the opened gate. To provide the required clearance, the gate must be raised high, thus requiring large towers to house the machinery. Lift gates have a simple structural configuration, but require large machinery to provide the lifting force.

Sector gates are used for very low lift navigation gates, such as encountered in coastal areas. These gates consist of two leaves that join at the center of the lock and rotate into recesses in the lock wall when open. 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 B-3 shows a typical sector gate.

There are several other types of gates used for locks. Most locks require culverts to fill and empty the lock chamber. These culverts require valves to control flows. The valves are usually fabricated steel gates, with sizes from 12 to 16 ft vertically and horizontally. Other types of lock chamber gates are occasionally used, such as a submersible tainter gate. Appendix C discusses design of lock gates in detail.

B.3.2. Tainter Gates. The most common type of gate for a spillway crest is a tainter gate. The shape of a tainter gate is similar to a sector gate, but with the axis horizontal. The gate is raised off the spillway to permit controlled flows over the dam. The curved shape of the gate provides favorable hydraulic discharge characteristics. The radial configuration means that hydraulic forces cause no moment about the axis of the gate, and thus do not affect operating loads. This reduces the required hoist capacity and can simplify machinery design. Appendix D discusses design of tainter gates in detail. Figure B-4 shows a typical tainter gate.

B.3.3. Spillway Lift Gates. Lift gates are also common for use as spillway crest gates. These are much narrower than lift gates used as lock gates. They usually consist of a single leaf that is raised to permit controlled flows. Sometimes the leaf is split into an upper and lower section, and the upper section may be raised to permit flows between the sections.

B.3.4. Flood Closure Gates. A wide variety of gate types is used to close openings in levees and floodwalls. This is largely due to the variety of widths, heights, and slopes of the openings (roadways are often not flat). There are other considerations such as 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. Common closure gates include:

- Swing Gate;
- Miter Gate;
- Rolling Gate;
- Trolley Gate;
- Stoplogs.

Swing gates are mounted on hinges, like a door. They are simply swung open and closed. For wider openings, there can be dual swing gates, that close against a removable post located at the center of the opening. 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. Rolling gates have wheels so they can simply be rolled open and closed. This requires a track for the wheels across the opening. Trolley gates are similar to rolling gates, but the gate is suspended from an overhead track. This eliminates the tracks across the roadway, but requires overhead support structure. Stoplogs provide the simplest closure gate configuration. Beams are simply stacked into slots on each side of the opening. For wider openings, removable posts can be used for use with short stoplogs, but the posts must be adequately supported. Appendix F further describes various closure gates.

B.3.5. Bulkheads. Bulkheads are steel gates used to permit dewatering of sections of a project to permit maintenance. Similar to a lift gates, bulkheads often consist of a single fabricated steel unit that is lowered into slots on each side of an opening. For larger openings, the maintenance closure gate usually consists of shallow units that are stacked to the necessary height. This seems to match the description of stoplogs used for flood closures. However, usage of the terms bulkheads and stoplogs is inconsistent among various offices. Sometimes the terms are used interchangeably for the same configuration of stackable units. Alternatively, the term bulkhead might be used only for a full-height, single unit. On the other hand, stoplog might be used to refer to simple stacked beams, while bulkhead is used for stacked units consisting of more elaborate horizontal trusses. When using either term, be sure that your meaning is clear to your audience. In this manual, bulkhead is used as a general term referring to all types of maintenance closure units.

For narrower openings, such as some spillway crests or hydropower discharges, bulkheads might consist of horizontal girders, with a skinplate between girders. For a narrow opening, it is more likely that taller units will be used, even a single full-height bulkhead. For wider openings, such as locks, bulkheads usually consist of stacked units, each several feet high, consisting of two horizontal trusses with a skinplate between the trusses. Figure B-6 shows this configuration.

Other configurations can also be used to provide a damming surface to permit maintenance dewatering. For wider openings and shallower water depths, removable frames are installed at intervals across the opening and attached to the concrete base. To provide the damming surface, panels span between the frames or beams span between the frames, with panels or beams spanning vertically between the beams. One version of this type of configuration is referred to as a poiree dam (Figure

B-7). This configuration can be dangerous, however, because the frames are attached to the concrete using embedded steel attach points. These attachments are underwater and cannot be inspected adequately as FCMs.

Appendix G further discusses bulkhead configurations.

B.3.6. Other Gate Types.

The types of gates described above are the most common types used in USACE projects, and they are further discussed in Appendices C through G. However, this list is not comprehensive. There are many other types of gates.

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

Dam crest gates come in many shapes. Only tainter and lift gates are discussed in detail in this manual. 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 mounted on the dam pier. Wicket gates are used on some dams where the gates only need to be closed for parts of a season. Wickets fold down on the top of the dam when not in use. They are propped up into position when in use. Bear trap, clamshell, inflatable, and other types of gates are also used on some dams.

The main body of this manual contains basic gate design requirements that apply to all types of gates. Appendixes C through G provide geometry, details, and other information about specific types of gates. Some of the information in those Appendixes might also be useful for design of other types of gates. The best source for geometry and detailing information on less common gate types is usually the design and construction records from previous projects. However, such information should be modified to comply with current design requirements, such as good joint detailing practice to resist fatigue.

Figure B-1. Miter Gate.

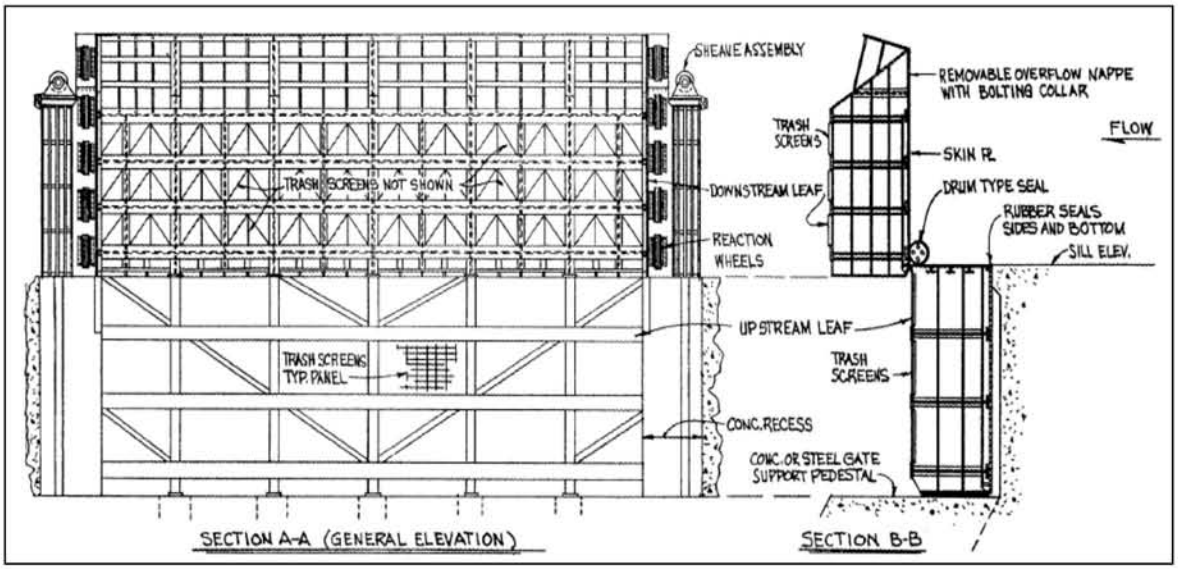



Figure B-2. Submersible Lift Gate.

Figure B-3. Sector Gate.

Figure B-4. Tainter Gate.

Figure B-5. One-Piece Bulkhead.

Figure B-6. Bulkhead.

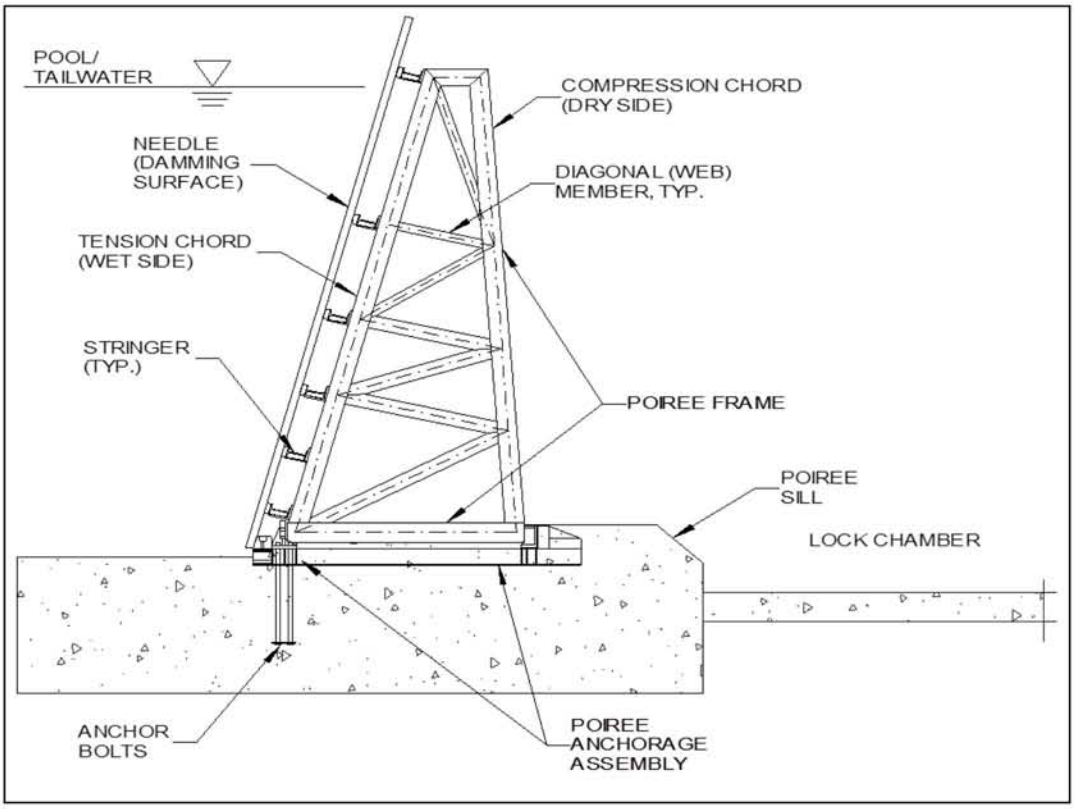



Figure B-7. Poiree Dam (Side View).

Figure B-8. Poiree Dam.

Figure B-9. Floating Bulkhead.

Figure B-10. Lifting Beam.

# APPENDIX C

## Miter Gates

C.1. Introduction. This appendix provides guidance for the structural design of miter gates used at navigation projects.

C.2. Miter Gate Configuration. A large percentage of the locks in the United States are equipped with double-leaf miter gates, which are used for moderate and high-lift locks. These gates are fairly simple in construction and operation and can be opened or closed more rapidly than other types of gates. Maintenance costs are generally low. A disadvantage of this gate is that it cannot be closed during an emergency situation with an appreciable flow through the chamber.

The two leaves of a miter gate form a shallow three-hinged arch angled upstream. In the closed position, under head, each leaf bears on a lock wall and on the other leaf at the center of the lock. This arch shape is very efficient for spanning larger distances between lock walls. Usually the angle of the leaf in plan, in the closed position is 1:3. Plate C-1 shows miter gate geometry for a horizontally framed gate.

Miter gates are framed either horizontally or vertically. The skin plate of a horizontally framed gate is supported by horizontal members that may be either straight girders acting as beams, or circular arches. Each horizontal member is supported by the vertical quoin post at the end near the lock wall and the miter post at the other end. All water load is transmitted through the girders to the quoin blocks and into the walls. 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. Vertically framed gates have been used primarily for long, shallow gates, usually when the height-to-width ratio of a leaf is less than about 0.5.

In the open position, miter gates 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 is recommended to help clear ice and debris from gate recesses.

C.3. Design Requirements. Chapter 3 and this appendix describe design requirements for gates, using Load and Resistance Factor Design (LRFD).

C.3.1. Loads. Chapter 3 describes loads for all gates. Loads that are applicable to miter gate design include self-weight, gravity loads, hydrostatic and hydrodynamic loads, operating loads, barge and other impact loads, ice loads, and earthquake loads:

- Self-weight. Dead load, D.

- Gravity loads, G. Gravity loads include mud weight (M), and ice weight (C), and shall be determined based on site-specific conditions.

- Hydrostatic loads. Hydrostatic load, $H_s$, consists of hydrostatic pressure on the gate considering both upper and lower pools.

- Hydrodynamic loads. Hydrodynamic, $H_d$, loads shall be equal to 30 psf (pounds per square foot) and shall be applied to produce maximum load effects.

- Operating loads. Operating load, Q, shall be the maximum load that can be exerted by the operating machinery (obtained from the mechanical Engineer that designed the machinery). See Section 3.2.3.3 for further discussion on operational loads.

- Barge impact. Barge impact load, IM, *shall* be specified as a point load (Figure C-1). The load shall be 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 at least 35 ft, or the standard barge width, from either lock wall. Both impact locations shall be investigated to determine the maximum structural effect. The magnitude of load shall be equal to 250 kips for unsymmetrical loading and 400 kips for symmetric loading.

- Earthquake design loads E. See Section 3.2.3.6 for earthquake loading.

- Diagonals. See attachment A.

C.3.2. Load cases. General loads and loading combinations for gates are described in Section 3.3 Miter gates shall be designed for the Strength, Extreme, and Fatigue Limit States for each of the following load cases and corresponding load combinations. Table C-1 lists load factors for miter gates. The Serviceability Limit State is addressed in Section 3.1.3.

- Case 1: Strength Limit State I, Gate Closed. Loads consist of unusual hydrostatic head differential and usual Hydrodynamic (temporal head) load as follows:

  - Upper gate subjected to unusual hydrostatic head with no tailwater (dewatered condition):

$$\gamma_{Hs2} H_{s2} \tag{C-1}$$

  - Lower gate subjected to unusual hydrostatic head differential with usual Hydrodynamic (temporal head) load:

$$\gamma_{Hs2} H_{s2} + \gamma_{Hd1} H_{d1} \tag{C-2}$$

- Case 2: Strength Limit State II, Gate Open, Table. Either gate subjected to dead, gravity, and hydrodynamic (temporal hydraulic load) or maximum machinery load, whichever produces the maximum effects:

$$\gamma_{D2} D_2 + \gamma_{G2} G_2 + (\gamma_{Hd1} H_{d1} \text{ or } \gamma_{Q2} Q_2) \tag{C-3}$$

- Case 3: Extreme Limit State, Gate Closed. Loads consist of unusual hydrostatic head differential and barge impact or earthquake, whichever produces the maximum effects:

$$\gamma_{Hs2}H_{s2} + (\gamma_{IM3}IM_3 \text{ or } \gamma_E E) \quad \text{(C-4)}$$

- Case 4a: Fatigue Limit State I. Stress range for member computed based on usual operating conditions to include emptying filling plus coincident temporal head:

$$\gamma_{Hs1}H_{s1} + \gamma_{Hd1}H_d \quad \text{(C-5)}$$

- Case 4b: Fatigue Limit State II. Stress range for member computed based on unusual operating conditions (hydrostatic head differential) to include emptying filling plus coincident temporal head:

$$\gamma_{Hs1}H_{s2} + \gamma_{Hd1}H_d \quad \text{(C-6)}$$

Table C-1. Load Factors for Miter Gates.

| **Load Cases** | | | **Loads/Load Factors** | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | **D** | **G** | **$H_s$** | **$H_d$** | **Q** | **EV** | **IM** | **EQ** |
| Limit State | Description | Case | $\gamma_D$ | $\gamma_G$ | $\gamma_{Hs}$ | $\gamma_{Hd}$ | $\gamma_Q$ | $\gamma_{EV}$ | $\gamma_{IM}$ | $\gamma_{EQ}$ |
| Strength I | Gate Closed | 1a | 0 | 0 | 1.4 | 0 | 0 | 0 | 0 | 0 |
| Strength I | Gate Closed | 1b | 0 | 0 | 1.4 | 1.0 | 0 | 0 | 0 | 0 |
| Strength II | Gate Open | 2 | 1.2 | 1.6 | 0 | 1.0(1) | 1.2(1) | 0 | 0 | 0 |
| Extreme I | Gate Closed | 3 | 0 | 0 | 1.4 | 0 | 0 | 0 | 1.0(1) | 1.0(1) |
| Fatigue I | Finite Life | 4a | 0 | 0 | 1.0 | 1.0 | 0 | 0 | 0 | 0 |
| Fatigue II | Infinite Life | 4b | 0 | 0 | 1.0 | 1.0 | 0 | 0 | 0 | 0 |
| Notes: (1) Select one at a time | | | | | | | | | | |

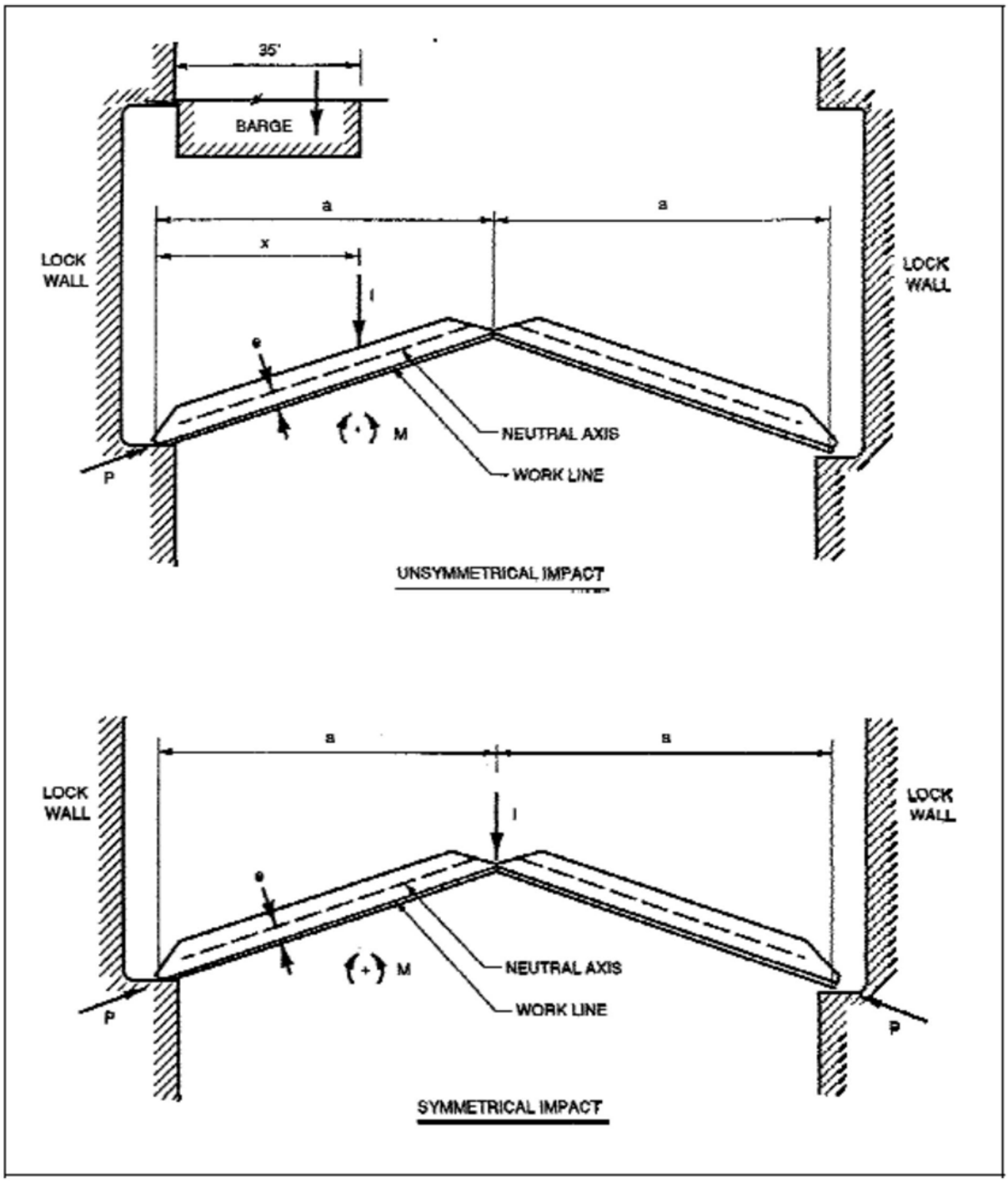



Figure C-1. Point load impact for miter gate girders.

C.4. Miter Gate Types.

C.4.1. Horizontally Framed Miter Gates.

C.4.1.1. Girder geometry. The primary structural elements of a single gate leaf consist of a series of horizontal girders (Plate C-1). The horizontal girders are a series of three-hinged arches that transmit the water pressures to the lock walls through the quoins. Figure C-2 and Plate C-3 show a horizontal girder (half of a three-hinged arch) acted on by water pressure, and illustrate a way to calculate end reactions. The following symbols are used in these figures:

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 water force on the end of each girder, determined from the water pressure on the surface extending from the contact point to the upstream side of the skin plate

W = total water force on each girder.

The three-hinged arch formed by the two leaves is symmetrical about the center line of the lock, therefore, 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.

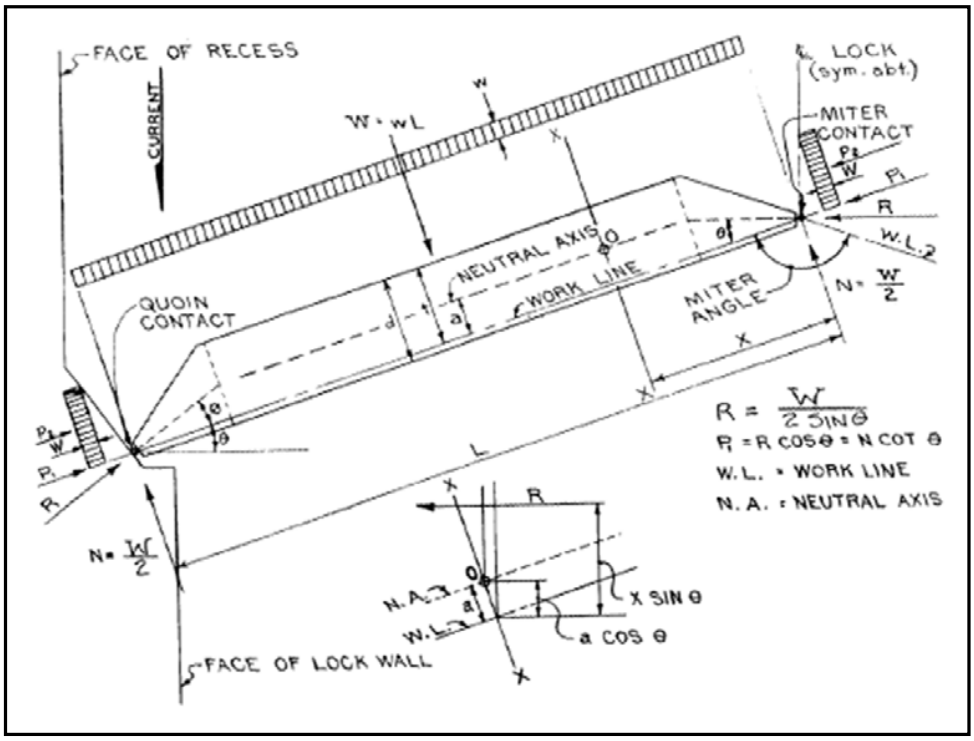



Figure C-2. Girder Data, Horizontally Framed Miter Gate.

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

Efficient girder design can be achieved through trial and error. Initial approximate dimensions may be taken as follows:

- The gate angle can be 1:3, thus $\theta$ = arc tan 1/3.

- The length of the leaf is then 0.527 times the distance between contact points of the gate at each wall.

- For the girder depth for gates of moderate height, a first guess can be 0.07 times L.

- The distance from the downstream girder flange face to the work line may be set at 4 in.

C.4.1.2. Loads and Reactions. Required loads and load combinations are defined in the main body of this manual. Prestressing loads in the gate diagonals should be treated as an external applied load. Reactions at the hinge and pintle are only those due to gravity and machinery loads. Reactions to water and impact loads are assumed to be entirely at the pintle and quoin blocks (Plate C-8).

C.4.1.3. Skin plate. The skin plate is located on the upstream face of the girders and is designed for the water load, with the edges of panels assumed fixed at the center line of intercostals and the edge of girder flanges, except that where the 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. Plate-bending stresses can be obtained from textbooks, e.g., Roark and Young. The skin plate is also considered an effective part of the upstream girder flange. When a section has a skin plate of a higher yield than the rest of the girder, the effective width of skin plate shall be determined by the higher yield point. Skinplate stresses should be checked for the combined effects of plate bending, plus stresses due to bending as part of the girder flange.

C.4.1.4. Intercostals. Intercostals are designed as vertical fixed-end beams supported at the center lines of girder webs. An effective section of skin plate is assumed as acting with the intercostal. The effective width shall be based on a stiffened, compact or noncompact section. A uniform water pressure is used for design of the intercostal, with the loading extending between the flanges of the girders.

C.4.1.5. Diaphragms. The end diaphragms are designed as panels acting as skin plate, with the effective panel being between the stiffener angle and the next lower girder. The stiffener is located at midpoint between girders. The head at the center of the effective panel is used as the design pressure. Intermediate diaphragms should be spaced and sized as follows:

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

- For shear forces resulting from the diaphragms tending to equalize differential deflections between adjacent horizontal girders due to variation of hydrostatic and impact loads;

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

Generally the end diaphragms are a minimum of ½ in. for all sizes of gates. Intermediate diaphragms are usually a minimum of ⅜ in, or ½ in for larger gates. Diaphragms are made as deep as the girder webs, and usually have vertical stiffeners similar to longitudinal stiffeners on the girder webs. 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).

C.4.1.6. Horizontal Girders. Horizontal girders are subject to bending plus axial compression. They are spaced so that variation in the girder flange sizes and skin plate thicknesses are held to a minimum. 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. Girder loading is primarily the differential water load on the gate. Boat impact loads usually govern the uniformly spaced upper girders.

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 should have horizontal stiffeners to meet the criteria for web buckling for axial loaded columns, using diaphragm spacing as the effective column length. Minimum horizontal stiffeners are generally used on girder webs even though not required by web buckling (Plates C-3 and C-5).

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 downstream flange should be checked where that flange is in compression near the end diaphragms.

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 transitions should be configured as a good fatigue category details (Plate C-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 shall include provisions for this additional eccentric axial load. A discussion of the distribution of this load among the girders may be found in Technical Report ITL-87-4, Report 7 (USAEWES 1987).

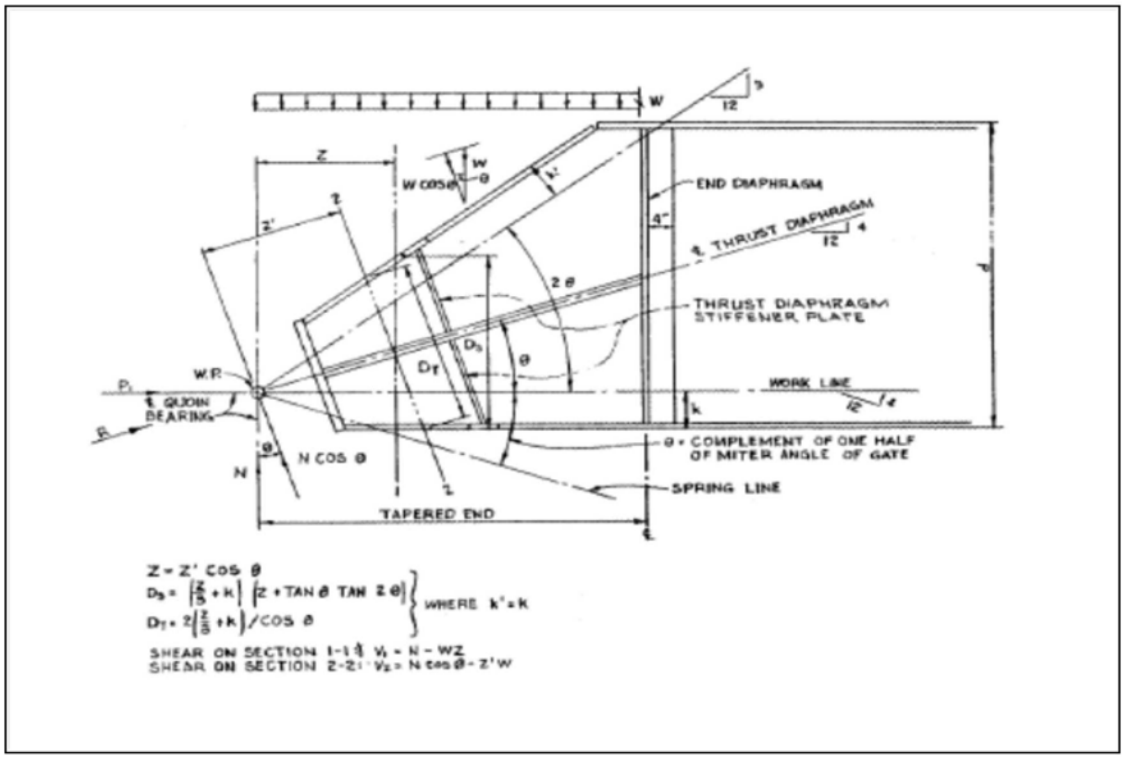



Figure C-3. Girder Tapered End Section.

C.4.1.7. The tapered end sections are used to transfer assumed linear bearing loads along the quoin and miter bearing blocks to discrete loads on individual 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. Finite element analysis can be used to provide a more thorough understanding of actual stress distribution. The minimum bottom girder web thickness over the pintle is usually ¾ in. and is machined to a 250 finish or to match the machine finish to the top of the pintle socket casting (Figure C-3). The top and bottom webs are wider at the quoin end to accommodate the gudgeon pin and pintle. Plates C-3, C-4, and C-5 present additional information on girders. Drain holes shall be provided in all girder webs. 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.

C.4.1.8. Thrust Diaphragm. The thrust diaphragm distributes the reaction of the girders from the quoin block into the girder webs. It also acts as the damming surface between the end plate and the end diaphragm. Part of the thrust diaphragm is also considered effective in the quoin post, making it subject to horizontal and vertical compression and plate-bending stresses. It is subject to shear stresses as it transfers gate reactions from bearing blocks to girder webs. Analysis of this complex behavior should be based on conservative assumptions and liberal use of stiffeners, or on finite element analyses as mentioned above for the girder tapered end sections.

C.4.1.9. Quoin Post. A section of the thrust diaphragms, vertically from top to bottom girders, forms a column to support the dead weight of the leaf. The end plate and two vertical stiffeners form one flange of the column. A plate perpendicular to the thrust diaphragm with vertical stiffeners

on the outside edges forms the other flange. Plate C-5 shows a typical layout of the quoin post. The axial load on the quoin post consists of the dead weight of the leaf plus ice and mud load. Due to the eccentricity of the pintle and gudgeon pin with respect to the centroid of the quoin post, the quoin post is also subjected to bending stresses, plus the skin plate action of the thrust plate. 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.

C.4.1.10. Gudgeon Pin Hood. The gudgeon pin hood is an arrangement of plates forming the hinge connection at the top of the miter gate leaf (Plate C-6). The recommended distance between the center lines of the top web and the top pin plate is 1 ft, 6¾ in. This is with a 1-in. top pin plate and 1¼ in. pin plate welded to the top girder web. The top pin plate has sections of it sloping from the top height down to the girder web. The downstream edge of the top pin plate is attached to the bulkhead plate. The upstream part of the hood is formed by a vertical plate, normally ¾-in. minimum thickness, that overlaps the upstream girder flange, with the edge of the vertical hood plate being welded on the center line of the horizontal girder web. Steel rings varying in thickness from $^{1}/_{16}$ in. to ¼ in. are used to adjust the vertical clearance between the gudgeon pin barrel and the pin hood.

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 C-7 illustrates formulas from Seely and Smith (1952). 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 made of forged alloy steel, normalized and tempered. The bushing is normally of bronze with the bearing pressure kept below 1,500 psi.

C.4.1.11. Gudgeon Anchorage. The upper anchorage system supporting the miter gate leaves is comprised of: gudgeon pin barrel, anchorage links, embedded anchorage. 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. To develop maximum operating strut forces, the leaf is assumed obstructed near its miter end. Plates C-11 and C-15 show the layout of typical anchorage systems.

- Gudgeon pin barrel. The gudgeon pin barrel, composed of welded carbon steel plates or forged alloy steel plates, is designed as a continuous beam supported by vertical stiffeners, and at the same time as a curved beam, which is 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 C-9 shows a typical barrel arrangement and formulas. The alternate method of analysis (shown in Plate C-10) may be used in lieu of the more precise method beginning in Plate C-9. Since the barrel is a critical item, the design stresses should be kept low, in the range of approximately 0.33 $F_y$.

- Anchorage links. The pinned end links connect to the embedded anchorage with a threaded 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 C-11). Plate C-15 shows an alternate top anchorage. This assembly is made up of two anchor arms and two gudgeon links. The links are welded to the arm, which is normal to the face of the lock wall. Adjustment of this anchorage assembly is accomplished by means of wedges. The links acting as a unit are assumed to have a maximum misalignment of 2½ in. at point B, shown in Plate C-11. This introduces a bending stress in conjunction with the axial load. Allowable stresses are usually governed by the

fatigue design requirements. The threaded section of each link, made up of a forged steel section with a minimum diameter of 6 in., and a hexagonal sleeve nut are used for adjustment of the gate leaf. Right- and left-hand threads, giving a turnbuckle effect, are recommended, with ½-in. square threads being used for the sleeve nuts. After all adjustments to the gate leaf have been made, a channel may be welded between the sleeve nuts to lock them in place. 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, a minimum of 6 by 4 in., is also made of forged steel. The pin-connected ends of the rectangular sections are designed as eye bars. Pins should be designed for both bending and bearing. The dimensions and sizes shown in Plate C-11 are recommended as a minimum.

- Embedded anchorage. The embedded anchorage distributes the top reaction of the leaf into the lock wall. It is designed as a triangular frame. The vertical and horizontal sides of the triangle are normally 9 ft. The horizontal member is for fabrication and construction and is assumed to carry no design load. Plates C-12 and C-15 show a typical layout of embedded anchorage. The reactions of the triangular unit are applied to the concrete through plates or pads on the lower points of the triangle. Bolts are used in conjunction with the bearing plates, with the bolts prestressed so that bearing on the concrete will never be completely relieved by the loads from the gate leaf. The prestressed bolts should have an anchor at the ends to carry the full load, assuming that there is no load transfer through bond and that mastic is used to prevent bond on the bolts. Bolt length should be sufficient to extend into at least two lifts of concrete. The use of strain gages or an ultrasonic bolt stress monitor is recommended for determining the loads in the prestressed bolts, as the nuts sometimes bind on one edge and thereby distort torque readings and make the turn of the nut method unreliable.

C.4.1.12. Pintle Assembly. The pintle and related components support the dead weight of each leaf of the miter gate. The unit is made up of four major components: pintle socket, pintle, pintle shoe, and pintle base (Plates C-13 and C-14). The center line of the pintle (vertical axis of rotation) is located eccentric (upstream) relative to the center of curvature of the bearing face of the quoin contact block. This center of curvature is on the thrust line. The center line of the pintle should be located on the point of intersection of the bisector of the angle formed by the mitered and recessed gate leaf work lines and the perpendicular line from the bisector to the quoin contact point resulting in an offset of approximately 7 in. as in the details shown in Plate C-4. Studies and experience show that these eccentricities will minimize interference and binding between the bearing blocks.

- Pintle Socket. The pintle socket is made of cast steel and is connected to the bottom of the lower girder web with turned Monel or stainless steel bolts. The bolts are sized to carry the gate leaf reaction in shear, but, as an added safety factor, a thrust plate should be welded to the underside of the bottom girder web, with a milled contact surface between the plate and pintle socket. The minimum plate size should be 1¼-in. thick and 12 in. wide, with a length as required by the girder web. The socket encloses the bronze bushing, which fits over the pintle ball. A maximum bearing stress of 1,500 psi is desirable, but may not always be practical. An automatic greasing system allows a higher bearing stress, but it should not exceed 2,500 psi. Plate C-13 presents additional information.

- Pintle. The pintle is generally made of cast alloy steel with a nickel content of 3 to 5%. It usually has a diameter of 10 to 20 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 should be of forged alloy steel

with bearing surfaces of corrosion-resisting steel deposited in weld passes to a thickness of not less than ⅛ in. and machined to the required shape. The pintle ball and bushing are finished to a 16-microinch (μin) finish where the two come in contact.

- Pintle Shoe. There are two types of pintle assemblies for horizontally framed miter gates: fixed and floating.

  ○ Fixed pintle. Plate C-16 shows a typical fixed pintle. This type of pintle is recommended for new construction and major gate rehabilitation. 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.

  ○ Floating pintle. Plate C-13 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 in the shoe. 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 preventing 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. However, 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 attach 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.

- 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.

C.4.1.13. Operating Strut Connection.

- Hood. Plate C-27 shows typical details for the hood-type connection. This connection is attached above 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. The fabrication cost for the hood type will generally be higher than for the other types of connections.

- 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.

- 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 C-27) 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 for the hood or vertical-shaft connections, the strut would have to be reduced in length, thereby creating potential interference between parts of the strut.

C.4.1.14. Diagonals. Each miter gate leaf is similar to a horizontal cantilever beam. Resistance to vertical loads is provided by the skinplate, 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. 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 C-sections are very flexible torsionally, diagonals are added to the back side of the leaf to increase the torsional stiffness and the diagonals are prestressed to adjust the plumbness of the miter post. Plate C-17 (a and b) shows an example diagonal configuration.

The diagonals may be pin connected or welded to the gusset plates. Turnbuckles or brackets on the end of the diagonals are used to prestress the diagonals. An advantage of the brackets is that no compression can be placed in the diagonal during prestressing. Strain gages should be used for determining the stress in each diagonal. The maximum stress, for temporary conditions, should not exceed 0.75 $F_y$.

Information for the design of diagonals is provided in *Torsional Deflection of Miter-Type Lock Gates and Design of the Diagonals* (USAED 1960) and in the Attachment to this appendix. 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.

C.4.1.15. Miter and Wall Quoins. Plate C-18 shows typical quoin and miter block details.

- Miter blocks. Miter blocks form a bearing and sealing surface between the miter ends of the leaves. Jacking and holding bolts are used for temporary supports and adjustment of the miter blocks to assure full contact between leaves in the mitered position. Miter blocks are usually 8 x 5½-in. rectangular blocks with one miter block having a concave surface with a radius of 1 ft, 6 in. and the other having a convex surface with a radius of 1 ft, 4½ in. These blocks are made up of 15 to 20-ft-long sections with transverse joints occurring at the center lines of horizontal girder webs.

- 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 with a 1 ft, 6-in. radius and the quoin block on each leaf having a convex surface with a radius of 1-ft, 4½-in. There are two recommended types of wall quoin systems. The first system, an adjustable type, consists of a 10 x 3½-in. bar, welded to a 1¼ x 1 ft, 5-in. base plate. 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 system, a fixed type, consists of a 10 x 3½-in. bar, welded to a vertical beam as described previously.

- Quoin materials. Adjustable and replaceable corrosion-resisting clad steel or solid corrosion-resisting steel blocks are recommended for both miter and wall quoins. When carbon steel quoin and miter blocks are installed, they are bolted to the gate with zinc or epoxy filler behind them. The blocks and the zinc can be protected with cathodic protection. As a minimum, the sides of the blocks can and should be painted.

- Backing material. After final adjustments have been made to the miter and quoin blocks, a gap of about ½ 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. In the past, epoxy was easier and safer to work with, but new types of equipment for heating zinc and preheating the ends of the gate leaves have greatly reduced many objections to its use. The initial investment in the equipment needed in using zinc is expensive and the placement may be slightly more expensive, but with the life expectancy of zinc being 2 to 4 times that of epoxy, the use of zinc will be less expensive during the life of a project. Precautions should be taken to prevent leakage of either filler, and to prevent air entrapment. 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.

C.4.1.16. Seals. Rubber seals should be installed on the bottom of each leaf to seal the gate to the miter sill, as shown in Plate C-20. 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. Seals might also be required above ends of the quoin and miter blocks if a water tight surface is required above that point.

C.4.1.17. 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 C-19 shows typical details.

C.4.1.18. Walkway. Miter gates should be equipped with a walkway across the top of the gate. It should have a width of 4 ft, with the top of the walkway flush with the top of the lock wall. Instead of a walkway, a maintenance bridgeway may be provided over (and supported by) the lower miter gates to accommodate a mobile crane, thereby eliminating the frequent need for a floating plant for many maintenance and repair operations.

C.4.1.19. 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 an impact loading. Generally, this extends from a point at or slightly below the 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.

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.

C.4.1.20. Gate Latches. Latches should be provided to hold each leaf in the recess against temporal hydraulic loads and in case of an emergency. A single latch at the top of the leaf is normally sufficient. Where the lock is used as a floodway during high flows, additional latches may be required. Latches should be so constructed that the leaf is held snug against the bumpers so the potential vibration is kept to a minimum. A latch or tie should also be provided to tie the leaves in the miter position, again with the ability to pull the leaves together so as to reduce the probability of vibration. Plates C-21, C-22, and C-23 show a variety of latching devices.

C.4.2. Arch Type Miter Gates. A horizontally framed gate can use arches instead of girders. For high-head gates, the arch should be more efficient by minimizing bending moments. Other than the arches, the basic components are the same as for a gate with girders. Plate C-24 shows a typical horizontal arch layout.

C.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 have been used primarily for long, shallow gates, usually when the height-to-width ratio of a leaf is less than about 0.5. Plate C-2 shows an example of a vertically framed miter gate.

C.4.3.1. Skinplate 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 skinplate is generally on the downstream side of the vertical beams, to minimize uplift forces on the gate. However, that position also maximizes opportunity for silt to accumulate above the bottom girder.

C.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 make the bottom girder flanges more directly effective in taking the components of the diagonals.

C.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.

C.4.3.4. Top horizontal girder. The top horizontal girder is designed 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.

C.4.3.5. Bottom horizontal girder. 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. 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 skinplate.

C.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, turnbuckles are recommended on all diagonals to allow for easier adjustments.

C.4.3.7. Miter and Wall Quoins. 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 2 ft, 0 in. high and 1 ft, 8 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. The beam is generally placed horizontally in first-pour concrete with the bearing being detachable, with provisions for adjustment.

C.4.3.8. Seals. Plates C-25 and C-26 show some seal details for vertically framed gates. J-seals are used along the vertical edges of the leaf. At the miter end, below the web of the bottom girder, two rectangular rubber blocks, one on each leaf, run from the vertical J-seal to the sill. 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 solid rubber block seal attached to the leaf is used between the leaf and pintle base.

C.4.3.9. Other Components. Other components such as pintles, strut connections, walkways, fenders and latches, are similar to those used for horizontally framed gates.

C.5. Erection and Testing. Miter gates should be completely shop assembled, if size permits, with adjoining pieces fitted together to ensure satisfactory field connections. The tolerances should not exceed $^{1}/_{16}$ in. for individual members up to 30 ft long and not more than 1/8 in. for members over 30 ft long. 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.

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.

After diagonals have been prestressed and final adjustments have been made to the anchorage, the leaves are mitered and securely held in this position while the contact blocks at the quoin and miter ends are brought into firm contact by adjusting the bolts behind the blocks. 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 shall 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 shall 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.

After completion of the gate, including prestressing of the diagonals, installation of all seals, and all adjustment, the gate leaves should be swung through a sufficient number of opening and closing operations to assure that 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.

The miter guide should be installed after the trial operation and second-pour concreting has been completed. The guide bracket and roller bracket assemblies should be mounted on their respective leaves with the gate in the mitered position. Adjustments should be made to the brackets so that either leaf may be mitered or opened without disturbing the other leaf.

The final test on the gate 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.

C.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 that the machinery can exert on the gate.

C.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 this 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.

Output pressure must be high enough to overcome hydrostatic pressure at the submergence depth, frictional losses in the supply and distribution lines, and still to 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. 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.

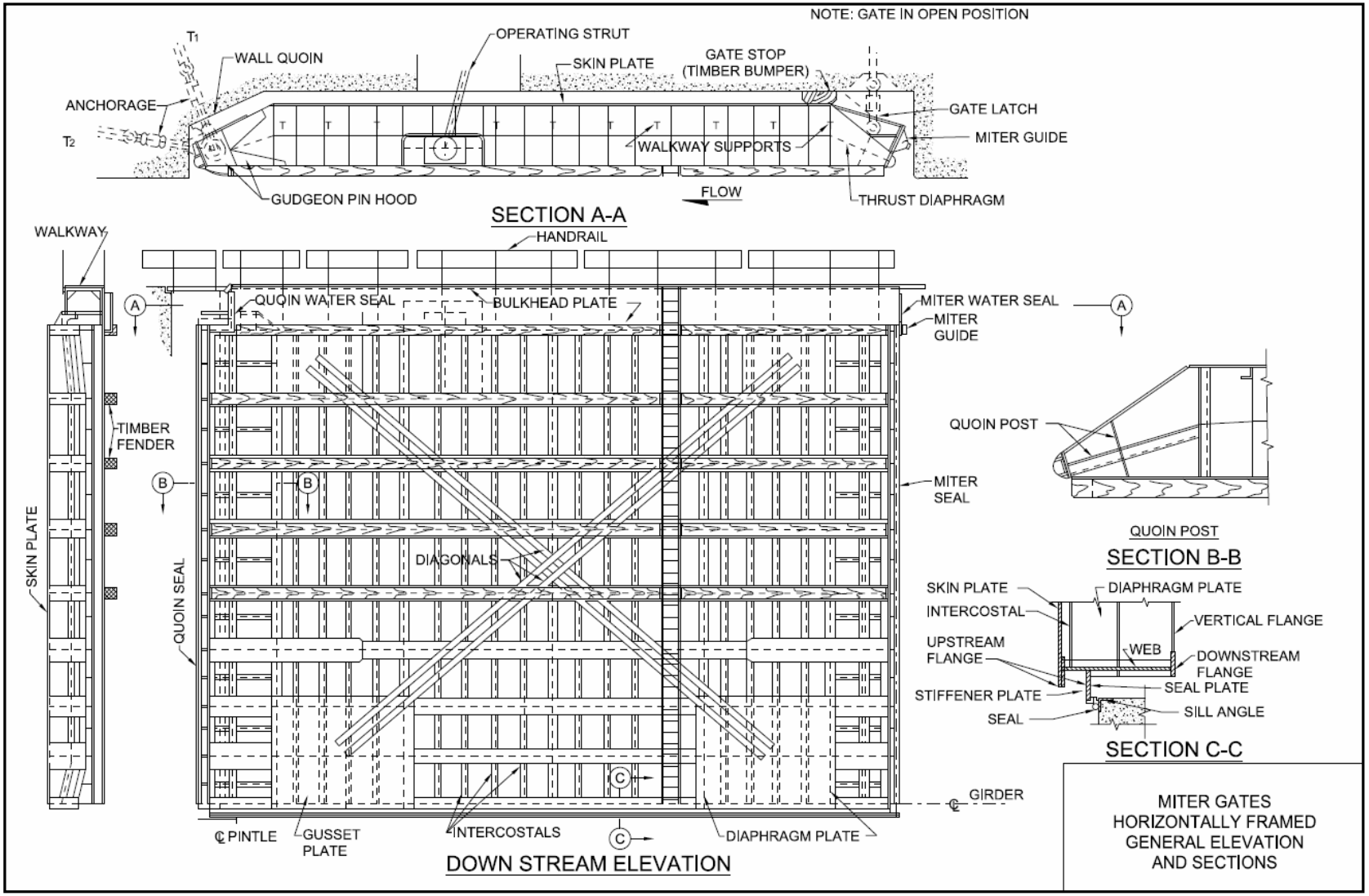



Plate C-1. General Elevation and Sections, Horizontally Framed Miter Gates.

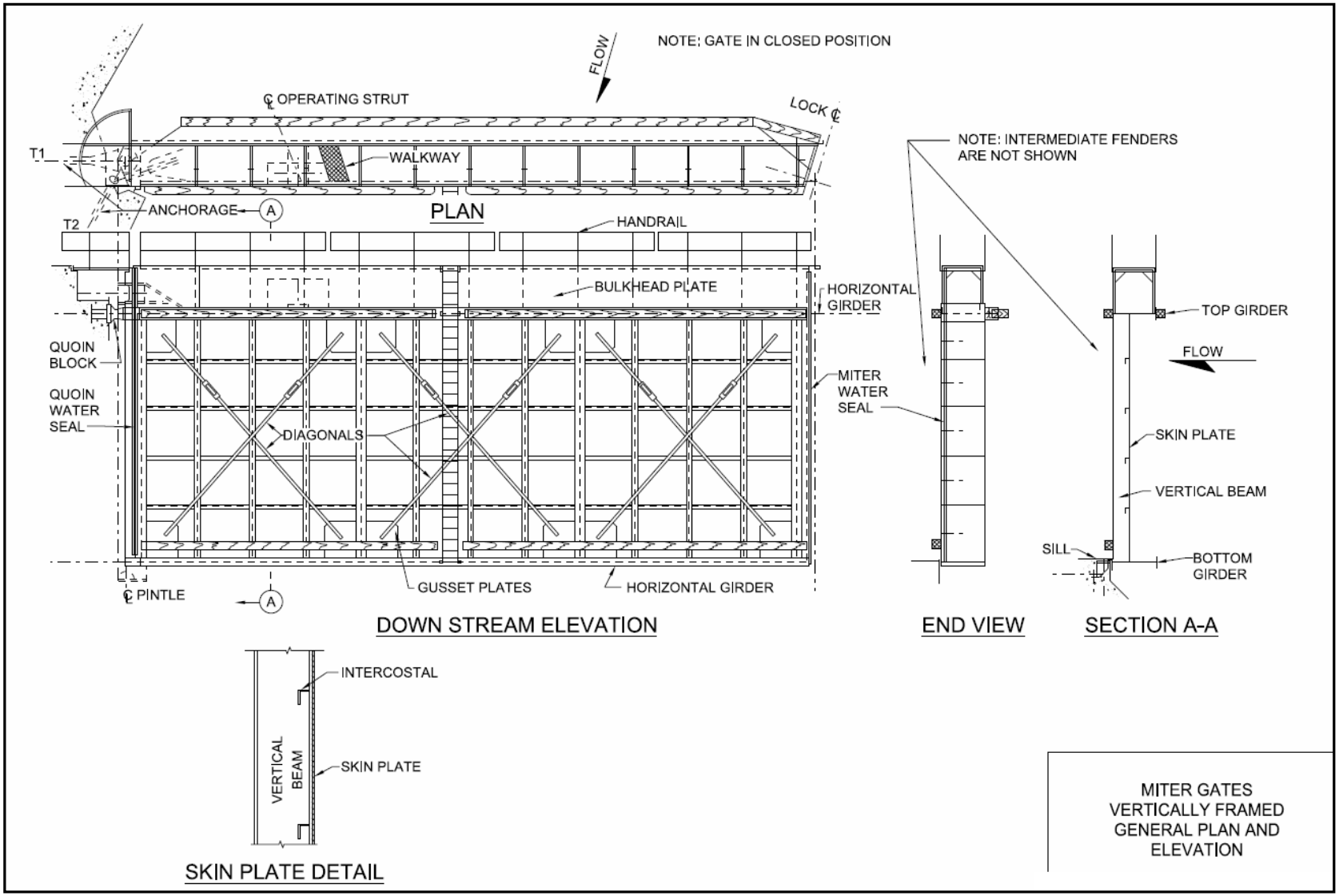



Plate C-2. General Plan and Elevation, Vertically Framed Miter Gates.

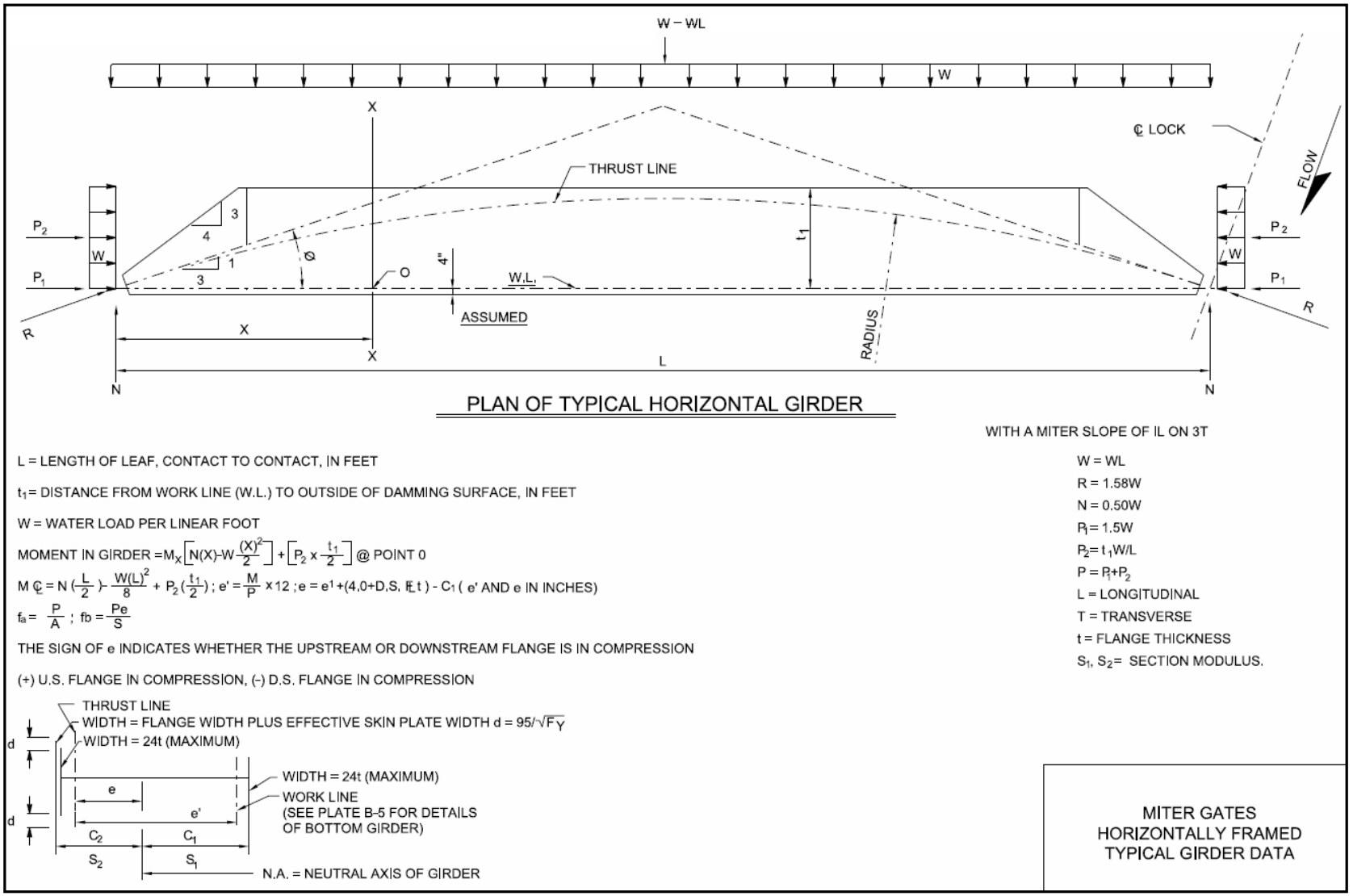



Plate C-3. Typical Girder Data, Horizontally Framed Miter Gates.

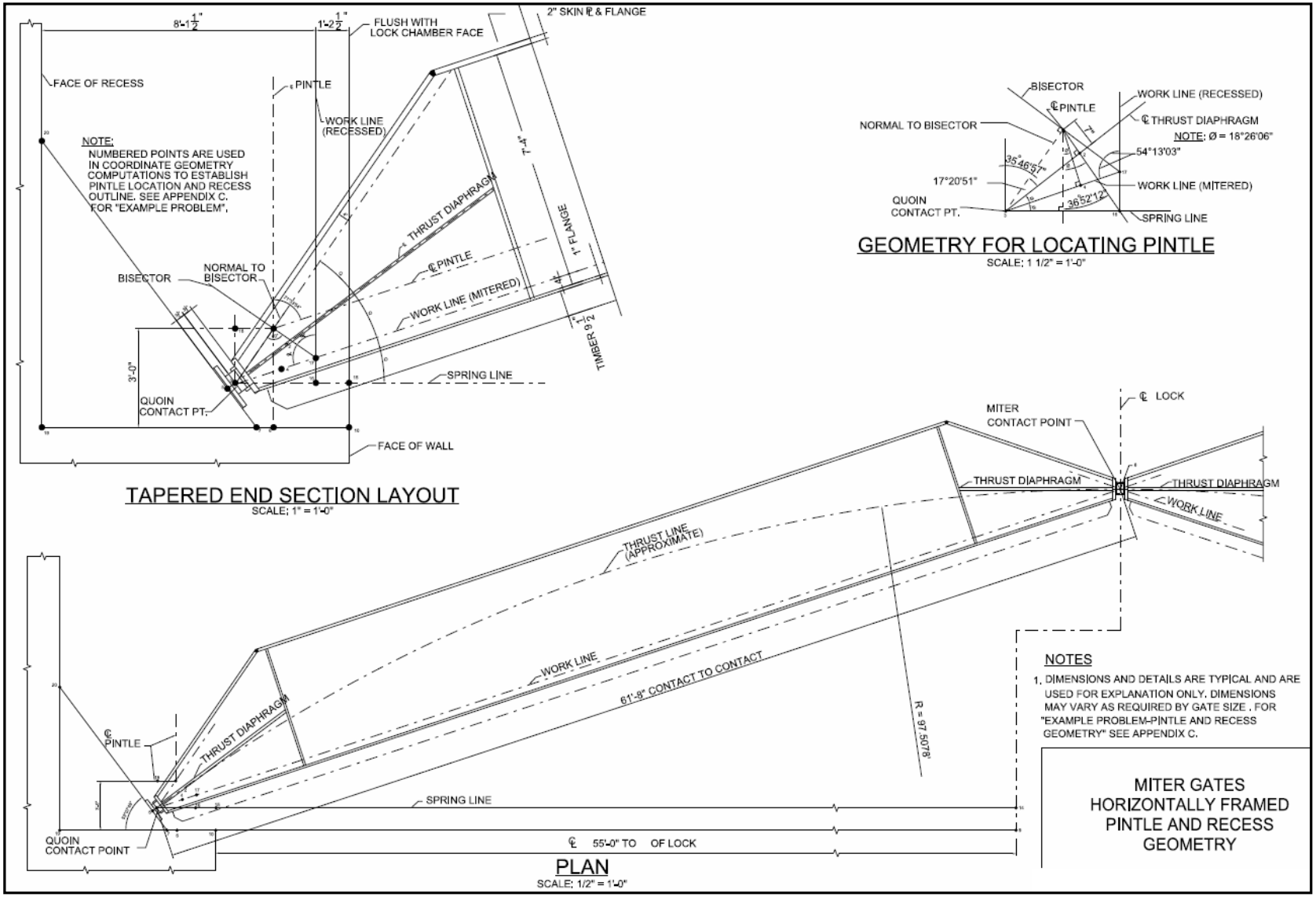



Plate C-4. Pintle and Recess Geometry, Horizontally Framed Miter Gates.

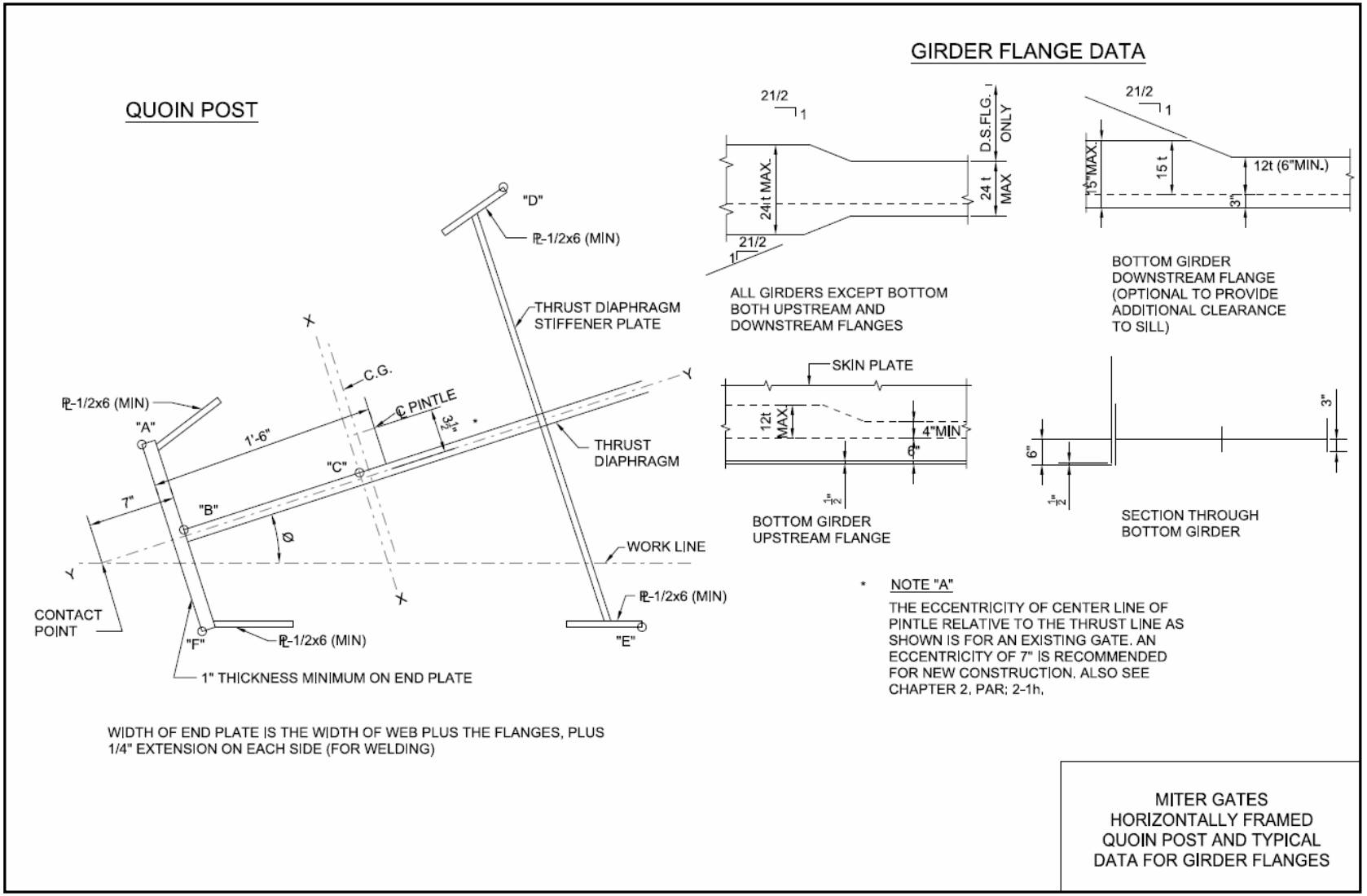



Plate C-5. Quoin Post, Horizontally Framed Miter Gates.

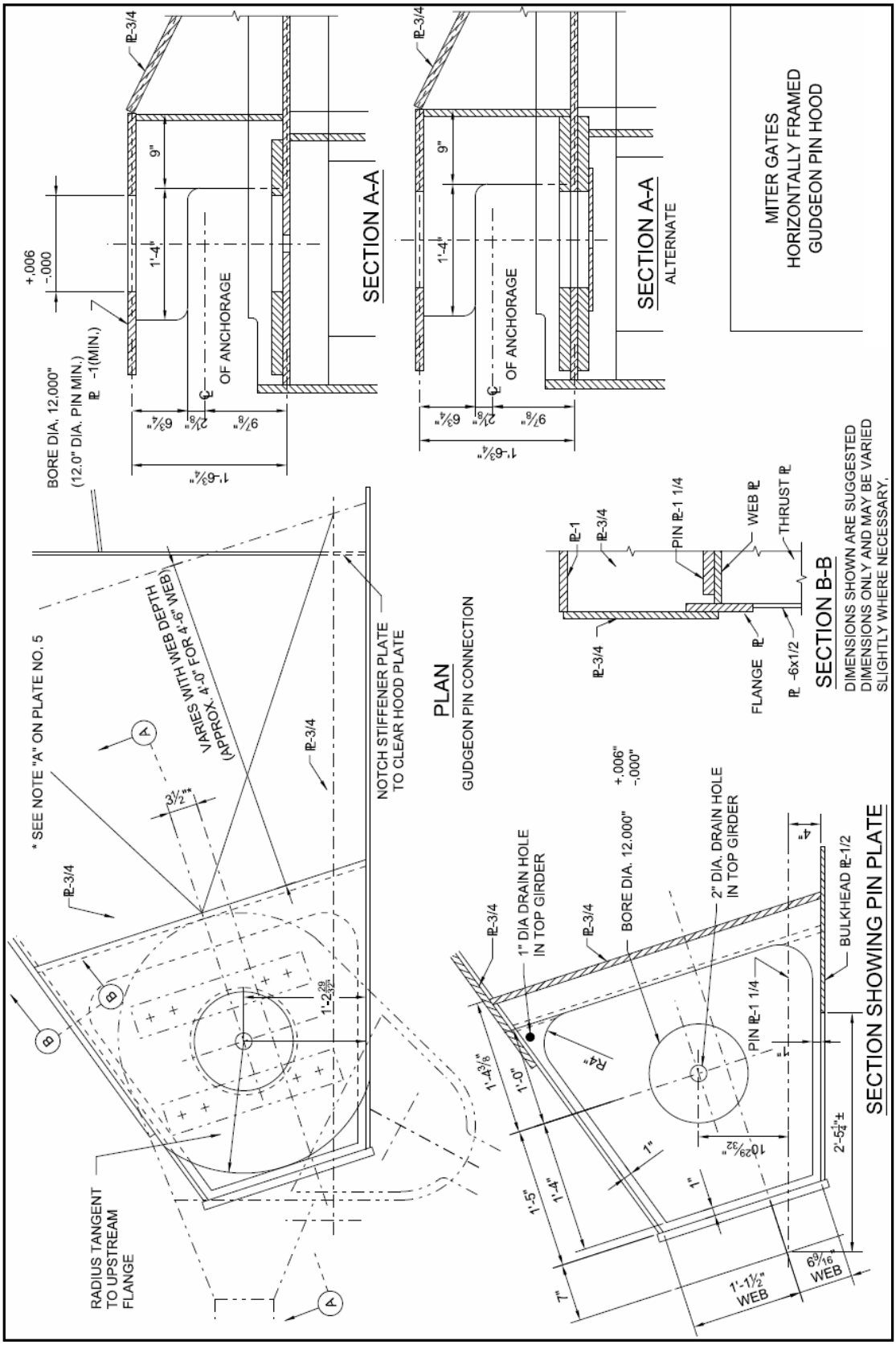



Plate C-6. Gudgeon Pin Hood, Horizontally Framed Miter Gates.

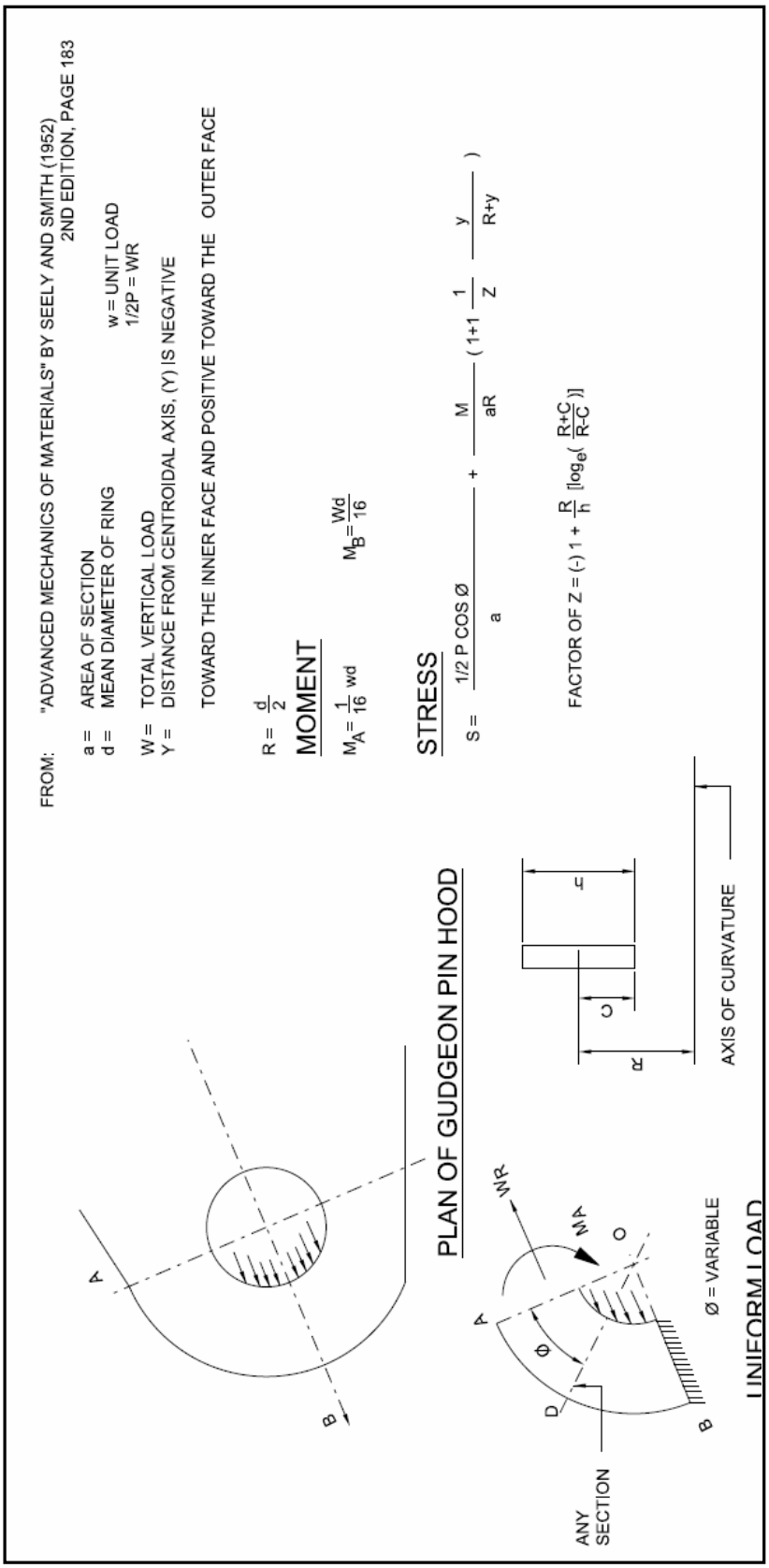



Plate C-7. Gudgeon Pin Hood Design Data, Horizontally and Vertically Framed Miter Gates.

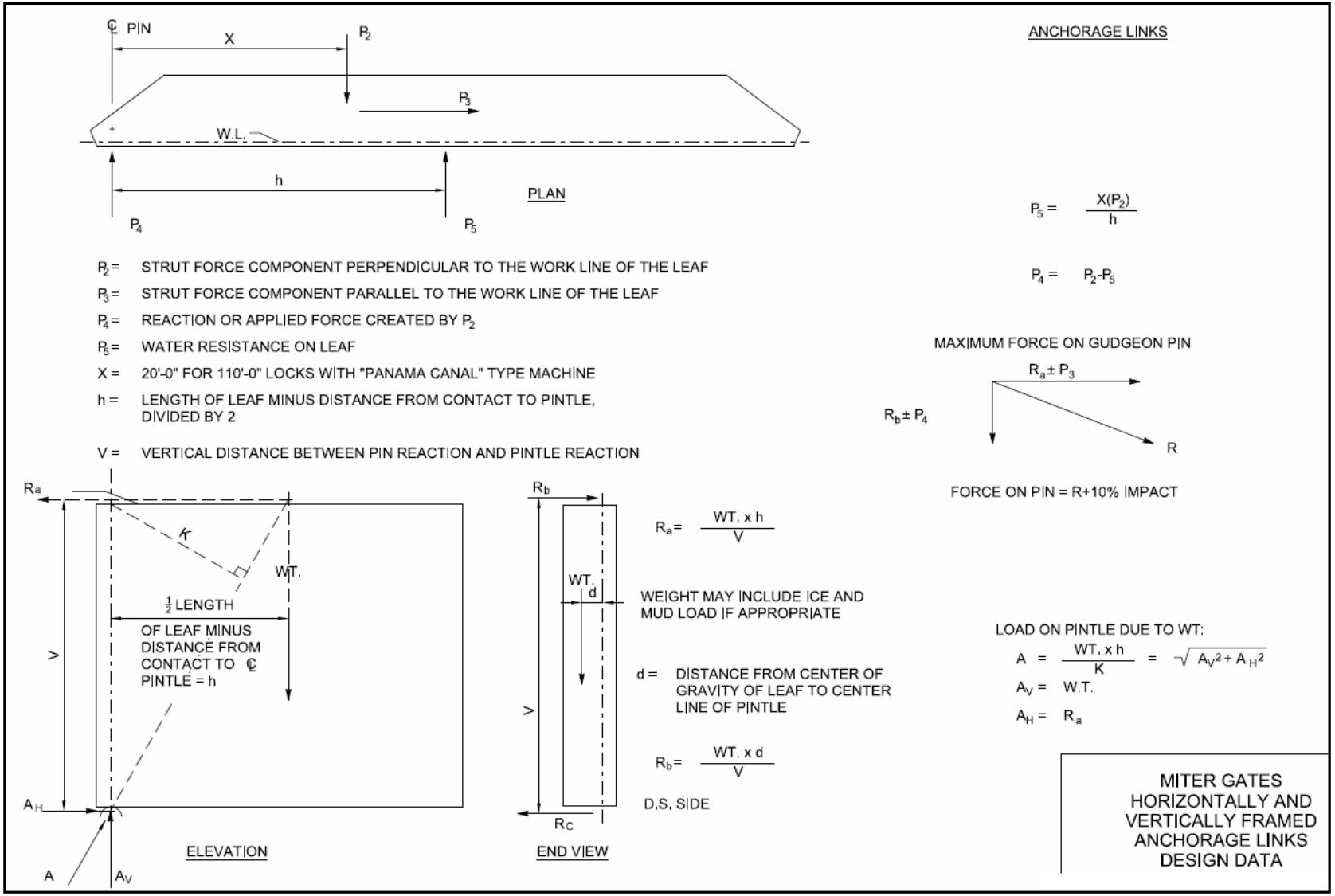



Plate C-8. Anchorage Links Design Data, Horizontally and Vertically Framed Miter Gates.

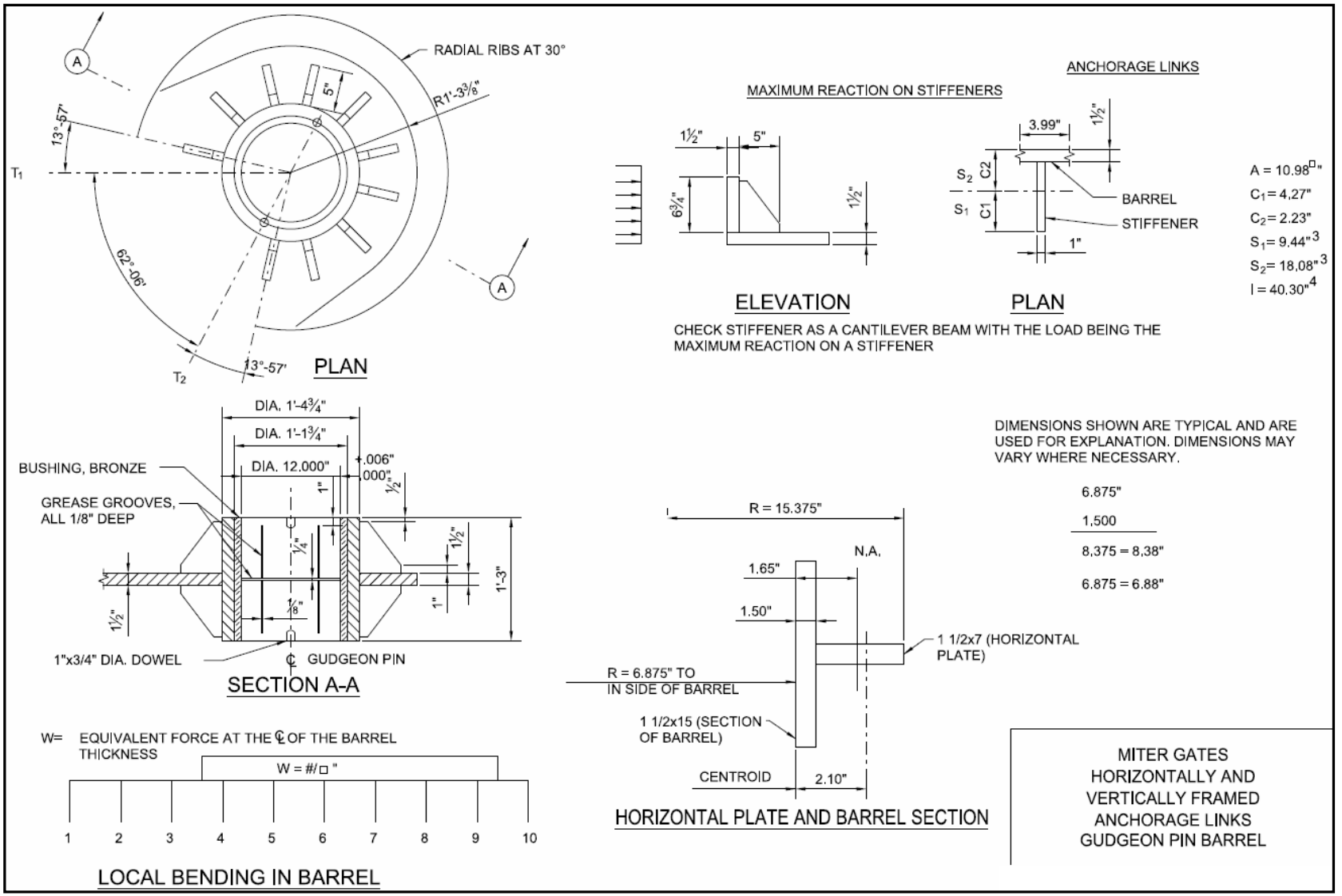



Plate C-9. Anchorage Links Gudgeon Pin Barrel, Horizontally and Vertically Framed Miter Gates.

# ANCHORAGE LINKS

## ALTERNATE METHOD OF PIN BARREL ANALYSIS

$r_0$ = RADIUS TO NEUTRAL AXIS

$$r_0 = \frac{A}{\int \frac{dA}{r}} = \frac{(7x1\ 1/2)+(15x1\ 1/2)}{15\int_{6,88}^{8.38}\frac{dr}{r} + 1.5\int_{8,38}^{15.38}\frac{dr}{r}}$$

$$r_0 = \frac{33,0}{(15 x \text{LOG}_e\ \frac{8.38}{6.88}) + (1.5\ \text{LOG}_e\ \frac{15.38}{8.38})} = 8,53"$$

Y = DISTANCE FROM NEUTRAL AXIS TO THE POINT OF STRESS

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

$$S = \frac{\text{WIDTH OF BARREL}\ R[15\int_{6.88}^{8.38} rdr - 127.95 \quad \text{WIDTH} x r_0\ M_Y\ 8.38\int_{6.88} dr + 1.5 \quad \text{WIDTH OF HORIZONTAL PL}\ 15.38\int_{8.38} rdr - 12.80 \quad \text{WIDTH} x r_0\ 15.38\int_{8.38} dr\ ]}{(171,68)\qquad (-191,93)\qquad (124,74)\qquad (-89,60)}$$

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

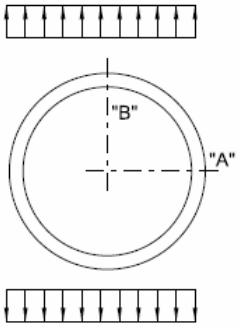
$$15\int_{6,88}^{8,38} r\,dr = 15\left[\frac{(8,38)^2}{2} - \frac{(6,88)^2}{2}\right] = 171,68$$

$$127.95\int_{6,88}^{8,38} dr = 127.95\left[(8,38-6,88)\right] = 191,93$$

$$1.5\int_{8.38}^{15,38} r\,dr = 1.5\left[\frac{(15,38)^2}{2} - \frac{(8,38)^2}{2}\right] = 124.74$$

$$12,80\int_{8.38}^{15.38} dr = 12,80\left[(15.38-8.38)\right] = 89.60$$

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)




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
DESIGNDATA
ANCHORAGE LINKS
GUDGEON PIN BARREL

Plate C-10. Design Data on Gudgeon Pin Barrel, Horizontally and Vertically Framed Miter Gates.

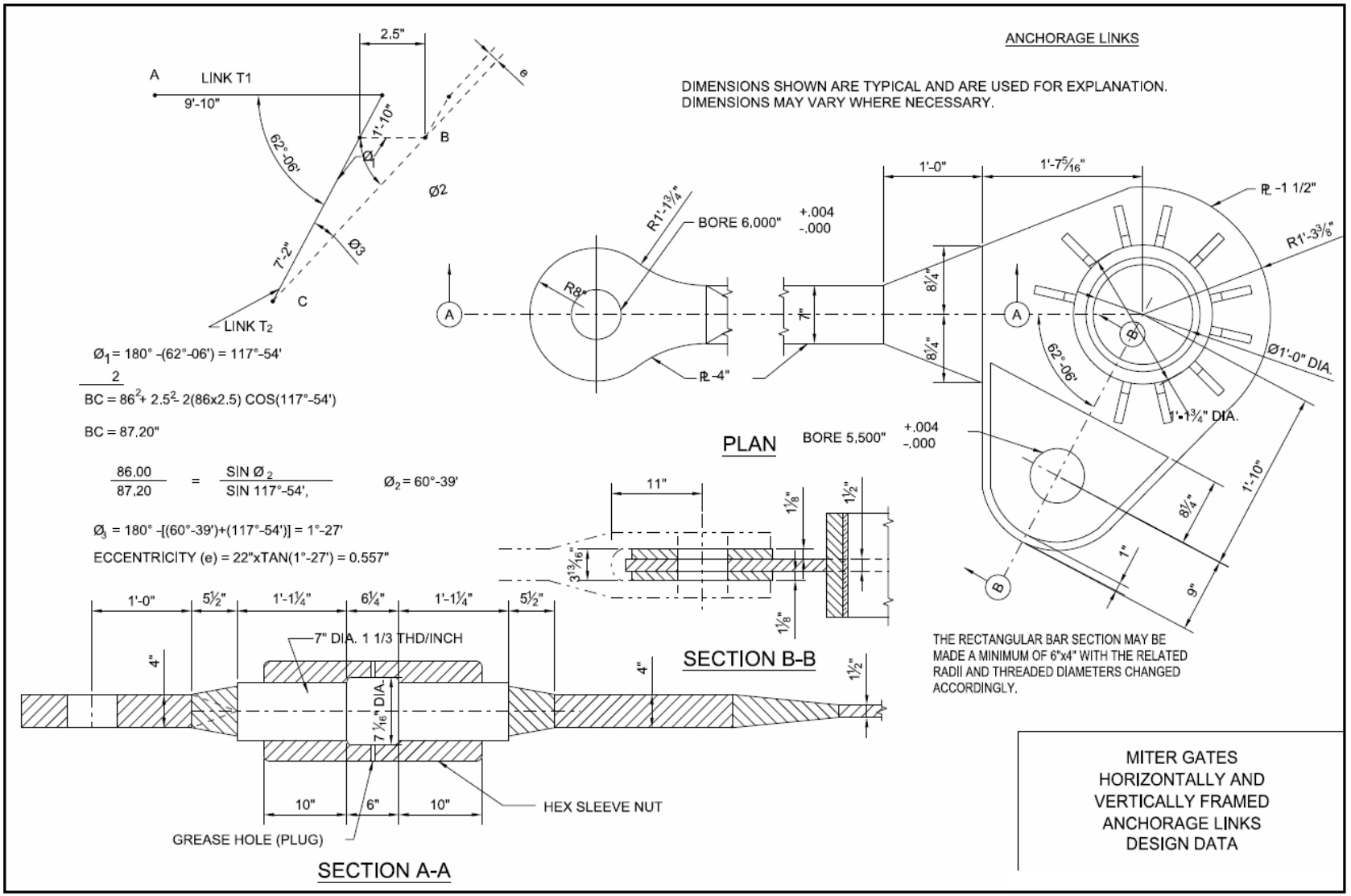



Plate C-11. Anchorage Links Design Data, Horizontally and Vertically Framed Miter Gates.

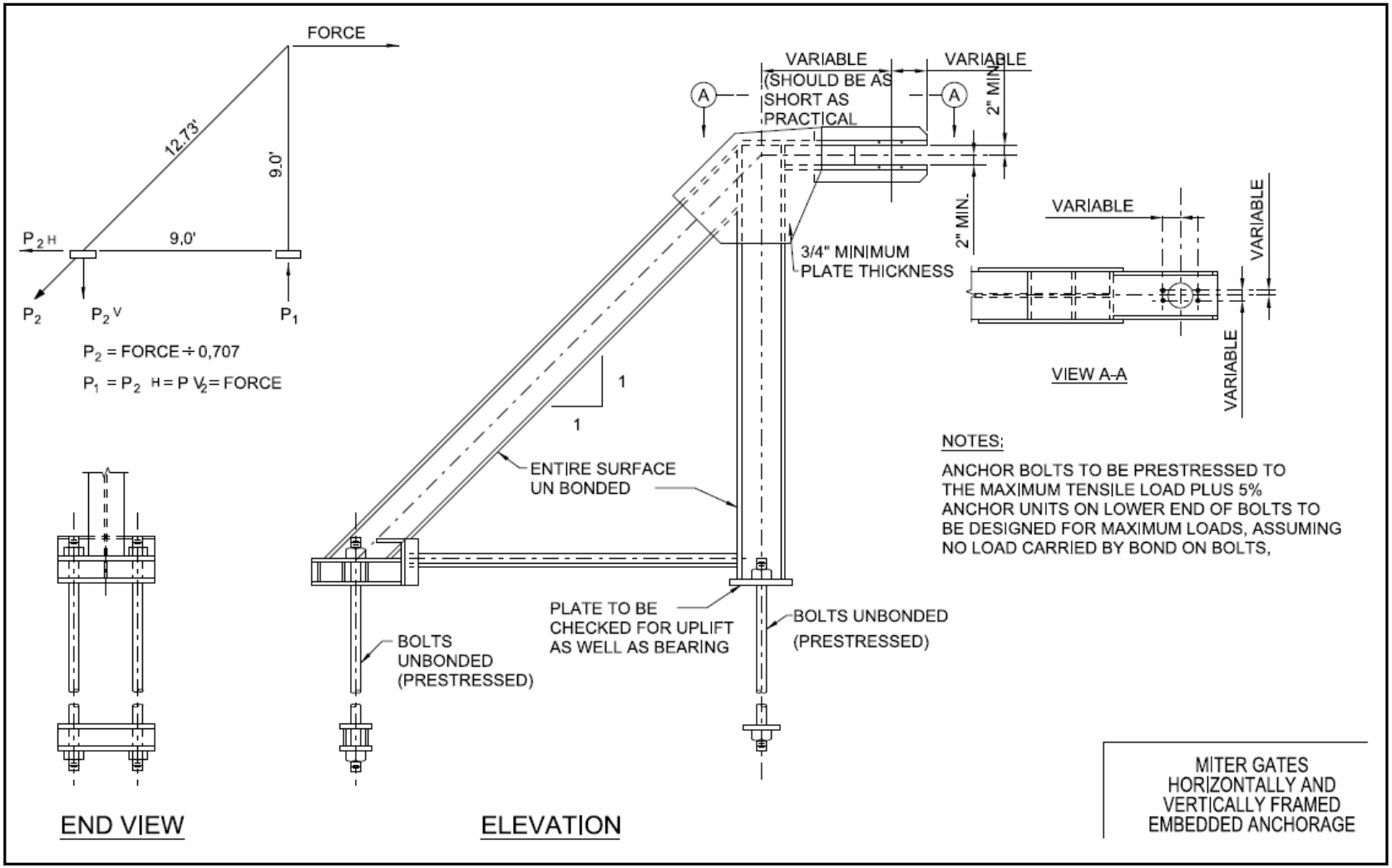



Plate C-12. Embedded Anchorage, Horizontally and Vertically Framed Miter Gates.

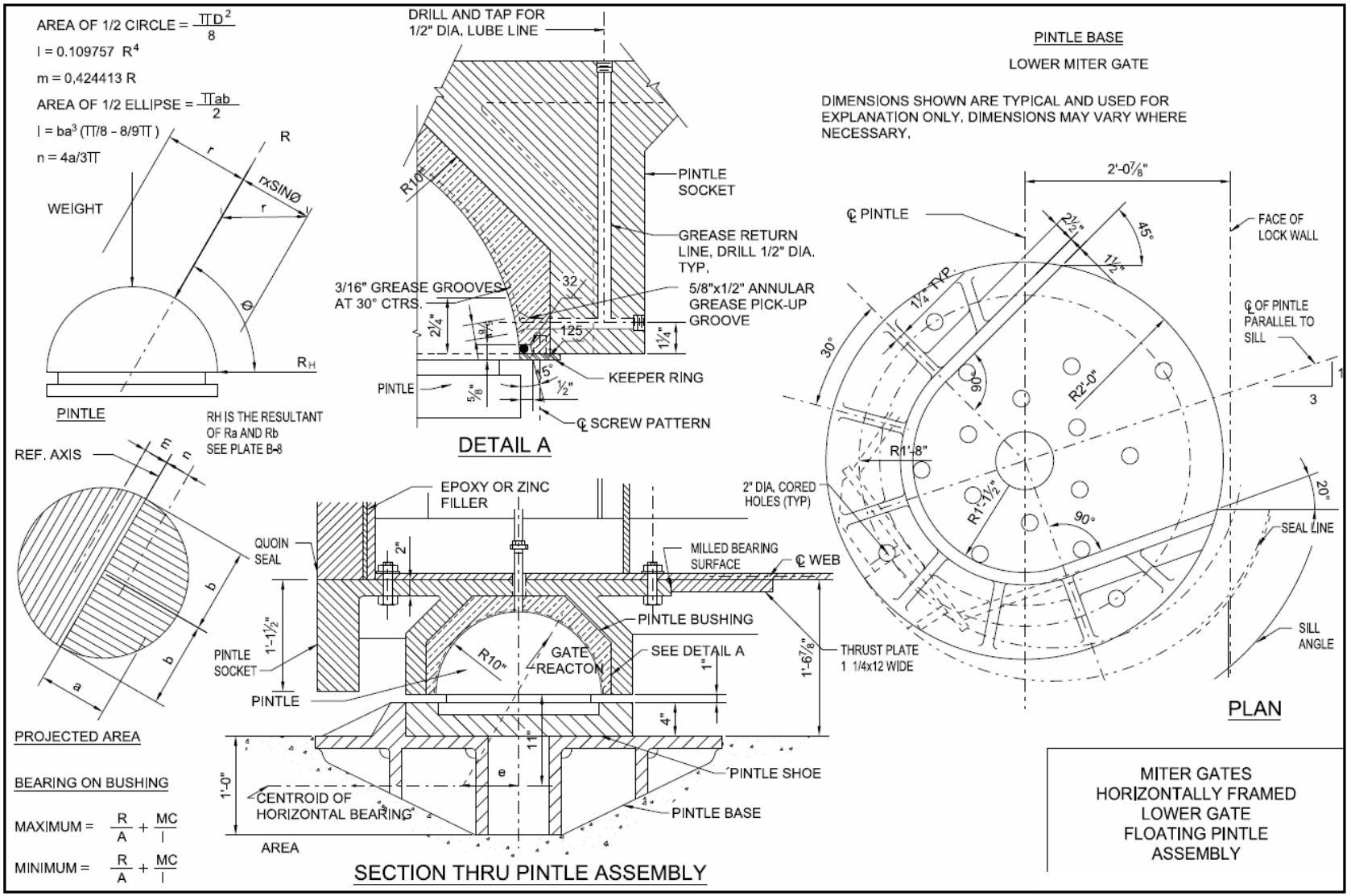



Plate C-13. Lower Gate Pintle Assembly, Horizontally Framed Miter Gates.

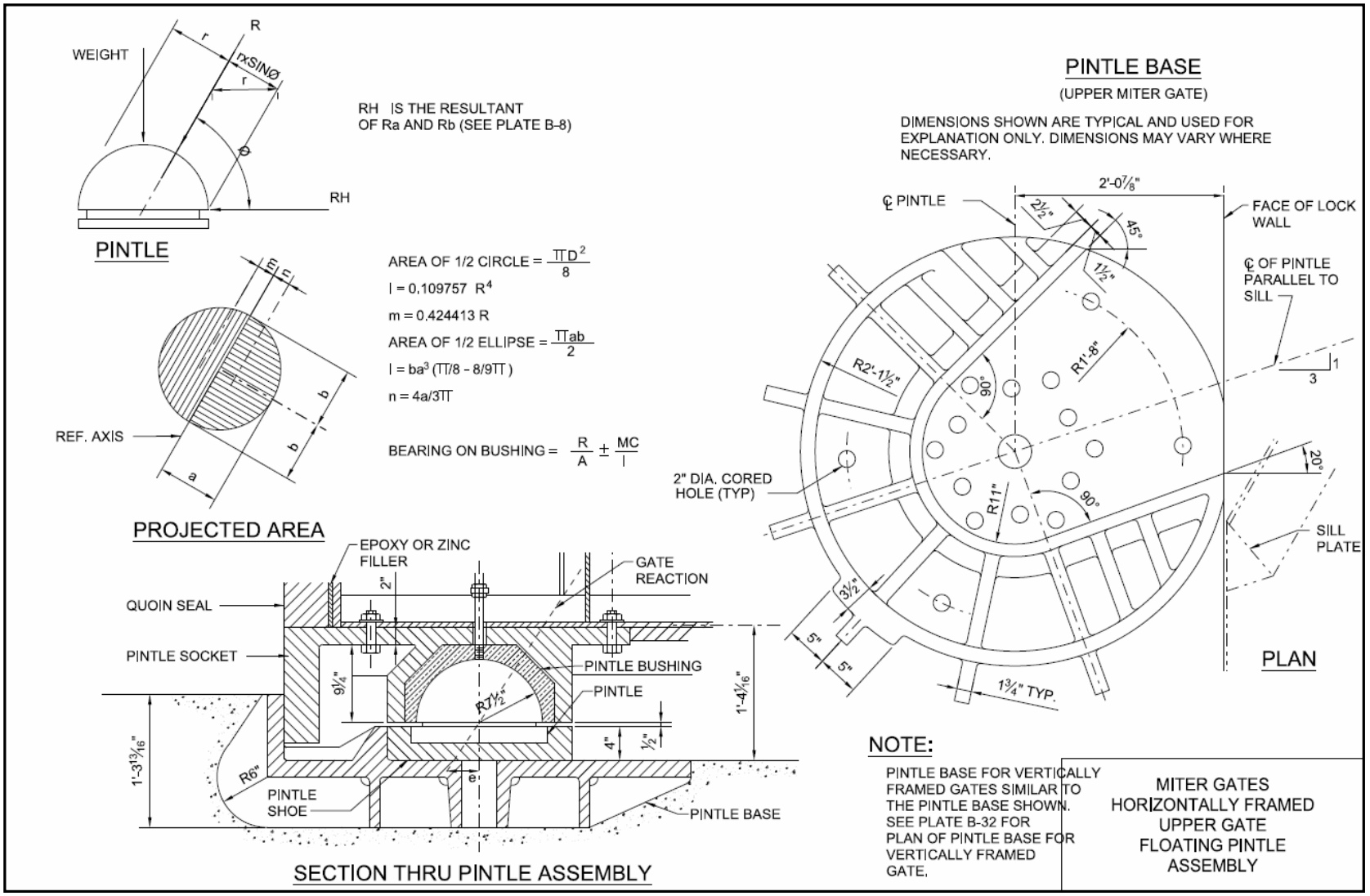



Plate C-14. Upper Gate Pintle Assembly, Horizontally Framed Miter Gates.

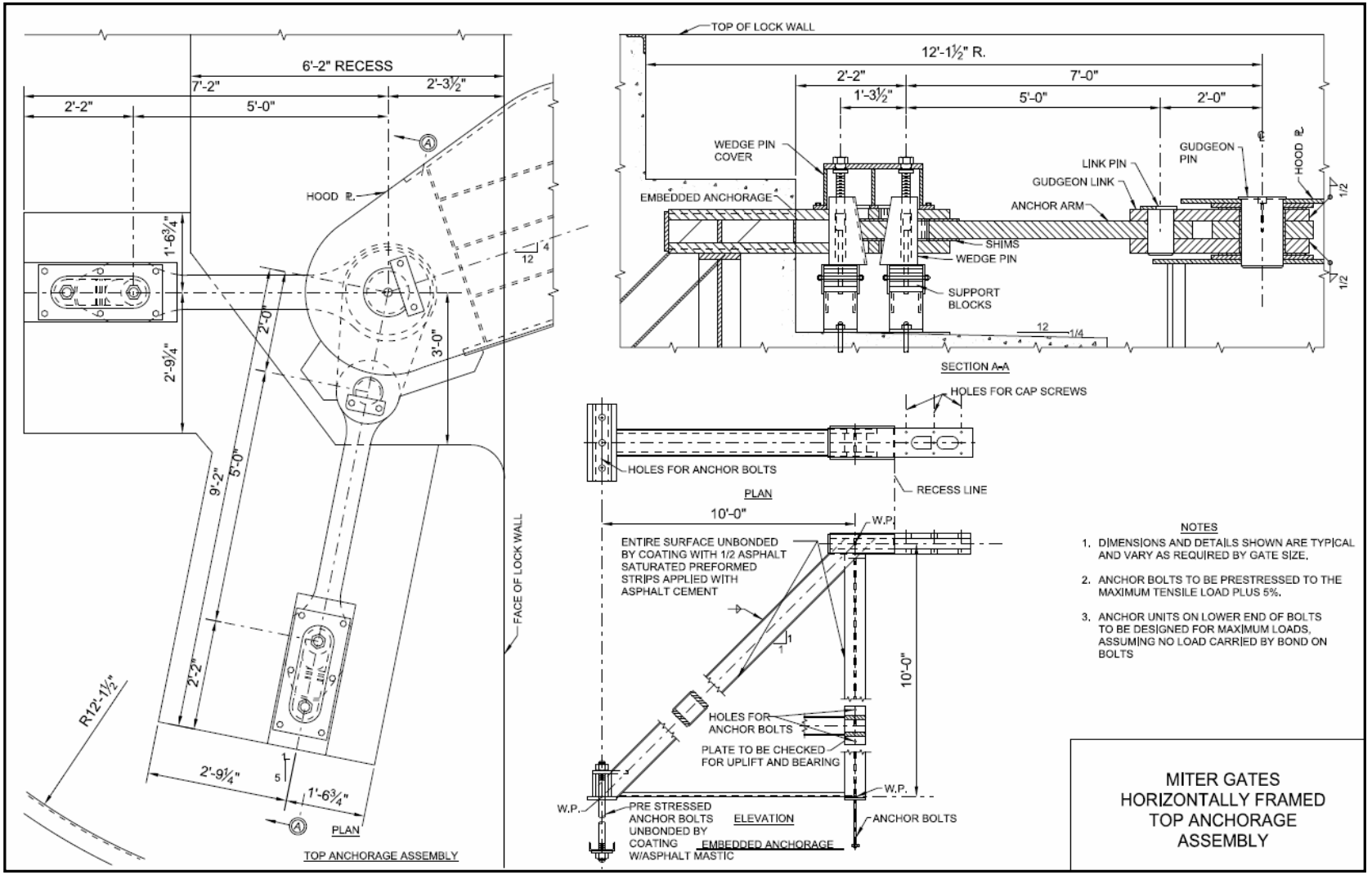



Plate C-15. Top Anchorage Assembly, Horizontally Framed Miter Gates.

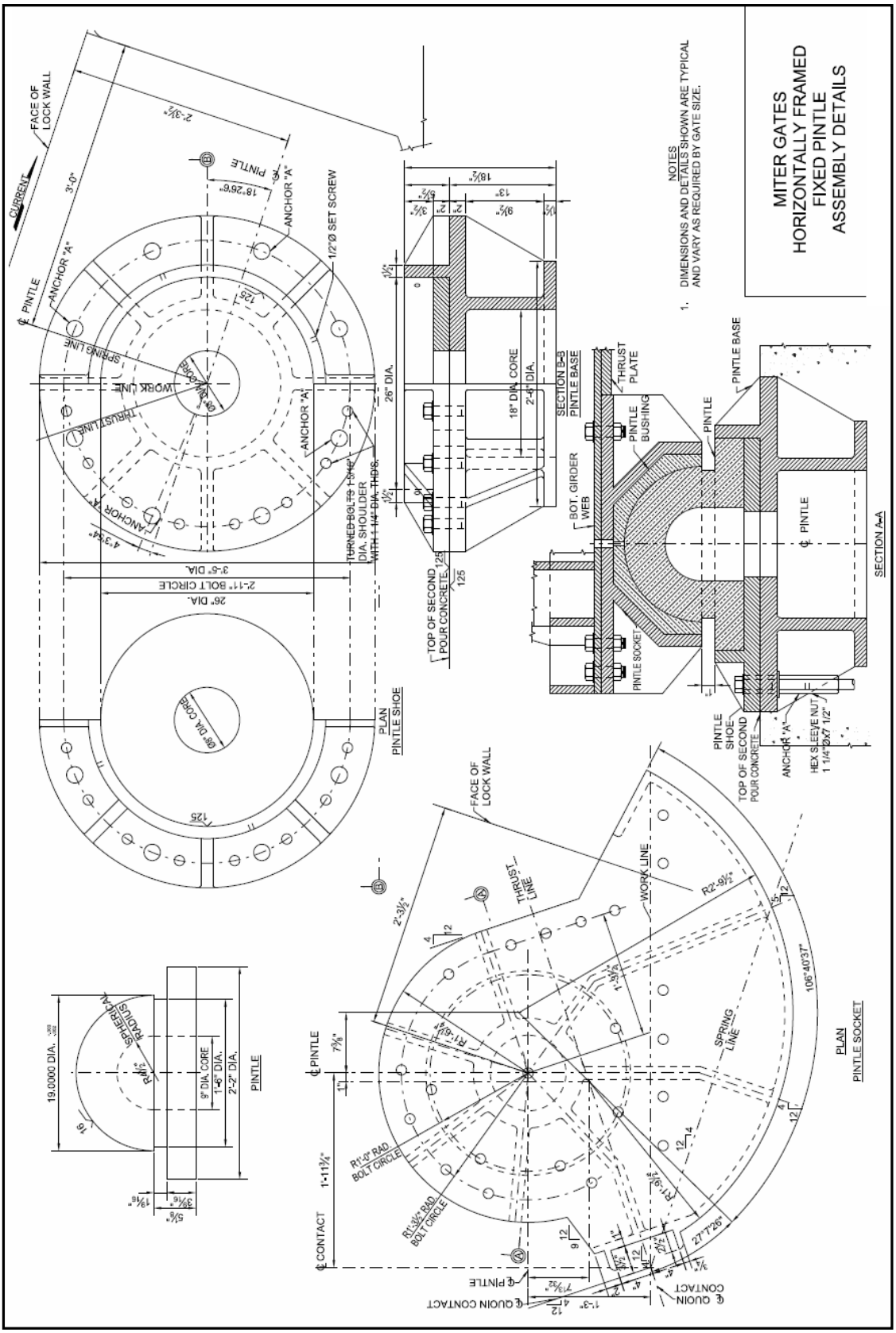



Plate C-16. Fixed Pintle Assembly, Horizontally Framed Miter Gates.

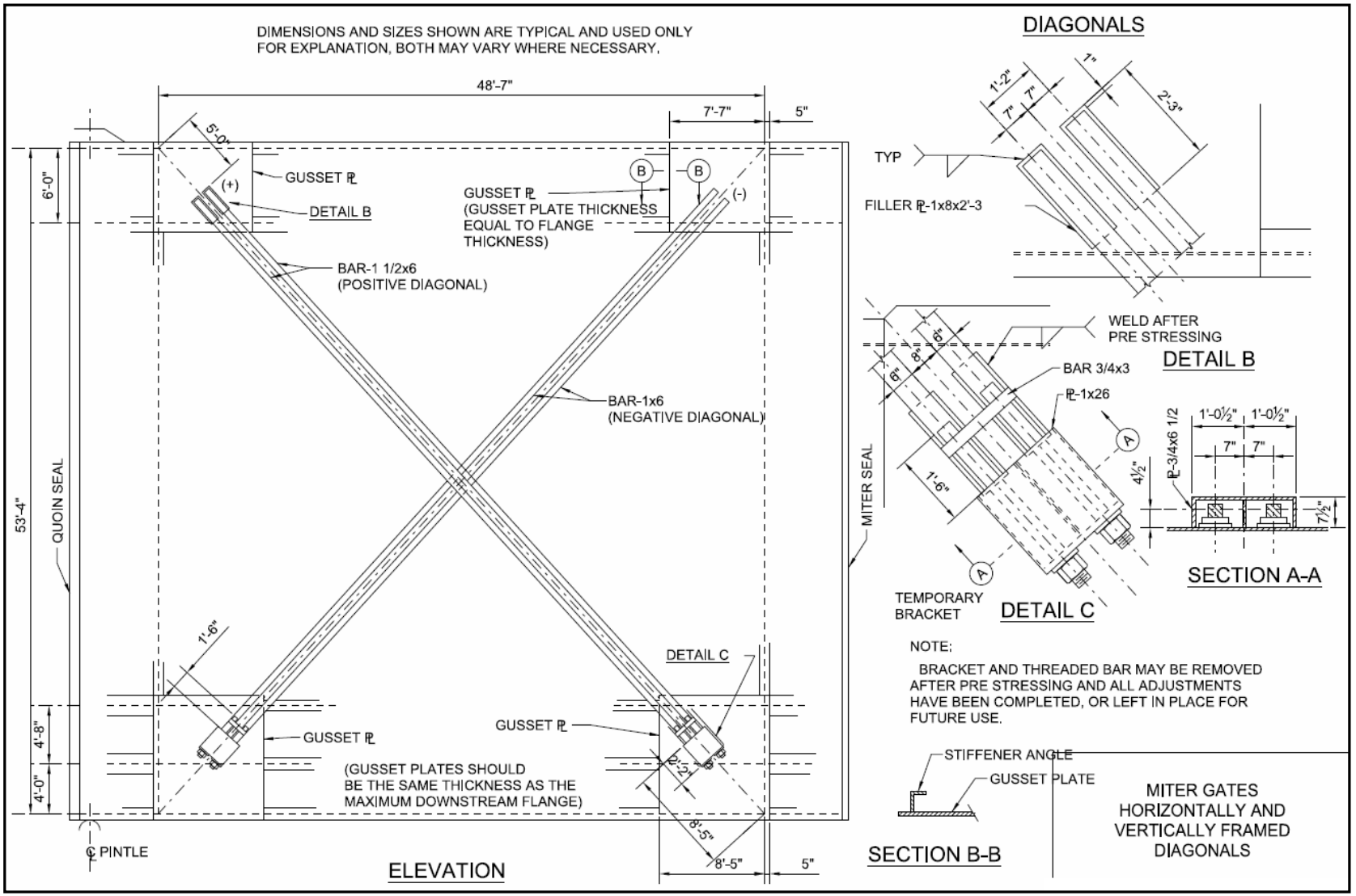



Plate C-17. Diagonals, Horizontally and Vertically Framed Miter Gates (Sheet 1 of 2).

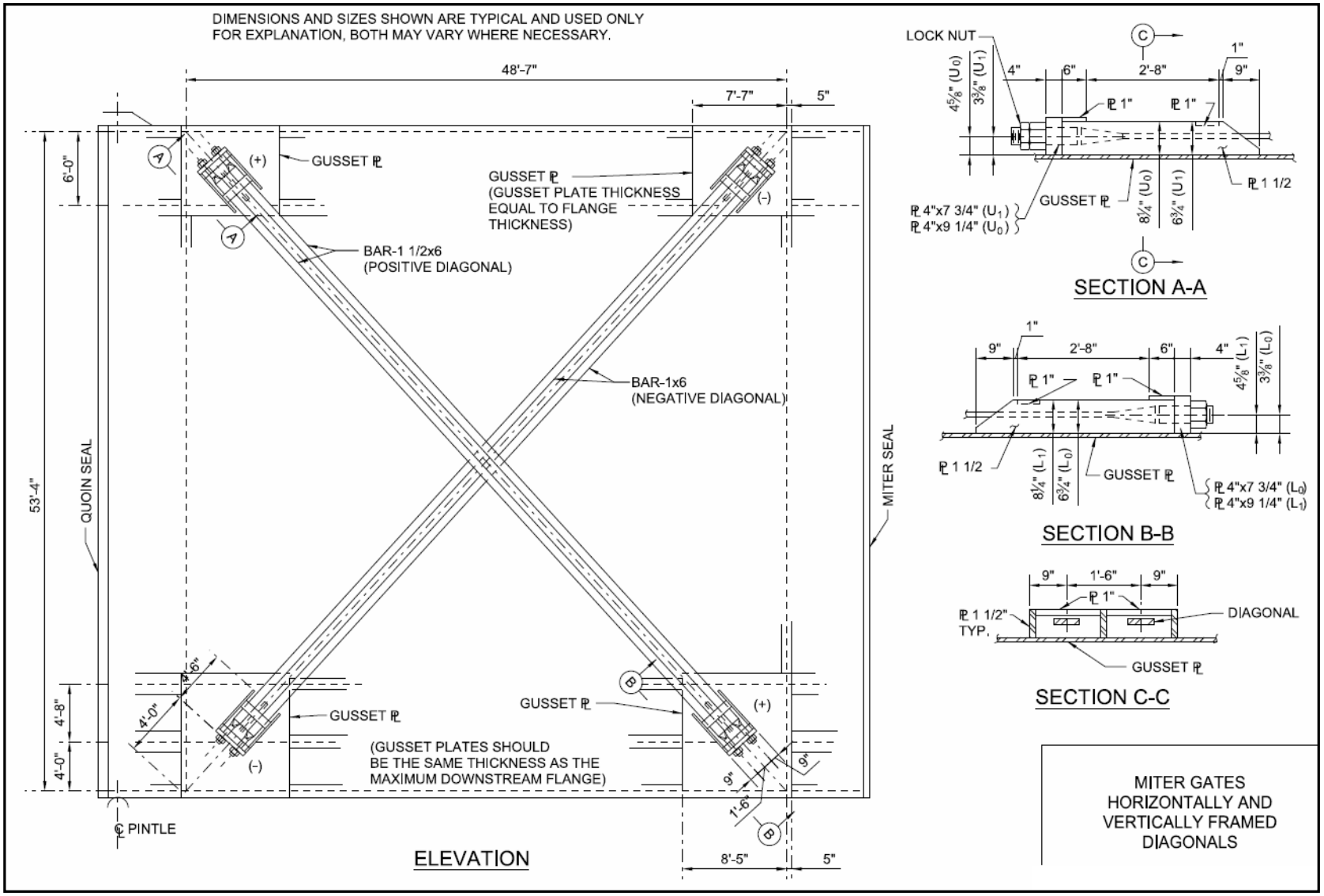



Plate C-17. Diagonals, Horizontally and Vertically Framed Miter Gates (Sheet 2 of 2).

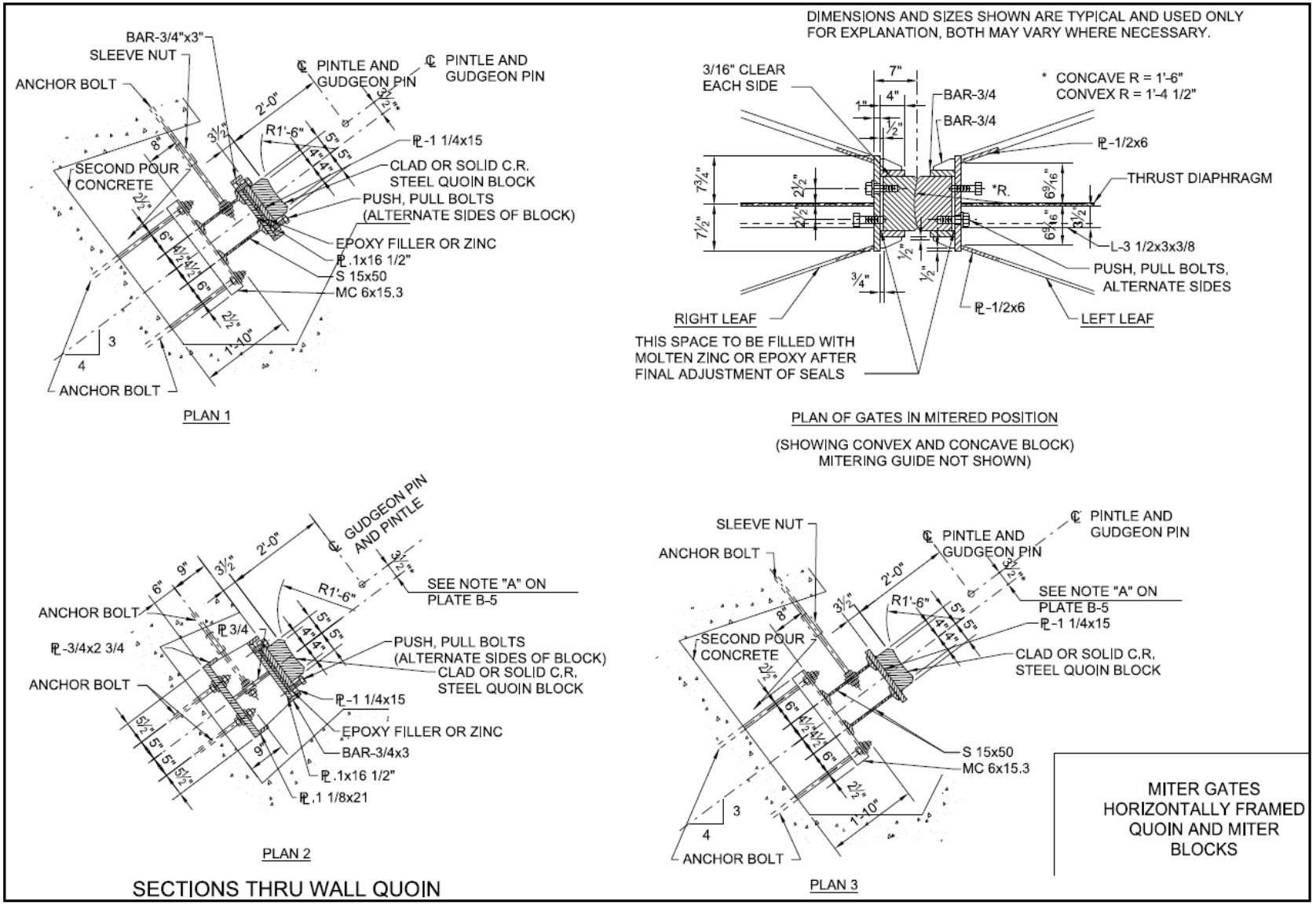



Plate C-18. Quoin and Miter Blocks, Horizontally Framed Miter Gates.

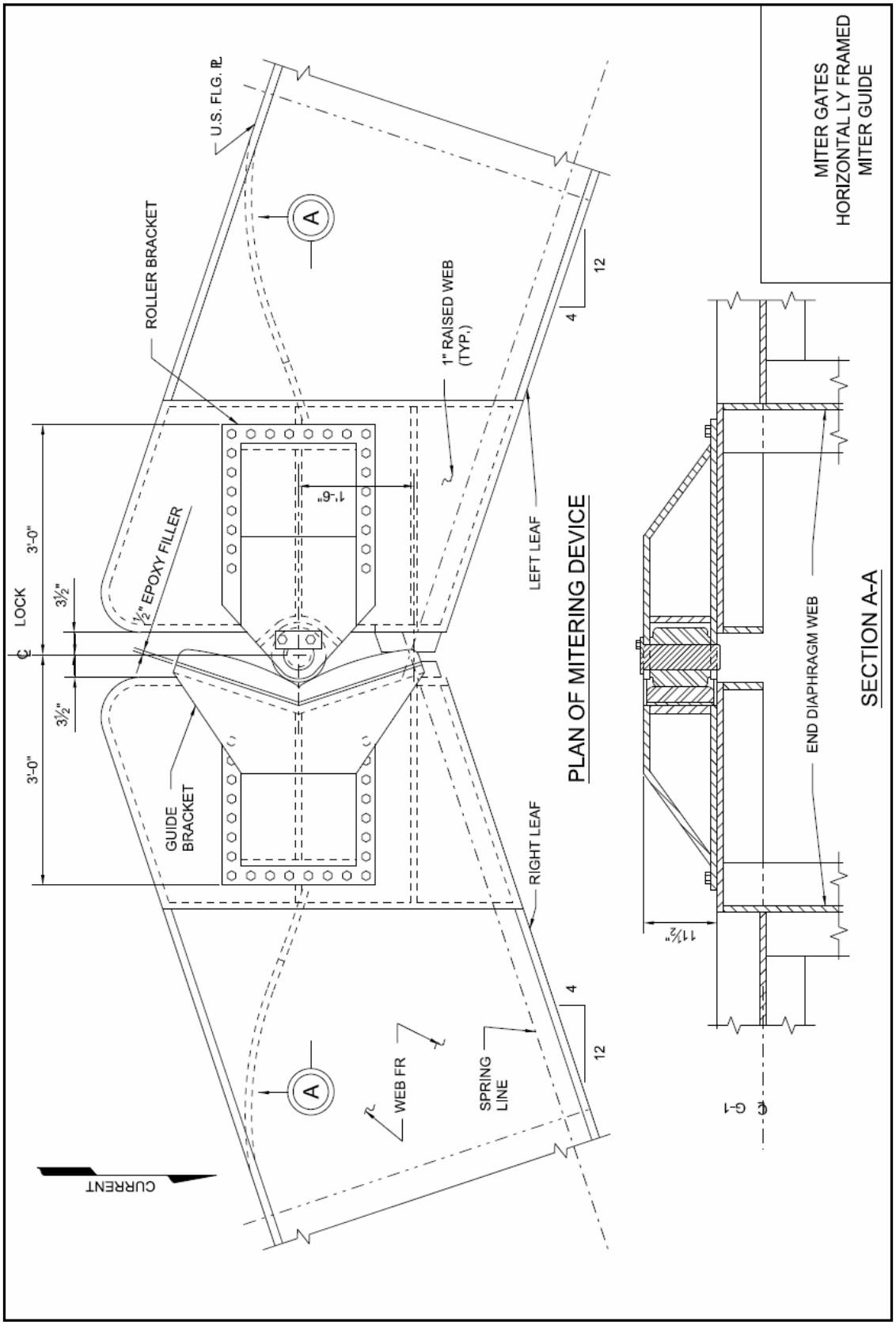



Plate C-19. Miter Guide, Horizontally Framed Miter Gates.

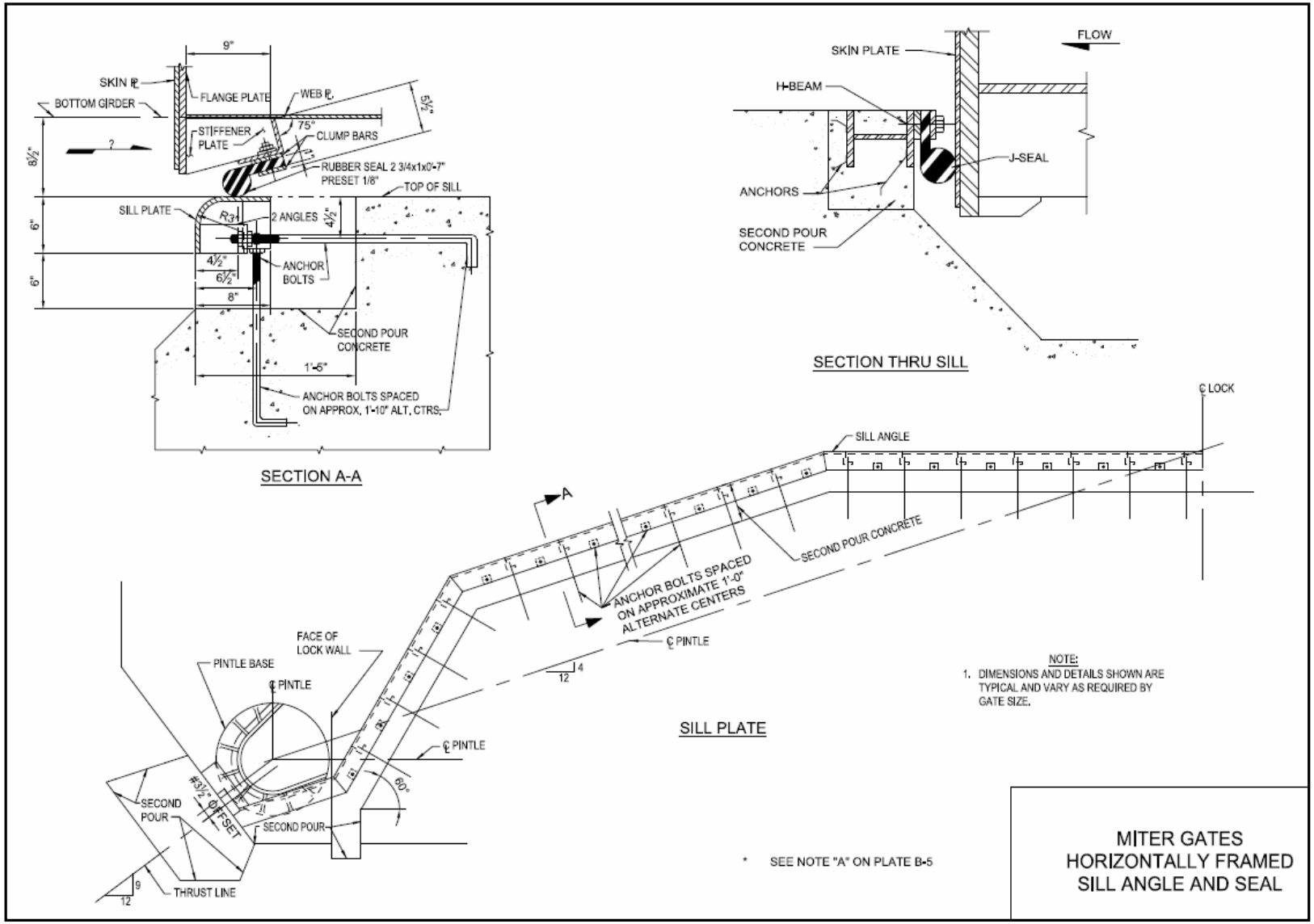



Plate C-20. Sill Angle and Seal, Horizontally Framed Miter Gates.

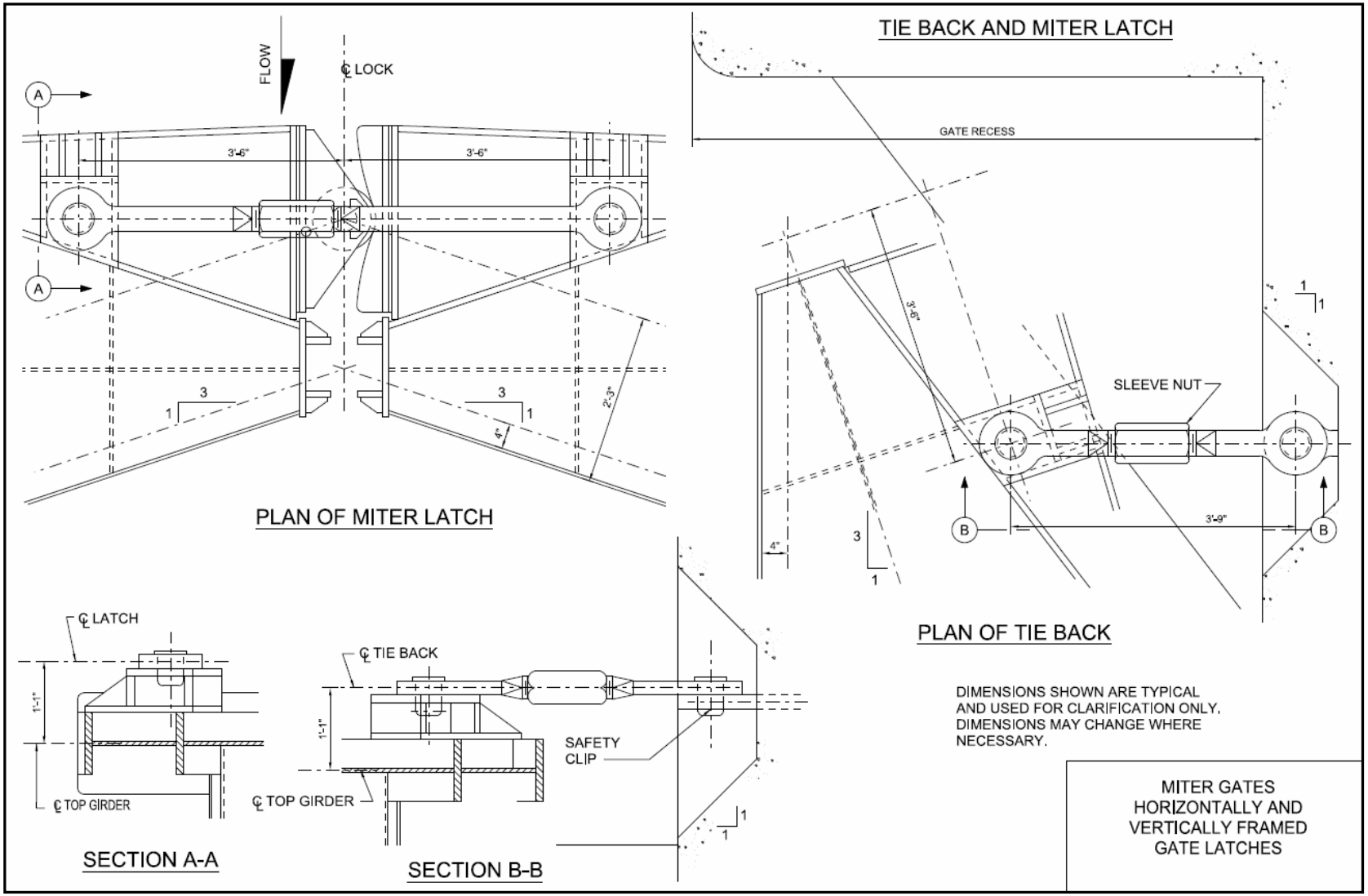



Plate C-21. Gate Latches, Horizontally and Vertically Framed Miter Gates.

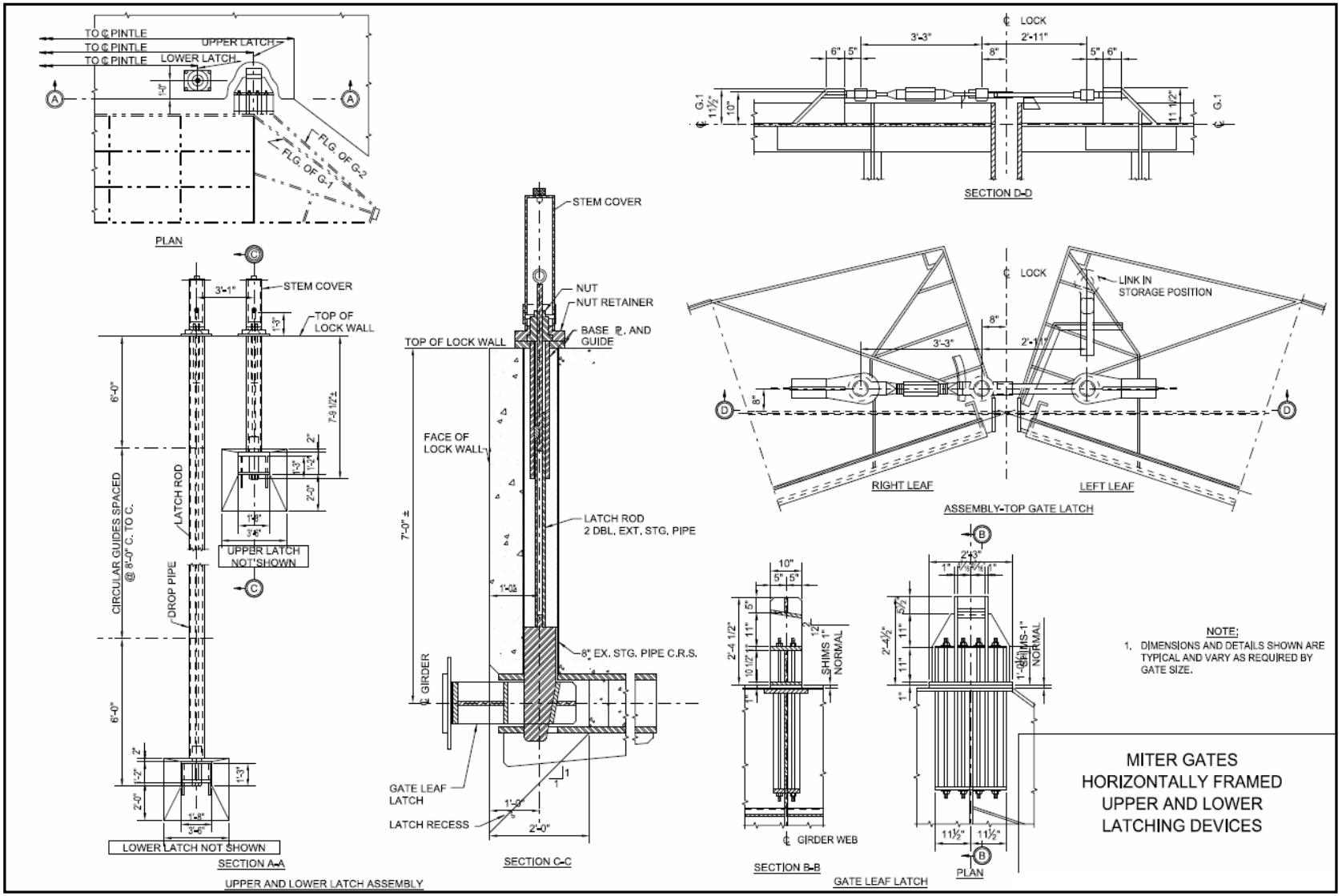



Plate C-22. Upper and Lower Latching Devices, Horizontally Framed Miter Gates.

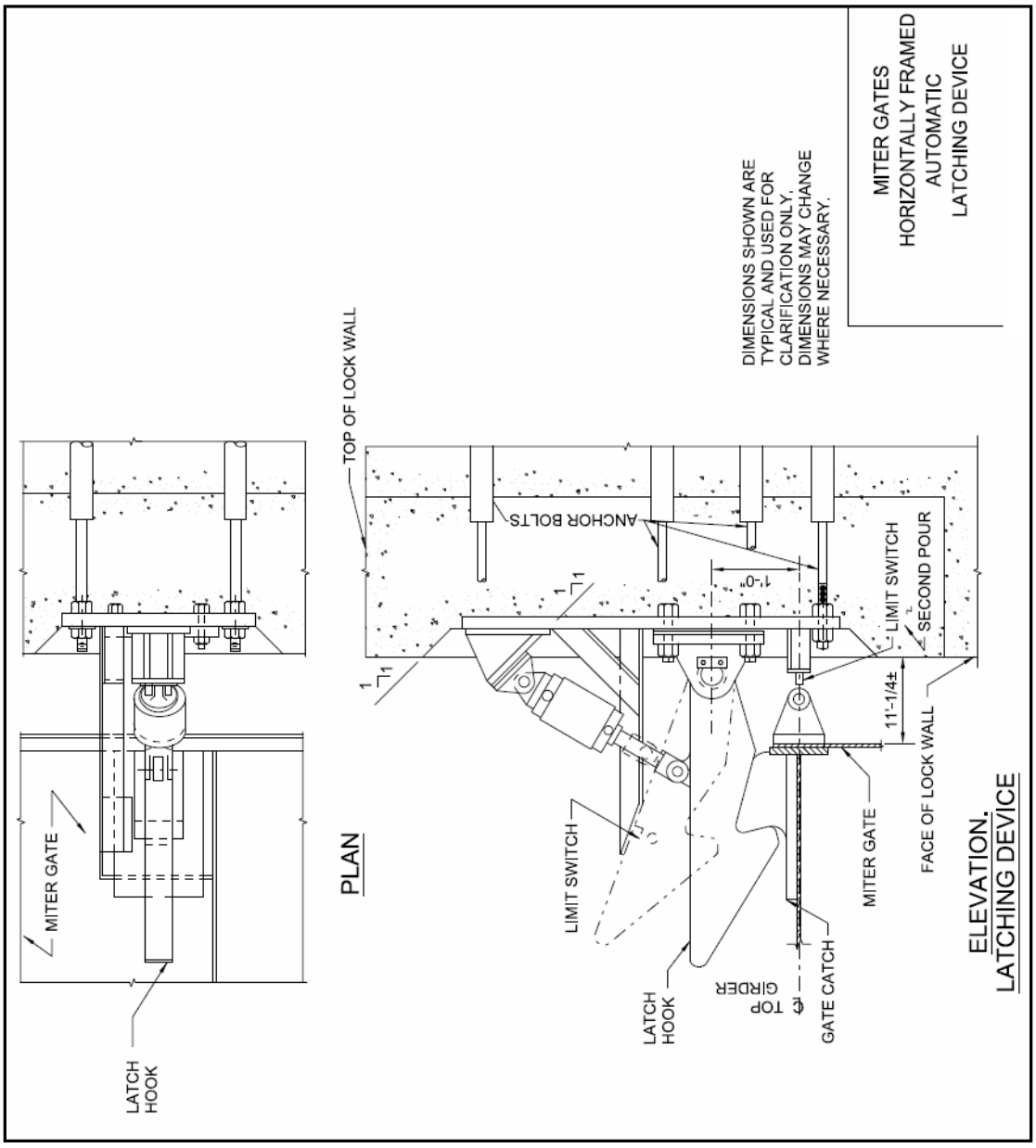



Plate C-23. Automatic Latching Device, Horizontally Framed Miter Gates.

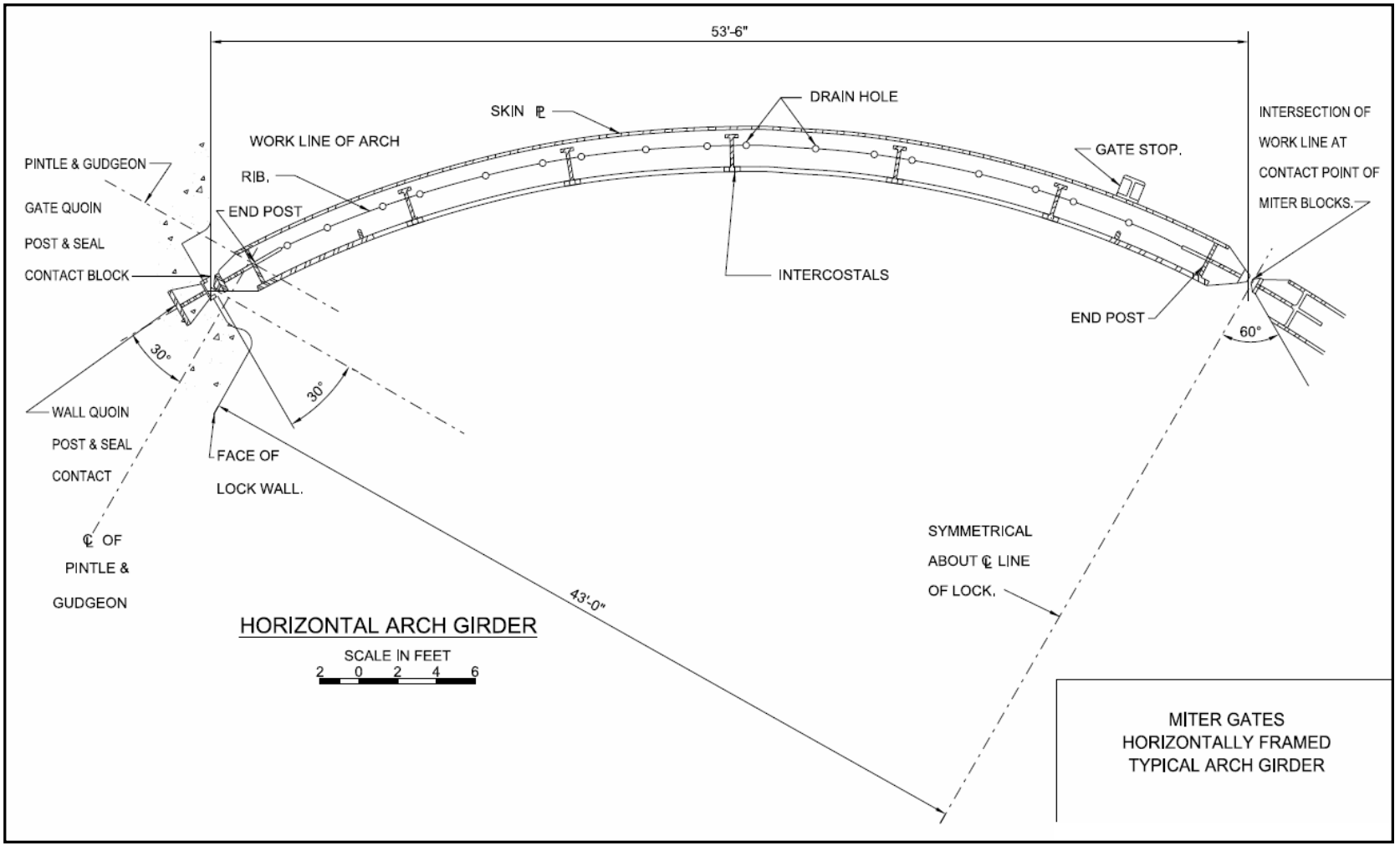



Plate C-24. Typical Arch Girder, Horizontally Framed Miter Gates.

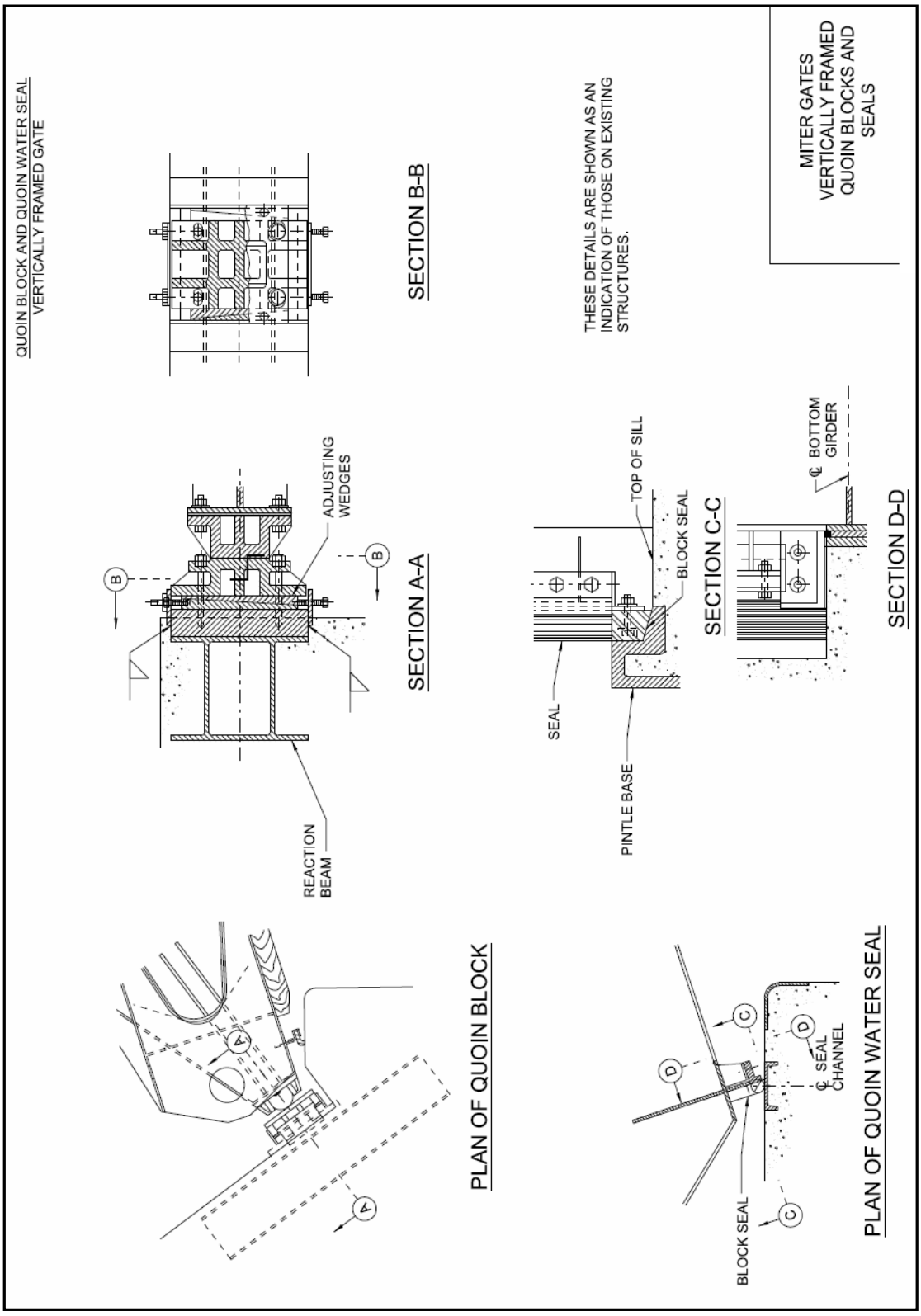



Plate C-25. Quoin Block and Seals, Vertically Framed Miter Gates.

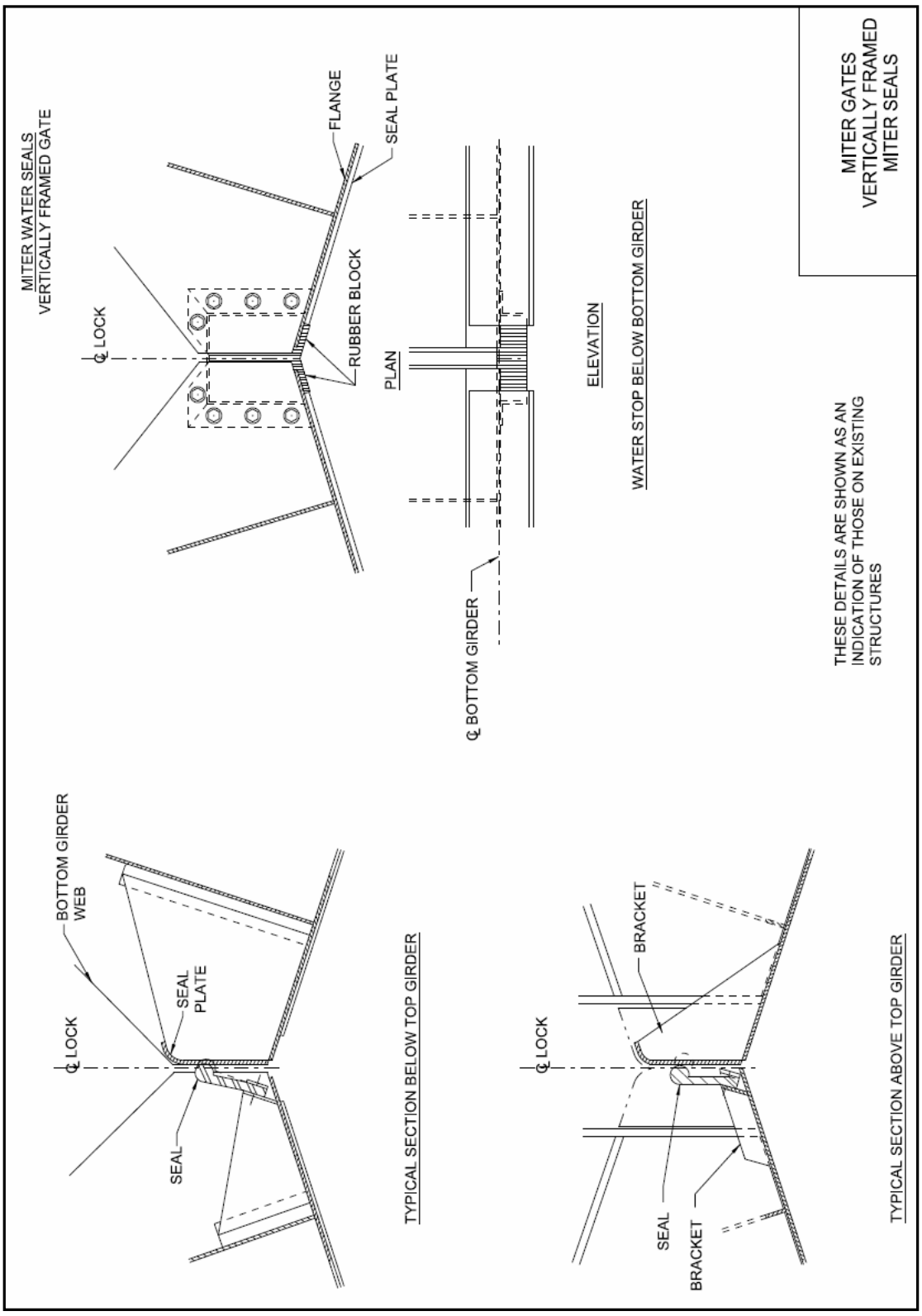



Plate C-26. Miter Seals, Vertically Framed Miter Gates.

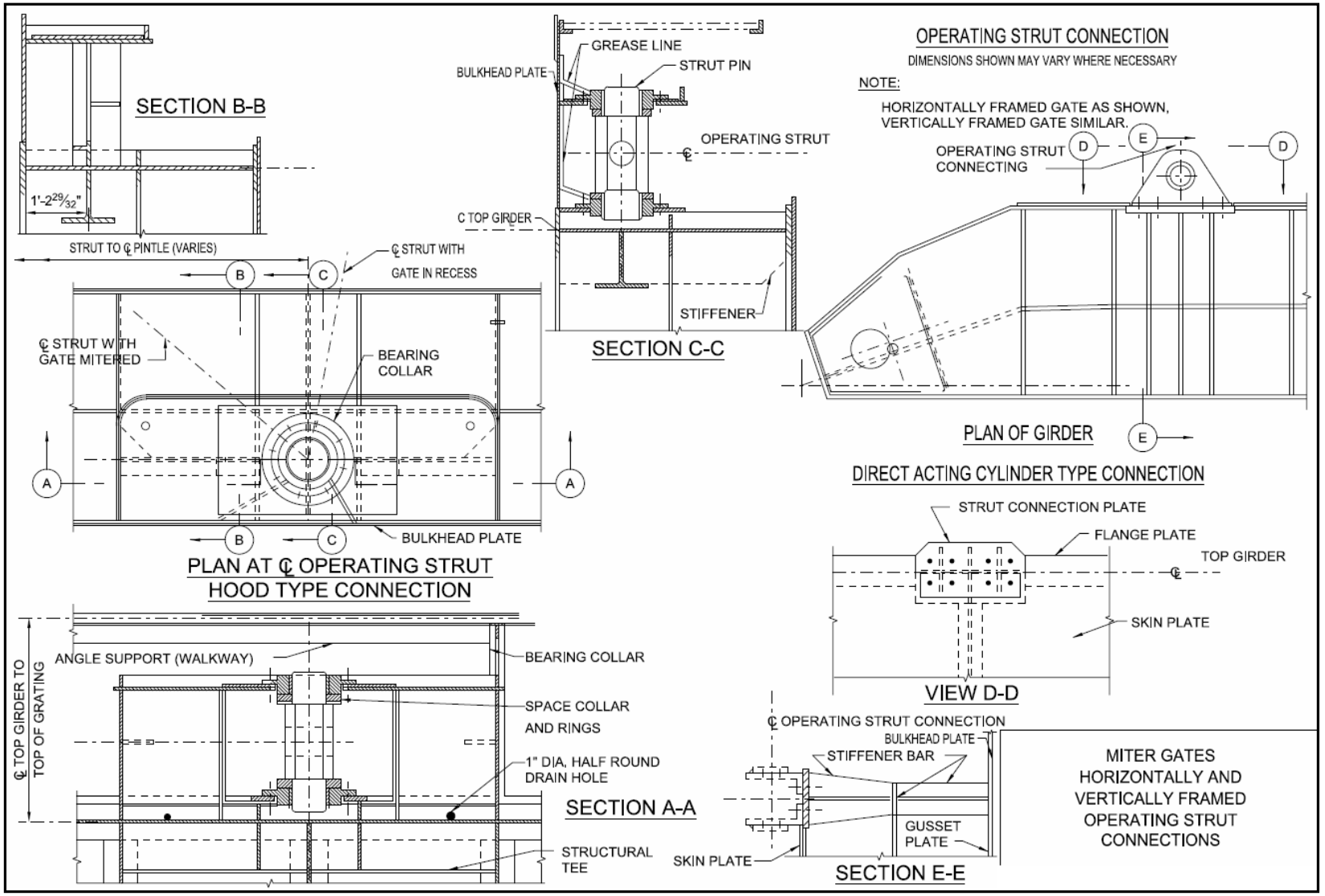



Plate C-27. Operating Strut Connections, Horizontally and Vertically Framed Miter Gates.

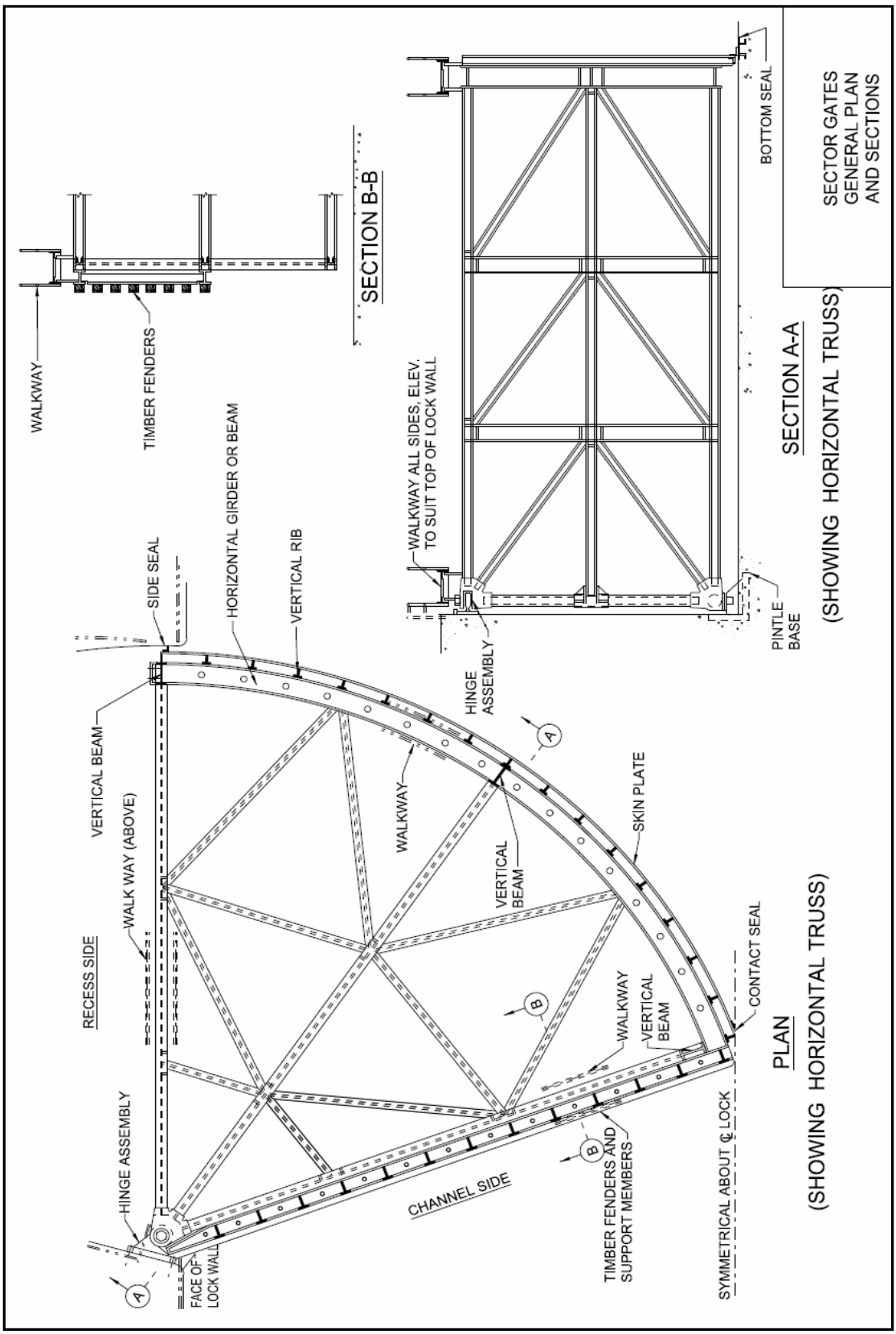



Plate C-28. General Plan and Sections, Sector Gates.

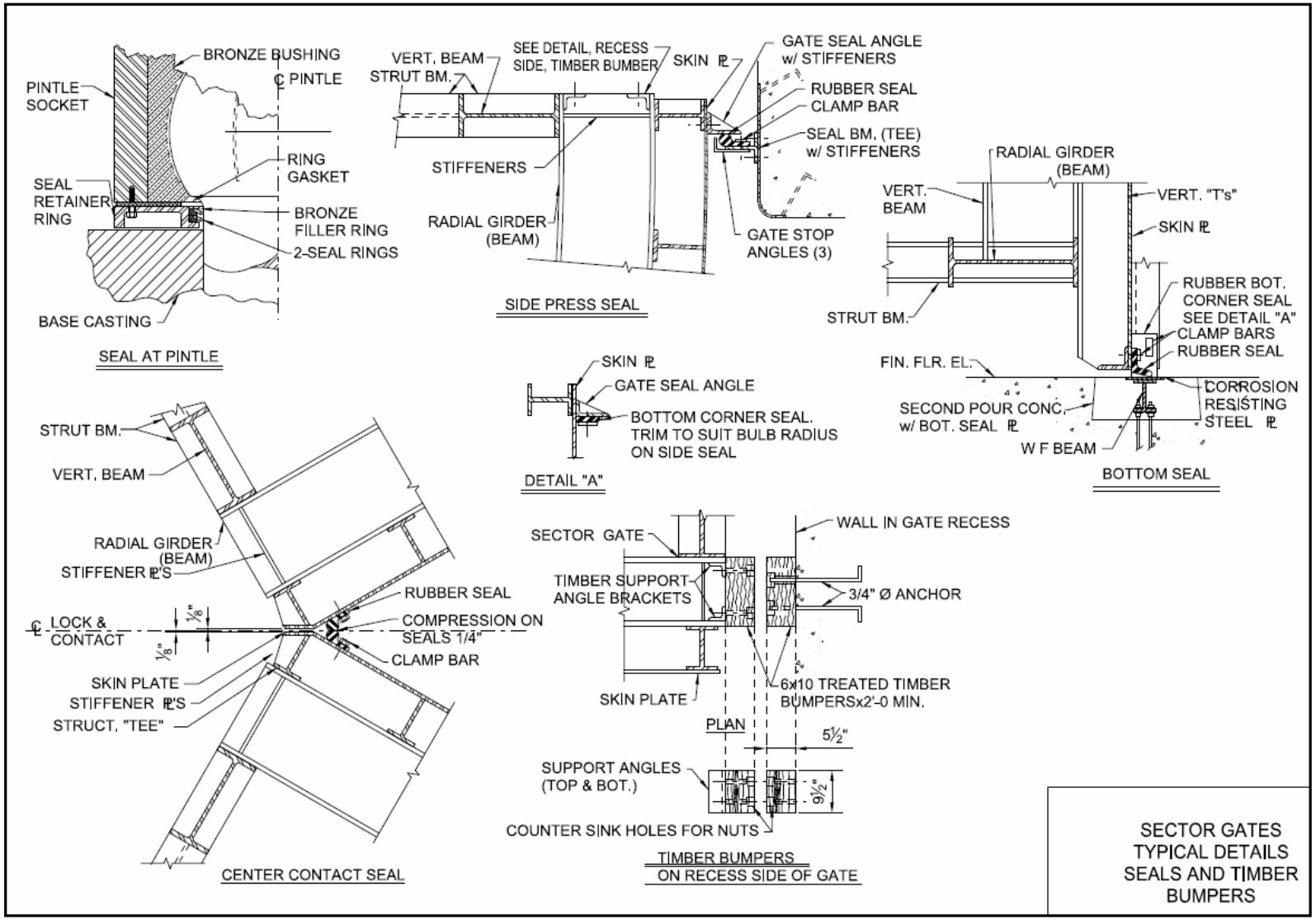



Plate C-29. Seals and Timber Bumpers, Typical Details, Sector Gates.

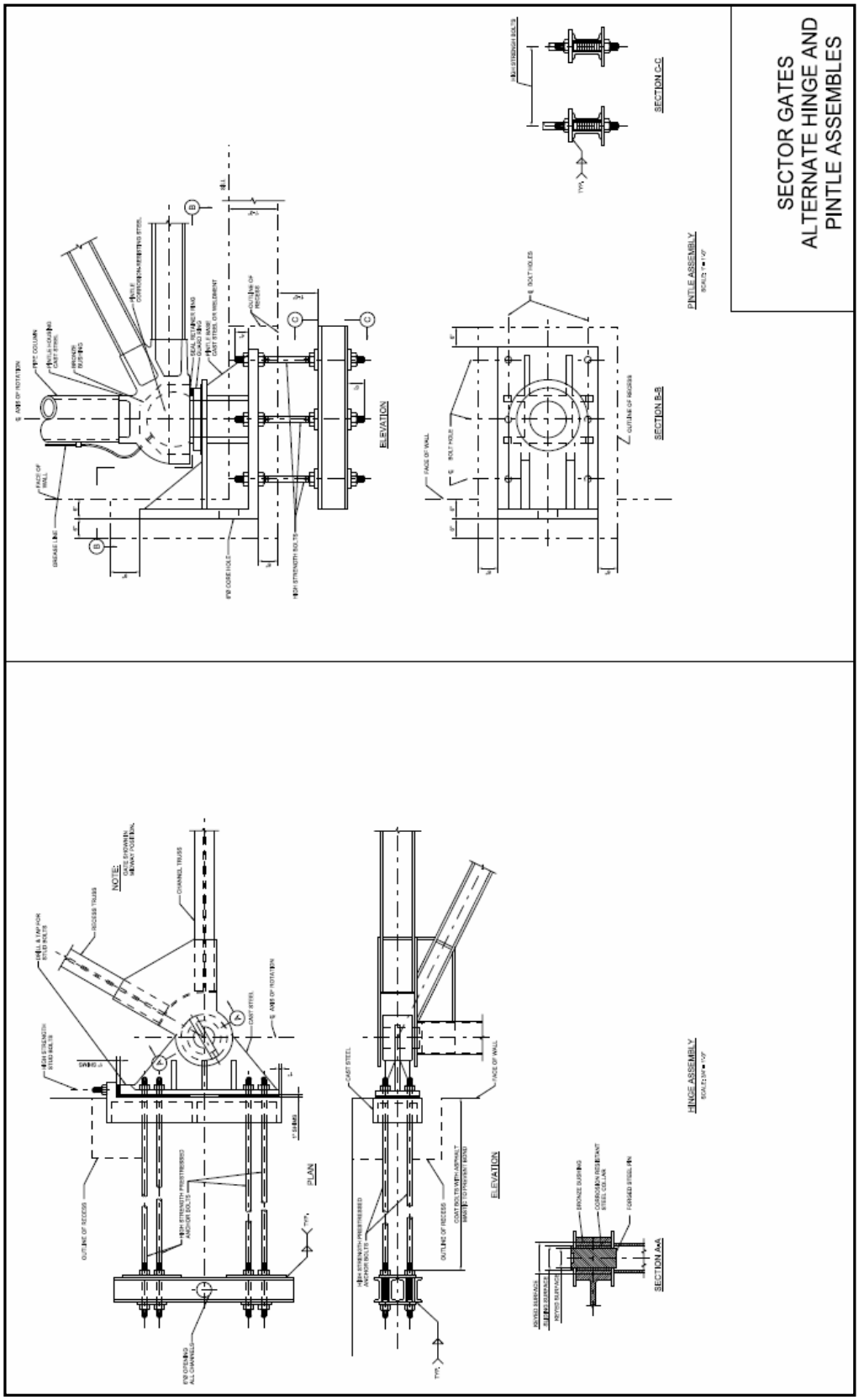



Plate C-30. Alternate Hinge and Pintle Assemblies, Sector Gates.

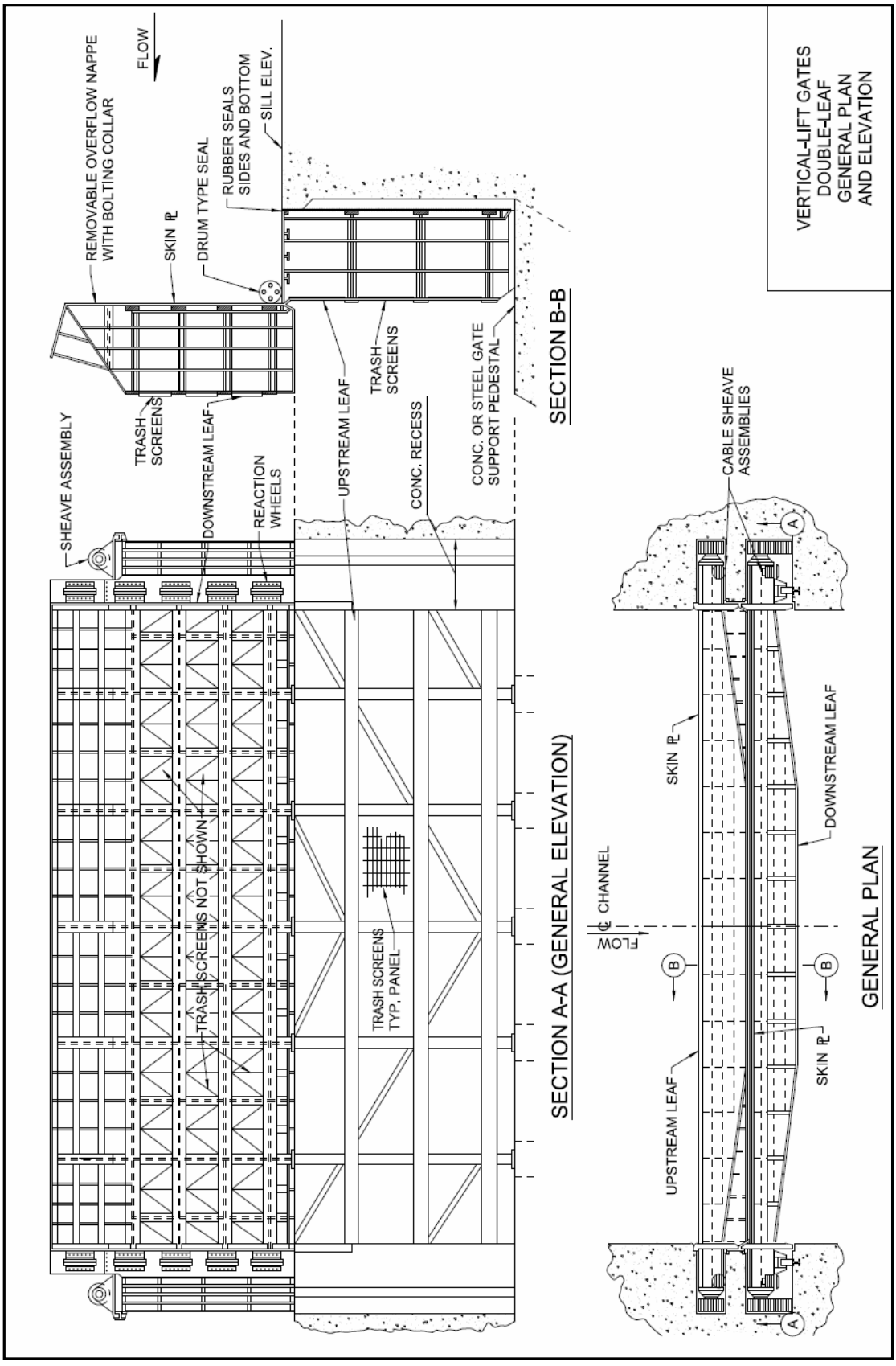



Plate C-31. General Plan and Elevation, Double-Leaf, Vertical Lift Gates.

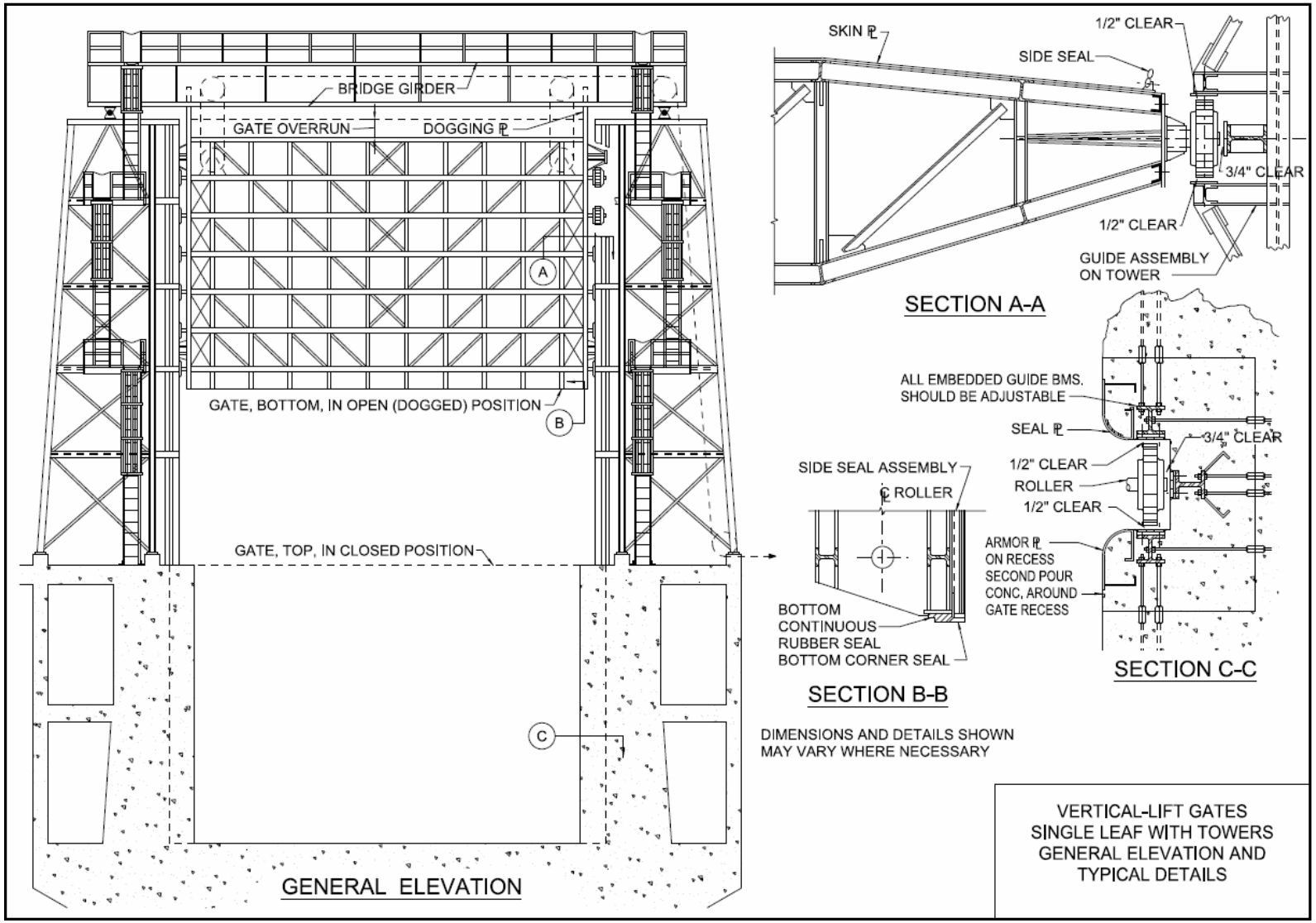



Plate C-32. General Elevation and Typical Details, Single Leaf, Vertical Lift Gates.

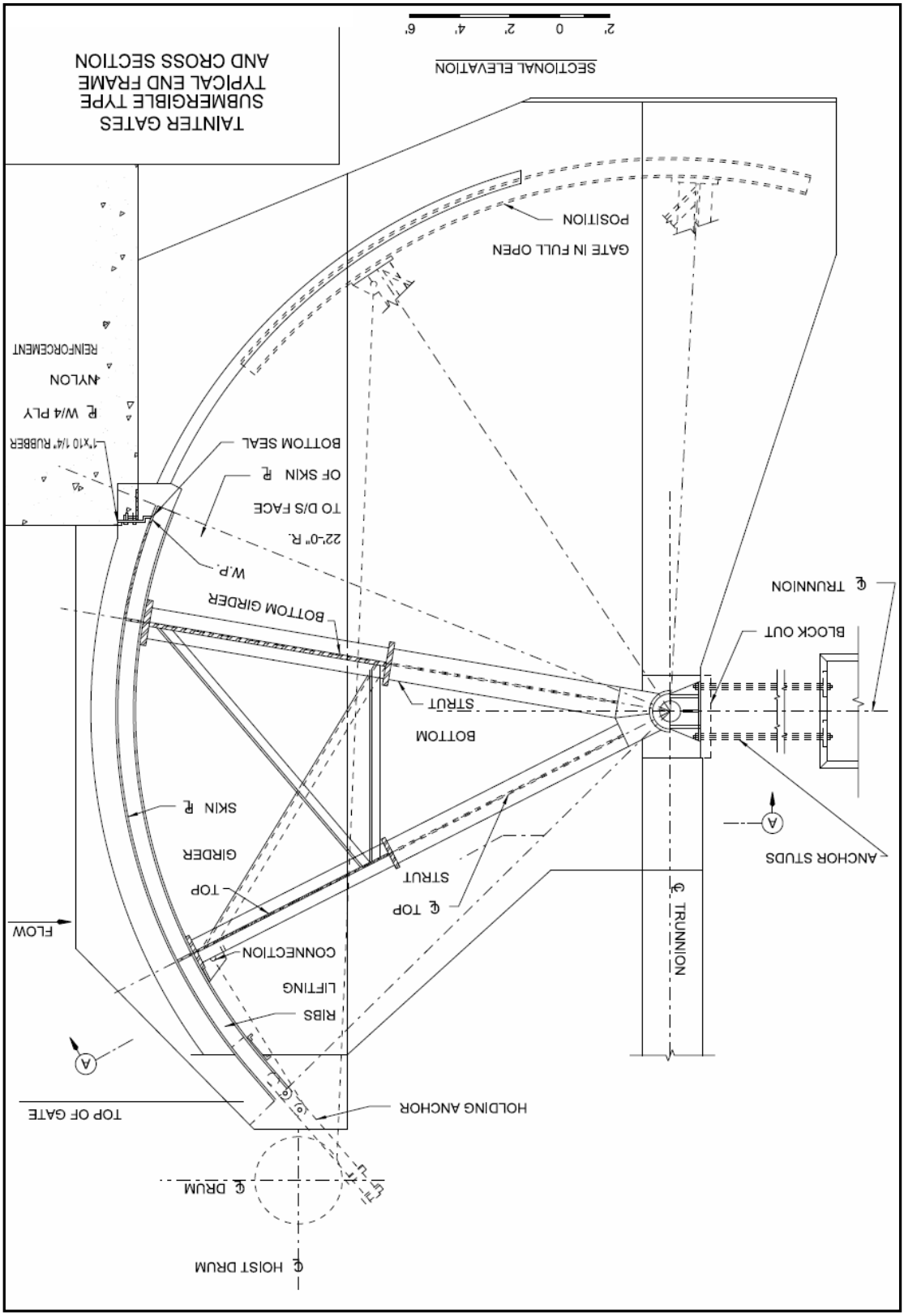



Plate C-33. Typical End Frame and Cross-Section, Submergible-Type Tainter Gates.

# ATTACHMENT 1

## Miter Gate Diagonal Design

### A1.1. 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.

### A1.2. Definitions of Terms and Symbols

Deviations from these symbols are noted at the places of exception:

$\Delta$ = 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 A1-1). 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 A1-4).

$a$ = The cross-sectional area of that part of a horizontal girder that lies outside the midpoint between the skin and the flange (see Figure A1-6).

$A$ = 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,

1/20 for riveted vertically framed leaves with skin of buckle plate (see paragraph A1.4.9.1).

$b$ = Distance from the center line of the skin plate to the flange of a horizontal girder (see Figure A1-6).

$c$ = 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 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 A1-5).

$J$ = Modified polar moment of inertia of the horizontal and vertical members of the leaf.

$K$ = A constant, taken equal to 4 (see paragraph A1.4.9.1).

$l$ = The larger dimension of a rectangular cross section.

$L$ = Length of a diagonal, center to center of pins.

$M$ = Torque required to turn the sleeve nut to prestress diagonal (refer to Equation A1-28).

$n$ = Number of threads per inch in sleeve nut of diagonal.

$N$ = Number of turns of nut to prestress diagonal (refer to Equation A1-27).

$Q_0$ = Elasticity constant of a leaf without diagonals (see paragraph A1.4.9.2).

$Q$ = Elasticity constant of diagonal defined by Equation A1-18.

$R_0$ = Ratio of change in length of diagonal to deflection of leaf when diagonal offers no resistance (refer to Equation A1-11). $R_0$ 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 A1-13). $R$ is positive for positive diagonals and negative for negative diagonals.

$s$ = Unit stress in diagonal.

$S$ = Total force in diagonal.

$t$ = Distance from center line of skin plate to center line of diagonal (for curved skin plate, see paragraph A1.4.8).

$T_z$ = 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 center line of pintle to extreme miter end of leaf.

$w$ = Width of panel (refer to Figure A1-1).

X = Distance from center line of skin plate to vertical shear center axis of leaf (refer to Equation A1-30).

$y$ = 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 A1-29).

## A1.3. 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. 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.

## A1.4. 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.

A1.4.1. Diagonal deformation. In Figures A1-2 and A1-3, the panel ak of Figure A1-1 is considered separately. As the leaf twists the panel ak twists as indicated by the dotted lines. In Figure A1-3, movements of all points are computed relative to the three reference axes gf, gb, and gk shown in Figure A1-2. 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 $\delta_0$ equal the change in length of either diagonal of Figure A1-3.

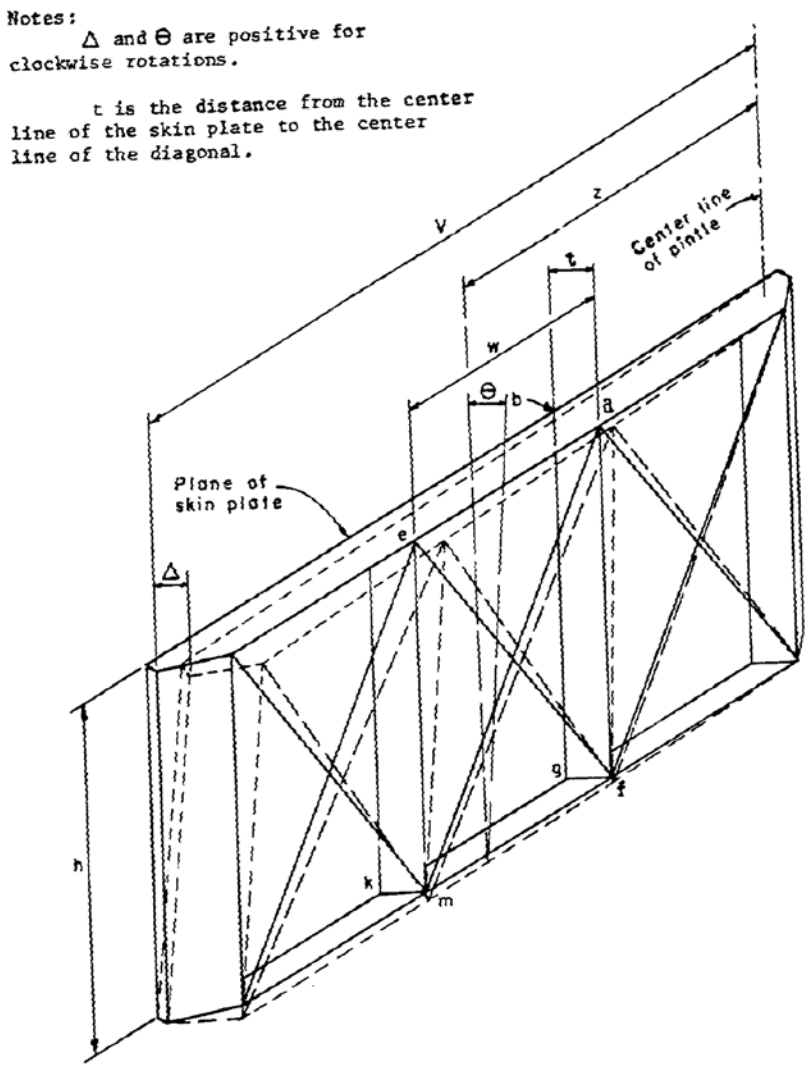



Figure A1-1. Schematic Drawing of a Typical Miter-Type Lock-Gate Leaf.

$$\begin{aligned}\delta_o &= \frac{d}{w}t\cos\alpha + \left(\frac{d}{h}t\sin\alpha\right) \\ &= \frac{dt}{w}\frac{w}{(w^2+h^2)^{1/2}} + \frac{dt}{h}\frac{w}{(w^2+h^2)^{1/2}} \\ &= \frac{2dt}{\dfrac{w}{(w^2+h^2)^{1/2}}}\end{aligned} \tag{A1-1}$$

A1.4.2. 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.

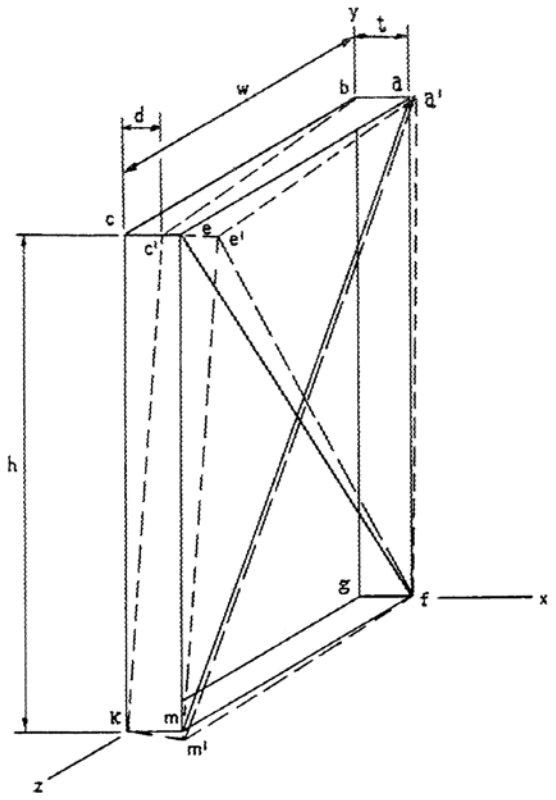



Figure A1-2. Schematic Drawing of Panel ak.

A1.4.3. 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:

$$r_o = \frac{\delta_o}{d} \tag{A1-2}$$

which, from Equation A1-1, becomes:

$$r_o = \pm \frac{2t}{(w^2 + h^2)^{1/2}} \tag{A1-3}$$

$r_0$ is positive for positive diagonals and negative for negative diagonals. Figure A1-4 illustrates the positive and negative diagonals of a typical leaf.

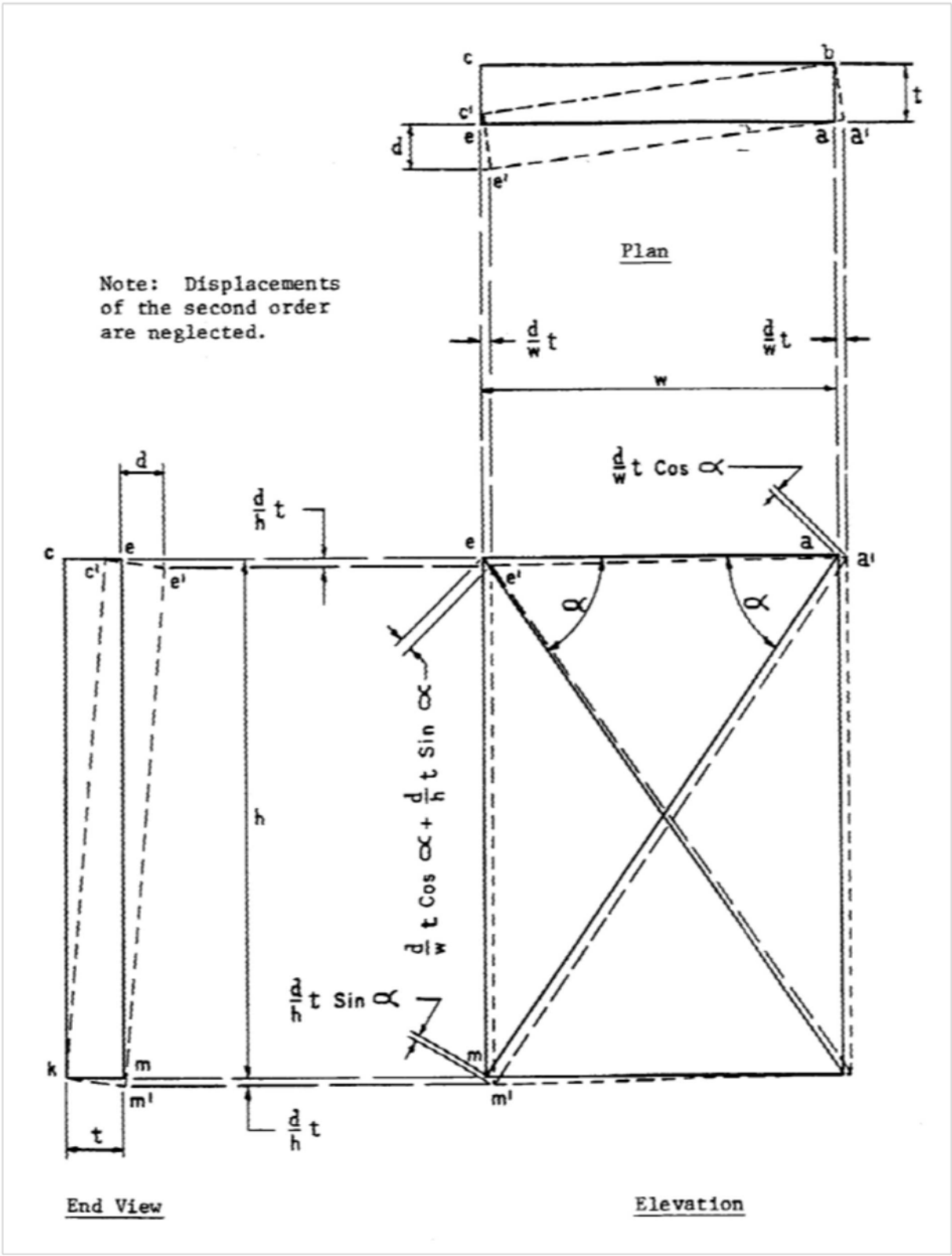



Figure A1-3. Displacements of Points of Panel ak.

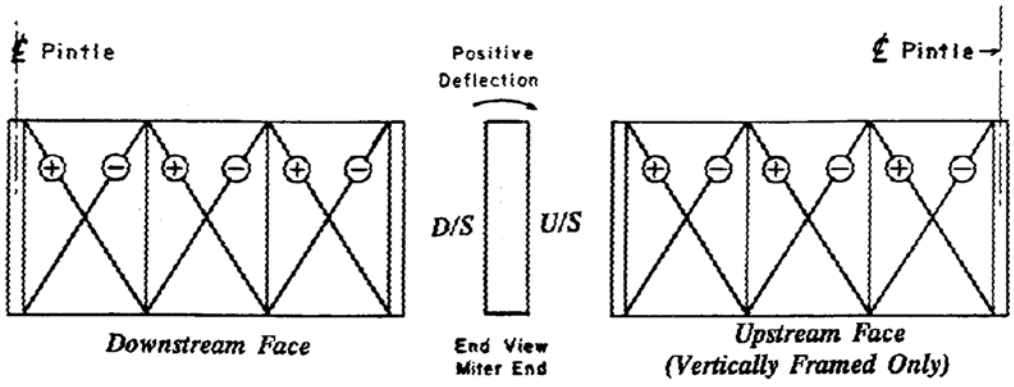



Figure A1-4. Positive and Negative Diagonals of a Typical Leaf.

A1.4.4. Diagonal restraint. Up to this point, the restraint offered by the diagonal members has not been considered. Equation A1-1 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 $\delta_0$. The actual deformation is $\delta$, which is less than $\delta_0$ by some amount $\delta'$:

$$\delta = \delta_o - \delta' \tag{A1-4}$$

It is evident that $\delta$ is inversely proportional to the resistance of the diagonal and that $\delta'$ 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:

$$\frac{\delta}{\delta_1} = \frac{A'}{A} \tag{A1-5}$$

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 A1-4 for $\delta'$ and substituting its value in Equation A1-5:

$$\frac{\delta}{\delta_o - \delta} = \frac{A'}{A} \tag{A1-6}$$

Let $r$ be defined as the ratio of the actual deformation of the diagonal to the deflection of the panel:

$$r = \frac{\delta}{d} \tag{A1-7}$$

Using Equations A1-2 and A1-7, Equation A1-6 can be written:

$$\frac{rd}{r_o d - rd} = \frac{A'}{A}$$

and solving for $r$:

$$r = \frac{A'}{A + A'} r_o \tag{A1-8}$$

(note) when the diagonal offers no restraint (that is to say that $A = o$), $r = r_0$.

Let $\Delta$ be defined as the torsional deflection of the whole leaf; see Figure A1-1. 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:

$$d = \frac{w\Delta}{v} \tag{A1-9}$$

Let $R_0$ be defined as follows:

$$R_o = \frac{\delta_o}{\Delta} \tag{A1-10}$$

Substituting the values of $\delta_0$ and $\Delta$ from Equations A1-2 and A1-9, respectively:

$$R_o = \pm\left( \frac{2wt}{v(w^2 + h^2)^{1/2}} \right)$$

which, from Equation A1-3, becomes:

$$R_o = \pm\left( \frac{2wt}{v(w^2 + h^2)^{1/2}} \right) \tag{A1-11}$$

Let $R$ be defined by:

$$R = \frac{\delta}{\Delta} \tag{A1-12}$$

Substituting in Equation A1-12 the values of $\delta$ and $\Delta$ obtained from Equations A1-7 and A1-9, respectively:

$$R = \frac{rd}{(v/w)d} = \frac{w}{v} r$$

which, from Equation A1-8 becomes:

$$R = \frac{w}{v} r_o \frac{A'}{A + A'} = R_o \frac{A'}{A + A'} \tag{A1-13}$$

A1.4.5. 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 Section A1.5 and A1.6 in this attachment). It can be seen from the definition of a positive diagonal that *D* is positive for positive diagonals and negative for negative diagonals.

A1.4.5.1. Deflection of leaf. Referring to Equation A1-12, 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:

$$\delta = R(-D) + R(\Delta) = R(\Delta - D) \tag{A1-14}$$

and similarly:

$$\delta_o = R_o(\Delta - D) \tag{A1-14a}$$

Since a positive value of δ represents a decrease in length, the elongation of a diagonal is (-δ) and the total force is:

$$S = \frac{(-\delta)EA}{L}$$

which, from Equation A1-14, becomes:

$$S = \frac{-REA}{L}(\Delta - D) \tag{A1-15}$$

If the diagonal offered no resistance to change in length, its deformation would be, from Eq. A-4, $\delta_o = \delta + \delta'$. The force in the diagonal, therefore, not only elongates 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 A1-14a, becomes:

$$W_D = \frac{1}{2} SR_o(\Delta - D) \tag{A1-15a}$$

Substituting the value of S from Equation A1-15:

$$W_D = \frac{-RR_oEA}{2L}(\Delta - D)^2 \tag{A1-16}$$

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 A1-1 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 $W_T$:

$$W_T = \frac{(\Delta - D)}{2hv} T_z \tag{A1-17}$$

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 A1-16 and A1-17, respectively, to zero and simplifying:

$$T_z - \frac{RR_oEAhv}{L}(\Delta - D) = 0$$

Let:

$$Q = \frac{RR_oEAhv}{L} \tag{A1-18}$$

Then:

$$T_z + Q(D - \Delta) = 0 \tag{A1-19}$$

Since $T_z$ is the torque-area of the external load, the quantity $Q(D\text{-}\Delta)$ 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 A1-19 may be written as follows:

$$\Sigma(T_z) - Q_o\Delta + \Sigma[Q(D-\Delta)] = 0 \tag{A1-20}$$

in which $\Sigma[Q(D\text{-}\Delta)]$ includes all diagonals of the leaf.

Since $\Delta$ is a constant for any condition of loading, Equation A1-20 may be solved for $\Delta$:

$$\Delta = \frac{\Sigma(T_z) + \Sigma Q(D)}{Q_o + \Sigma Q} \tag{A1-21}$$

which 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 A1-21 must equal zero:

$$\Sigma(T_z)_{D.L.} + \Sigma(QD) = 0 \tag{A1-22}$$

Equation A1-22 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 A1-21 may be written:

$$\Delta = \frac{\Sigma(T_z)_{L.L.} + \Sigma(T_z)_{D.L.} + \Sigma(QD)}{Q_o + \Sigma Q}$$

But if Equation A1-22 is satisfied, $\Sigma(T_Z)_{D.L.} + \Sigma\,(QD) = 0$,

therefore:

$$\Delta = \frac{\Sigma(T_z)_{L.L.}}{Q_o + \Sigma Q} \tag{A1-23}$$

which is the fundamental equation for deflection of a leaf with all diagonals prestressed. Equation A1-23 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 A1-15 by A:

$$s = \frac{RE}{L}(D - \Delta) \tag{A1-24}$$

which is the fundamental equation for unit stress in a diagonal.

A1.4.5.2. Maximum numerical value of *D*. If the maximum allowable unit stress is substituted for s in Equation A1-24, the maximum allowable numerical value of (*D*-Δ) will be obtained. Since the maximum values of Δ are known from Equation A1-23, the maximum numerical value of *D* for any diagonal can be determined.

*A1.4.5.3.* Minimum numerical value of *D*. The diagonals of a gate leaf should be prestressed so that all of them are always in tension (see paragraph A1.4.10). If this is to be so, the quantity (*D*-Δ) 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*.

A1.4.5.4. Values of *D*. Values of *D* shall be selected such that they satisfy Equation A1-22 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.

A1.4.6. 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 Equations A1-15a and A-17:

$$\frac{1}{2}SR_o(\Delta - D) = -\frac{(\Delta - D)}{2hv}T_z$$

Treating $R_o$ as equal for all diagonals, substituting $sA$ for $S$, and taking Σ for all diagonals in a set:

$$\sum A = -\frac{\Sigma T_z}{R_o hv} \qquad \text{(A1-25)}$$

With the above, the maximum positive $\Sigma T_z$ will give the total area required in the set of negative diagonals and the maximum negative $\Sigma T_z$, the area for the positive diagonals.

A1.4.7. 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 A1.6.8shows 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:

$$\frac{\text{Greater dimension}}{\text{Lesser dimension}} = (2)^{1/2}$$

A1.4.8. 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 center line of the diagonals, and the side boundaries of the panel.

A1.4.9. Discussion.

A1.4.9.1. 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:

$$R = \frac{A'}{A + A'} R_o \qquad \text{(from Eq. A1-13)}$$

$$Q = \frac{RR_oEAhv}{L} + \frac{R_o 2Ahv}{L} \times \frac{A'}{A + A'} \qquad \text{(from Eq. A1-18)}$$

1. 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, $\delta$ was determined from strain gage readings on the diagonal and $\Delta$ was measured directly as the leaf was twisted. Equation A1-12 gave the value of $R$. $A'$ was then calculated from Equation A1-12 in which the theoretical value of $R_o$ obtained from Equation A1-11 was substituted.* Values of $A'$ obtained are:

Sault Ste. Marie $A'$ = 0.025 in.$^2$ (model)
= 0.025 x $(32)^2$ = 26 in.$^2$ (prototype)
Louisville = 13 in.$^2$
Mississippi River Gates = 10 in.$^2$

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.

3. 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

* 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.

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 A1-24 where an error in $A'$ produces an error $R$, which is opposite to that produced in $(D - \Delta)$. Thus, stress is nearly independent of $A'$.

A1.4.9.2. 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 $Q_o$.

1. When a leaf twists, the horizontal and vertical members rotate through angles of $\Delta/h$ and $\Delta/v$, respectively. The work done in any member is:

$$W = \frac{1}{2}\frac{E_s J}{v}\frac{(\Delta)^2}{h^2}, \text{ for horizontal members}$$

$$W = \frac{1}{2}\frac{E_s J}{h}\frac{(\Delta)^2}{v^2}, \text{ for vertical members}$$

$E_s$ = shearing modulus of elasticity

$J$ = modified polar moment of inertia.

The work done by an external torque is, from Equation A1-17

$$W_T = \frac{\Delta}{2hv} T_z$$

In this case, the value of $D$ in Equation A1-17 is zero since the members are not supplying a resisting torque when the deflection is zero. Equating $W_T$ to $W$ and solving $T_z$:

$$\mathrm{Tz} = \frac{E_S J}{H}\Delta, \text{ for horizontal members}$$

$$\mathrm{Tz} = \frac{E_S J}{H}\Delta, \text{ for vertical members}$$

The quantities $E_sJ/h$ and $E_sJ/v$ might be called the values of $Q_o$ for horizontal and vertical members, respectively, hence:

$$Q_o = K\, E_s\, \Sigma\,(J/h + J/V) \qquad \text{(A1-26)}$$

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.

2. Nearly all members of a leaf subject to torsion are made up of narrow rectangles. For these, the value of $J$ is:

$$\frac{\Sigma\, l\,(3)^3}{3}$$

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.

3. Using Equation A1-26, $Q_o$ 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 dead weight. $Q_o$ 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.

A1.4.9.3. 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 A1.4.11. 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.* 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 paragraphs A1.4.2 and A1.8, respectively.

A1.4.9.4. Skin plate consisting of buckle plates. The theory is based on the assumption 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 within the restrictions imposed on $D$ (see paragraph A1.4.5). The reader is referred to paragraph A1.6, Example 2.

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* The operating strut mechanism should also then be designed for these larger forces.

A1.4.10. Methods for prestressing diagonals. It is essential that all diagonals be prestressed. With all diagonals prestressed, none will ever alternately bow out and then snap back into position during operation of the leaf. It is certain that this buckling was responsible for some of the failures of diagonals that occurred in the past. Prestressing also reduces the torsional deflection of the leaf to a minimum, since all diagonals are always acting. 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. In the other, the sleeve nut on the diagonal is turned a precomputed amount. Caution should be taken when using the twist of the leaf method where the leaf has top and bottom torque tubes. Due to the increased leaf stiffness, there is the 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 damage such as localized buckling of plates, excessive deflection in the quoin post, and damage to the grease seals, pintle, and pintle socket, etc. These two methods are discussed in the following paragraphs.

A1.4.10.1. 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 tying the miter end to the sill or 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 *D* for one set of diagonals and the slack is removed from this set. During this operation, the other set of diagonals must be maintained slack. The leaf is then twisted in the opposite direction the computed *D* for the other set of diagonals, and the slack is removed from them (see paragraph A1.6, Example 2). 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 he felt. As a further precaution, a strain gage is recommended on the diagonal being tightened. The maintained deflection of the leaf should also be watched, since more than a slight tension in the diagonal will cause a change in deflection of the leaf. On existing gates in which the diagonals were not designed by this method, it may be necessary to overstress some diagonals during the prestress operation. For this one-time load, a stress of $0.67F_y$ is considered permissible where $F_y$ is the yield strength of the diagonal material. The prestressing force required (normal to the leaf, at the upper miter corner) is obtained from Equation A1-21 as:

$$p = \frac{\Delta(Q_o = \Sigma Q) - \Sigma Q - G\Sigma T_z)\ D.L.}{hv}$$

where $Q$ includes only the active diagonals (see paragraph A1.6.8 Example 2).

A1.4.10.2. Turn-of-the-nut-method. In this method, it is essential that the nut be very well lubricated with a heavy lubricant. Initially, all diagonals must be slack and, during the prestressing operation, each diagonal must be maintained slack until it is reached in the prestressing sequence. Then the slack is removed from the first diagonal to be prestressed and the diagonal is clamped to the leaf, as close to both ends of the nut as possible, to prevent twisting of the diagonal during the nut-turning operation. The clamping should restrain twisting of the diagonal without preventing elongation of the full length. In removing the slack, the same precautions should be observed as in the previous method. The nut is then turned the precomputed *N* for the diagonal. This procedure is repeated for each succeeding diagonal. (See paragraph A1.5, Example 1) The large torque required to fully tighten the nut can be provided by a mechanically supplied force at the end of a long wrench. The nut must be turned to shorten the diagonal an amount = $R_o$ ($D$-$\Delta$). Therefore, if $n$ is the number of threads per inch, the number of turns required is:

$$N = \frac{nR_o(D-\Delta)}{2} \tag{A1-27}$$

in which Δ is the initial deflection measured in the field. From textbooks on machine design, the torque $M$ required to turn the nut to obtain the desired prestress, $sA$, is:

$$M = sA\tan(\theta+\alpha)d$$

where $d$ is the pitch diameter of the threads. θ is the friction angle, which from tests, may be taken equal to $\tan^{-1}(0.15) = 8°30'$, and α is the helix angle, which, within the size range that would be used on diagonals, may be taken as a constant angle of 1°30'. Further the maximum unit stress s is given by Equation A1-24.

Therefore:

$$M = 0.18sAd = \frac{0.18\,REAd(D-\Delta)}{L} \tag{A1-28}$$

in which Δ is determined from Equation A1-21, with only the active diagonals included.

A1.4.10.3. Comparison of methods. The twist-of-the-leaf method has been used, with excellent results, considerably more than the turn-of-the-nut method. While the turn-of-the-nut method appears to have some merit, such as reduction in setup time, the elimination of overstressing any diagonal during prestressing, and the elimination of strain gages, this method is not recommended due to the difficulties encountered during prestressing. The diagonal bar tends to twist and it is extremely difficult to provide sufficient torque to the sleeve nut or turnbuckle without first deflecting the leaf. The turn-of-the-nut method is included for information but for normal installations the twist-of-the-leaf method should be used.

A1.4.11. 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.

A1.4.11.1. 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:

$$Y = \frac{\Sigma(I_n Y)}{\Sigma I_n} \tag{A1-29}$$

in which $I_n$, is the moment of inertia of any horizontal girder about its vertical centroidal axis.

A1.4.11.2. 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.

1. A shear diagram with arrows to indicate the direction of the shear is shown in Figure A1-5. 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.

2. The shear diagram for the web of the right-hand part of the top girder is redrawn to a larger scale in Figure A1-6. 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 A1-6. 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:

$$T = \Sigma\frac{Vay}{I}by_n = \frac{V}{I}\Sigma(ayby_n)$$

If the leaf is not to twist, the sum of the moments of the vertical and horizontal forces must equal zero:

$$VX + \frac{V}{I}\Sigma(ayby_n) = 0$$

and solving:

$$X = -\left[\frac{\Sigma(ayby_n)}{I}\right] \qquad \text{(A1-30)}$$

which is the horizontal distance from the center line 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.

3. Equations A1-29 and A1-30 are general expressions, independent of the number of horizontal girders, and as such apply equally well to horizontally framed gates.

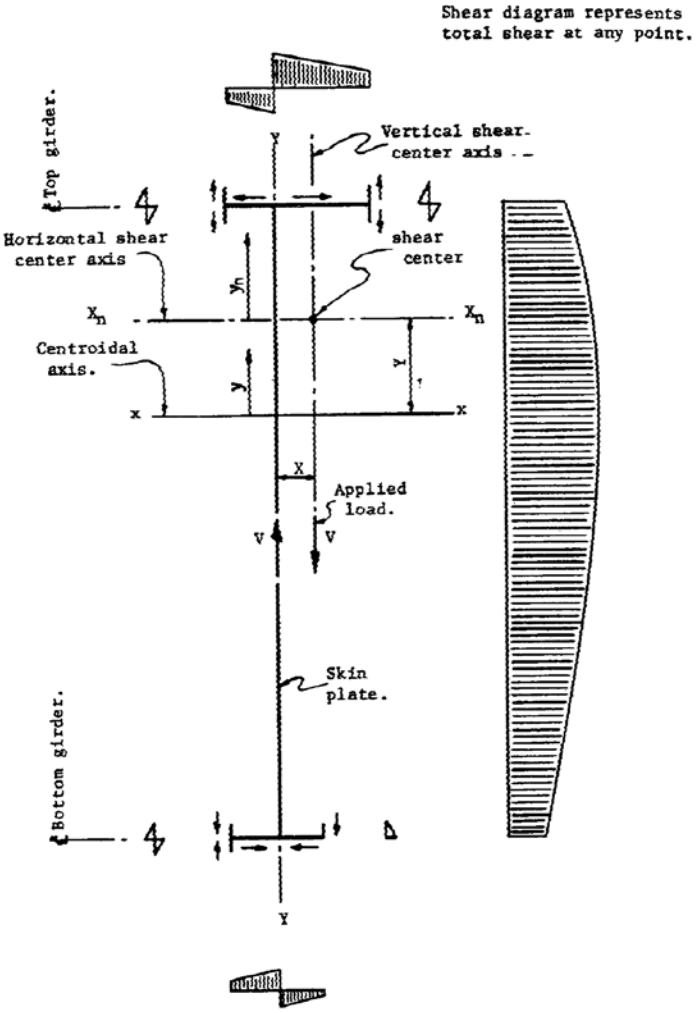



Figure A1-5. Shear Diagram for Typical Vertically Framed Lock-Gate Leaf.

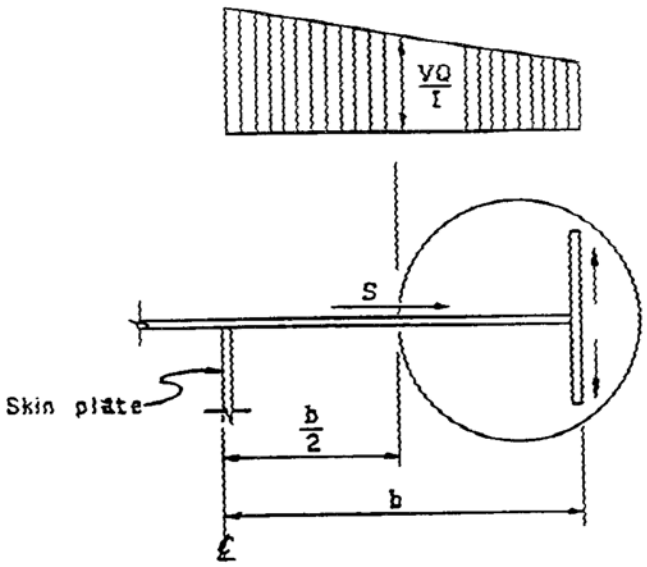



Figure A1-6. Shear Diagram for Web of the Right-Hand Part of the Top Girder.

A1.5. Example 1, Horizontally Framed Gate

Lower operating gates, MacArthur Lock, Sault Ste. Marie (see Figure A1-7).

A1.5.1. 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 A1.4.9.1):

$$A' = 1/8\ (4 \times 44.16) = 22\ \text{in.}^2$$

A1.5.2. Evaluation of $Q_o$ (see paragraph A1.4.9.2and Table A1-1).

$$Q_o = K\ E_s\ \Sigma(j/h + j/v)$$

$$Q_o = 4 \times 12 \times 10^6 \frac{4320.0}{3 \times 684.0} + \frac{590}{3 \times 529}$$

$$= 120.0 \times 10^6\ \text{in.lb.} \qquad \text{(from Eq. A1-26)}$$

A1.5.3. Location of shear center (see Figure A1-5). Computations for the centroidal axis and moment of inertia of the vertical section through the leaf (see Figure A1-7) are not given. Tables A1-2 and A1-3, respectively list computations of distances $x$ and $y$:

$$Y = 310\ \text{in.}\ I = 42.6\ x\ 106\ \text{in.}^4$$

Horizontal shear center axis:

$$Y = \frac{\Sigma(I_n y)}{\Sigma I_n} = \frac{-1.61 \times 10^6}{162{,}000} = -10.0\ \text{in.} \qquad \text{(From Eq. A1-29)}$$

Vertical shear center axis:

The value of $b$ for all girders is -36.1 in.

$$X = \frac{b}{I}\Sigma(ayy_n) = -\left(\frac{-36.1}{42.6\times10^6}\right)\times13.54\times10^6 = 11.4 \text{ in.} \quad \text{(from Eq. A1-30)}$$

A1.5.4. Load torque areas (see paragraph A1.4.9.3). The forces that produce twisting of the leaf are shown in Figure A1-8. Table A1-4 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.

A1.5.5. Evaluation of $R_o$, R, and Q.

$$R_o = \pm\frac{2wt}{v(w^2+h^2)^{1/2}} = \pm\frac{2\times483\times37.8}{529(483^2+684^2)^{1/2}} = \pm0.0822$$

Required size of diagonals:

For diagonal $U_oL_1$,

$$A = -\Sigma\frac{T_z}{sR_ohv} = -\left(\frac{-11{,}570\times10^6}{18{,}000\times0.0822\times684\times529}\right) = 21.5 \text{ in.}^2 \quad \text{(from Eq. A1-25)}$$

For diagonal $L_oU_1$,

$$A = -\left(\frac{9{,}200\times10^6}{18{,}000\times0.0822\times684\times529}\right) = 17.1 \text{ in.}^2$$

For diagonal $L_oU_1$, the dead load torque is not now 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:

$$R = \frac{A'}{A+A'}R_o = \pm\frac{22}{A+22}\times0.822 \quad \text{(from Eq. A1-13)}$$

$$Q = \frac{RR_oEAhv}{L} = \frac{R\times0.0822\times29\times10^6\times A\times684\times529}{771} = 112\times10^7\times RA$$

Table A1-5 lists computation of the constant $Q$.

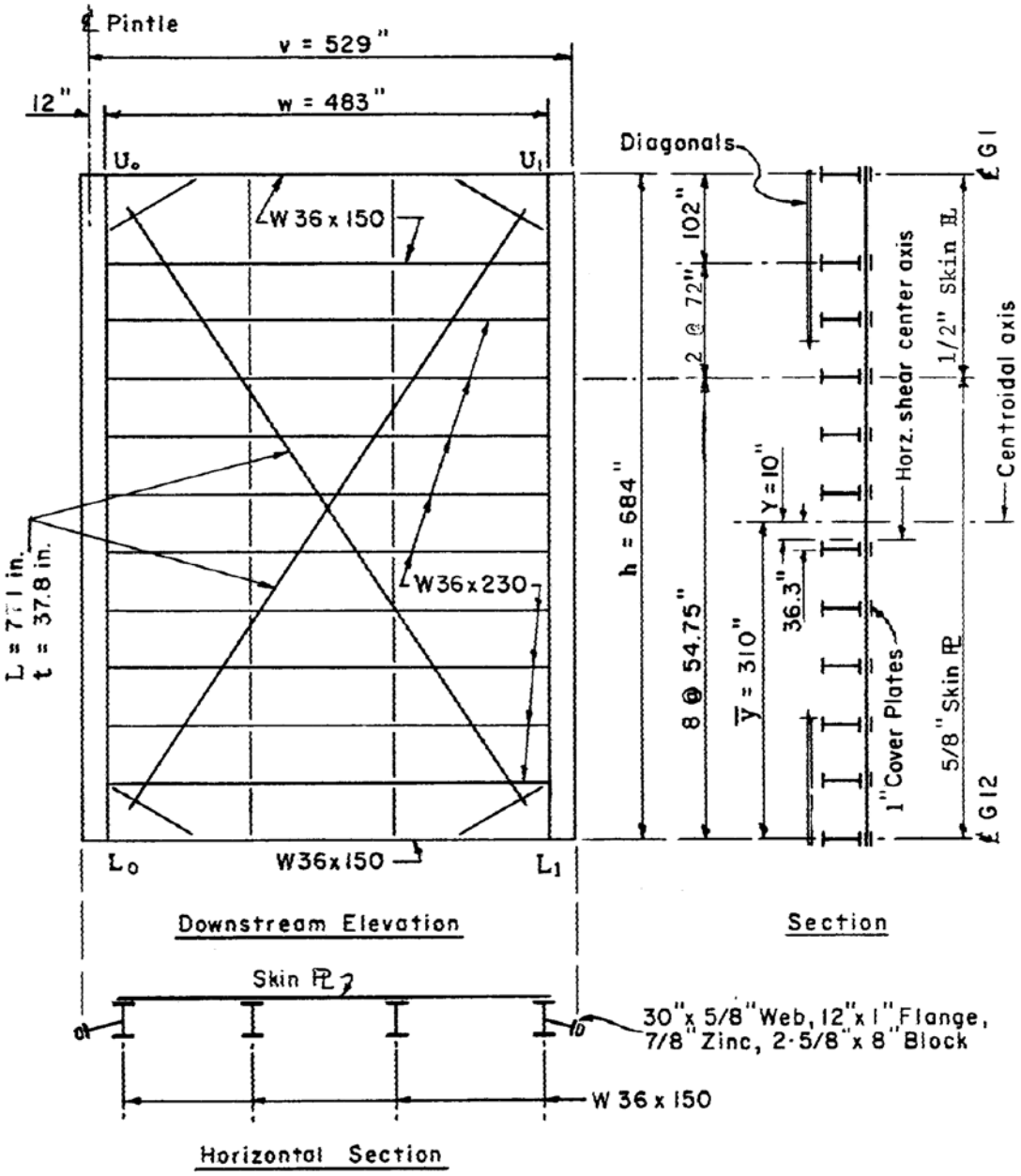



Figure A1-7. Lower Gate Leaf, MacArthur Lock, Sault Ste. Marie.

Table A1-1. 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 | 9 | 16.5 | 1.26 | 300.0 | — |
| through G11) | | | | | |
| Skin (between flanges) | | | | | |
| ½—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 A1-2. Computation of Distance Y.

| Girder | Ln (in.$^4$) | y (in.) | $l_{n.y}$ (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 A1-3. 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 |

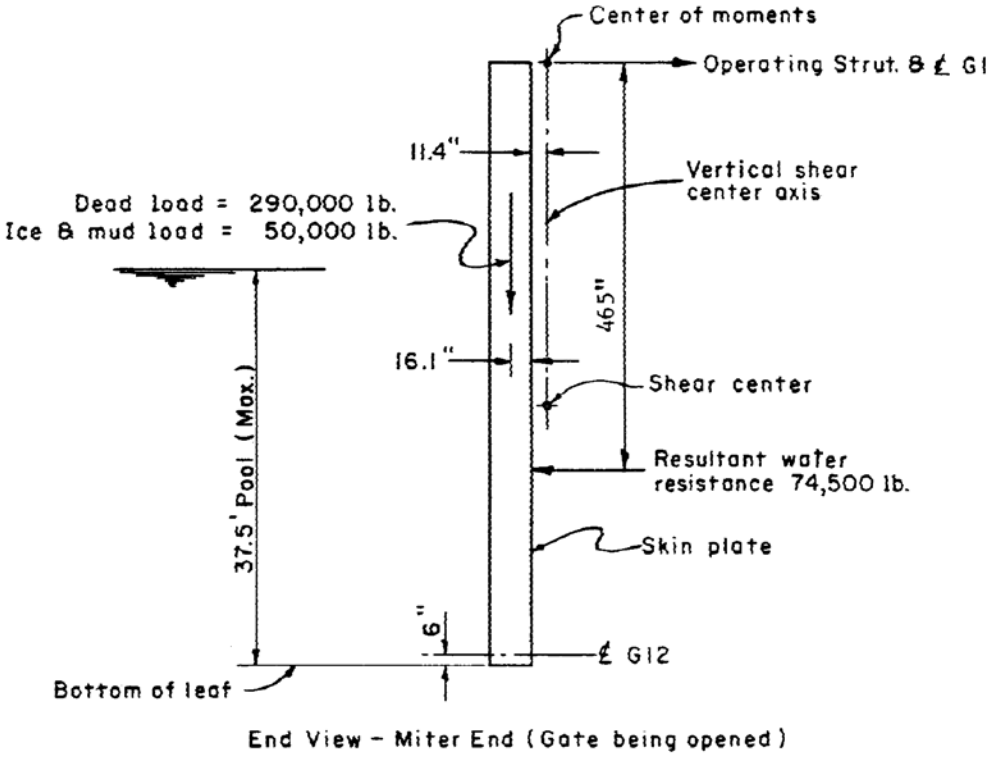



Figure A1-8. Forces Acting on Leaf Being Opened.

Table A1-4. 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 A1-5. 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 |

A1.5.6. Deflection of leaf.

$$\text{Gate opening } \Delta = \frac{\Sigma T_z}{Q_o + \Sigma\Theta} = \frac{9{,}200 \times 10^6}{(120 + 1{,}960) \times 10^6} = 4.4 \qquad \text{(from Eq. A1-23)}$$

$$\text{Gate closing } \Delta = \frac{(-9{,}200 - 350) \times 10^6}{(120 + 1{,}960) \times 10^6} = -4.6 \frac{(-9{,}200 - 350) \times 10^6}{(120 + 1{,}960) \times 10^6} = -4.6$$

A1.5.7. Prestressed deflections and stresses in diagonals. Table A1-6 lists prestress deflections. The minimum numerical values of *D* (line 3) are the maximum deflections of the leaf. Maximum numerical values of (*D* - Δ) are found by solving Equation A1-24.

$$(D - \Delta) = \frac{sL}{RE} = \frac{18{,}000 \times 771}{R \times 29 \times 10^6} = \frac{0.478}{R}$$

Having the maximum numerical values of (*D* - Δ), 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 A1-22 is satisfied that is, Σ(*QD*) must equal +2,020 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

$$s = \frac{RE}{L}(D - \Delta) \qquad \text{(from Eq. A1-24)}$$

and are placed in lines 8, 9, and 10.

From Table A1-6, it is seen that the diagonal sizes chosen are quite satisfactory.

Table A1-6. Stresses in Diagonals during Normal Operation.

| Line | Positive Diagonal Parameter | Negative Diagonal $U_oL_1$ | $L_oU_1$ |
|---|---|---|---|
| 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-i) (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 | QO (in .2-lb. x 106) | +7,030 | -5,000 |
| | | Σ(QD) = 2,030 x $10^6$ 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 |

A1.5.8. Method of prestressing. The turn-of-the-nut method will be used. After the diagonals are made slack, the deflection of the leaf is measured in the field. Since this actual initial deflection is unknown at this time, the theoretical value will be used (with diagonals slack $Q$ - zero):

$$\Delta = \frac{\Sigma(T_z) + \Sigma Q(D)}{Q_o + \Sigma Q} = \frac{\Sigma T_z}{Q_o} = \frac{-2{,}020 \times 10^6}{120 \times 10^6} = -16.8 \text{ in.} \quad \text{(from Eq. A1-21)}$$

A1.5.8.1. Diagonal $U_oL_1$. The slack is removed from this diagonal only and the diagonal is clamped. The required prestress is then obtained by tightening the sleeve nut the following number of turns:

$$N = \frac{nR_o(D - \Delta)}{2} = \frac{2.5 \times 0.0822}{2}[+6.7 - (-16.8)] = 2.41 \text{ turns} \quad \text{(from Eq. A1-27)}$$

The torque required to accomplish this is found from Equation A1-28 after determining the resulting leaf deflection from:

$$\Delta = \frac{\Sigma T_z + \Sigma Q_D}{Q_o + \Sigma Q} = \frac{(-2{,}020 + 1{,}050 \times 6.7) \times 10^6}{(120 + 1{,}050) \times 10^6} = 4.4 \text{ in.} \quad \text{(from Eq. A1-21)}$$

$$M = \frac{0.18 REA_d(D - \Delta)}{L}$$

$$= \frac{0.18 \times 0.0393 \times 29 \times 10^6 \times 12 \times 4.75(6.7 - 4.4)}{771} = 35{,}000 \text{ in.-lb} \quad \text{(from Eq. A1-28)}$$

or 490 lb required at the end of a 6-ft wrench. In this option, it is assumed that both members of diagonal $U_oL$ are prestressed simultaneously.

A1.5.8.2. Diagonal $L_oU_1$. The theoretical initial deflection of the leaf for this diagonal is the final leaf deflection of 4.4 in. after prestressing the previous diagonal. To prestress this diagonal the required amount, it is necessary to tighten the nut through the following turns, after first removing the slack:

$$N = \frac{2.75(-0.0822)(-5.5 - 4.4)}{2} = 1.12 \text{ turns} \quad \text{(from Eq. A1-27)}$$

This tightening will make the leaf plumb ($\Delta = 0$) and will require a maximum torque of:

$$M = \frac{0.18\,(-0.0452) \times 29 \times 10^6 \times 9 \times 4.25(-5.5 - 0)}{771} = 64.000 \text{ in.-lb} \qquad \text{(from Eq. A1-28)}$$

or 900 lb required at the end of a 6-ft wrench.

A1.5.8.3. 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 A1-21 with $\Delta$ equal and opposite to the out-of-plumb deflection. This prestress deflection can then be substituted in Equation A1-27 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 in., the corrected prestress deflections would be found from $\Sigma QD = (\Delta Q_o + \Sigma Q) - (T_Z)D.L.$ to be 980 in.$^2$1b x $10^6$. With $D$ for diagonal $L_oU_1$ maintained at -5.5 in., the $D$ then required for diagonal $U_oL_1$ would be +5.7 in. and $N$ for this diagonal would become 2.30 turns. The remainder of the computations would be repeated.

A1.6. Example 2, Vertically Framed Gate

See Figures A1-9 and A1-10.

A1.6.1. Evaluation of A'. The cross-sectional area of the bottom girder (see Figure A1-10) is 36.7 in$^2$, the cross-sectional area of any vertical girder is 37.0 in.$^2$, (see Figure A1-9), 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:

$$A' = (1/20)\,(36.7 + 74.0 + 112.5) = 11.0 \text{ in.}^2$$

A1.6.2. Evaluation of $R_o$, R, and Q. Since this is an existing lock, the diagonal sizes are fixed.

$$R_o = \pm\frac{2wt}{v(w^2 + h^2)^{1/2}} = \pm\frac{2 \times 232t}{723(232^2 + 535^2)^{1/2}} = \pm 000110t \qquad \text{(from Eq. A1-11)}$$

$$R = \frac{A'}{A + A'}R_o = \pm 0.0121\frac{t}{(A + 11)} = \frac{11}{(A + 11)}R_o \qquad \text{(from Eq. A1-13)}$$

$$Q = \frac{RR_oEAhv}{L} = \frac{RR_o \times 29 \times 10^6 \times A \times 535 \times 723}{471} = 238 \times 10^8 \times RR_oA \qquad \text{(from Eq. A1-18)}$$

Table A1-7 lists computation of the elasticity constant $Q$.

Table A1-7. 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

A1.6.2.1. 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 A1-9 (see paragraph A1.4.8 for the determination of $t$ for skin not in a plane).

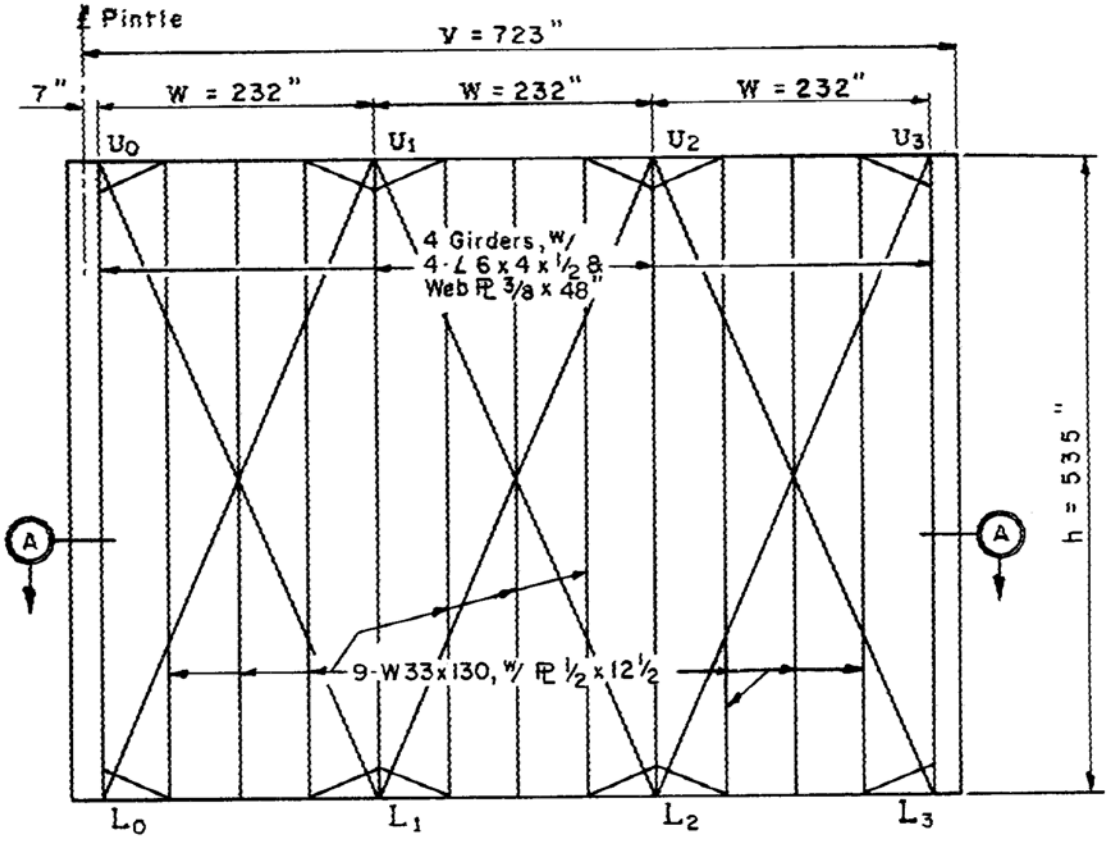
A1.6.2.2. This example provides a good illustration of the inefficiency of past designs. The upstream diagonals are quite ineffective because they are so close to the skin plate. If all the upstream diagonals were omitted (in other words, the number of diagonals cut in hail) and the skin plate placed in their location instead, the leaf would he 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).

A1.6.3. Evaluation of $Q_0$ (see paragraph A1.4.9.2 and Table A1-8).

$$Q_o = K \times E_s \times \Sigma(J/h + J/v)$$
$$= 4 \times 12 \times 10^6 \left( \frac{310}{3 \times 535} + \frac{700}{3 \times 723} \right) = 25 \times 10^6 \text{ in.-lb} \quad \text{(from Eq. A1-26)}$$

Table A1-8. Computation of Modified Polar Moment of Inertia J.

| Elements | No. of Elements | 1 (in.) | c (in.) | Horizontal Elements | Vertical Members |
|---|---|---|---|---|---|
| Horizontal Girders | | | | | |
| U/S flange, | 1 | 18.0 | 2.38 | 240 | |
| Web, (Top) | 1 | 72.0 | 0.50 | 10 | |
| D/S Flange, | 2 | 14.0 | 0.88 | 20 | |
| U/S flange, | 1 | 12.0 | 0.50 | 0 | |
| Web, (Bottom) | 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 | 9 | 11.5 | 1.73 | | 540 |
| US flange | 9 | 31.4 | 0.58 | | 60 |
| Web | 9 | 11.5 | 0.86 | | 70 |
| DIS flange | | | | | |
| | | | Total = | 310 | 700 |




Diagonals on both US and DS faces.
Pin-to-pin length of all diagonals is 471 in.

Downstream Elevation

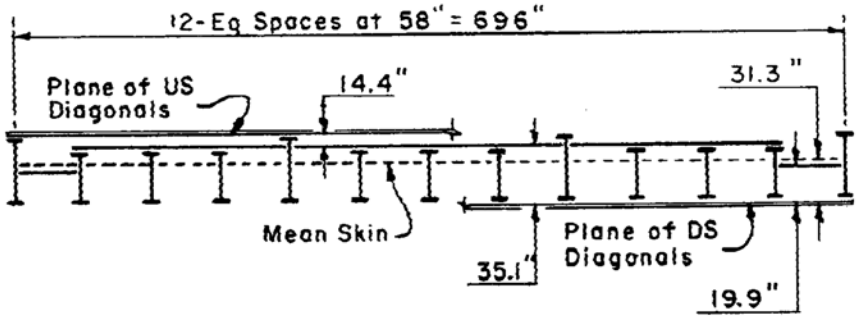



Figure A1-9. Schematic Drawing of a Vertically Framed Leaf.

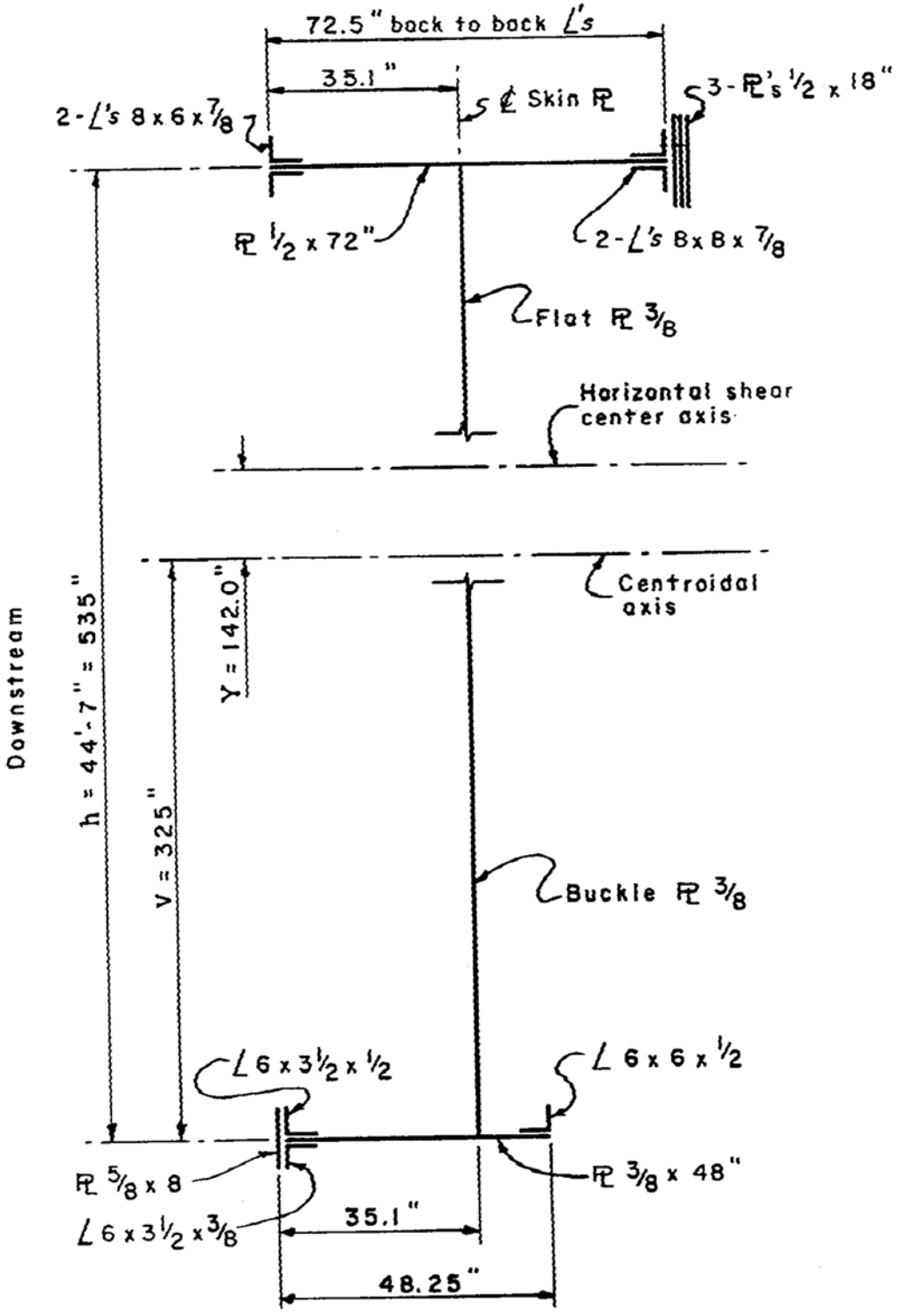



Figure A1-10. Average Vertical Section through Leaf.

A1.6.4. Location of shear center (see Equation A1-26). Computations for the centroidal axis and the moment of inertia of the vertical section through the leaf are not shown (see Figure A1-9).

$y$ = 325 in.

$I$ = 14.3 x $10^6$in.$^4$

Horizontal shear center axis:

Moment of inertia of Top girder = 84,100 in $^4$

$$Y = \frac{\sum(I_n \times y)}{\sum I_n} = \frac{84{,}100 \times 210 - 12{,}100 \times 325}{96{,}200} = +142 \qquad \text{(from Eq. A1-29)}$$

Vertical shear center axis:

Table A1-9 lists computation of the distance $X$.

$$X = -\left[\frac{\sum(ayby_n)}{I}\right] = -\left(\frac{-69.9 \times 10^6}{14.3 \times 10^6}\right) = +4.9 \text{ in.} \qquad \text{(from Eq. A1-30)}$$

Table A1-9. 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 |

A1.6.5. Load torque areas (see discussion in paragraph A1.4.9.3). The forces that produce twisting of the leaf are shown in Figure A1-11. 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 106 in.$^2$-lb

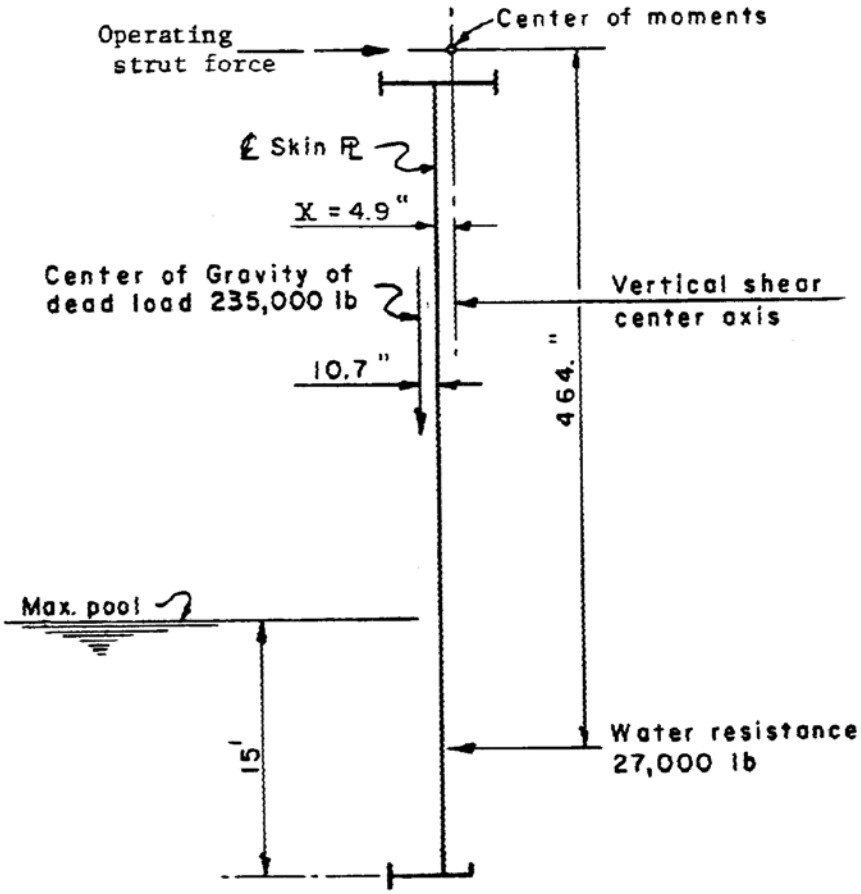



Figure A1-11. Torsional Forces on Leaf.

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:

$$\Delta = \frac{\Sigma T_z}{Q_o + \Sigma Q} = \frac{\pm 4{,}530 \times 10^6}{(25+895) \times 10^6} = \pm 4.9 \text{ in.} \qquad \text{(from Eq. A1-23)}$$

where positive value is for gate opening.

A1.6.6. Prestress deflections and stresses in diagonals. Table A1-10 lists the prestress deflections. The minimum numerical values of $D$ (column 4) are the maximum deflections of the leaf. Maximum numerical values of ($D$ - Δ) are found by solving Equation A1-24:

$$(D-\Delta)\max = \frac{sL}{RE} = \frac{18{,}000 \times 471}{R \times 29 \times 10^6} = \frac{0.292}{R}$$

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 A1-22 is satisfied; that is, $\Sigma QD$ 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 A1.4.9.4), 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:

$$s = \frac{RE}{L}(D-\Delta) \qquad \text{(from Eq. A1-24)}$$

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.

A1.6.7. 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 A1-11 lists the dead load shear remaining in the skin (buckle plates).

A1.6.8. Method of prestressing. The twist-of-the-leaf method will be used as outlined in paragraph A1.4.10.1. The maximum force will be required when the leaf is deflected +10.0 in. 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 A1-21 with $\Delta$ 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 in., revised values of $D$ would be selected to make $\Sigma QD$ equal to +840 x $10^6$ in.$^2$-lb. The leaf would then hang plumb. Repeat computations, if necessary.

Table A1-10. Computation of Diagonal Stresses.

| 1 | DIAGONAL | | POS. DIAGONALS | | | | | | NEG. DIAGONALS | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 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<br>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<br>$IN.^2$ - LB. X $10^6$ | | +1500 | +1240 | +590 | | +600 | | -830 | | -140 | | -1,650 | |
| 9 | STRESS, LB / SQ. IN. | DURING PRESTRESSING<br>Δ = + 10.0 IN. | PRESTRESSED LAST | | | | | | 13,400 | 11,500 | 12,500 | 23,900 | 26,400 a | 23,600 |
| 10 | | GATE STATIONARY<br>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 | | GATE BEING OPENED<br>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 | | GATE BEING CLOSED<br>Δ = -4.9 IN. | 16,700 | 17,100 | 17,100 | 10,900 | 8,500 | 10,700 | 4,300 | 3,700 | 300 | 600 | 600 | 600 |

QD = +1,310

a TOO HIGH, BUT INHERENT IN THIS EXISTING GATE.

Table A1-11. 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 |
|---|---|---|---|---|---|---|---|
| | $DSU_0L_1$ | 10.0 | 11,200 | +112,000 | | | |
| 0-1 | $USU_0L_1$ | 10.0 | 7,300 | + 73,000 | | | |
| | $USL_0U_1$ | 4.5 | 6,600 | - 29,000 | +119,000 lb | - 196,000 lb | +77000 lb |
| | $DSL_0U_1$ | 4.5 | 8,300 | - 37,000 | | | |
| | $DSU_1L_2$ | 8.0 | 10,300 | + 82,000 | | | |
| 1-2 | $USU_1L_2$ | 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 | | | |
| | $DSU_2L_3$ | 4.5 | 9,800 | + 44,000 | | | |
| 2-3 | $USU_2L_3$ | 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 | | | |

A1.7. 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 A1-12 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 Equation A1-11:

$$R_o = \pm\left(\frac{2w \cdot h \cdot t}{H \cdot v \cdot (w^2+h^2)^{\frac{1}{2}}}\right)$$

This value of $R_0$ replaces that given in Equation A1-11, being a more general expression. It is seen that for a value of $h = H$ (no vertical paneling) the derivative from Equation A1-11 (p C-79) reverts to Equation A1-11. With the above value of $R_0$, all the other expressions and the method of analysis remain identical to that previously outlined.

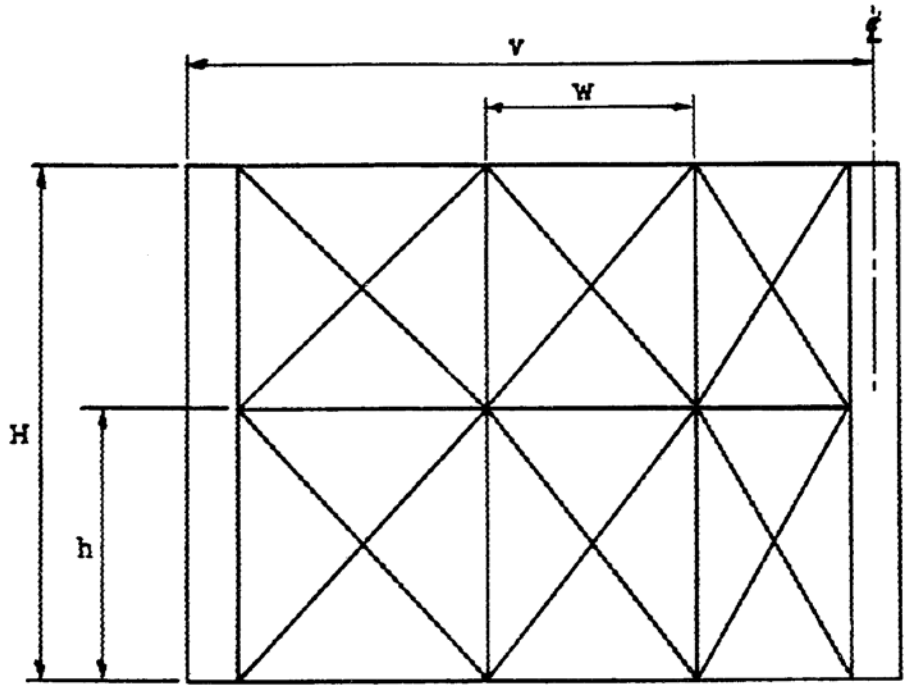



Figure A1-12. Vertical and Longitudinal Arrangement of Leaf Panels.

A1.8. Derivation of Equation A-11 (p C-79)

The general value of $R_0$ can be found as follows (refer to paragraph A1.4.4). Let $d$ = deflection of panel; other symbols are as defined previously. Figure A1-13 illustrates the displacements of points of a vertical divided panel.

Let $\delta_0$ = 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 A1-13)

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 Eq. A1-11)

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 $Q_0$ (Equations A1-18 and A1-26, respectively) still apply with $H$ substituted for $h$.

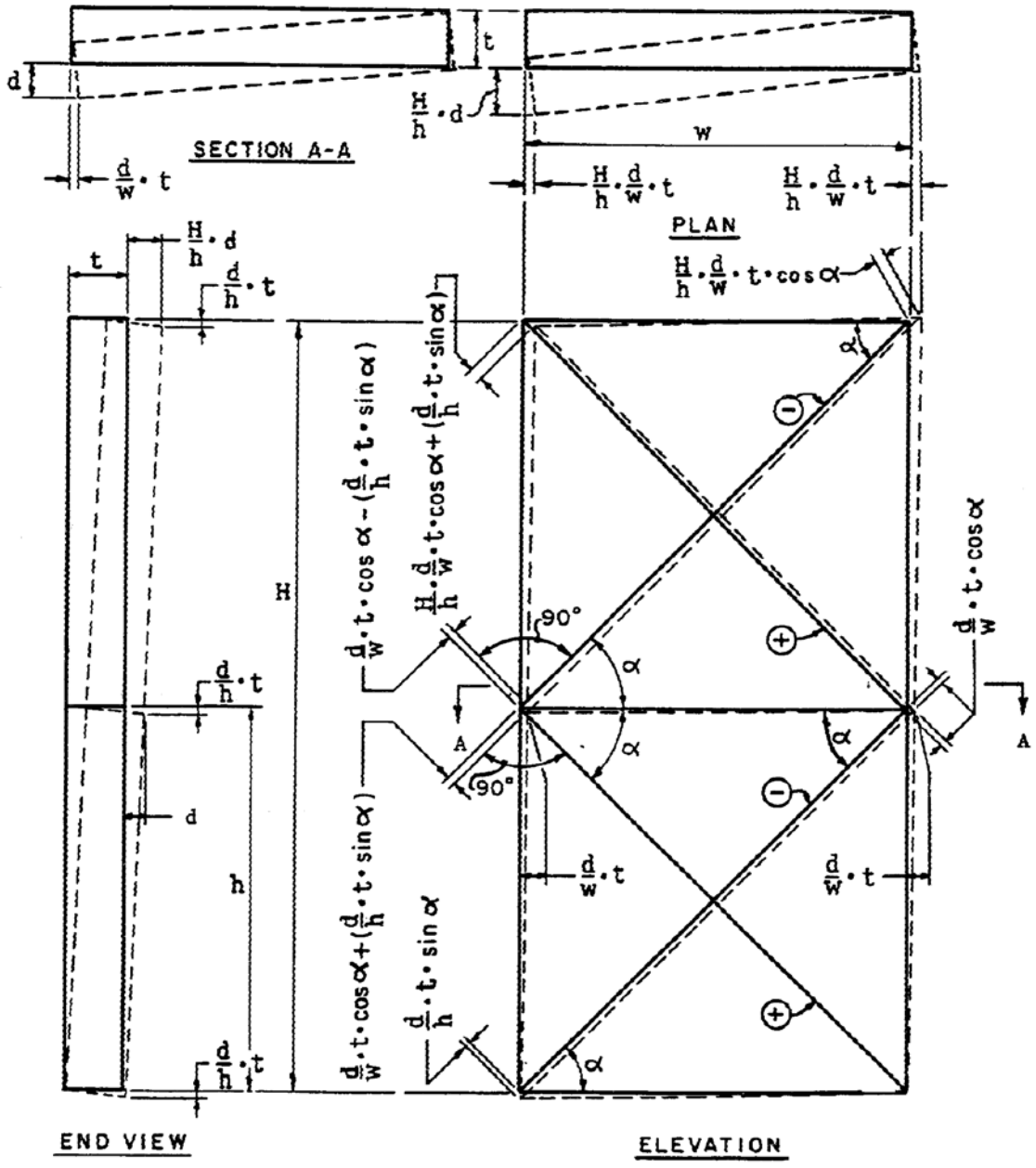



Figure A1-13. Displacement of Points of a Vertical Divided Panel.

A1.9. Temporal Hydraulic Loads

The effect of temporal hydraulic loads on the miter gate diagonal design will be evaluated at each lock with appropriate conditions selected for the design. A minimum temporal hydraulic load of 1.25 ft (with a period exceeding 30 sec) will be used for gate diagonal design if it governs, with a leaf submergence corresponding to normal navigation pool conditions. For this load condition, a 33⅓% overstress is allowed for diagonal design. Temporal hydraulic loads in the lock chamber and/or lock approaches may be caused singly or in combination by the following:

- Wind waves and setup;
- Ship waves;
- Propeller wash;
- Lock overfill and/or over empty;
- Lock upstream intake and downstream exit discharges.
- Landslide waves;
- Tributary and/or distributary flow near lock;
- Surges and reflected waves in canals;
- Seiches;
- Changes in spillway or powerhouse discharges;
- Tides.

A1.10. Procedure for Prestressing Diagonals

A1.10.1. The following steps establish a procedure for prestressing diagonals. There are different procedures for stressing diagonals, this being just one. Use Figure A1-14 with this procedure:

- With all diagonals slack, adjust anchorage bars so quoin end is plumb and bottom girder is horizontal. Pintle shoe shall be fully seated against the back of the pintle base.
- Lubricate the nuts on the diagonals so they can turn easily.
- Place rosettes for strain gages on all diagonals a minimum of 20 hours before prestressing unless approved quick-setting cement is used.
- Without the restraint of any guys or jacks, the leaf will deflect in a negative direction under its own dead load weight. Measure this deflection.
- Guy the leaf at its miter end to the tieback anchor and place jacks at the miter end.

- Jack the miter end away from the wall until the leaf has a deflection equal to $D_1$.

- Hold the deflection and tighten diagonals 1 and 3. Tighten these diagonals so that there is no horizontal bow. Do not attempt to remove all vertical sag.

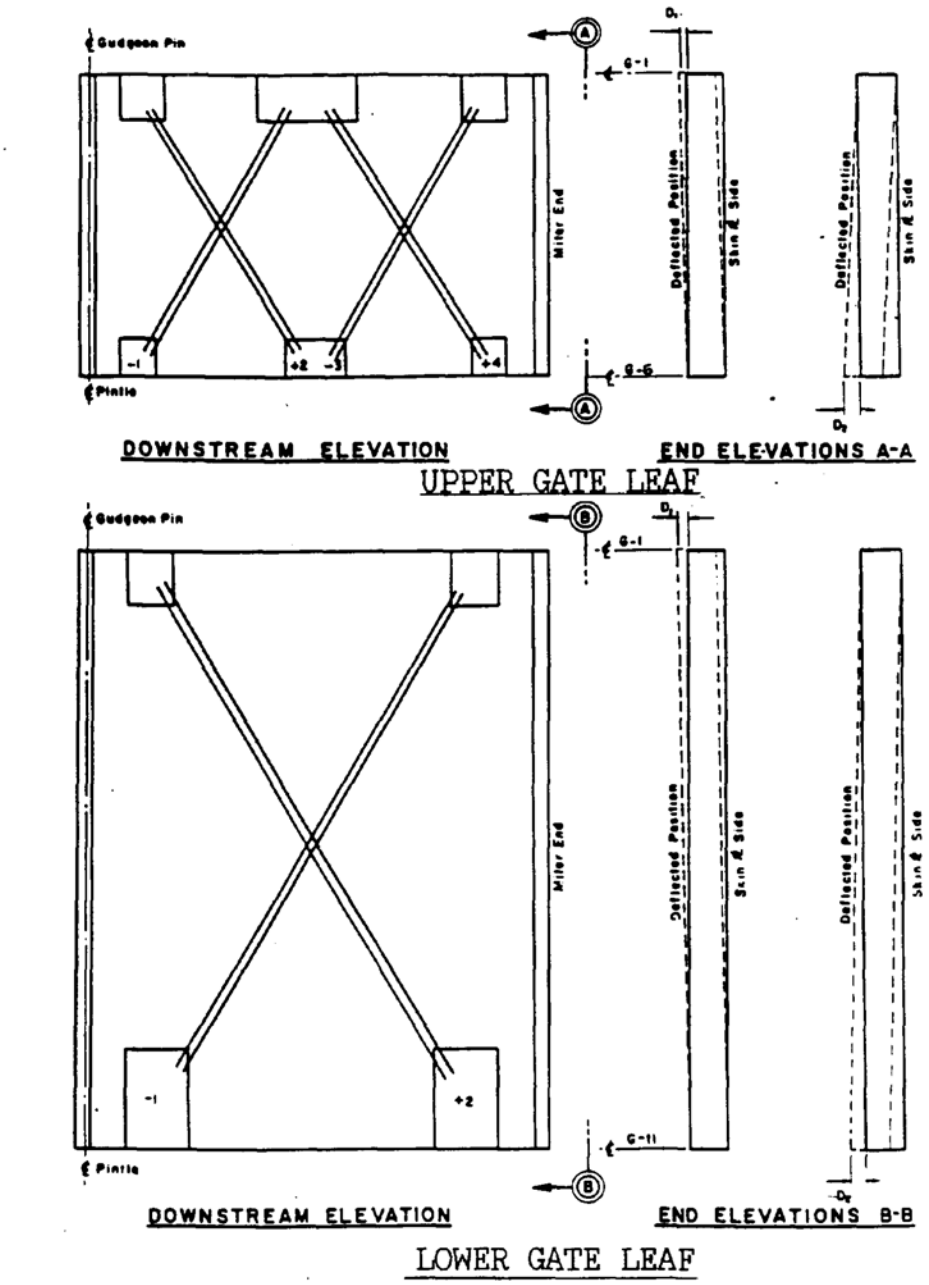
- Tighten diagonals 2 and 4.

- Proceed with the jacking until a deflection $D_2$ is obtained. During this operation do not change the adjustment of diagonals 1 and 3. However, continue tightening diagonals 2 and 4 until there is a slight tension in the members when the leaf is in its final deflection position.

- During the prestressing operation use a strain gage to determine the stress in the diagonals. The maximum allowable stress shall be $0.75F_y$.

- After the final adjustments of the diagonals remove the guys and jacks. The leaf should return to the plumb position. 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.

- Final minimum and maximum stresses, unless otherwise approved by the Contracting Officer, shall be $0.45F_y$ minimum and $0.55F_y$ maximum for all diagonals.




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 A1-14. Methods for Prestressing Diagonal.

A1.11. New Information on Diagonal Design

A1.11.1. New preliminary information has been gained through the finite element study made by Drs. L. Z. Elkin, K. M. Will, and B. J. Goodno of the Georgia Institute of Technology regarding torque tubes and leaf stiffness (USAEWES 1987). For all current gates designed with the 2.5-ft differential head, it appears that the values arrived at through the finite element analysis of Bankhead Lock lower gate in Tuscaloosa, AL, are realistic. This includes the values of leaf stiffness without diagonals, with diagonals, and with horizontal top and bottom torque tubes. These values are only a recommendation and consideration should be given to any variation in leaf configuration and modifications made to adjust the design factors accordingly.

A1.11.2. The use of top and bottom torque tubes is suggested as a suitable means of increasing leaf stiffness, although it appears that the conventional method of prestressing by twisting the leaves with a jack may need to be altered. On the Oliver Lock in Tuscaloosa, AL, where the torque tubes were used and diagonals sized for surge loading, it appeared that the twist-of-the-leaf method of prestressing the diagonals had about reached its maximum. Due to the increased leaf stiffness and corresponding jack capacity (±150 tons), it appeared that damage to the leaf, such as localized buckling of plates, excessive deflection of the quoin post, damage to the grease seals, pintle, pintle socket, etc., could be imminent.

The values representing leaf stiffness for this particular study were determined to be:

$Q_0$ = stiffness factor of leaves without diagonals

$D$ = stiffness factor of diagonals

$Q_t$ = stiffness factor of top and bottom torque tubes
(one 6-ft girder space at top and one 4-ft girder space at bottom)

$D = 2.4Q_0$

$Qt = Q_0$.

A1.11.3. It is recommended that consideration be given to prestressing new gate leaves with torque tubes by turning the nuts on the ends of the diagonals and using suitable means to prevent twisting of the diagonals. This would simplify the prestressing and reduce the risk of damage to the gate leaves as well as reduce the risk to personnel. There may be commercial sources that have equipment available that could be readily adapted to this means of prestressing, as has been the case in prestressing the anchor bolts of the embedded anchorage.

A1.11.4. Additional studies are needed to advance the understanding of miter gate leaf stiffness. Significant factors are dead load deflection, jack loads, if used, strain gage readings, problems encountered, alignment of gudgeon pin over pintle, and any other information thought to possibly be pertinent. For additional information, see USAEWES (1987).

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# APPENDIX D

## Spillway Tainter Gates

D.1. General. This appendix provides guidance for the design, fabrication, and inspection of spillway tainter gates, and associated trunnion girders and trunnion-girder anchorages, for navigation and flood control projects.

Controlled spillways include crest gates that 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. A tainter gate is a segment of a cylinder mounted on radial arms that rotate vertically on trunnions anchored to the piers. Spillway flow is regulated by raising or lowering the gate to adjust the discharge under the gate.

Submergible tainter gates can be used as a lock gate, typically at the upstream end of the lock, or as a spillway gate. 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 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. Figures D-27 and D-28 show typical end frame and cross-section of this type of gate.

D.1.1. Configuration. This manual describes a conventional tainter gate configuration. However, there are numerous unique variations of tainter gate types. Figure D-1 shows a dam with tainter gates. Figure D-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. 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.

The configuration and design of a submergible tainter gate is similar to that of a spillway tainter gate, as described in Appendix D of this manual. Navigation lock gates are usually wider and might have lower heads than 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. Plate C-33 illustrates a submergible tainter lock gate.

D.1.2. Advantages of Tainter Gates.

Advantages of tainter gates include that:

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

- A lower hoist capacity is required.

- Tainter gates operate relatively quickly and efficiently.

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

- Tainter gate geometry provides favorable hydraulic discharge characteristics.

D.1.3. Disadvantages of Tainter Gates.

- 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 since they necessarily contain more concrete, which usually gives the structures a less favorable seismic resistance due to their greater height and mass.

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

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

D.1.4. Use on Corps of Engineers Projects. Spillway tainter gates are used on projects for flood control, navigation, and hydropower. Although navigation and flood control 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 control projects are loaded significantly only during flood events. These differences may influence selection of the lifting hoist system, emphasis on detailing for resistance to possible vibration loading, and selection of corrosion protection systems.

D.1.4.1. Navigation Projects. 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. 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. Under normal conditions, navigation gates are often partially submerged and significantly loaded with the upstream-downstream hydrostatic head. Figure D-3 shows a typical cross-section of a navigation dam with tainter gates.

D.1.4.2. Flood Control and Hydropower Projects. Flood control projects provide temporary storage of flood flows. Many projects include gated spillways to regulate outflow. On flood control 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. Trunnions are typically located at an elevation approximately one-third the height of the gate above the sill. Some unique multipurpose projects (projects that provide flood control and reservoir storage) and most hydropower projects include aspects of flood control 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. Figure D-4 shows a typical cross-section of a flood control or hydropower dam with tainter gates.

D.2. Gate Design.

D.2.1. Gate Geometry. Sizing of the gates is an important early step in the design process. Gate size affects other project components, project cost, and operation and maintenance. This section includes various considerations for selecting a practical and economical tainter gate size. Related guidance can be found in EM 1110-2-1603, EM 1110-2-1605, and EM 1110-2-2607.

D.2.1.1. Gate size. The Hydraulic Engineer will normally establish the limiting parameters for gate height and width. Within those limits, various height-to-width ratios should be studied to find the most suitable gate size for the project. The structural designer should coordinate closely with the Hydraulic Engineer in determining the basic limiting requirements for size and shape. The size, shape, and framing system of the gates should be selected to minimize the overall cost of the spillway, rather than the gate itself.

D.2.1.2. Gate width. The gate width will be determined based on such factors as maximum desirable width of monoliths, length of spillway, bridge spans, drift loading, overall monolith stability, and loads on trunnions and anchorages. On navigation projects, the gates may be set equal to the width of the lock, so that one set of bulkheads can serve both structures. It is usually desirable to use tall rather than shallow gates for a given discharge, since the overall spillway width is reduced and results in a more economical spillway.

D.2.1.3. Gate radius. The skin plate radius will normally be set equal to or greater than the height of the gate. The radius of the gate will also be affected by operational requirements concerning clearance between the bottom of the gate and the water surface profile. This is often the case for navigation dams on rivers where the gate must clear the flood-stage water surface profile to pass floating debris. On such projects requiring larger vertical openings, it is common to use a larger radius, up to four times the gate height, to allow for a greater range of opening. This will require longer piers for satisfactory location of the trunnion girder.

D.2.1.4. Trunnion location. It is generally desirable to locate the trunnion above the maximum water surface profile 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. It will usually be advantageous to locate the trunnion so that the maximum reaction is approximately horizontal (with the trunnion girder about one-third the height of the gate above the sill). This will allow

for simplified design and construction by allowing the trunnion post-tensioned anchorage to be placed in horizontal layers.

D.2.1.5. Operating equipment location. 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 D-3) 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 D-5). Figure D-6 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.

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. 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. 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. Many gates have non-vertical wire ropes and many gates include ropes that are non-tangent at or near the closed and/or full opened positions.

D.2.1.6. Hydraulic cylinder hoist system. Some newer gate designs use hydraulic cylinder hoist systems. However, these 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. 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. 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.

D.2.1.7. 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. Additional considerations could include standardization of gate sizes on a project or among nearby projects. The standardization of sizes could result in savings by eliminating multiple sets of bulkheads, standardizing machinery, and reducing stored replacement parts, etc.

D.2.2. Standard Gate Structure.

D.2.2.1. Primary gate components. The principal elements of a conventional tainter gate are the skin plate assembly, horizontal girders, end frames, and trunnions (Figure D-7). The skin plate assembly, which forms a cylindrical damming surface, consists of a skin plate stiffened and supported by curved vertical ribs. Structurally, 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. The horizontal girders are supported by the end frames. End frames consist of radial struts or strut arms and bracing members that converge at the trunnion, which is anchored to the pier through the trunnion girder. The end frames may be parallel to the face of the pier (support the horizontal girders at the ends) or inclined to the face of the pier (support 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.

D.2.2.2. Other structural members.

- Horizontal girder lateral bracing. Bracing is generally 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. Lateral bracing, which carries significant vertical forces from the skin plate assembly to the end frame, is often considered a part of the end frame (Figure D-8). Intermediate bracing, which is located away from the end frames, provides girder lateral stability. It can be considered as one of the secondary members.

- Downstream vertical truss. The downstream vertical truss consists of bracing provided between the downstream flanges of the horizontal girders. Various configurations have been used depending on the gate size and configuration (Figure D-9). For gates with more than two girders, the downstream vertical truss does not lie in a single plane. Since the horizontal girders are arranged along the arc of the skin plate assembly, the downstream girder flanges do not lie in the same plane. This out-of-plane geometry is commonly ignored for design purposes.

- End frame bracing. Figure D-10 shows 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.

- 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, which is often a tubular member that extends across the gate bay from one end frame to the other, is attached to each end frame near the trunnion (Figure D-11).

D.2.3. Alternative Gate Structure. 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.

D.2.3.1. 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.

D.2.3.2. Vertically framed gates. In vertically framed gates, vertical girders support ribs that are placed horizontally on the skinplate. 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.

D.2.3.3. 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, but fabrication and maintenance costs are often higher. Its use has been very limited.

D.2.3.4. 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 are often higher.

D.2.3.5. 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 early gates were designed as a series 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 very high. For this reason, they were not adopted as a standard design. More recently, the use of computer designed 3-D space frame gates constructed with tubular sections has been investigated and may be practical in some situations.

D.2.3.6. Overflow/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 gate.

D.2.4. Loads. Loads and loading combinations for gates are described in the main body of this manual. 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. Reactions loads are not factored since they are determined from equilibrium with factored loads applied. As a result, reaction forces reflect the load factors of the applied loads:

- Self-weight. Dead load, (D).

- Gravity loads. Gravity loads include dead load or weight of the gate (D), mud weight (M), and ice weight (C), and shall be determined based on site-specific conditions.

- Hydrostatic loads. Hydrostatic loads consist of hydrostatic pressure on the gate considering both upper and lower pools. Three levels of hydrostatic loads are considered as described in Section 3.2.3.1.

- Gate-lifting system (operating machinery) loads. See Section 3.2.3.3.3 for further discussion on operational loads. Operating machinery is provided to support gates during lifting or lowering operations. Under normal operating conditions, the machinery provides forces necessary to support the gate, and for the load cases described herein, these forces are treated as reaction forces. Loads Q are machinery loads for conditions where the machinery exerts applied forces on an otherwise supported gate. There are three levels of loads applied by the operating machinery to the gate. The hydraulic cylinder maximum downward load Q1 is the maximum compressive downward load that a hydraulic hoist system can exert if the gate jams while closing, or if when the gate comes to rest on the sill. The hydraulic cylinder at-rest load Q2 is the 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). Loads Q1 and Q2 do not exist for wire rope hoist systems. The maximum upward operating machinery load Q3 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. The gate-lifting systems exert forces on specific gate members whether the forces are reactions or applied loads. For example, 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. If the wire rope is not tangent to the skin plate, the rope will exert an additional concentrated load on the gate (Figure D-6). Concentrated forces that typically vary with gate position in magnitude and direction are present at the attachment points for both gate-lifting systems. Operating machinery loads must be quantified in consultation with the mechanical Engineer that designs the machinery. Determination of load magnitudes and suggested coordination are discussed in Section D.6.

- Ice-impact load IM. The ice-impact load is specified to account for impact of debris (timber, ice, and other foreign objects) or lateral loading due to thermal expansion of ice sheets. Additionally, this load provides the overall structure with a margin of safety against collapse under barge impact. (Barge impact is an accidental event that is not practical to design for and is not specifically considered in design.) IM is specified as a uniform distributed load of 5.0 kips/ft that acts in the downstream direction and is applied along the width of the gate at the upper pool elevation. Ice shall be placed to produce maximum effects.

- 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. 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.) 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 D-1.

$$F_s = \mu_s S l + \mu_s \gamma_w \, {}^{d}/_{2} \left( l_1 \, {}^{h}/_{2} + h l_2 \right) \quad \text{(D-1)}$$

where:

$\mu_s$ = coefficient of side-seal friction

l = total length of the side seal

l1 = 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

l2 = 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 = 3δEI/d3, where δ is the seal preset distance

γw = 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

- Trunnion pin friction, Ft. 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. The friction moment at the end of the hub is a function of a coefficient of friction, the trunnion reaction force component Rz that acts normal to the end of the pin (normal to the pier face), and the average radius of the hub. A coefficient of friction of 0.3 shall be used. This is a reasonable value that applies for any bushing material that may be slightly worn or improperly maintained and includes effects thrust washer friction and bearing misalignment.

- Earthquake design loads E. See Section 3.2.3.6 for earthquake loading.

- Hydrodynamic Loads. The primary hydrodynamic load considered in design is the wave load WA. See Section 3.2.3.2 for determining wave loads. Flow induced vibration shall be limited as discussed in Section 3.2.3.2 and this appendix.

- Environmental Loads. The environmental load considered for tainter gate design is wind, W. Wind loads shall be based on a 300-year return period per ASCE 7, but need not be taken greater

than 50 psf. Wind loads are small when compared to hydrostatic loads and only affect gate reactions when the gate is in an open position.

D.2.5. Load and Resistance Factor Design. Design requirements using LRFD are provided in the main body of this manual and as follows. These provisions apply to non-submergible gates only. For submergible tainter gates, see Section 3.4.2 for applicable loads and Section 3.4 design requirements.

D.2.5.1. Load cases. Tainter gates shall be designed for the Strength and Extreme Limit States for each of the following load cases and corresponding load combinations. Table D-1 lists load factors. The Serviceability Limit State is addressed in Section D.2.6

- Case 1: Strength Limit State I, Gate Closed. Loads consist of unusual hydrostatic head differential with gate subjected to self-weight, gravity loads, maximum hydraulic cylinder pressure, Q1 alone or hydraulic cylinder residual pressure and weight, Q2 plus wave or impact, whichever controls:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs}2H_{s2} + [\gamma_{Q2}Q_1 \text{ OR } \gamma_{Q2}Q_2 + (\gamma_{Hd2}H_{d2} \text{ or } \gamma_{IM2}IM_2)] \quad \text{(D-2)}$$

- Case 2.a: Strength Limit State II, Gate Open, Gate operating on two hoists. Loads consist of extreme hydrostatic head differential with gate subjected to self-weight, gravity loads, and side seal and trunnion friction:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s3} + \gamma_{Q2}Q_{Fs2} + \gamma_{Q1}Q_{Ft2} \quad \text{(D-3)}$$

- Case 2.b: Strength Limit State II, Gate Open, Gate operating on two hoists. Loads consist of unusual hydrostatic head differential with gate subjected to self-weight, gravity loads, and side seal and trunnion friction plus wave or impact, whichever controls:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} + \gamma_{Q2}Q_{Fs2} + \gamma_{Q1}Q_{Ft2} + (\gamma_{Hd2}H_{d2} \text{ or } \gamma_{IM2}IM_2) \quad \text{(D-4)}$$

- Case 3: Strength Limit State II, Gate Open, Gate operating on one hoist. Loads consist of unusual hydrostatic head differential with gate subjected to self-weight, gravity loads, and side seal and trunnion friction:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} + \gamma_{Q2}Q_{Fs2} + \gamma_{Q1}Q_{Ft2} \quad \text{(D-5)}$$

- Case 4.a: Extreme Limit State, Gate Closed. Loads consist of extreme hydrostatic head differential with gate subjected self-weight, and hydraulic cylinder residual pressure and weight, Q2:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s3} + \gamma_{Q2}Q_2 \quad \text{(D-6)}$$

- Case 4.b: Extreme Limit State, Gate Closed. Loads consist of usual hydrostatic head differential with gate subjected to self-weight, gravity loads, and earthquake:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s1} + \gamma_{EQ}EQ \quad \text{(D-7)}$$

- Case 5: Extreme Limit State, Gate Jammed. Loads consist of usual hydrostatic head differential with gate subjected to self-weight, gravity loads, and maximum hydraulic cylinder OR wire rope forces, whichever is applicable:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs}2H_{s1} + \gamma_{Q2}Q_3 \quad \text{(D-8)}$$

- Case 6: Extreme, Gate Fully Opened. Loads consist of self-weight and gravity loads plus wind, earthquake, OR maximum hydraulic cylinder OR wire rope forces (whichever is applicable), whichever controls:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + (\gamma_{EV}EV + \gamma_{EQ}EQ + \gamma_{Q2}Q_3) \quad \text{(D-9)}$$

Table D-1. Load Factors for Tainter Gates.

| Load Cases | | | Loads/Load Factors | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | D | G | Hs | Hd | Q | EV | IM | EQ |
| Limit State | Description | Case | γD | γG | γHs | γHd | γQ | γEV | γIM | γEQ |
| Strength I | Gate Closed | 1 | 1.2 | 1.6 | 1.4Hs2 | 0 | 1.2Q1 | 0 | 0 | 0 |
| Strength I | Gate Closed | 1 | 1.2 | 1.6 | 1.4Hs2 | 1.2(1) | 1.2Q2 | 0 | 1.6(1) | 0 |
| Strength II | Usual Operation | 2.a | 1.2 | 1.6 | 1.4Hs3 | 0 | 1.4QFs2+ 1.0QFt2 | 0 | 0 | 0 |
| Strength II | Usual Operation | 2.b | 1.2 | 1.6 | 1.4Hs2 | 1.2(1) | 1.4QFs2+ 1.0QFt2 | 0 | 1.6(1) | 0 |
| Strength II | Unusual Operation | 3 | 1.2 | 1.6 | 1.4Hs2 | 0 | 1.4QFs2+ 1.0QFt2 | 0 | 0 | 0 |
| Extreme I | Gate Closed | 4.a | 1.2 | 1.6 | 1.4Hs3 | 0 | 1.2Q2 | 0 | 0 | 0 |
| Extreme I | Gate Closed | 4.b | 1.2 | 1.6 | 1.4Hs1 | 0 | 0 | 0 | 0 | 1.0EQ |
| Extreme I | Gate Jammed | 5 | 1.2 | 1.6 | 1.4Hs2 | 0 | 1.2Q3 | 0 | 0 | 0 |
| Extreme I | Gate Opened | 6 | γD2(2) | γG2(2) | 0 | 0 | 1.2Q3(1) | 1.3W(1) | 0 | 1.0EQ(1) |

Notes: (1) Select one at a time
(2) Use max or min values, whichever produces maximum effects

D.2.6. Serviceability Requirements. Serviceability requirements for gates are provided in Section 3.1.3. Consider whether bulkhead slots are needed to permit gate maintenance. Following are a few operational issues specific to tainter gates.

D.2.6.1. 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 the condition where the gate is supported on only one side, the gate may rotate so that the gate 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 piers due to frictional forces that occur with gate movement. A 3-D finite element analysis may be required to determine the normal forces and subsequent potential for binding. If operational requirements include lifting or closing the gate when it is supported on one side only, the designer should consider possibilities of roller failure or degraded embedded metal surface conditions (due to corrosion or presence of foreign materials/growths) on the effective roller drag or frictional resistance.

D.2.6.2. Ice control. Where ice may accumulate and inhibit gate operation, heaters shall be considered in the design.

D.2.6.3. Vibration. Vibration due to flow under the gate shall be considered in the design and detailing of the tainter gate. To limit vibration, the bottom lip of the tainter gate and sill should be detailed as described in the design details section below.

D.2.6.4. 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 lighter members such as bracing 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 debris impact damage and binding of lodged debris.

D.2.7. Fatigue and Fracture Control.

D.2.7.1. Chapter 4 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.

D.2.7.2. Trunnion yoke plates, trunnion bushing assembly, cable attachment brackets, steel trunnion girders, and built-up members may include weldments with thick plates and/or high constraint. These areas are often especially susceptible to poor material properties and cracking. Appropriate fabrication requirements, including weld sequence and inspection, should be specified for thick plate weldments or highly constrained weldments. See Chapters 4, 5, and 7 for guidance on controlling cracking in these members.

D.2.8. Material Selection. Material selection guidelines for gates are provided in Section 2.3.

D.2.9. Analysis and Design Considerations.

D.2.9.1. Two-dimensional Analytical Models. In the design of tainter gate structural members, it has been common practice to model the 3-D behavior with several independent 2-D models. With the 2-D approach, the overall behavior is simulated by modeling separately the behavior of the skin plate assembly (composed of the skin plate and supporting ribs), horizontal girder frames (composed of a horizontal girder and the two adjacent struts), vertical end frames (composed of struts and braces), and the vertical downstream truss. Analysis of the 2-D models is interdependent. Various loads on one model can be reactions from another (girder frame loads are obtained from the rib model reactions), and many of the same loads are applied to each model. Additionally, struts include member forces from separate models. (The strong axis flexural behavior of the struts is simulated with the girder frame model. Axial and weak axis flexural forces are provided by the end frame model.) An alternative for each 2-D model is described in the following subsections. In the discussion for each model, loads for all load conditions are described. Various loads that are applied to the models are not factored, since they are calculated as reactions caused by applied factored loads.

- Skin plate assembly. For 2-D analysis, the skin plate and ribs are assumed to have zero curvature. The skin plate serves two functions: each unit width of skin plate is assumed to act as a continuous beam spanning between ribs. An effective portion of the skin plate also acts as the upstream flange of the ribs. The ribs, with the skin plate flange, are continuous beams supported by the horizontal girders. Analysis of the skinplate should include hydrostatic loads, plus wire rope

reaction forces acting normal to the skinplate. Analysis of the ribs should include hydrostatic loading, plus the sill reaction load at the bottom of the skin plate and wire rope reaction forces acting normal to the skinplate.

- Girder frame. The analytical model is a simple frame representing the horizontal girder and end struts (Figure D-12). The model is symmetric, but since the frame is not fully restrained laterally, for certain conditions, the frame should be subjected to a calculated sidesway. 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. Girder loading is based on reaction forces from the rib analysis. There might be additional concentrated loads if hydraulic cylinders are used to operate the gate. It is assumed that girder lateral bracing resists girder torsional forces that are caused by gravity loads. All reactions from this analysis should be considered when calculating effects of friction at the trunnion.

- End frame model. Although the analytical model for the end frame consists of struts and strut bracing, it should also include the bracing members in the plane of the end frame between the horizontal girders and members representing the girder webs. The end frame model is used to determine: sill reaction load, operating machinery reaction load, trunnion reaction, and end frame member forces. Since there is no moment restraint at the trunnion, the model must be made stable by providing some rotational reaction representing the sill, operating equipment, or gate stops. Applied loads represent girder reactions, gravity loads, and operating equipment forces. Once trunnion reactions are determined, there should be a moment applied at the trunnion to represent friction forces when the gate is being moved. Since trunnion friction can affect equipment forces, and equipment forces affect trunnion reactions, it might be necessary to iterate a few times to determine a consistent set of trunnion friction forces and equipment forces.

- 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. 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 at the opposite upper corner. Applied loads should include only gravity loads. This should result in conservative member forces since the skinplate assembly would certainly help carry these loads.

D.2.9.2. Skin Plate Assembly. The skin plate assembly consists of the skin plate and vertical ribs. Horizontal intercostals are not used because of higher fabrication and maintenance costs. The required skin plate thickness is dependent on the rib spacing (skin plate span), and the required rib size is dependent on the skin plate thickness since an effective portion of skin plate acts as a rib flange.

- Skin plate. At equally spaced interior ribs, the skinplate can be designed for the fixed-end moment, spanning between ribs. Spacing between the exterior ribs at the ends of the gate should be adjusted such that the moment does not exceed the 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 ⅜ in., while a thickness greater than ¾ in. will rarely be required for any gate.

- Ribs. Ribs are usually structural tee sections with the web welded to the skin plate. An effective width of skin plate acts as the upstream flange of the rib. Girder spacing is adjusted to equalize maximum rib bending moments at various locations along the rib.

- Fabrication and maintenance. Skin plate splices are full penetration welds. Due to expected wear and deterioration, it is appropriate to increase the skin plate thickness along the bottom of the gate and under wire rope locations. Ribs must be proportioned to provide adequate clearances for welding and painting. The minimum rib depth is usually 8 in.

D.2.9.3. Horizontal Girder. The girders are generally located to permit an economical design for the ribs. However, the location of girders also affects the load on each girder.

- Design. Horizontal girders may be rolled sections or built-up plate girders. They are designed primarily for flexure about their major axis, which is significantly influenced by where the struts intersect the girders. With inclined struts, lateral torsional buckling of the girder should be checked since a significant length of the downstream flange of the girder will be in compression. The downstream vertical truss provides lateral stability and resistance to torsional buckling for the girders. The tension flange of some girders might be considered FCMs.

- Fabrication and maintenance. 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 most appropriate for drainage.

D.2.9.4. End Frames. End frames include the struts, associated bracing, and trunnion hub flange plates. The arrangement and orientation of the end frames affects the magnitude and distribution of end frame and horizontal girder forces, trunnion fabrication, trunnion pin binding, and thrust forces into the pier.

- Design. Struts must resist significant axial forces and flexure about both axes. End frame bracing should be spaced to achieve adequate weak axis slenderness ratios for the struts. Bracing members may experience significant flexure depending on end frame geometry and trunnion friction. Trunnion hub flanges are proportioned to resist the strut flexural, shear, and axial loads.

- Fabrication and maintenance. Strut bracing are usually wide flange sections with the same depth as the struts, to simplify connections. Struts are welded to trunnion hub flanges, with clearance provided between the ends of intersecting struts (Figure D-13). These connections generally involve full penetration butt splices involving thick plates, thus complicating fabrication requirements.

- 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.

- 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 control projects, they are often not feasible for navigation dam projects where floating debris is a concern. The struts are usually positioned in a single vertical plane. In this case, only two strut/girder connections can be at the same horizontal distance from the pier face. 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. Struts can also be positioned such that the girder end of each strut is an equal horizontal distance from the pier face. This complicates geometry since the struts would then lie on a conical surface with the apex at the trunnion. This results in complex fabrication of the strut-to-trunnion hub flange connection, since the struts are rotated with respect to one another and do not lie in one plane.

D.2.9.5. 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, it may not be practical to design a bracing system that would prevent significant lateral displacements if the gate were supported on only one side. In these cases, it may be necessary to provide side bumpers to limit lateral movement.

D.2.10. Design Details.

D.2.10.1. 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.

- 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.

- 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 D-14). For most gates, the preferred seal configuration is provided by direct contact between the skin plate edge and the sill plate. It is recommended that rubber seals not be used on the gate bottom unless leakage cannot be tolerated. If leakage is critical, 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.

D.2.10.2. Lifting Attachments. Lifting attachments are often treated as FCMs for design. The force in the attachment due to machinery operating at maximum stall pull normally governs the design. The magnitude of this loading will be obtained from the mechanical Engineer responsible for the machinery design and will be based on the capabilities of the lifting equipment. The wire rope attachment often

must be designed with a rotating attachment to allow the cable to pull away from the skin plate as the gate approaches the full open position. Many gates also have skin plate extensions of smaller radius at the top to allow the rope to wrap over the top of the gate when fully closed. Figure D-15 shows a typical wire rope attachment detail.

D.2.10.3. 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. Typical hole size has a diameter of 2 in. Additionally, half round holes can be provided in stiffener plates, along with extra large corner copes to avoid pockets of water between stiffeners. Holes in flanges should generally be avoided.

D.2.10.4. Gate Stops. Many structures are provided with gate stops to limit the gate movement. The machinery is designed to stop before the gate contacting 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.

D.2.10.5. Bumpers. 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 are usually fitted with a bronze rubbing surface. Figure D-16 shows a common bumper detail. Rollers or ultra-high molecular weight plastic rubbing surfaces may be used to reduce friction to prevent gate binding.

D.2.10.6. 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.

D.3. Trunnion Assembly.

D.3.1. General Description. The trunnion assembly provides support for the tainter gate while allowing for rotation for operational use. The design of lubrication systems, determination of tolerance and finish requirements, material selection, and determination of allowable stresses should all be coordinated with a mechanical Engineer.

D.3.1.1. Conventional Trunnion System. The trunnion assembly is usually bolted to the upsteam face of the trunnion girder. It is made up of a fixed trunnion yoke, a trunnion hub, 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 loads directly to the trunnion girder. Figures D-17 and D-18 show typical details for cylindrical and spherical bearing assemblies, respectively. 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. When compared to cylindrical bearings, spherical bearings are generally narrower and the use of spherical bearings produces a more uniform pressure distribution over the trunnion pin, potentially reduces trunnion pin moments and gate arm stresses due to misalignment. Spherical bearings will accommodate an angular rotation transverse to the pin centerline in the range of 6 to 10 degrees depending on bearing size. A tradeoff exists with the use

of spherical bearings over cylindrical bushings in that the gate arms associated with spherical bearing are usually heavier due to an increased buckling length.

D.3.1.2. Center-Mounted Trunnion Pin. Center-mounted trunnions are commonly used in combination with steel box trunnion girders. The trunnion pin is supported at the geometric centroid of the girder by plates that are oriented perpendicular to the pin centerline. The pin can bear directly on the supporting plates or within a housing tube attached to the plates. Use of the tube provides for a more accurate bore for the pin. This arrangement can significantly reduce torsion applied to the trunnion girder since load eccentricity is reduced or eliminated. Figure D-19 shows a center-mounted arrangement.

D.3.2. Structural Components. Figure D-20 shows the layout of structural components of the trunnion assembly.

D.3.2.1. Trunnion Yoke. The yoke is typically fabricated of welded structural steel and consists of two parallel plates (yoke plates) welded to a stiffened base plate (Figure D-21). The yoke plates are machined to receive the trunnion pin and associated components. The assembly is bolted to the trunnion girder after final installation adjustments have been made by horizontal and vertical jackscrews. Potential shear loads at the interface with the trunnion girder might be large enough to require shear bars that are welded to the base plate, to transfer loads to the trunnion girder. The trunnion yoke 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. The yoke side plates are sized to resist trunnion pin bearing load and lateral gate loads. The base plate and stiffeners are be designed to resist contact pressure between the yoke bearing plate and trunnion girder based on gate reaction forces and bolt prestressing loads.

D.3.2.2. 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 gate arm extensions 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. The centerline of bearing of the trunnion hub is commonly offset with respect to the centerline of the gate arms (Figure D-20). This offset is recommended so that a uniform bearing stress distribution occurs under maximum loading. 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.

D.3.2.3. Trunnion Pin. The trunnion pin transfers the gate loads from the hub to the yoke side plates. A retainer plate that is welded to the pin is fitted with shear pin 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 D-22) shall be sized to prevent rotation. Trunnion pins may be designed with a hole drilled along the pin centerline for entry of a radioactive source to facilitate radiographic testing. The hole may also be tapped for handling, installation, and removal purposes.

D.3.2.4. Trunnion Bushing. Bushings are provided between the trunnion pin and hub and between the hub and yoke plates. The bushings provide a uniform bearing surface and reduce torsional loads due to friction. The required thickness will depend on the size of the trunnion pin. However, to maintain a true shape during machining, bushings should be at least ½-in. thick. Bushings are usually furnished with two side disk bushings and one cylindrical trunnion pin bushing. Shear pins between the side bushing and trunnion yoke plates are used to prevent bushing rotation. A light drive fit between the hub and cylindrical bushing is generally specified to prevent differential rotation between the hub and bushing. The bushing is usually given an overall finish of 63 μin except for the bore finish of 32 μin For lubrication, grease grooves are commonly provided on the inside face of the bushing. The size, length, and location of the grease groove must be sufficient to distribute lubricants uniformly to all bearing surfaces. A hole may be drilled through the hub and bearing to inject grease into the grooves.

D.3.2.5. Spherical Plain Bearings. Spherical plain bearings consist of an inner and outer ring and may contain intermediate sliding elements. The outer ring is fit within the trunnion hub and the inner ring is placed on the trunnion pin. The outer ring of the bearing is generally mounted inside the trunnion hub with an interference fit to prevent movement of ring seats. The inner ring may be mounted to the trunnion pin using an interference fit to prevent movement between the pin and the inner ring. The bearing internal clearance (between inner and outer rings) must account for deformations induced by the interference fits. Spherical bearings transmit radial forces (loads acting primarily in plane perpendicular to axis of the trunnion pin) and moderate axial forces simultaneously (loads acting parallel to the axis of the trunnion pin). Where axial forces are large, thrust washers or an additional thrust bearing may be required to transmit axial loads. A housing shoulder is generally mounted to the trunnion pin on the side adjacent to the pier and a locking plate or spacer sleeve retains the ring on the opposite side. The housing end cover generally retains the outer ring.

D.3.3. Material Selection.

D.3.3.1. 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).

D.3.3.2. Trunnion Bushing. Bushing materials are generally selected based on allowable bearing stresses, resistance to galling, and coefficient of friction. Aluminum bronze (ASTM B148) is commonly used where bearing pressures do not exceed 5000 psi. Manganese bronze or self-lubricating bronze (ASTM B22-90a) is used for applications where bearing pressures up to 8000 psi are required.

D.3.3.3. Spherical bearings. Spherical plain bearings are generally made of a high-strength carbon chromium steel treated with molybdenum disulfide. Maintenance free bearings may include a sinter bronze composite or a poly-tetra-flouro-ethylene compound.

D.3.3.4. Trunnion Pin. The material used for the trunnion pin must be compatible with the bushing material and be capable of high quality finishes to minimize friction. The trunnion pins shall be made from corrosion-resistant steel such as forged steel conforming to ASTM A705, Type 630, Condition H 1150. Historically, a carbon steel forging such as ASTM A668 was coated with a stainless steel weldment and then machined. This practice has become less economical due to high labor costs.

D.3.3.5. Shear Pin. Material for shear pins should be corrosion resistant and machinable. The shear pins are typically machined from cast or forged steel.

D.3.4. Design Requirements. All components of the trunnion assembly shall be designed based on ASD, using a uniform load factor of 1.0. Maximum allowable working stresses for forgings and casting shall be limited to 0.5Fy where Fy is the material yield stress. Allowable stresses and tolerances for bearings and bushings shall be established by the mechanical Engineer. The frictional resistance developed at the trunnion pin affects the end frame design and the required hoist capacity. The magnitude of the friction load is a function of the trunnion pin diameter, the coefficient of friction and the magnitude of the gate thrust. The coefficient of friction used for design must be consistent with the materials used. (A realistic coefficient of friction for systems with lubricated bronze or aluminum bronze bushings is 0.1 to 0.15. However, a coefficient of friction of 0.3 should be assumed for design purposes. This is conservative value accounts for potential wear or poor lubrication.) The bearing stress between the yoke base plate and the trunnion girder should include both the pre-tensioning force of the anchorage stud bolts and global gate reaction forces.

D.3.5. Serviceability Requirements. Serviceability requirements for gates are provided in the main body of this manual.

D.3.6. Trunnion Alignment. 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. 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 so that the trunnion hub axes are on a common horizontal line. Second-placement concrete or grout (zinc in the case of steel girders) 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.

D.4. Gate Anchorage Systems.

D.4.1. 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 includes high-strength, preloaded components while the non-prestressed system incorporates structural steel components.

D.4.1.1. 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. Figures D-23 and D-24 show a typical post-tensioned anchorage system. The anchors, which are placed inside ducts embedded in the concrete, are tensioned after the concrete has set and cured. Subsequent to tensioning, the annular space between the post-tensioning steel and duct is grouted for corrosion protection. Generally, two groups of anchorage steel are installed, one near each pier face. A limited bearing area is provided directly under each anchorage group. 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 reach. 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 and spalling stresses. Reinforcement must be provided to resist these stresses.

D.4.1.2. 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 appendix provides criteria only for prestressed systems.

D.4.2. 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. 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 throughout a cross-section some distance from the anchorage device.

D.4.2.1. 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. Couplers are available 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. 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.

D.4.2.2. 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, either galvanized ferrous metal or polyethylene. Polyethylene ducts are usually corrugated to increase crushing resistance and to interlock with surrounding concrete.

D.4.2.3. 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. 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. Additional information on ducts can be found in the *AASHTO LRFD Bridge Construction Specifications*.

D.4.3. Material Selection.

Post-tensioning bars shall be of high-tensile alloy steel, conforming to the requirements of ASTM Designation A722. Cable strands shall conform to ASTM A416 with a minimum strength of 270 ksi. The minimum compressive strength of concrete in anchorage zones shall be 4500 psi. The minimum compressive strength of concrete between the anchorage zones shall be 4000 psi. Higher concrete strengths may be used if required due to bearing, spalling, or bursting stresses. The maximum concrete aggregate size should be selected to pass between ducts and reinforcing bars.

D.4.4. Design Requirements.

A properly designed anchor system will prevent structural cracking of concrete, limit trunnion deflections, account for all time-dependent stress losses, and safely accommodate specified loads for the life of the structure. Excess deflections and structural cracking may occur if the trunnion girder looses 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 Strength and Service Limit States. The Strength Limit States ensure that the anchorage system will resist all factored loading combinations without failing. 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. The following design requirements are mandatory.

D.4.4.1. Design basis. Except as modified herein, the post-tensioned anchorage system shall be designed in accordance with current AASHTO requirements. Anchorage groups shall be sized such that the long-term effective anchorage force provides a minimum compressive bearing stress between the girder and the pier of 200 psi while the system is subject to the maximum trunnion reaction due to service loads. The anchorage zones, including spalling, bursting, and edge tension reinforcement shall be designed following procedures described by AASHTO, using factored jacking forces.

D.4.4.2. Loads. The maximum load will usually occur when one gate is raised just off the sill and the adjacent gate (if applicable) is unloaded. A uniform load factor of 1.9 shall be applied to each of the applied loads for the design of tendons. The initial jacking force is based on the required long-term effective stress and should include all expected prestress losses.

D.4.4.3. Resistance Factors. The resistance factor for tendons shall be 1.0. Strength design for the pier concrete shall be based on the criteria for an eccentrically loaded tension member, using a resistance factor of 0.9.

D.4.5. Analysis and Design Considerations.

D.4.5.1. Anchorage Force. The trunnion girder is assumed to behave as a simply supported beam, with cantilevered end spans. 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.

Anchorage forces are selected to maintain required pier contact pressures. The minimum residual bearing stresses between the pier and trunnion girder must also be sufficient to prevent sliding on the bearing surface due to trunnion reactions parallel to the bearing surface, unless other means such as dowels or mechanical confinement by adjacent concrete are provided to prevent relative movement between the trunnion girder and pier.

D.4.5.2. Anchor Depth. The depth of anchorage into the pier or abutment should be maximized to the extent possible 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.

D.4.5.3. 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 shall be provided where required in the tendon anchorage zones to resist bursting and edge tension forces induced by tendon anchorages. Design guidance regarding bursting, spalling, and edge tension is specified by AASHTO.

D.4.6. Serviceability.

Serviceability requirements for gates are provided in Section 3.1.3. Corrosion of the tendons and anchorage components is the primary serviceability concern regarding design of the anchorage system. The anchorage system shall be doubly protected against corrosion as described above.

D.4.7. Design Details.

D.4.7.1. Anchorage Layout. The post-tensioning anchorage steel should be installed in two groups with each group being located as close to the adjacent pier face as practicable. To place conventional pier steel reinforcement, clearance outside the prestressing bars of approximately 10 in. should be provided. Anchorage plates for alternate tendons should be installed in a staggered pattern 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.

D.4.7.2. Reinforcement. Spalling zone reinforcement that consists of conventional grid reinforcement designed to resist approximately 4% of the total prestressing force will be adequate for most cases. The reinforcement should be placed as near to the surface as practicable. It is suggested that the outer layer of reinforcement be welded to angles that are embedded along the vertical edges of the pier. For typical arrangements, tensile stresses in the bursting zone have been shown to be a maximum of about 18% of the unit compression stress due to prestressing. Reinforcement for these stresses should be provided from the downstream pier face into the pier for a distance of approximately one-half the width of the pier.

D.4.7.3. 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.

D.5. Trunnion Girder.

D.5.1. 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 are very stiff, 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.

D.5.2. Components.

D.5.2.1. 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 D-25 shows the upstream face of a typical concrete trunnion girder.

D.5.2.2. 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 (especially those that include center-mounted pins) 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 and provide support for center-mounted pins. Figure D-26 shows a steel trunnion girder. However, sufficient space is required between stiffeners to permit quality welds and weld inspection.

D.5.3. 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 shall be low-relaxation, high-tensile, seven-wire strands conforming to ASTM A416 with a minimum strength of 270 ksi. Bars are generally preferred over strands because they are less susceptible to stress corrosion. Steel trunnion girders are considered FCMs and require appropriate weld quality and material properties.

D.5.4. 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 orientation of adjacent gates shall be considered when evaluating the loading condition on the trunnion girder (i.e., when one is gate closed and the other gate is open, closed, or dewatered for maintenance).

D.5.4.1. Concrete. Tendons and concrete trunnion girders shall be designed based on the strength design method in accordance with current American Concrete Institute (ACI) requirements, except as

follows. The critical section for shear and torsion shall be taken as the first row of anchorage tendons closest to the trunnion. The minimum amount of web reinforcement should not be less than 0.003 bs, where b is the girder width and s the spacing of web reinforcement. A uniform load factor of 1.7 shall be applied to each of the specified loads for evaluating Strength Limit States and a load factor equal to 1.0 shall be applied to each of the loads for evaluating Service Limit States. A load factor of 1.2 shall be applied to the girder anchorage force.

D.5.4.2. Steel. Steel girders shall be designed per LRFD requirements in the main body of this manual. Fatigue is typically not considered in the design of steel trunnion girders, since the number of load cycles is generally low. Possible dynamic loading due to gate vibrations is not generally a concern because of the very low stress cycles this would cause on the trunnion girder. However, steel trunnion girders are considered fracture critical and proper material selection, welding procedures, joint details, and adequate quality control and testing is required. Lateral torsional buckling is generally not a concern for box girders due to the relatively stiff cross-sections. However, it might be a concern for I-shaped trunnion girders. In evaluating lateral torsional buckling, the unsupported length should be based on supports the centerlines of anchorage groups. Members shall be designed to resist jacking loads.

D.5.5. Analysis and Design Considerations. The size of the trunnion girder is dependent on the magnitude of the flexural, shear, and torsion 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, especially for yoke mounted pins. 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.

D.5.5.1. Stressing sequence. Depending on the tendon stressing order, the controlling design stage may occur when only some tendons are stressed. Consideration shall be given to concrete stresses and reinforcement requirements as each tendon is tensioned in sequence. Any special requirements regarding stressing order shall be described in the contract specifications or noted on the contract drawings.

D.5.5.2. Steel Girders. Beam flanges and webs should be proportioned to satisfy compact section requirements to avoid local buckling. Where compact sections are not practical, noncompact sections are allowed. However, slender elements shall not be used. Girders must be configured to resist the large anchorage post-tensioning loads. This might require plates, pipes, or tubes oriented parallel to the anchors.

D.5.6. Serviceability Requirements.

D.5.6.1. Serviceability requirements for gates are provided in Section 3.1.3. Chapter 6 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.

D.5.6.2. 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.

D.5.7. Design Details.

D.5.7.1. Concrete Girders. It has been common practice to require that the trunnion girder 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.

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 bars 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.

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 satisfactorily in previous designs.

D.5.7.2. Steel Girders. I-shaped girders are easier to fabricate than are 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 center-mount trunnions and tendon 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.

D.6. 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.

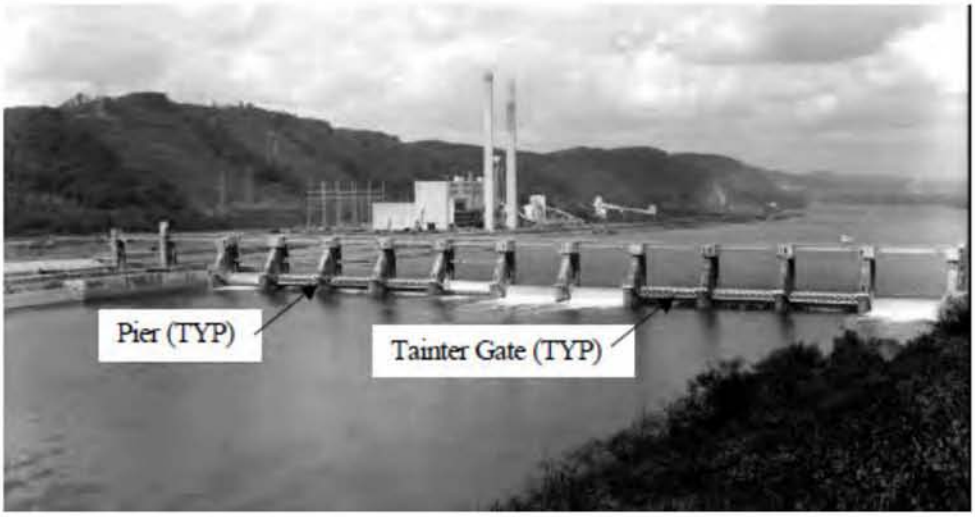


Figure D-1. Overall View of Navigation Dam from Downstream.

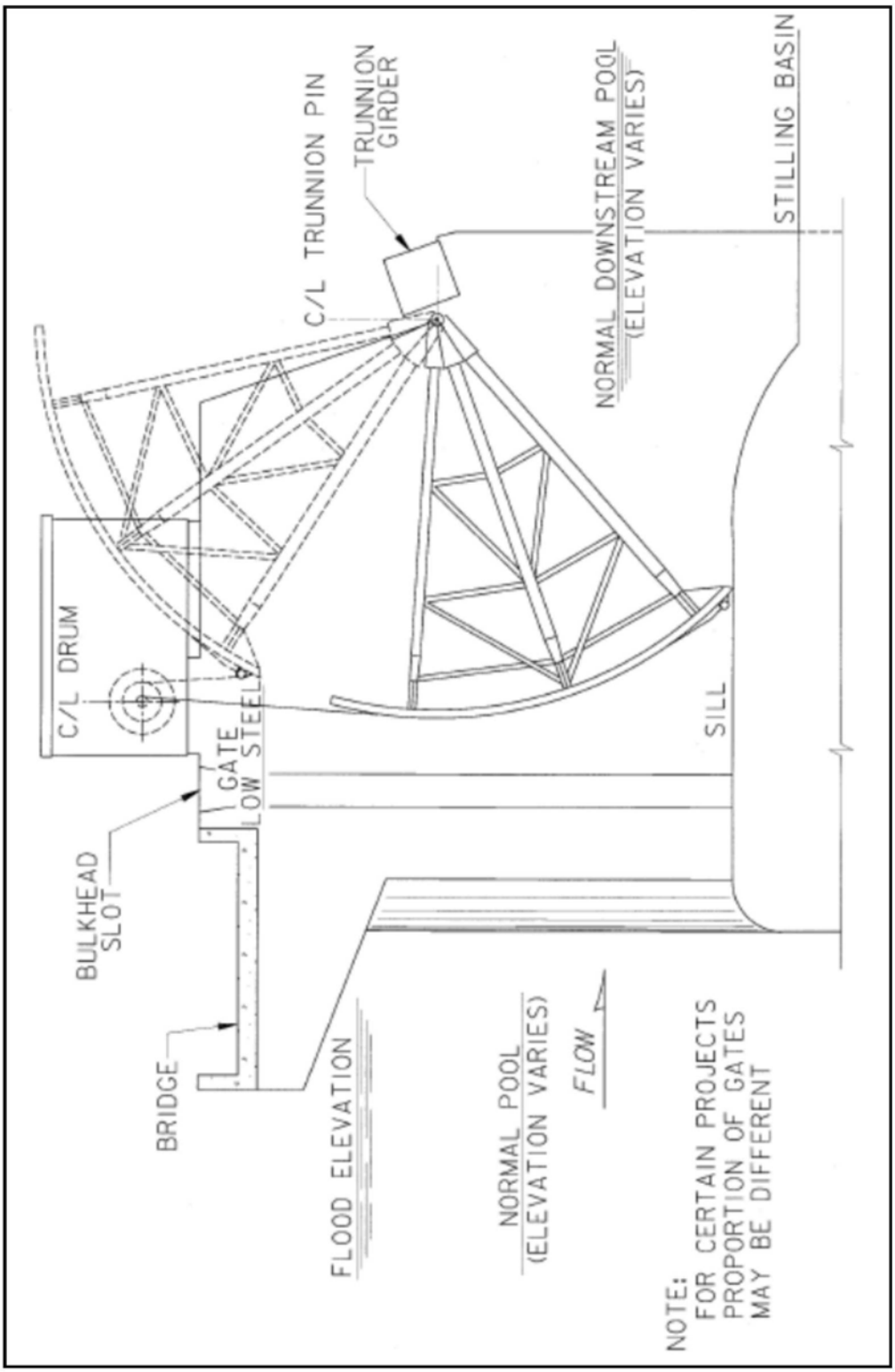


Figure D-2. Downstream View of a Typical Tainter Gate.

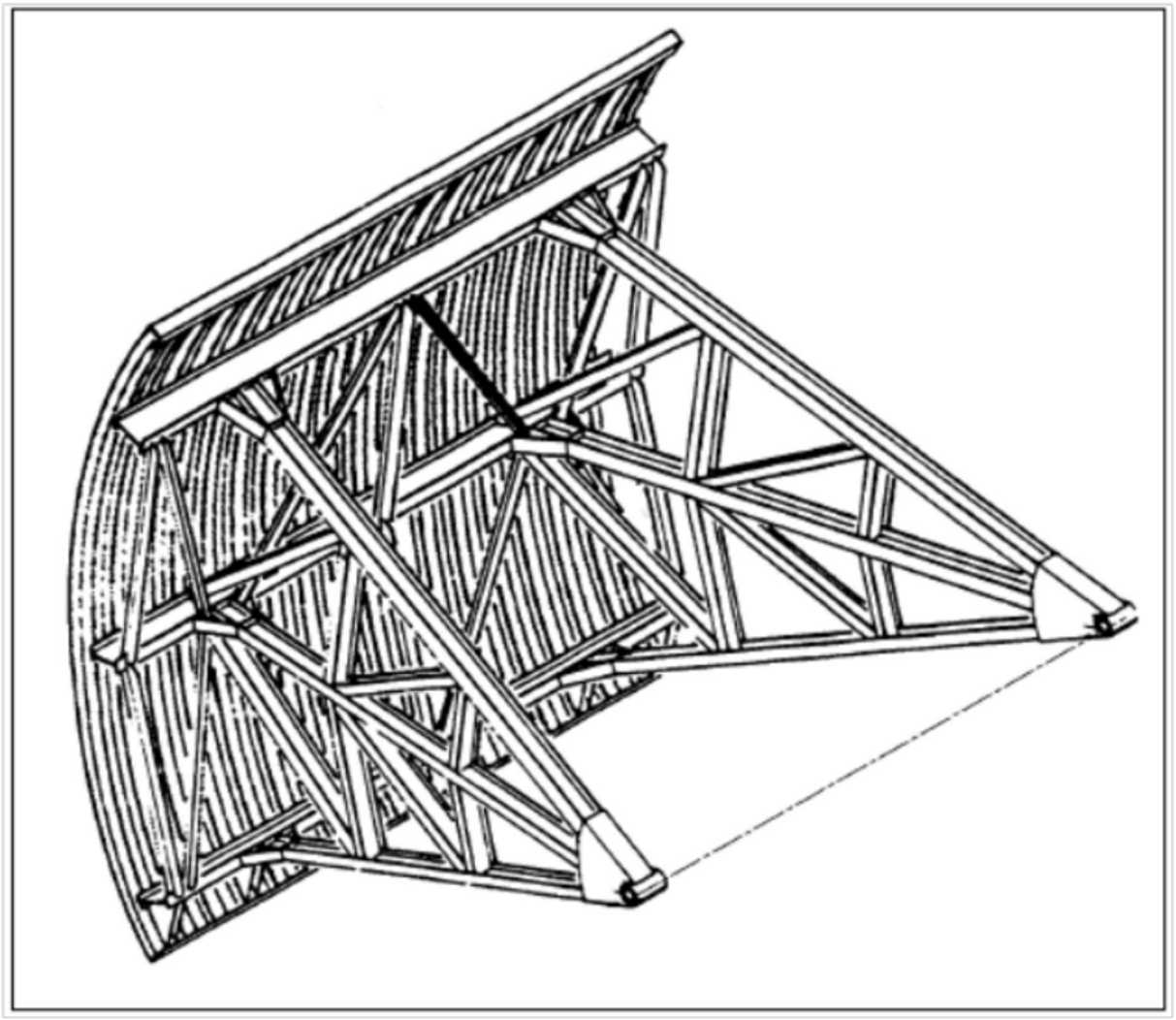

Figure D-3. Typical Navigation Tainter Gate

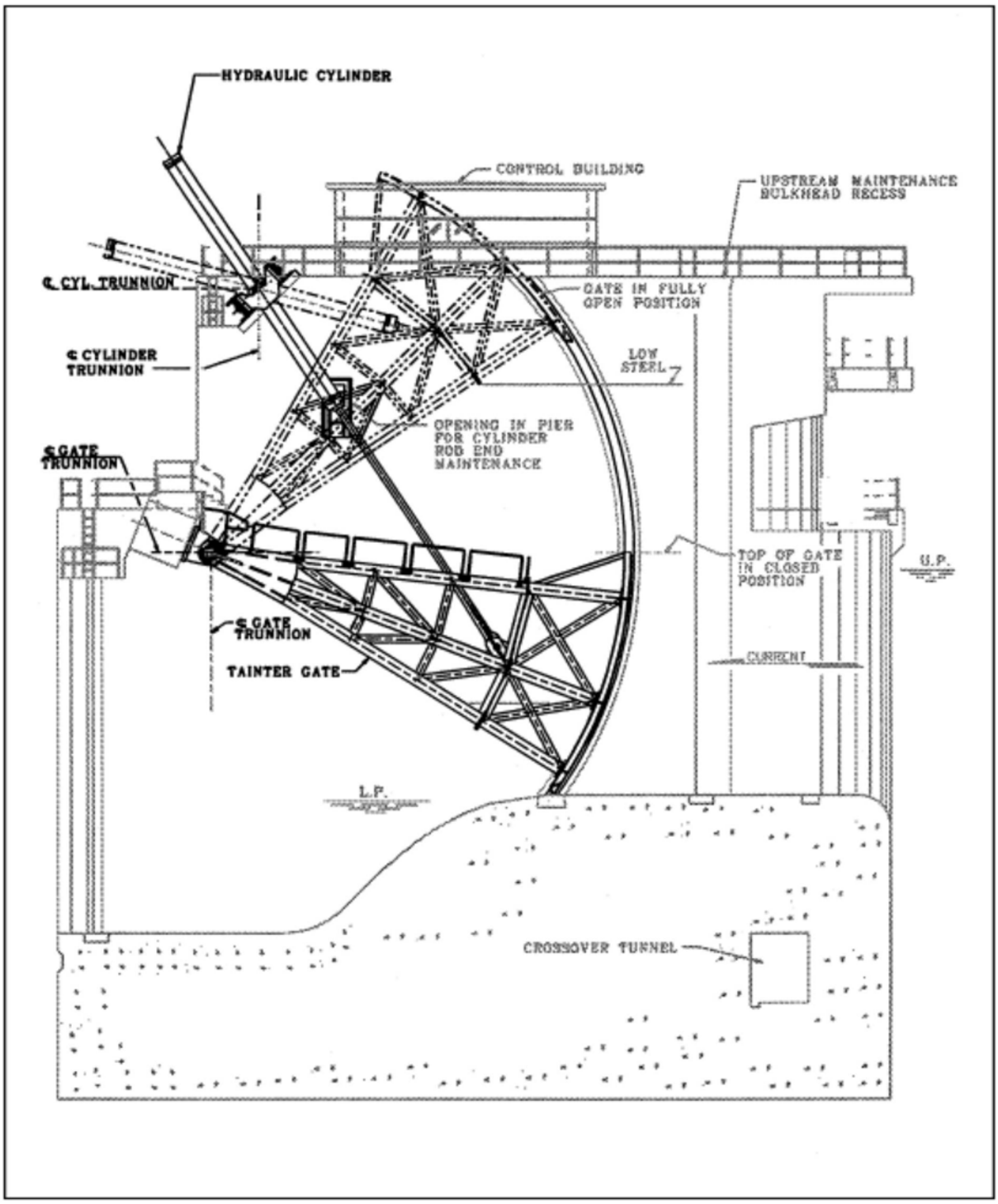


Figure D-4. Hydraulically Operated Tainter Gate.

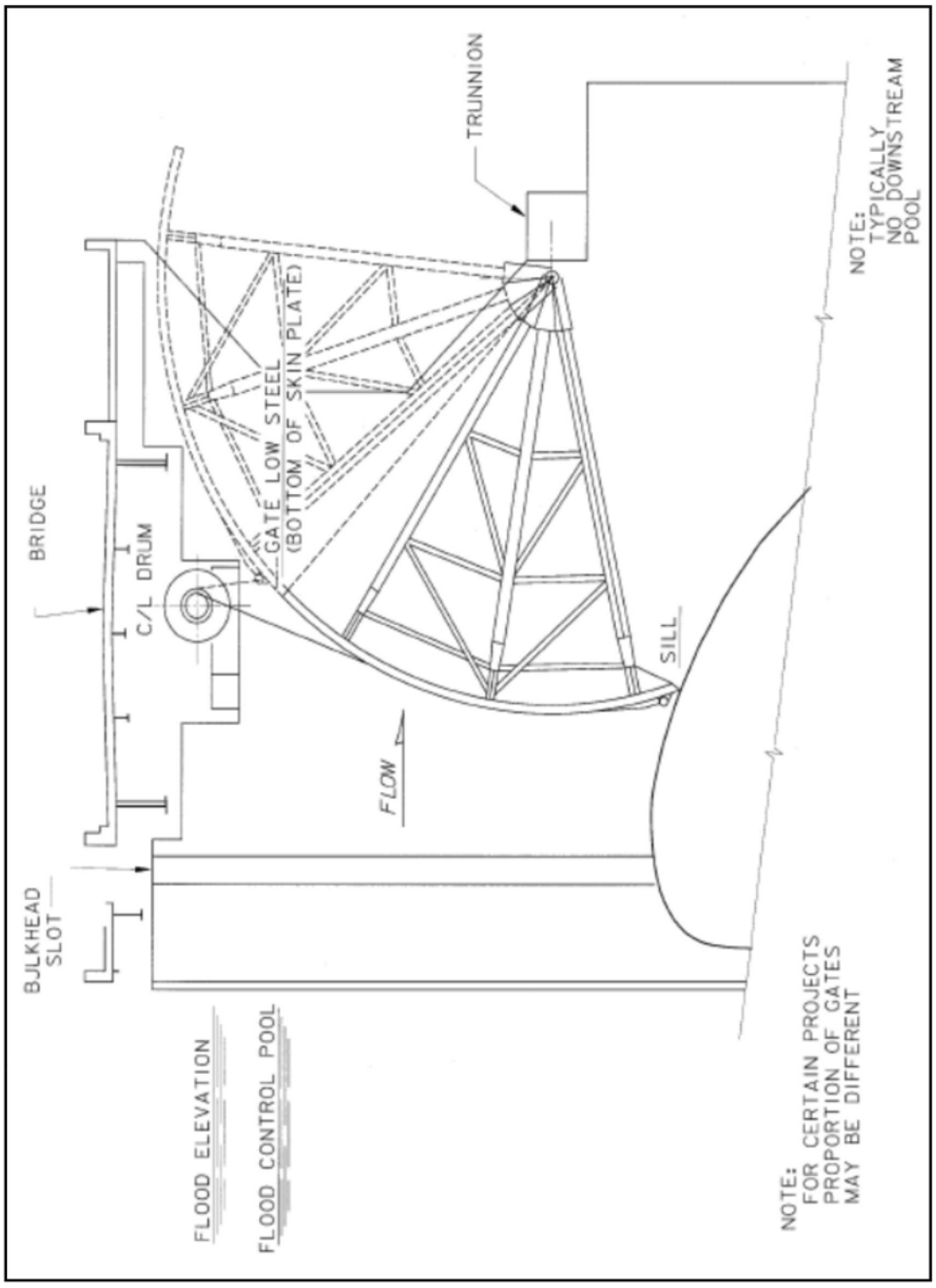


Figure D-5. Typical Flood Control or Hydropower Tainter Gate.

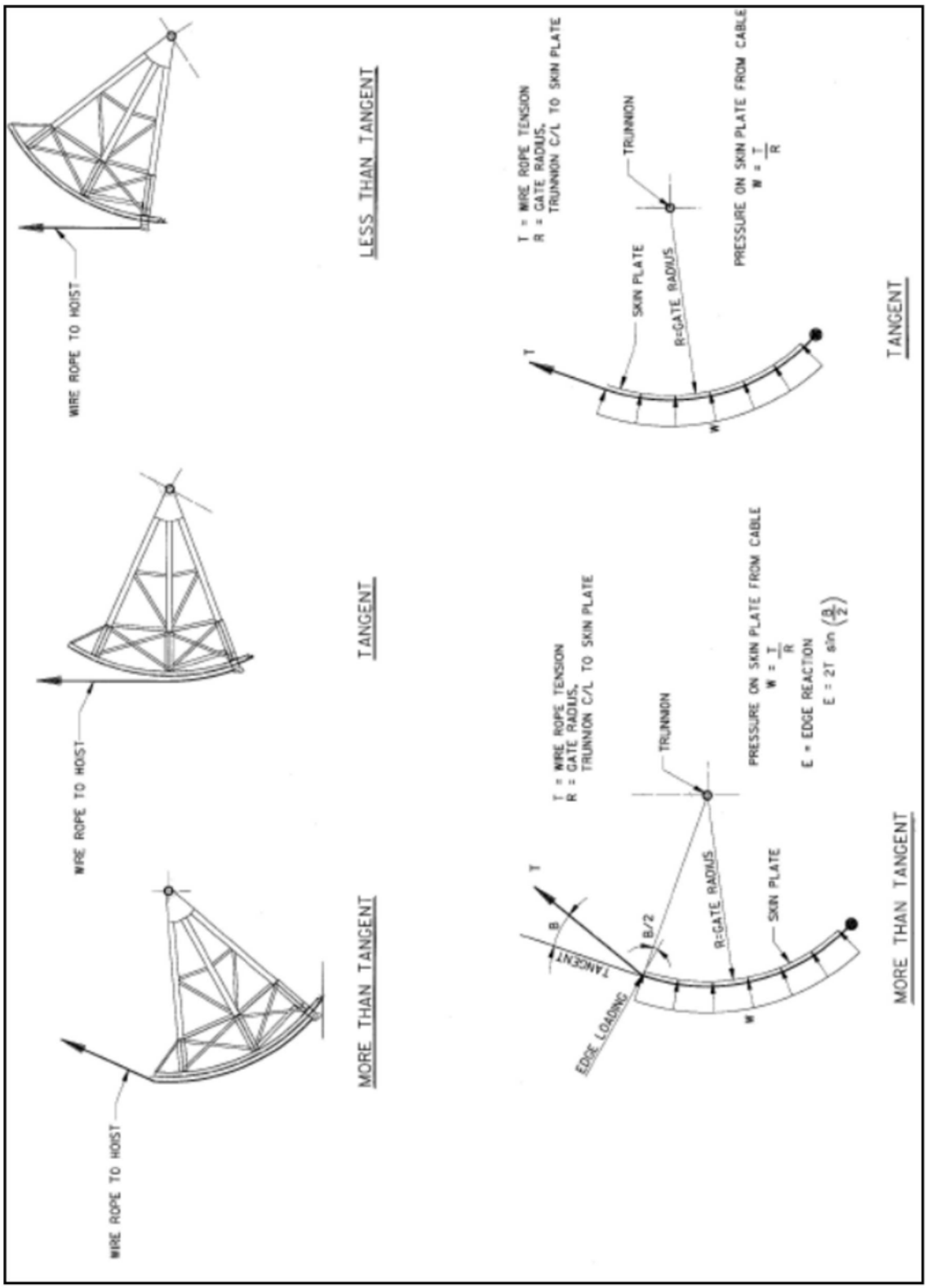


Figure D-6. Loads Due to Various Wire Rope Configurations

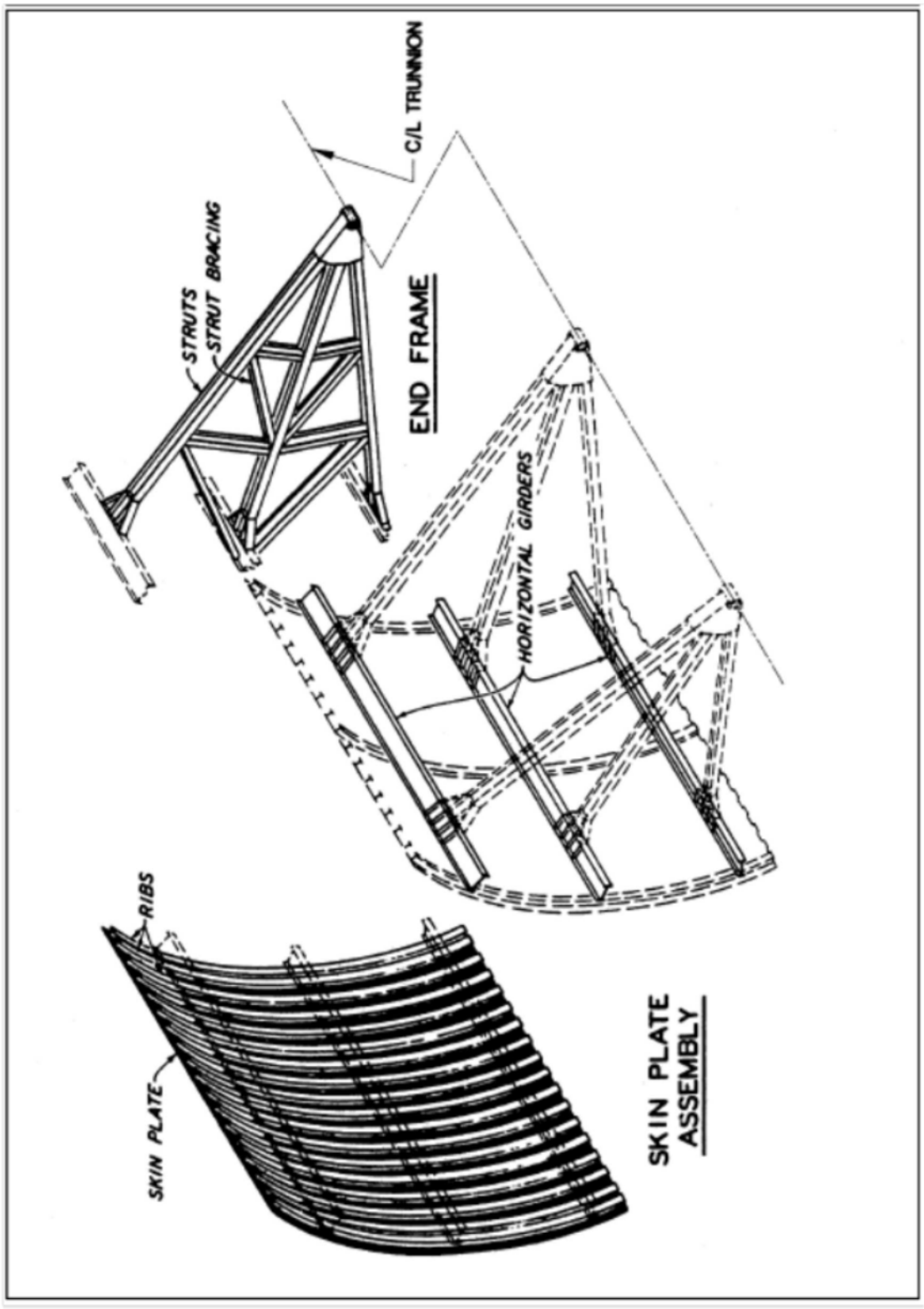


Figure D-7. Primary Tainter Gate Components.

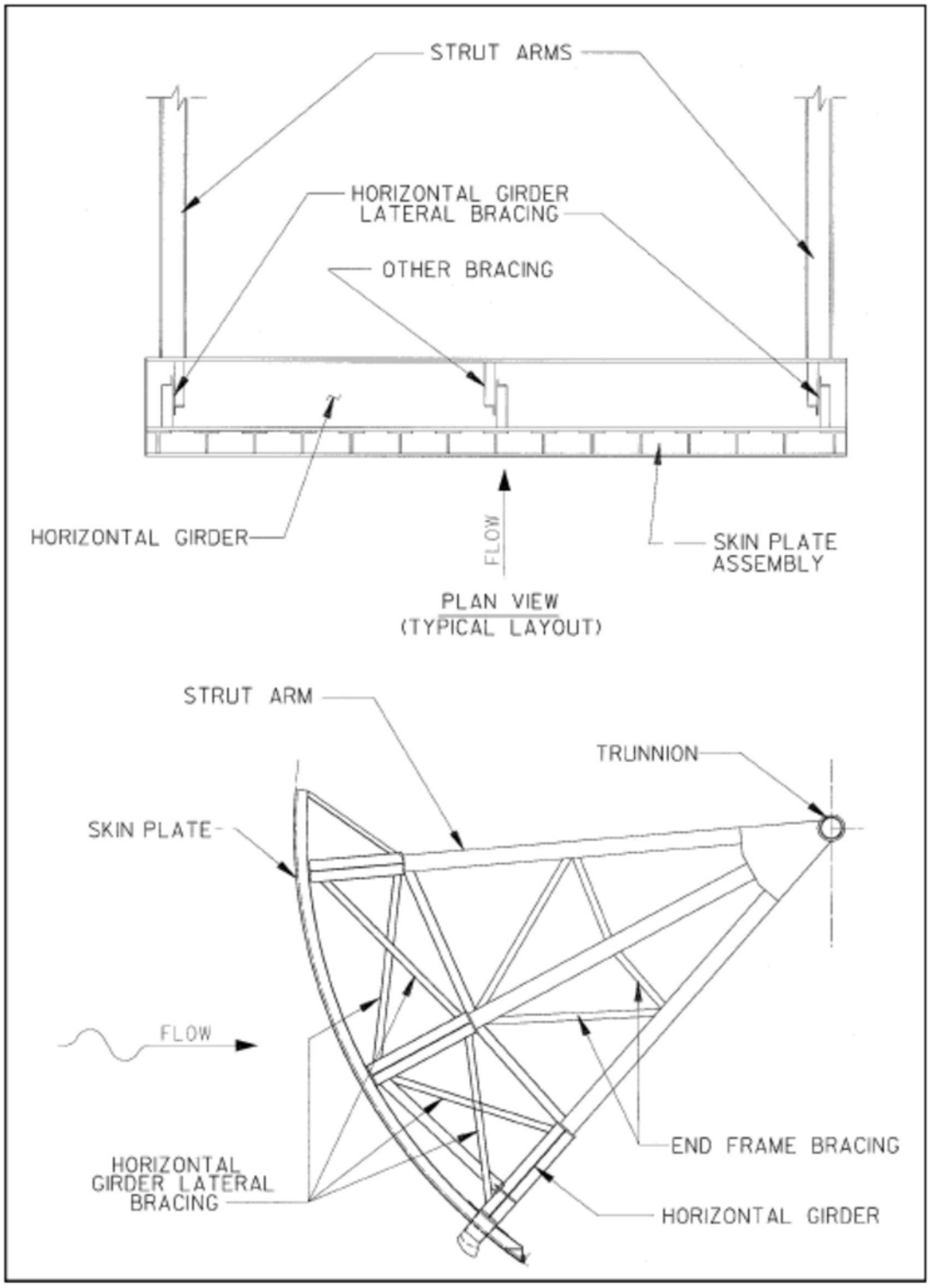


Figure D-8. Horizontal Girder Lateral Bracing.

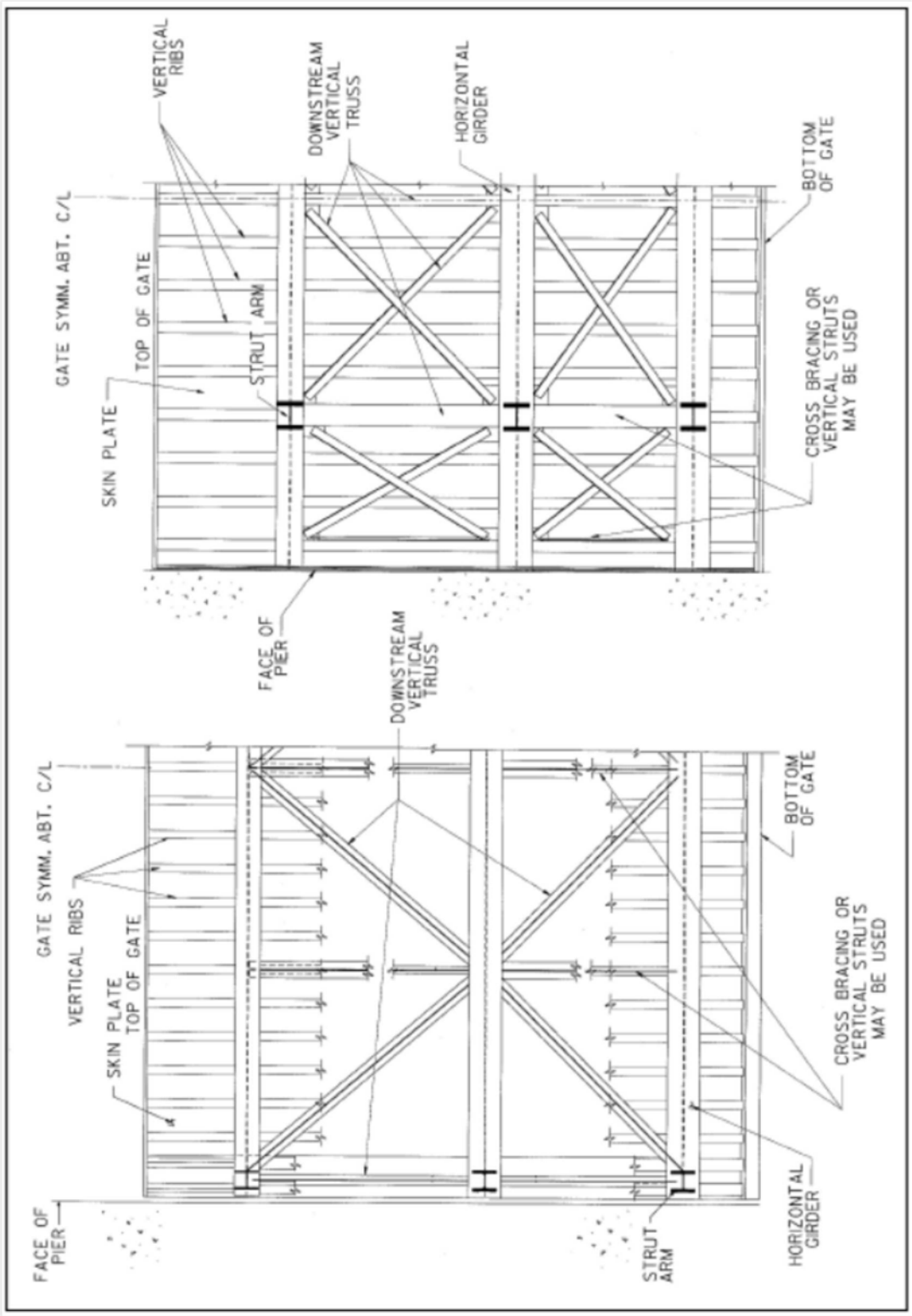


Figure D-9. Downstream Vertical Truss (Typical Configurations).

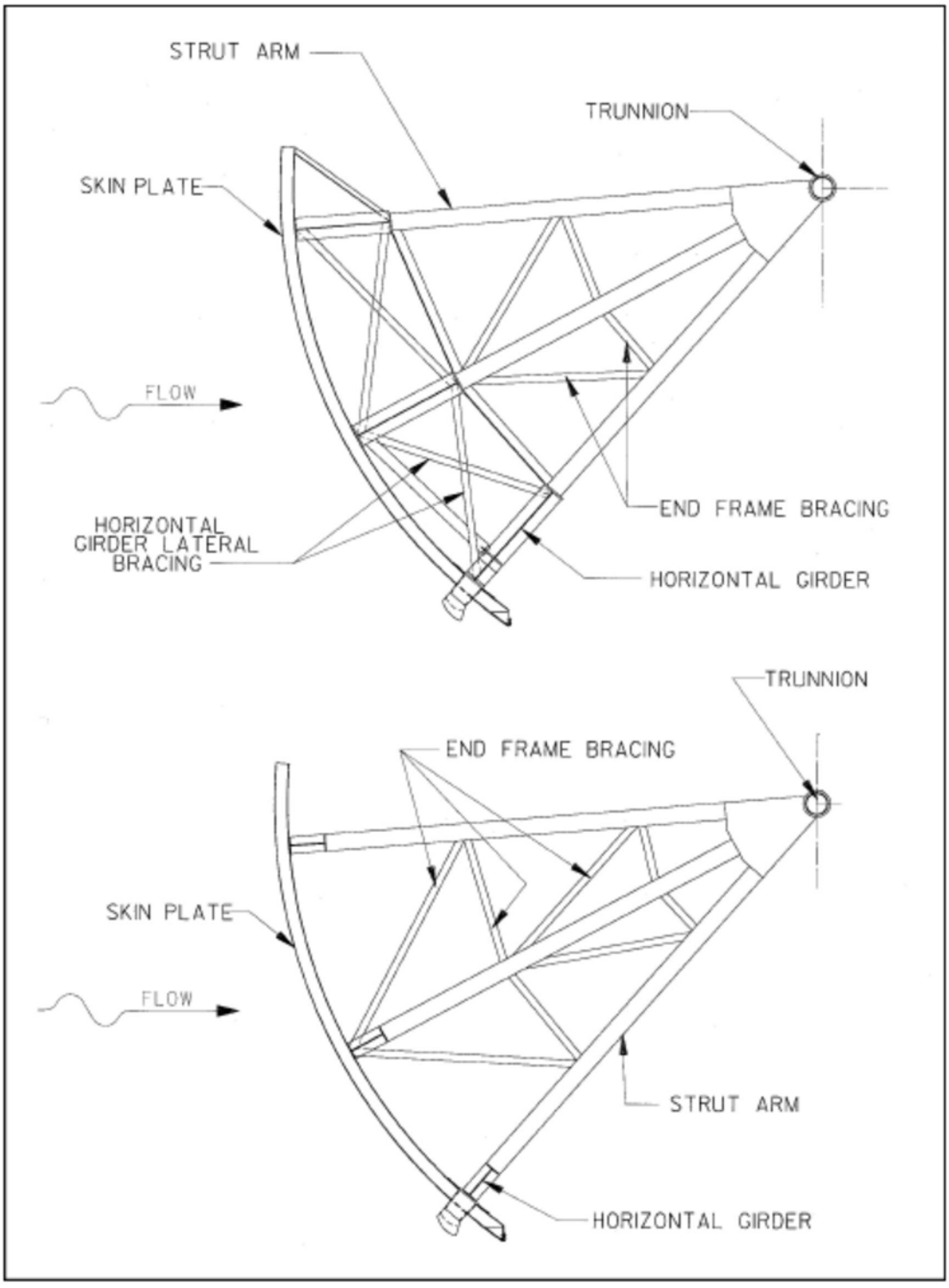


Figure D-10. End Frame Bracing (Typical Arrangements).

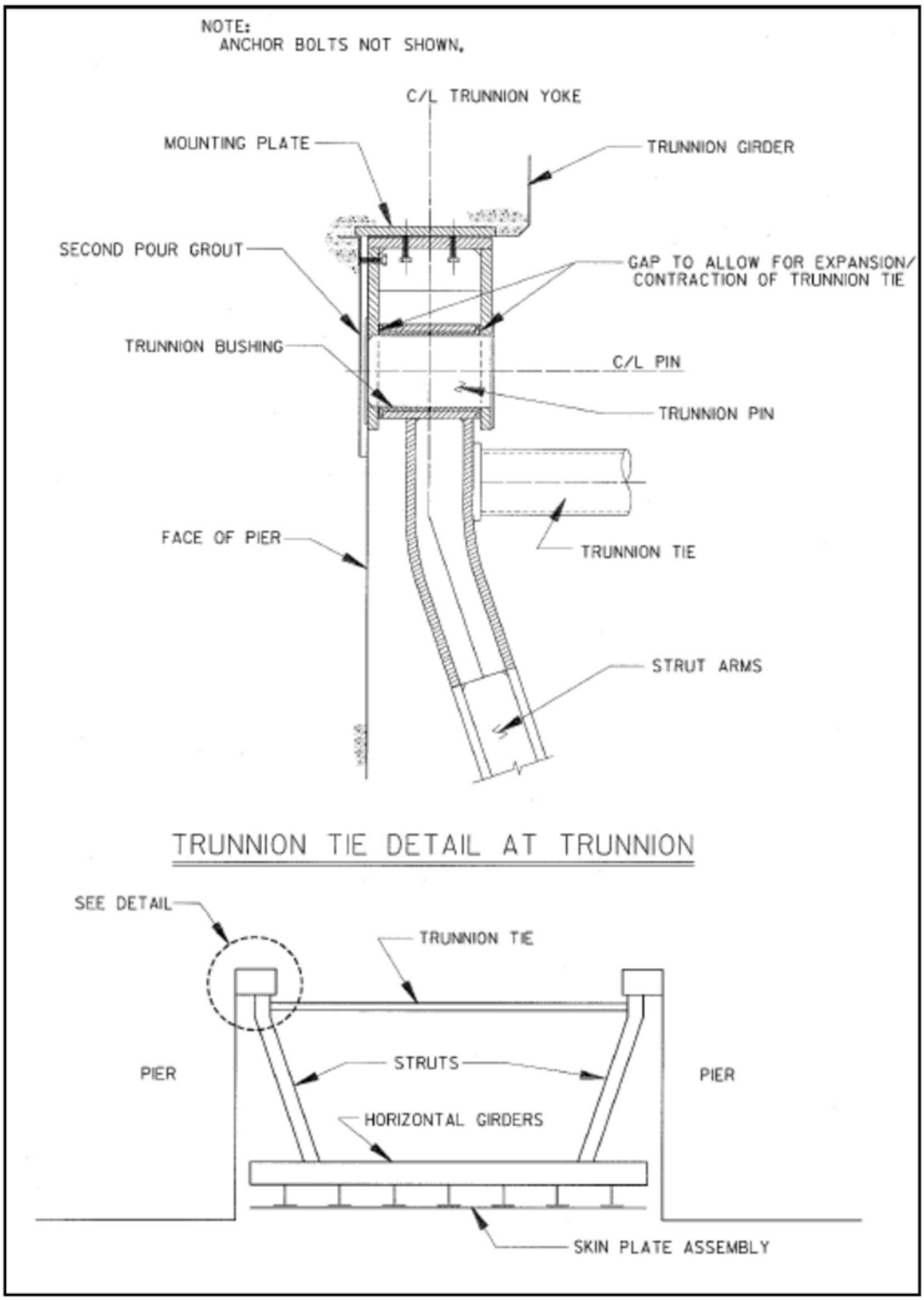


Figure D-11. Trunnion Tie.

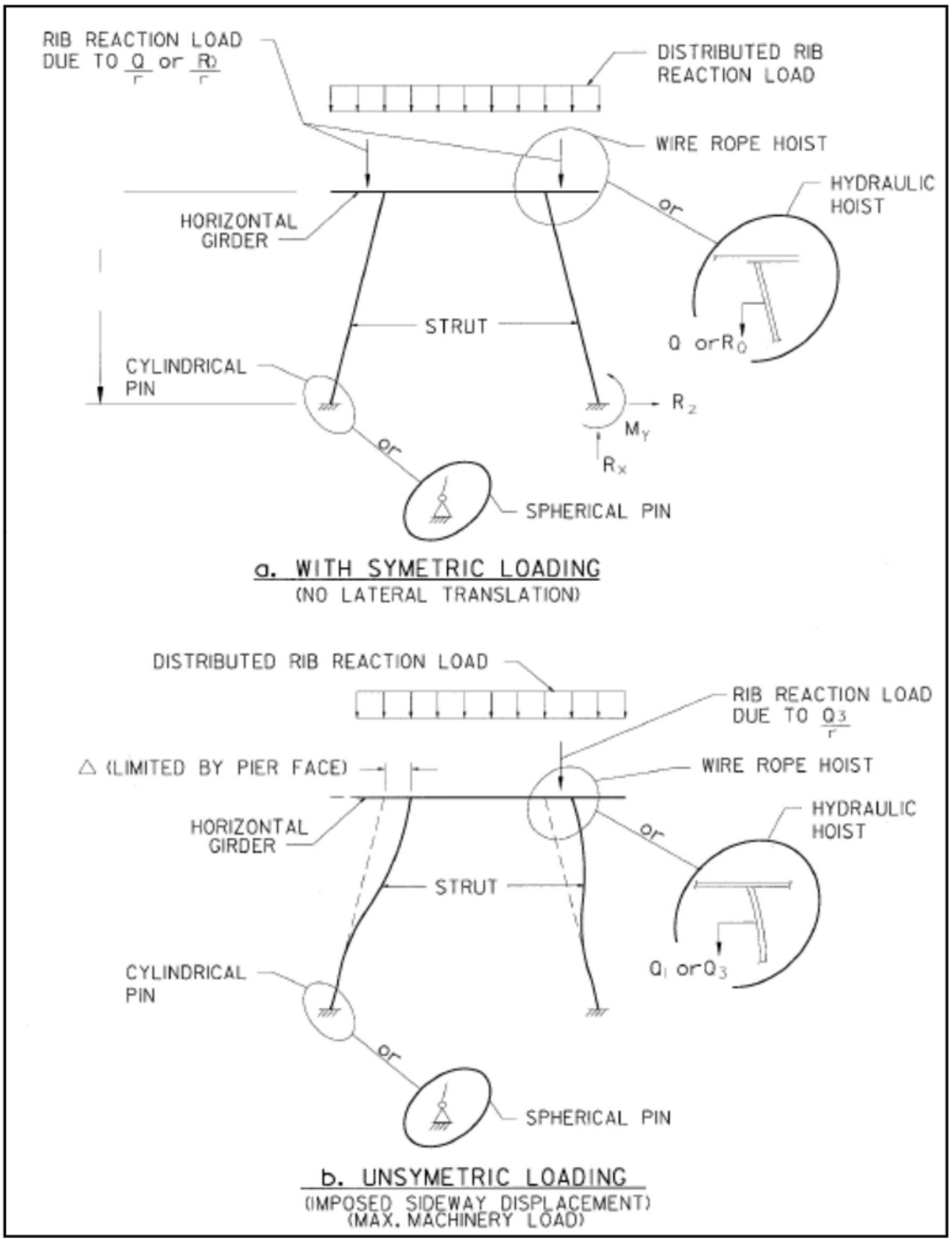


Figure D-12. Girder Frame Model Loads And Boundary Conditions.

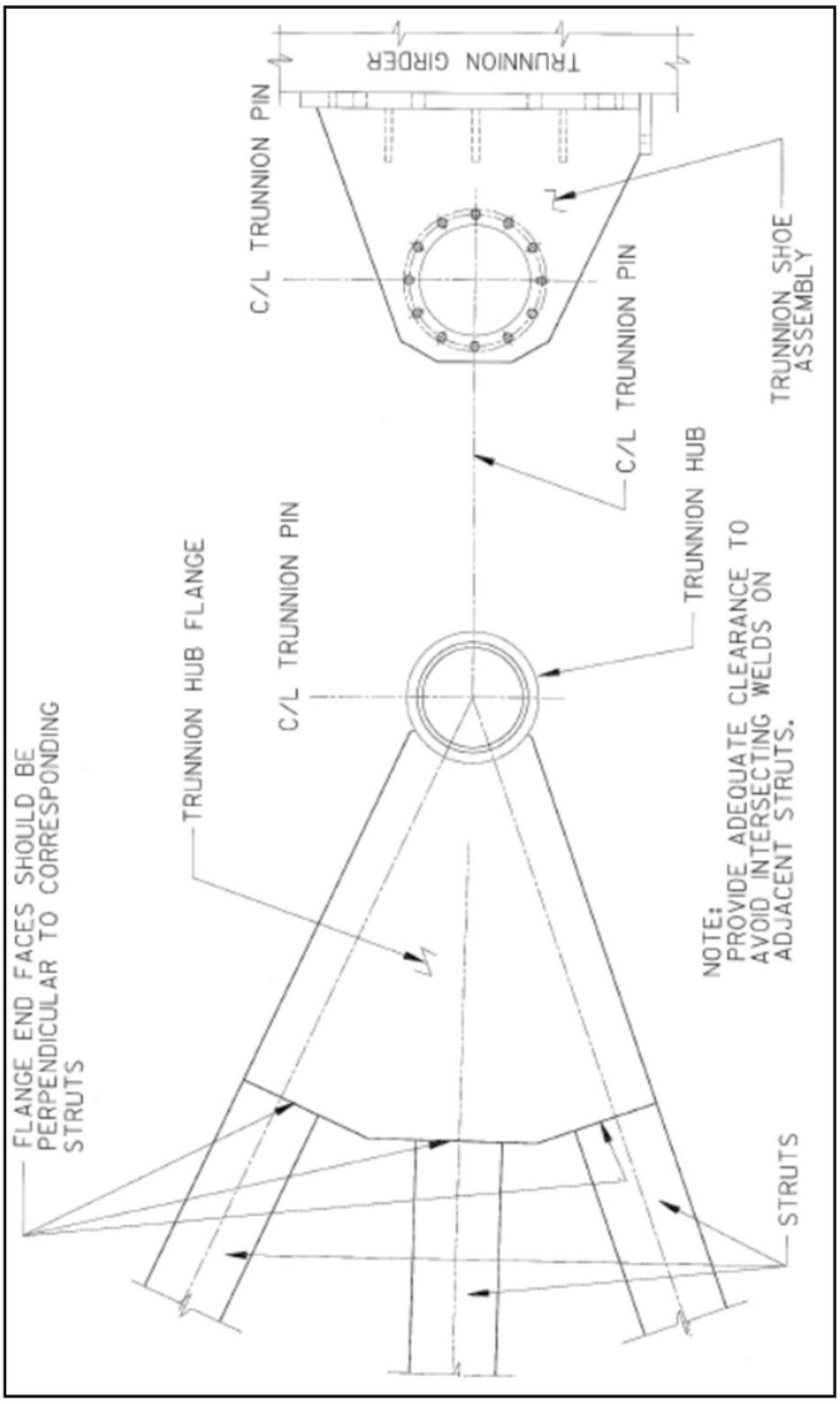


Figure D-13. Trunnion Hub Flange.

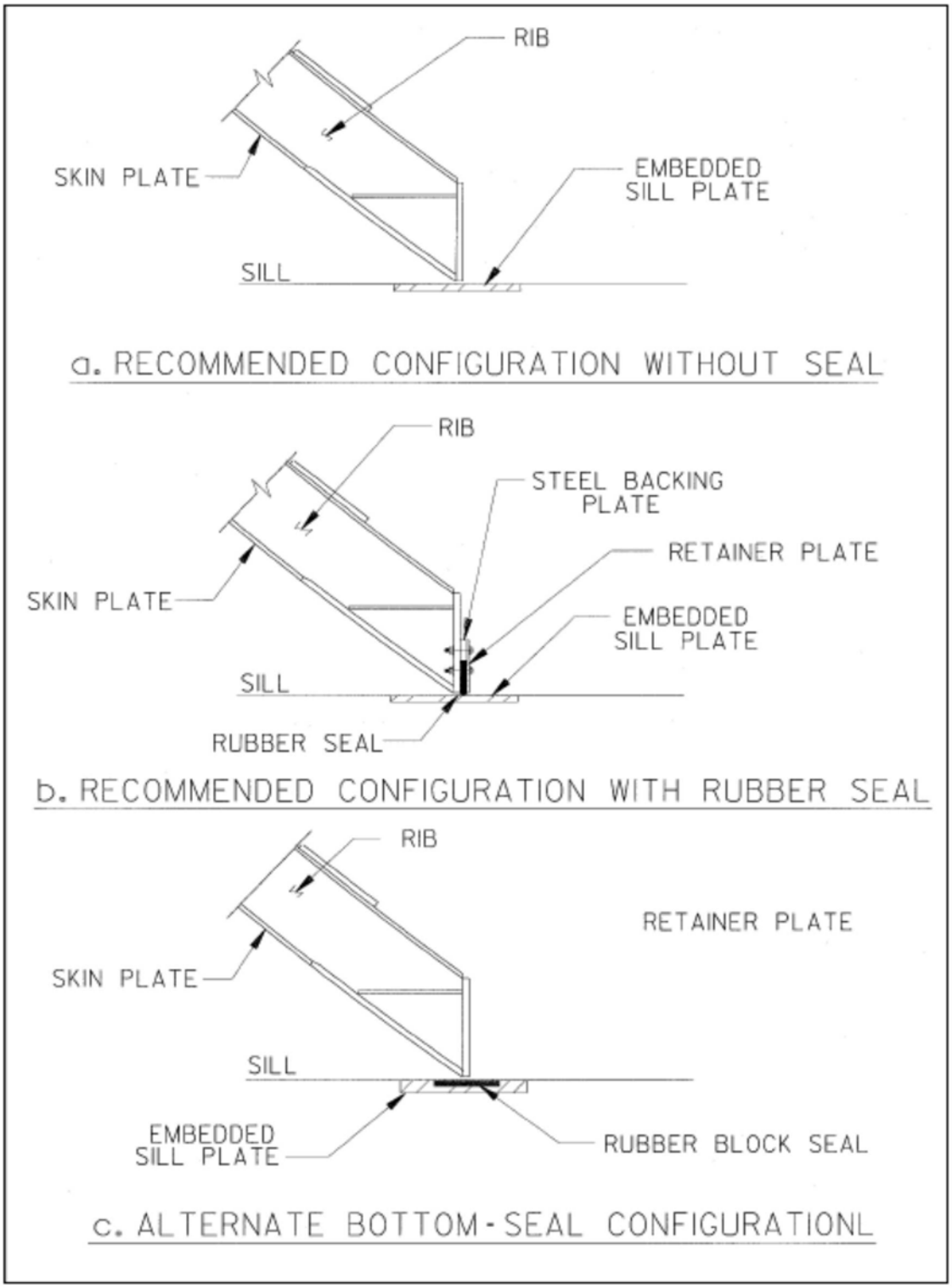


Figure D-14. Bottom Seal Configurations.

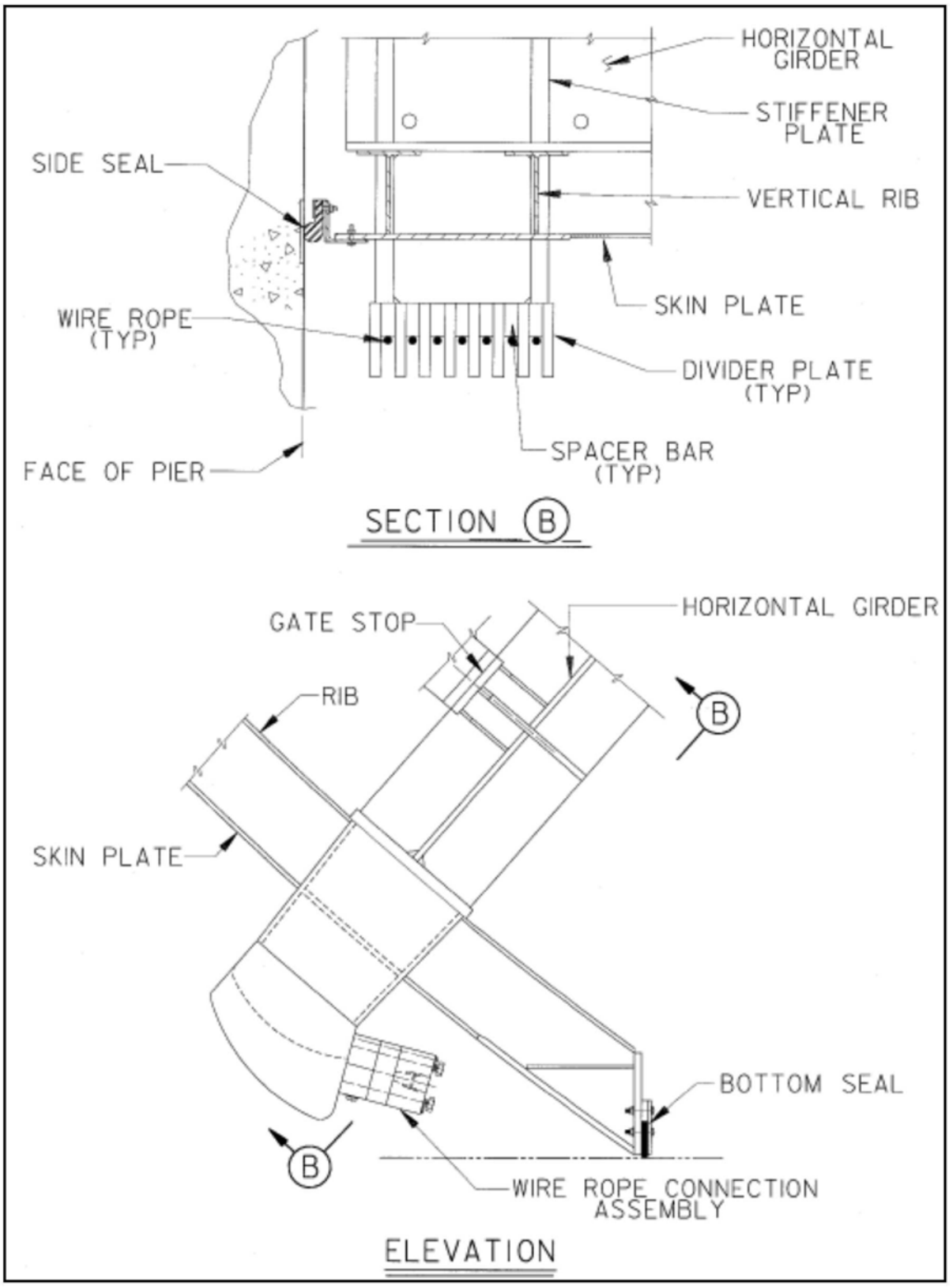


Figure D-15. Wire Rope Connection Bracket.

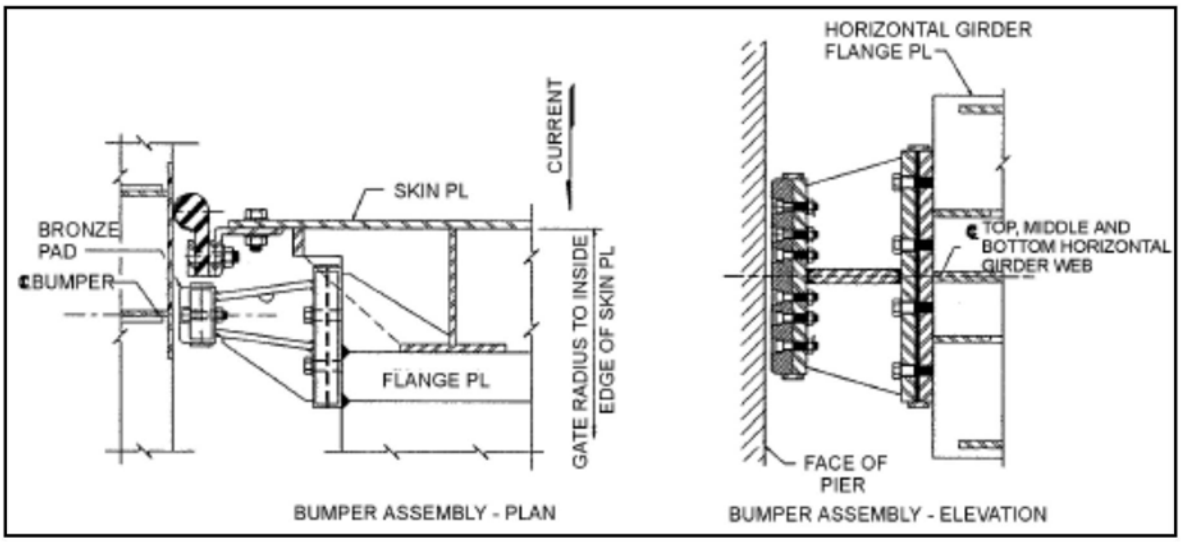


Figure D-16. Typical Gate Bumper Assembly.

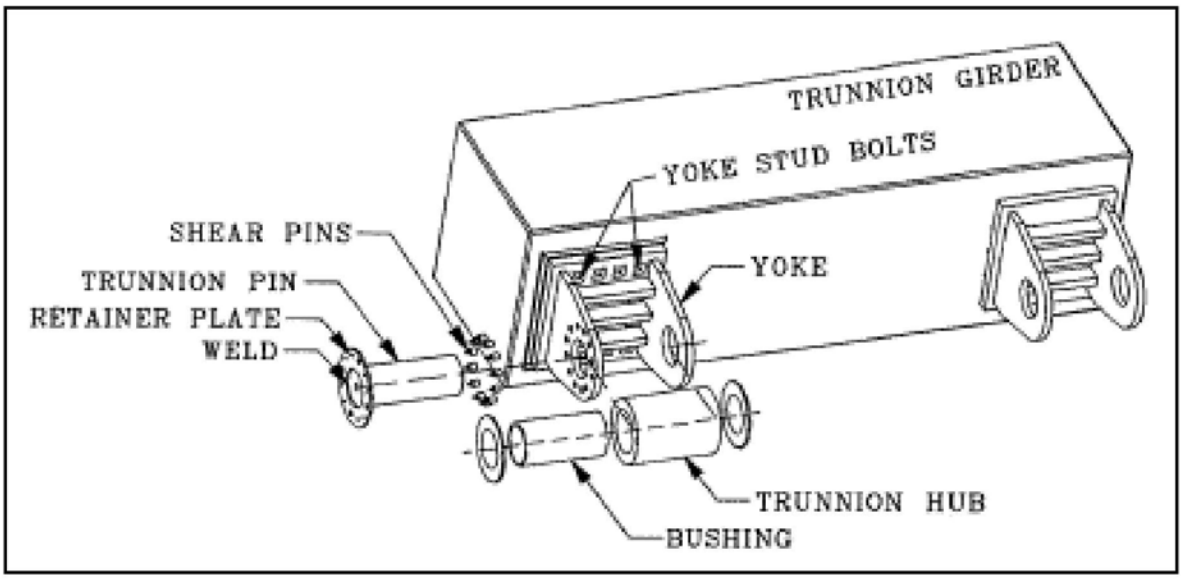


Figure D-17. Trunnion Assembly with Cylindrical Bushing.

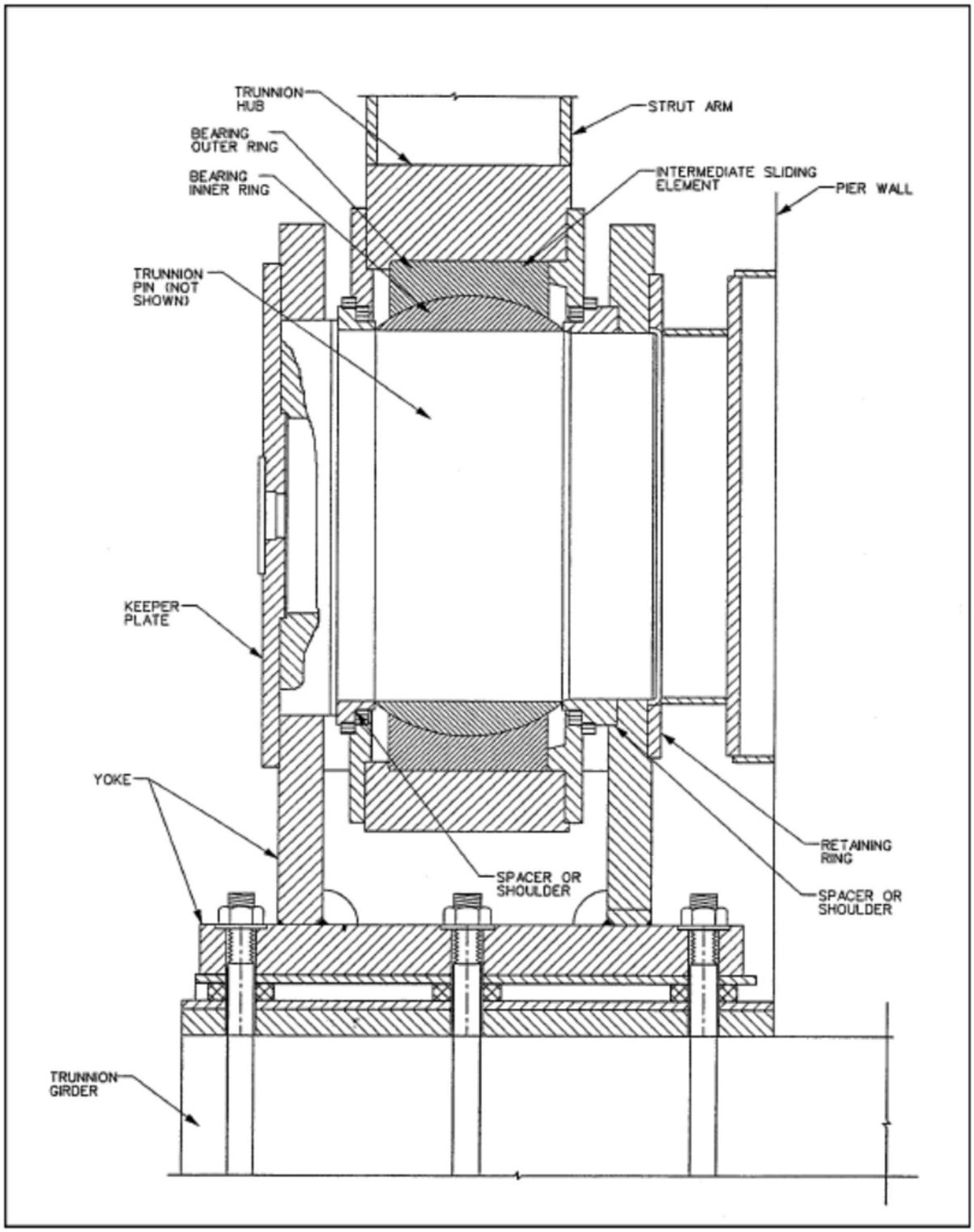


Figure D-18. Spherical Bearing.

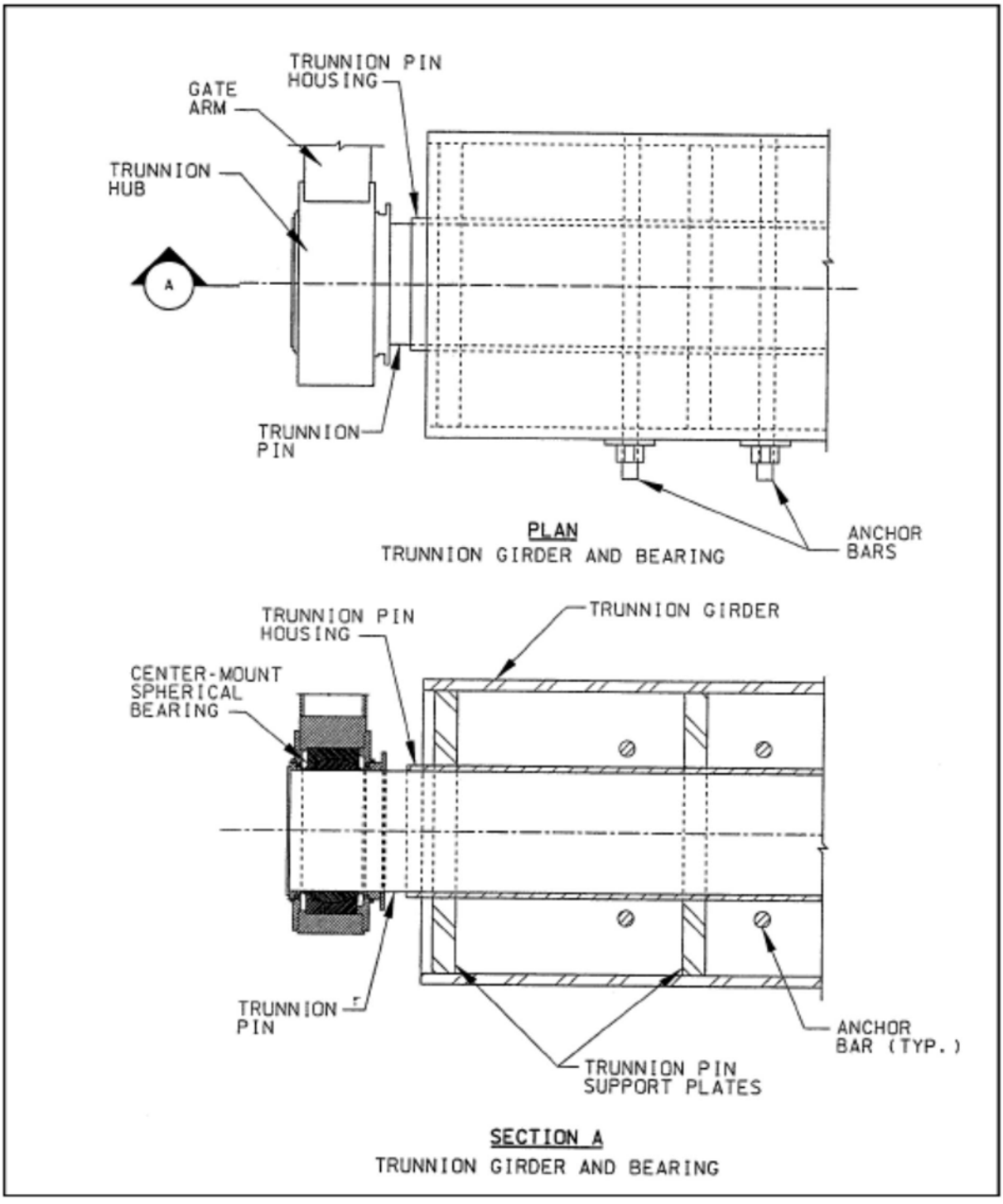


Figure D-19. Center-Pin Mount Bearing.

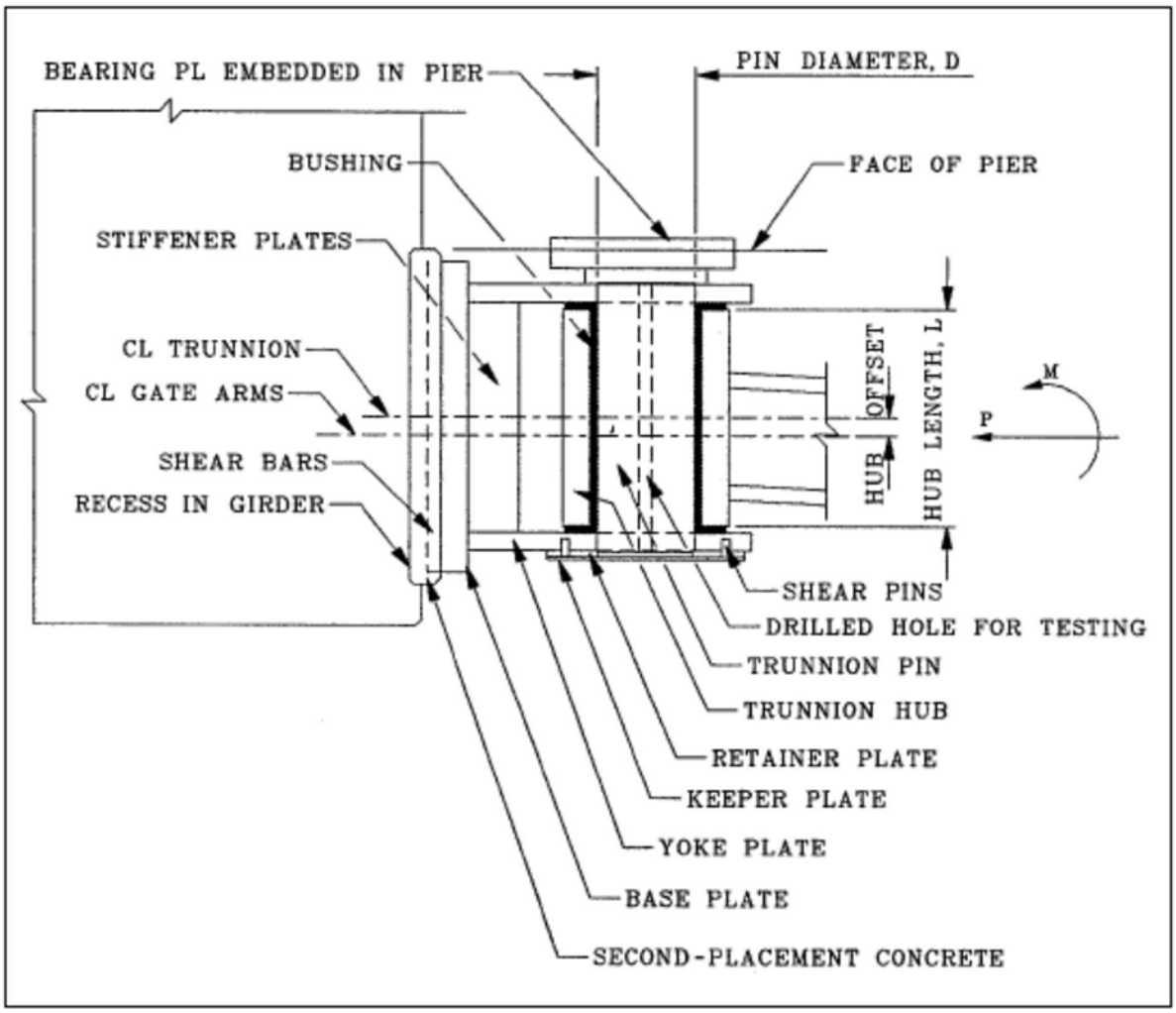


Figure D-20. Trunnion Assembly Structural Component Layout.

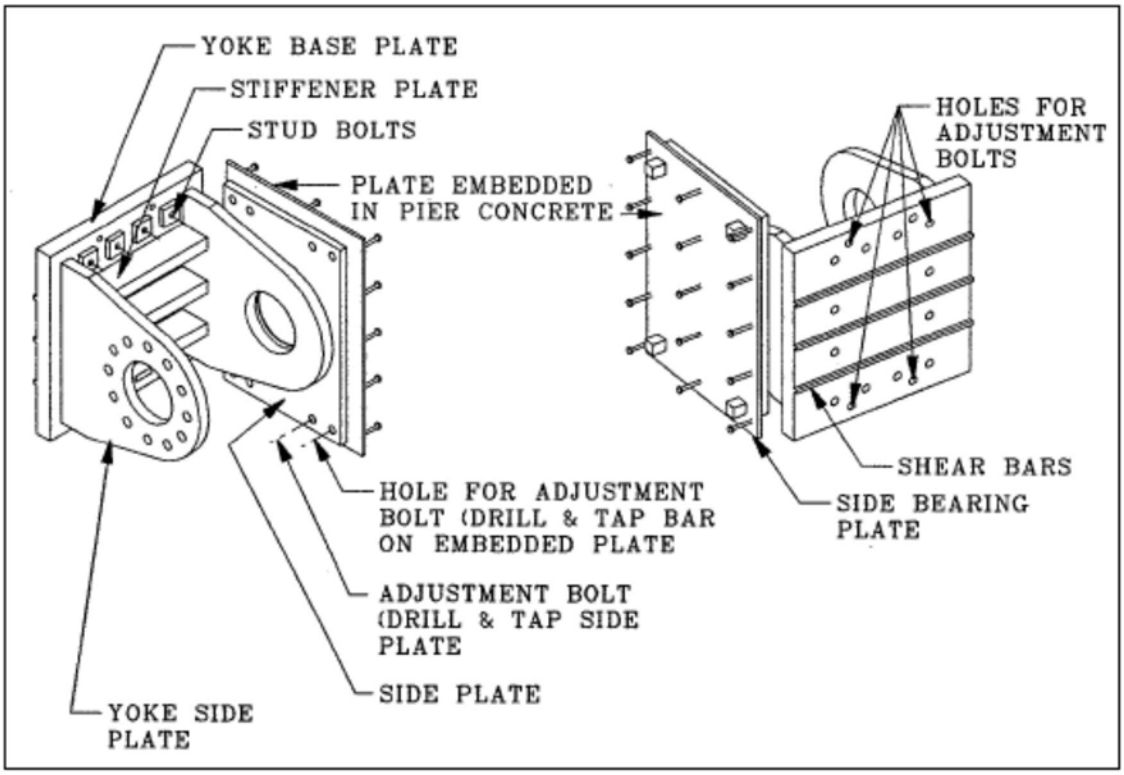


Figure D-21. Typical Trunnion Yoke Assembly.

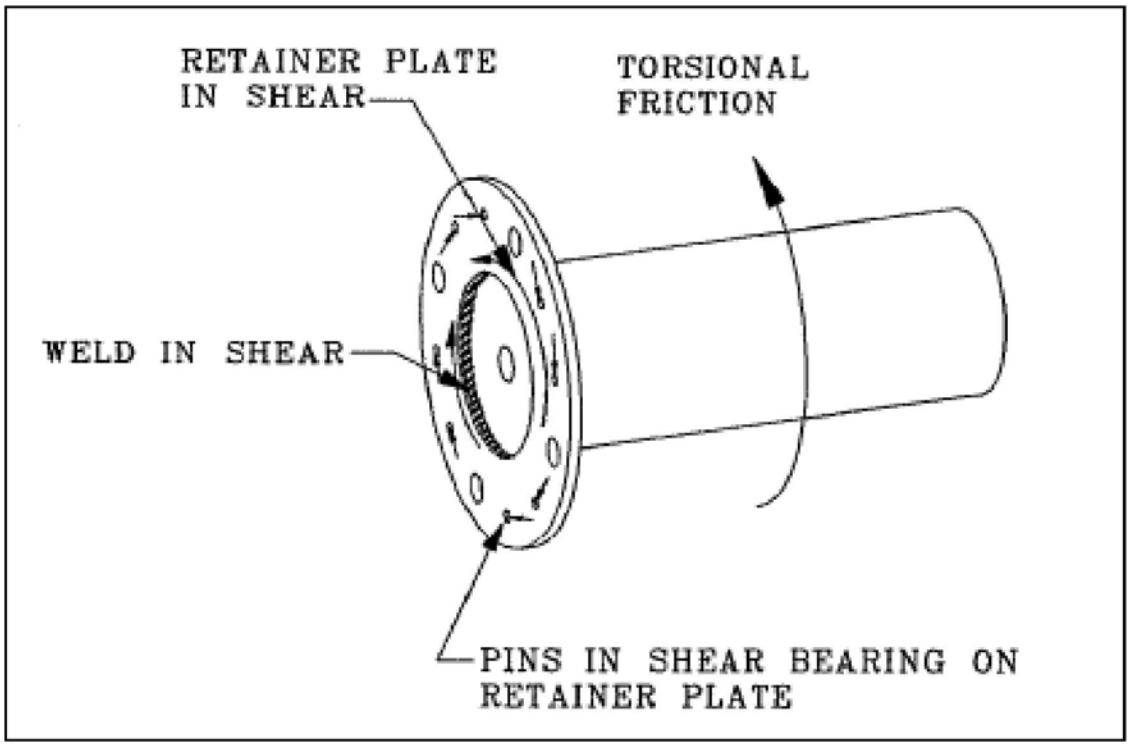


Figure D-22. Trunnion Pin and Retainer Plate.

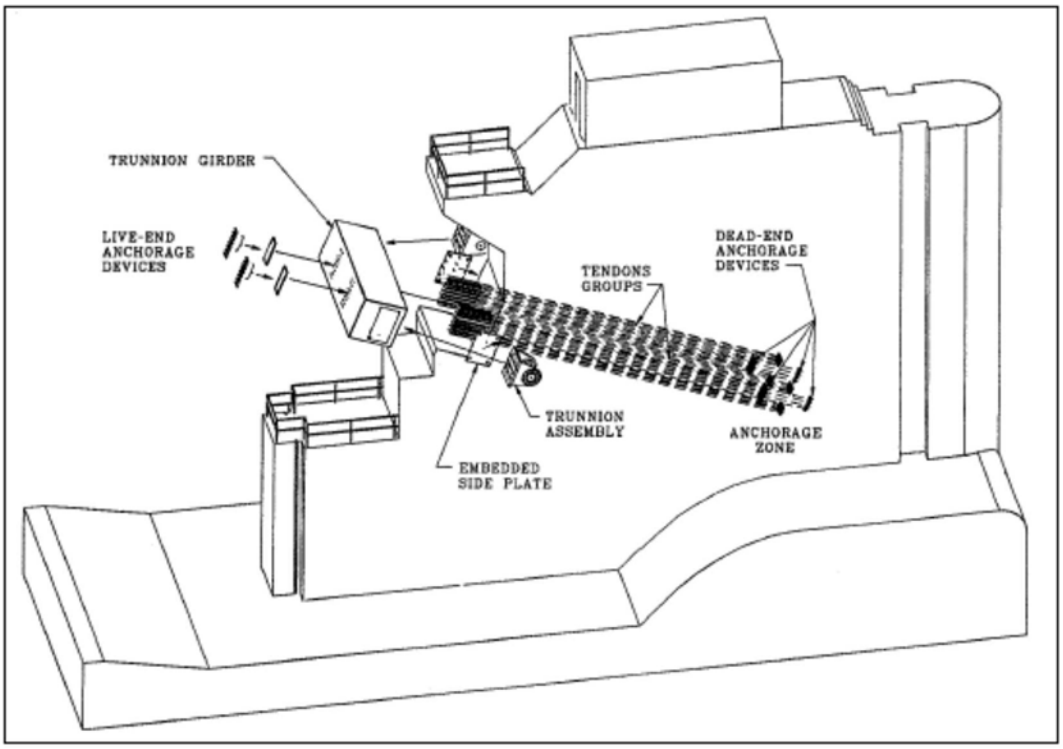


Figure D-23. General Arrangement of Gate Anchorage.

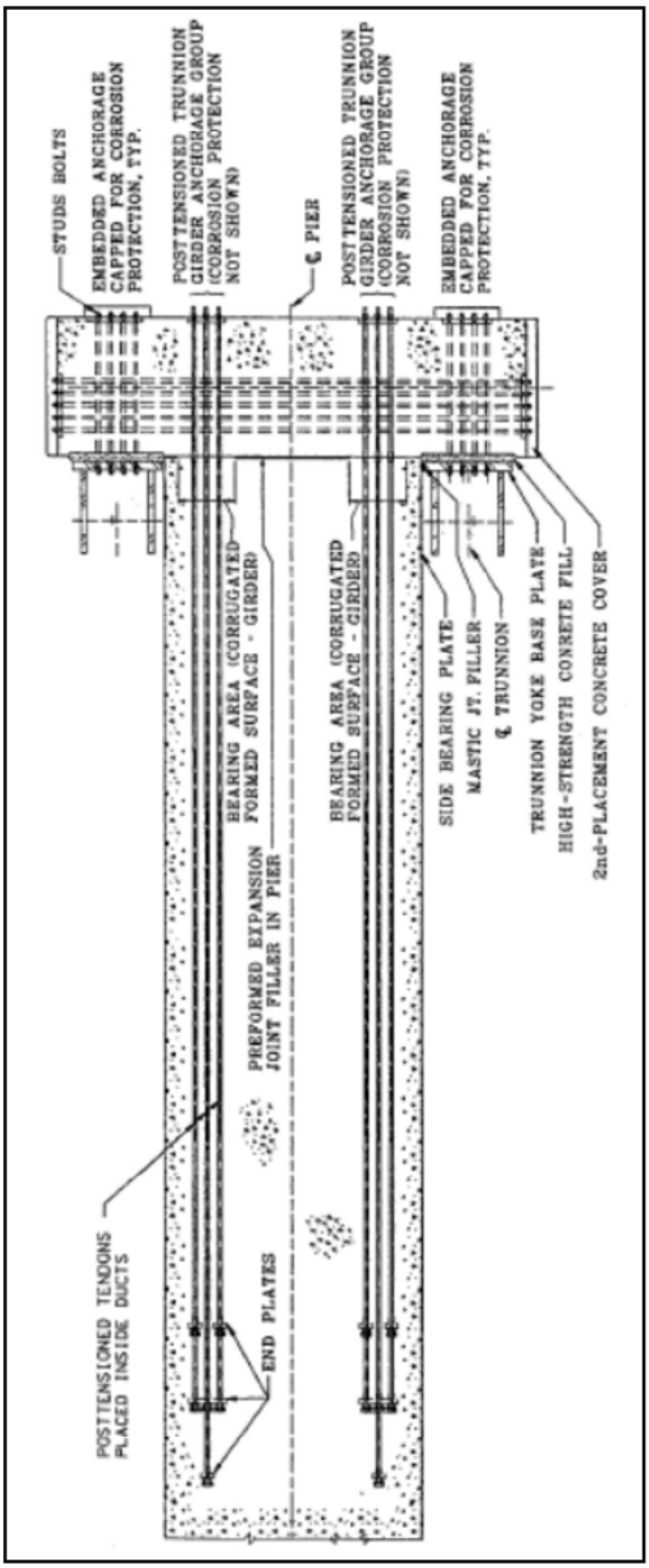


Figure D-24. Post-Tensioned Anchorage System.

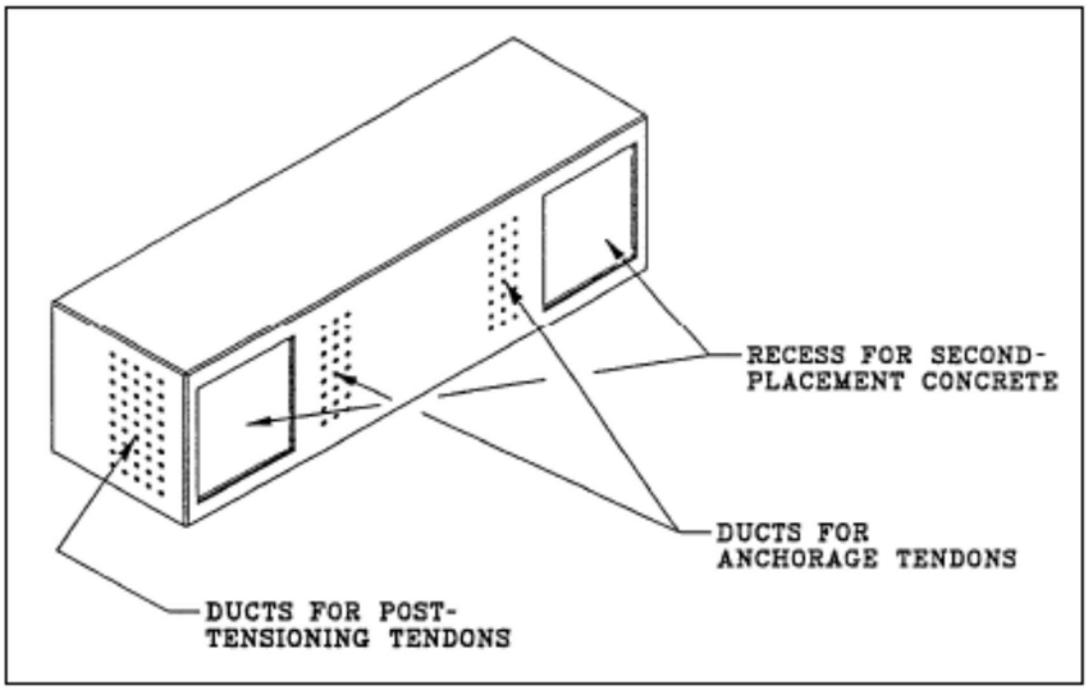


Figure D-25. Concrete Trunnion Girder.

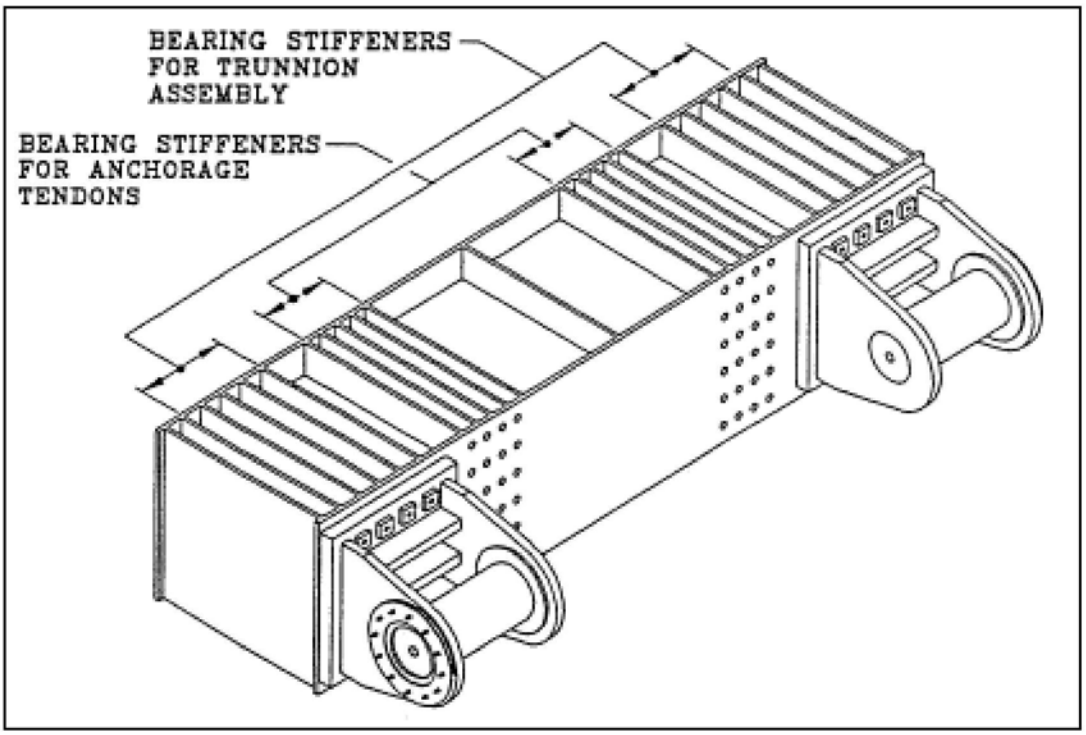


Figure D-26. Steel Trunnion Girder.

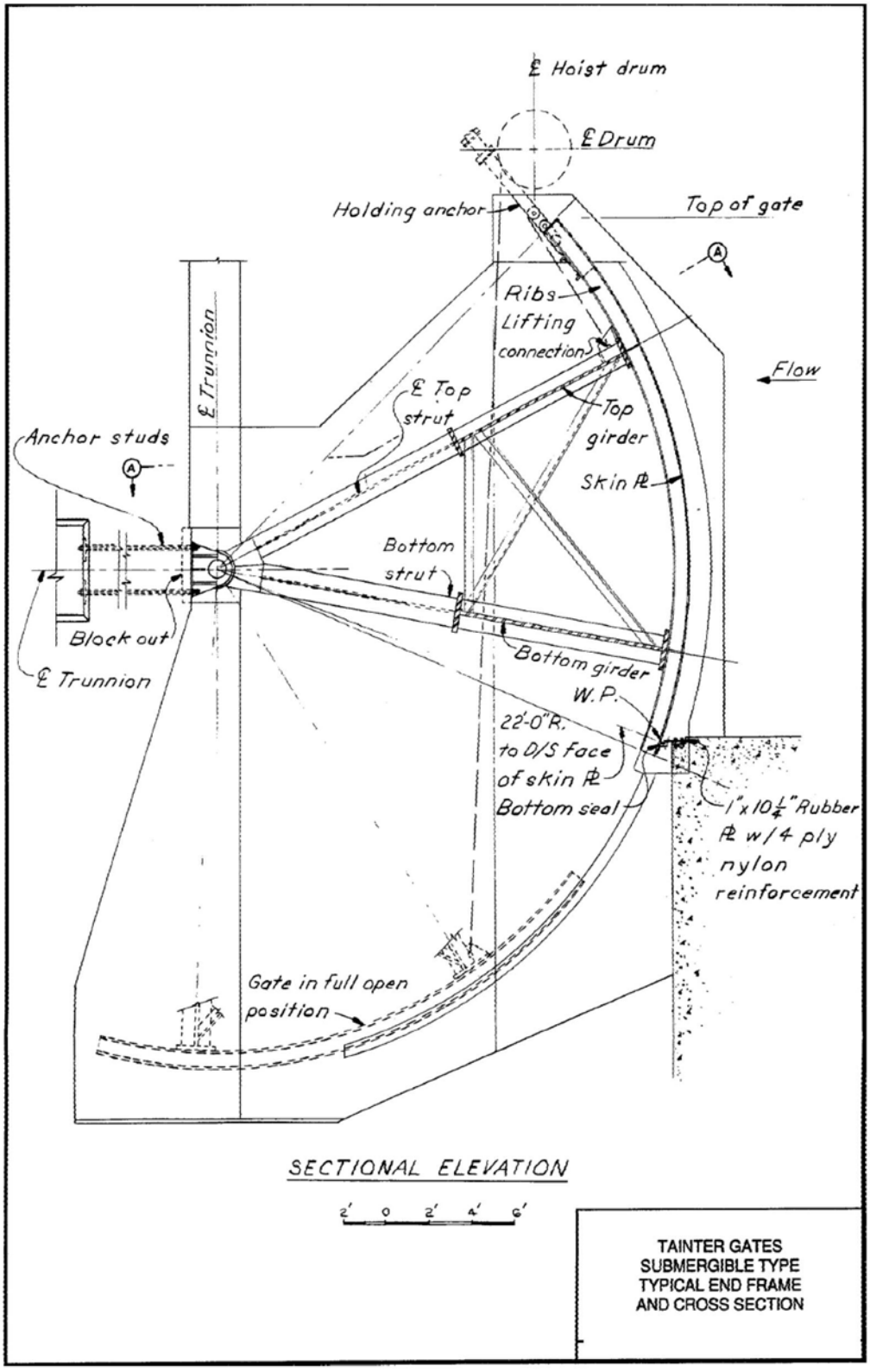


Figure D-27. Submergible Tainter Gate Typical Cross-section.

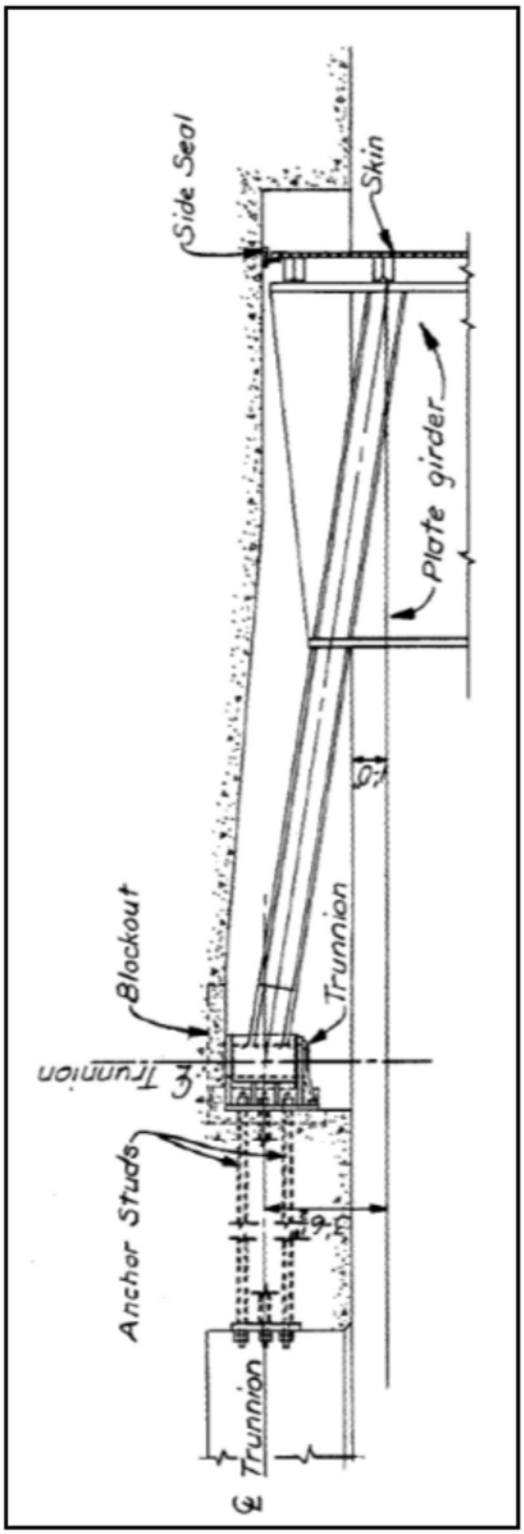


Figure D-28. Submergible Tainter Gate Typical End Frame.

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# APPENDIX E

## Vertical Lift Gates

E.1. Introduction.

E.1.1. Scope. This manual 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, maintenance bulkheads, and slide gates, specific criteria have not been developed.

E.2. Description and Application.

E.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. Each type of gate used has its advantages and disadvantages and is designed to accommodate special requirements for closure and retention of hydraulic head.

E.2.2. Gate Types and Applications.

E.2.2.1. 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 a miter gate at the other end. They can be raised or lowered under low to moderate heads, and are often used to skim ice or debris through the lock chamber. However, they are not used when there is reversed head. At the upstream end a lift gate might be a single leaf submergible gate, which 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 provided 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. When a lift gate is used at the downstream end of a lock, it is 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 E-1 shows a general configuration for a double-leaf submersible gate. Plate E-2 shows an overhead lift gate.

- 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. Plates E-3 to E-5. show examples of a horizontal truss and tied arch. These gates are often used as tide or hurricane gates along the seacoast and for inland navigation locks. When they are used as hurricane gates, they are normally raised to permit normal water traffic to pass underneath and lowered to protect harbors from tidal storm surges. This type of gate would be used 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.

- 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. Plates E-6 and E-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. 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.

E.2.2.2. Spillway Crest Gates. This type of gate is sometimes preferred over tainter gates because the spillway crest requires a shorter length of spillway pier and provides a more economical pier design. 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 are sometimes provided to engage projections spaced at intervals on the gate to hold the gate at the proper elevation. In some cases, it may be advantageous to mount the dogs in the gate and provide a dogging ladder in the gate slot. However, the other arrangement is preferred. Different types of spillway crest gates are:

- Single-section. This gate consists of one section that provides a variable discharge between the bottom of the gate and the sill. Single-section gates operate similarly to multiple-section gates, except that single-section gates are dogged off in the service slots.

- Multiple-section. 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. The latching mechanisms should be carefully designed so they do not stick or corrode. This has been a maintenance problem for some projects. Plate E-8 shows the top section of a multiple-section gate.

- Double-section. 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, because sealing is more complicated, and because additional length of pier is required. This type is useful for skimming ice and trash. However, that function can also be performed by shallow top sections of a multiple-section gate that are lifted clear of the pool.

E.2.2.3. Outlet Gates. Often, lift gates are used for emergency closure of water intake systems or outlet works. Their normal operation is in the open position. They are not used for throttling flows.

However, they are used to stop flow under operating conditions. They normally rest on dogging devices during normal operation. In emergencies, they are lowered into the closure slot to stop the flow of water.

- Powerhouse. Emergency gates are required for sudden closure of the turbine intakes to prevent subsequent damage to the turbines or powerhouse. These types of gates are normally framed with horizontal girders and a downstream skin plate. Upstream skin plates may be used when silt or mud fills up girder webs. Diaphragms are used to transfer vertical loads from the hoists. The hoisting system uses either hydraulic cylinder(s) 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. See EM-1110-2-4205 for additional information and requirements. Because these gates must be capable of operating under full head and flowing water, tractor type gates are used to reduce friction. See paragraph E2.3 for descriptions of types of end supports. Plate E-9 depicts this type of gate.

- Outlet works. Emergency closure gates for outlet works are similar to those used in powerhouses. They are often used for service gates and flow control. It is advantageous to use tractor gates for fully submerged outlet works, as in intake towers, due to the reduced friction under full head and flow. However, many gates use wheels where loading allows. The hoisting system may require the use of a gantry crane or its own hoisting system, either wire rope or hydraulic.

E.2.3. 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.

E.2.3.1. Fixed-Wheel. With this type of end support, the 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(s) 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. 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. 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.

E.2.3.2. Tractor (Caterpillar). This type of end support, also referred to as caterpillar, 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 E-9 shows this type of end support system. Plate E-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. Their main advantages over fixed wheels are 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. When compared to slide gates, the main advantage is reduced friction, which reduces the hoisting effort required for controlling flow. This reduced friction also reduces the wear and maintenance as compared with that of slide gate seal surfaces.

E.2.3.3. 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. 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. However, 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. 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, which use J-seals along a seal plate.

E.2.3.4. 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. 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. 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 gates.

E.2.4. Advantages/Disadvantages.

E.2.4.1. General. The use of overhead or submersible lift gates for navigation locks versus miter gates, sector gates, or submersible tainter gates would be based on economics, riverflow operational criteria, and navigation lock configuration. Some of the main advantages of using vertical lift gates are ease of fabrication, considerably shortened erection time, and in most cases, shorter monoliths or supporting piers for spillways, powerhouse intakes, and navigation locks compared with those of tainter or radial gates. The load from the gate to the supporting pier or monolith is in one direction, simplifying the design of the supports. One main disadvantage of using vertical lift gates that are under constant cyclic loading is that the main load-resisting frame relies on a tension flange or, in the case of an arch, on a tension tie. In these cases, fatigue plays a primary role in their design. The use of fixed-wheel, tractor, Stoney, or slide gates versus tainter gates for spillways and outlets also depends on head, size of gate, river flow operational criteria, and economics.

E.2.4.2. Navigation Locks. For high-lift requirements, or when the leaf of a submersible gate must rest on the bottom of the lock chamber or in a recessed sill, an overhead gate would be more desirable than a submersible gate. In the case where submersible leaves rest on the bottom of the lock chamber or in a recessed sill, silt would lead to hoisting problems, to weak axis loading to the girders, and to higher

maintenance costs. Where there are high-lift requirements, a submersible gate would require multiple leaves to obtain the lift required to pass river traffic. This may not be advantageous when considering hoisting arrangements and costs. Where debris and ice must be passed through the lock, a submersible gate would be more advantageous than an overhead, miter, or sector gate. For a multiple-leaf submersible gate, the downstream leaf is lowered to allow flow through the lock to pass ice or debris. A disadvantage of the use of overhead lift gates is that a tower is required to house the mechanical equipment and to gain sufficient lift for barge traffic to pass through the lock. This can cause increased design effort, in which flexibility in the tower must be accounted for in the design of the guide/hoisting system. Undesirable cracking of the monolith can occur if the tower/monolith interface is not designed to account for the tensile stresses developed from flexure of the tower.

E.2.4.3. Spillway. For spillway crest gates, tainter gates are preferred over vertical lift gates. This is due primarily to lower maintenance. When multiple-section vertical lift gates are required, the latching mechanisms can become inoperable unless continued maintenance is performed. Since this can increase maintenance, it should be avoided. However, vertical lift gates would be preferred to tainter gates when:

- The elevation of the maximum controlled pool is so far above the sill that excessively long piers would be required for tainter gates, or;

- Flood discharges or drift conditions are such that any obstruction to the flow below the bottom of the spillway bridge is undesirable, requiring the gate to be removed.

E.2.4.4. Outlet Gates. For powerhouse intake gates, the normal preference for use would be a vertical lift gate, due primarily to savings in the length of the intake pier and ease of construction. Time savings would occur for gates used for outlet works. Normal use for these types of gates is a tractor gate due to its low friction during operation. The size of gate and head requirements determines the feasibility of slide, fixed-wheel, or tractor gates. Slide gates require precise machined tolerances on the seal surfaces from the gate to the bearing guides. This requires careful quality control during field installation. Wear and damage to the slide and bearing surfaces due to use and cavitation can require higher maintenance to the slide gate. It may be more cost effective to replace wheels, rollers, or seals on a fixed-wheel or tractor gate than to fill and machine the gate and bearing surfaces of a slide gate.

E.3. Navigation Lock Lift Gates.

E.3.1. General. Almost all lift gates use a horizontal framing system. Vertical framing systems are not structurally efficient in transferring loads to the side bearing surfaces and require special framing to accommodate roller guides for hoisting operations. Vertical framing systems are not recommended for new vertical lift gates, except where being replaced in kind. For navigation locks, framing for either the upstream or downstream gate uses girders, trusses, or tied arches. The framing system selected will depend on span, hydrostatic head, and lift requirements.

E.3.2. Framing Systems.

E.3.2.1. 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. For short gates, it is not advantageous to vary the spacing of the girders. However, for taller gates where the change in

hydrostatic loading will be more significant from the bottom sill to the top, it is more economical to vary the spacing. Varying the spacing will require additional attention to design of the intercostals and skin plate to compensate for the varying hydrostatic pressure and span between girders. The girders frame into end posts that transfer end shear from the girders to bearing, either on the gate guides or through the types of end supports described in paragraph E2-3. Intercostals are framed plates or structural shapes that span the layers of horizontal girders used to create two-way plate-bending action for the skin plate. Diaphragms are used to provide continuity of the gate by distributing horizontal loads more uniformly, supporting and distributing vertical loads. Plates E-6 through E-9 show examples of horizontal girder framing.

E.3.2.2. Trusses. Trusses may be more economical or weigh less than plate girders. Horizontal trusses would be most economical for navigation locks with high-lift overhead gates or for long horizontal spans across navigation locks. It may be advantageous to vary the spacing of the main trusses to achieve an economical arrangement of the same truss and member sizes throughout the height of the gate. Plate E-5 shows a typical use of horizontal trusses for navigation lock framing. Common members used for the trusses are wide flanges and structural Ts. The main trusses frame into an end post supported by an end-bearing similar to a stiffened plate girder. 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.

E.3.2.3. Tied Arches. This type of framing, as with trusses, is normally employed for high-head and long span gates used in navigation locks. Because of the load transfer ability of the arch, this framing is generally more structurally efficient than plate girders. Particular care must be used in designing the main tension tie, as there is little redundancy if it fails. Therefore, fatigue design becomes most critical for these members, particularly in the connection of the arch girders to the main tension tie. The members can be made of rolled shapes, built-up members, solid plates, or plate girder members. Normally the front arch is framed with structural Ts, with the webs welded continuously to the skin plate. Plates E-3 and E-4 show a vertical lift gate of this type. As with girders, other framing members include intercostals, diaphragms, end posts, stiffeners, and skin plates.

E.3.2.4. Vertical Framing. This type of framing system is not very common and is not recommended for use. However, this type of gate may be more economical if it is being used to replace a gate of the same type. Vertically framed gates most commonly use stiffened plate girders. With this type of system, the main load is transferred from the skin plate to vertical girders that frame into a horizontal main girder at the top and bottom of the gate. The load transfer is through the top girders to the end wheels at the guide recess. This arrangement is unsuitable for large gates because of the concentration of load at the top and bottom of the end posts.

E.3.3. Navigation Lift Gate Loads. Chapter 3 describes loads for all gates. Loads that are applicable to lift gate design include self-weight, gravity loads, hydrostatic and hydrodynamic loads, operating loads, environmental loads, impact loads, and earthquake loads.

E.3.3.1. Self-weight. Dead load, D.

E.3.3.2. Gravity loads. Gravity loads include mud weight (M), and ice weight (C), and shall be determined based on site-specific conditions.

E.3.3.3. Hydrostatic loads. Hydrostatic load, $H_s$, consists of hydrostatic pressure on the gate considering both upper and lower pools.

- For submersible gates, consideration must be given to the operation of a multiple-leaf gate, with the gate seals effective and ineffective. Figure E-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. 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. Figures E-2 and E-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, $H_s$ represents the maximum head differential between upstream and navigation lock pool elevations. During normal operation, Figures E-4 and E-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 $H_s$ represents the maximum head differential 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.

- Figures E-6 and E-7, respectively, show the hydrostatic load $H_s$ and water seal arrangements for overhead gates with and without a crossover gallery. For both conditions, $H_s$ represents the maximum head differential 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 $H_s$ represents the maximum head differential between the upstream pool elevation and tailwater pool elevation, or upstream sill.

E.3.3.4. Hydrodynamic loads. Hydrodynamic, $H_d$, loads include overtopping of submersible gate leaves and wave load $W_A$. See Section 3.2.3.2 for determining wave loads. For submersible gates, Figure E-8 shows the operation of the downstream leaf when passing ice and debris. In this case, $H_d$ represents the head from the flow overtopping the downstream leaf.

E.3.3.5. Operating loads. Under normal operating conditions, operating loads are treated as reactions to all opposing forces including D, G, $H_d$, and friction. In the case of gate binding, the operating load, Q, shall be the maximum load that can be exerted by the operating machinery (obtained from the mechanical Engineer that designed the machinery). See Section 3.2.3.3 for further discussion on operational loads.

E.3.3.6. Barge impact. Barge impact load, IM, shall be specified as a point load and shall be applied to any portion of the gate exposed to barge traffic at locations that produce the maximum effects in the primary members of the gate. The magnitude of load shall be equal to 250 kips.

E.3.3.7. Environmental loads include thermal loads, T, and wind, W. The effects of extreme thermal differentials, T, caused by ambient air and water temperatures adjacent to the exposed faces of the gate, shall be determined based on the navigation lock at full pool, exposing the skin plate to the pool temperature and the downstream girders or tension ties to ambient conditions and tailwater. This shall include temperature differentials related to seasonal ambient and water temperatures. For moderate climates the ambient temperature range shall be from 0 to 120 °F, and for cold climates from –30 to 120 °F Pool temperatures shall be based on observed or recorded data

and applied to the season during which the maximum ambient temperatures are predicted to occur. Wind loads shall be based on a 300-year return period per ASCE 7. Wind load shall be applied normal to the projected surface of the gate. For submersible gates, wind loads need not be applied.

E.3.3.8. Earthquake design loads E. See Section 3.2.3.6 for earthquake loading.

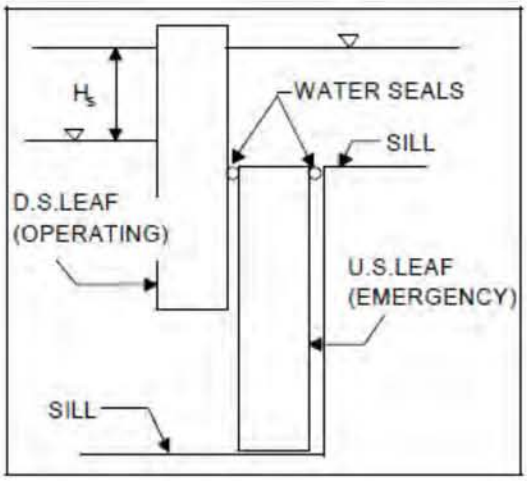


Figure E-1. Submersible Lift Gate, Normal Operation.

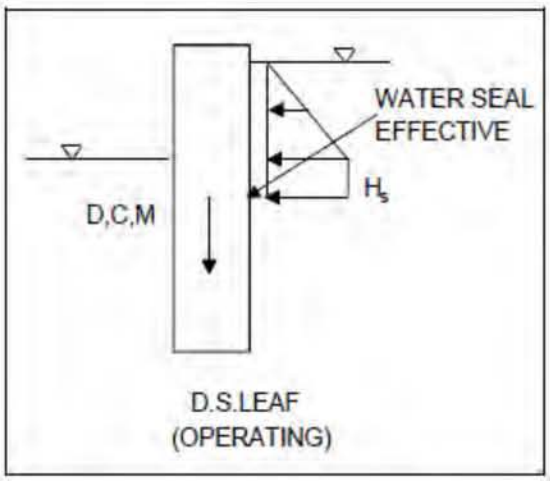


Figure E-2. Submersible Lift Gate, Hydrostatic Loading Diagram, Downstream Leaf, Seals Effective.

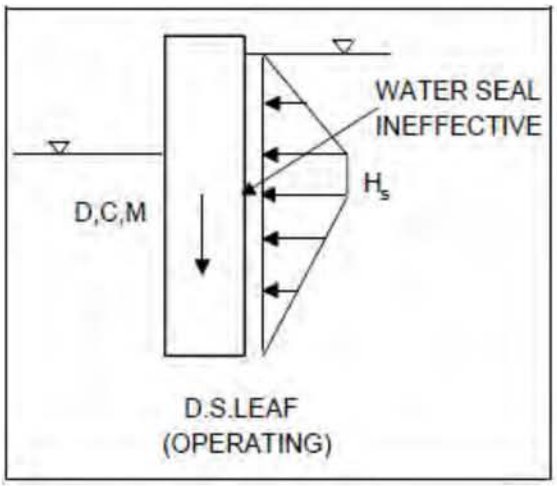


Figure E-3. Submersible Lift Gate, Hydrostatic Loading Diagram, Downstream Leaf, Seals Ineffective.

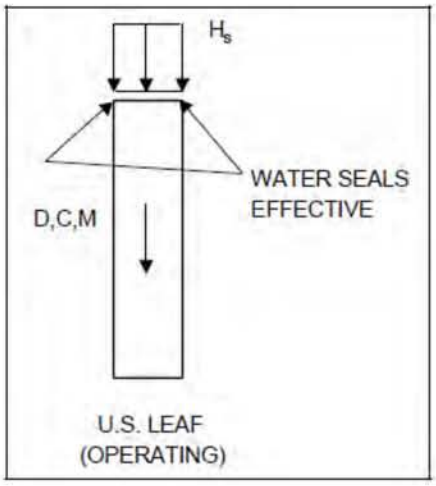


Figure E-4. Submersible Lift Gate, Hydrostatic Loading Diagram, Upstream Leaf, Seals Effective.

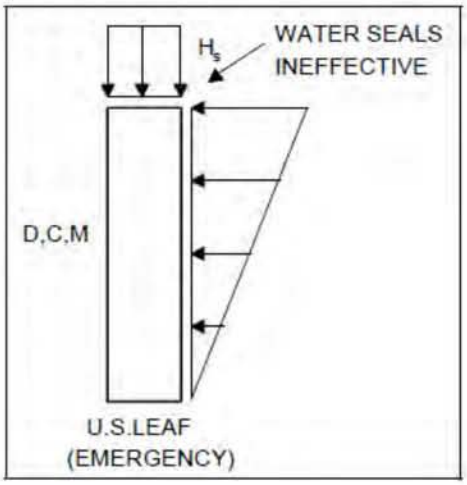


Figure E-5. Submersible Lift Gate, Hydrostatic Loading Diagram, Upstream Leaf, Seals Ineffective.

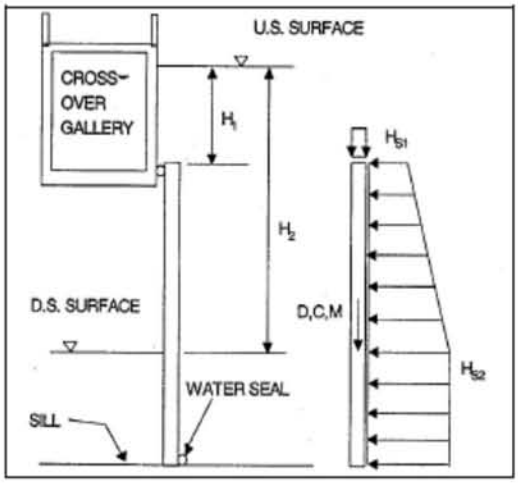


Figure E-6. Overhead Lift Gate with Crossover Gallery, Hydrostatic Loading.

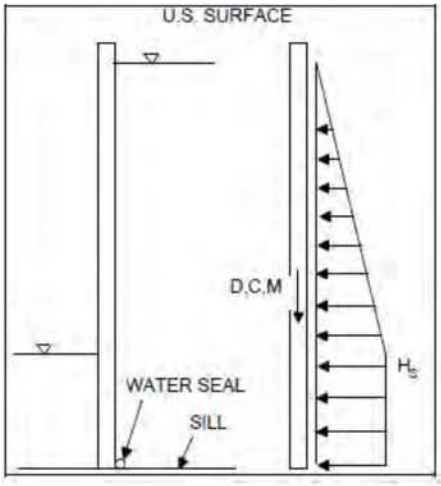


Figure E-7. Overhead Lift Gate without Crossover Gallery, Hydrostatic Loading.

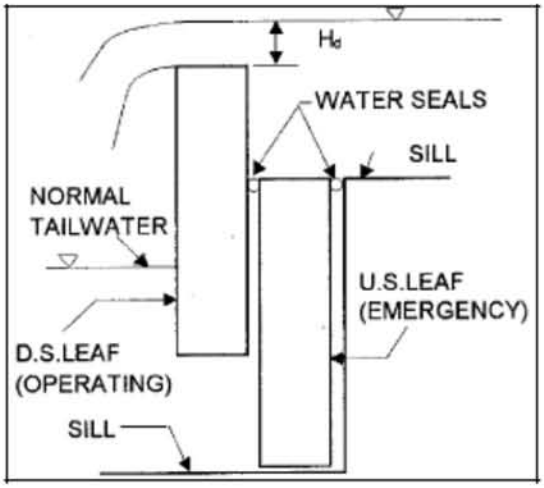


Figure E-8. Submersible Lift Gate, Hydrodynamic Loading For Passing Ice And Debris.

E.3.4. Load Cases.

General loads and loading combinations for gates are described in Section 3.3. Lift gates shall be designed for the Strength, Extreme, and Fatigue Limit States for each of the following load cases and corresponding load combinations. Table E-1 lists load factors for lift gates. The Serviceability Limit State is addressed in Section 3.1.3.

Case 1: Strength Limit State I, Gate Closed. Loads consist of self-weight and gravity loads plus unusual hydrostatic and unusual hydrodynamic loading, where hydrodynamic loading can be from either gate overtopping or waves.

$$\gamma_{D2}D_2 + \gamma_{G1}G_2 + \gamma_{Hs2}H_{s2} + \gamma_{Hd2}H_{d2} \quad \text{(E-1)}$$

Case 2: Strength Limit State I, Gate Closed. Loads consist of self-weight, gravity loads, unusual hydrostatic, and thermal loading.

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} + \gamma_{EV}EV \quad \text{(E-2)}$$

Case 3: Strength Limit State II, Gate Open. Loads consist of self-weight and gravity loads plus wind with wind applied in both upstream and downstream directions.

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma EVEV \quad \text{(E-3)}$$

Case 4: Extreme Limit State, Gate Jammed. Loads consist of self-weight and gravity loads plus maximum machinery loads.

$$\gamma_{D2}D_2 + \gamma_{G1}G_2 + \gamma_{Q2}Q_3 \quad \text{(E-4)}$$

Case 5: Extreme Limit State, Gate Closed. Loads consist of usual hydrostatic head differential with gate subjected to self-weight, usual gravity loads, and earthquake or impact, whichever produces the maximum effects.

$$\gamma_{D2}D_2 + \gamma_{G1}G_1 + \gamma_{Hs1}H_{s1} + (\gamma_{IM3}IM_3 \text{ or } \gamma_E E) \quad \text{(E-5)}$$

Case 6a: Fatigue Limit State: Fatigue Limit State I. Stress range for member computed based on usual operating conditions to include emptying filling plus coincident hydrodynamic loading and temperature changes.

$$\gamma_{Hs1}H_{s1} + \gamma_{Hd1}H_d + \gamma_{EV}EV \quad \text{(E-6)}$$

Case 6b: Fatigue Limit State II. Stress range for member computed based on unusual operating conditions (hydrostatic head differential) to include emptying filling plus coincident temporal head and temperature changes.

$$\gamma_{Hs1}H_{s2} + \gamma_{Hd1}H_d + \gamma_{EV}EV \quad \text{(E-7)}$$

Table E-1. Load Factors for Lift Gates.

| **Load Cases** | | | **Loads/Load Factors** | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | **D** | **G** | **$H_s$** | **$H_d$** | **Q** | **EV** | **IM** | **EQ** |
| Limit State | Description | Case | $\gamma_D$ | $\gamma_G$ | $\gamma_{Hs}$ | $\gamma_{Hd}$ | $\gamma_Q$ | $\gamma_{EV}$ | $\gamma_{IM}$ | $\gamma_{EQ}$ |
| Strength I | Gate Closed | 1 | 1.2 | 1.6 | 1.4 | 0 | 0 | 1.2 | 0 | 0 |
| Strength I | Gate Closed | 2 | 1.2 | 1.6 | 1.4 | 1.6 | 0 | 0 | 0 | 0 |
| Strength II | Gate Open | 3 | 1.2 | 1.6 | 0 | 0 | 0 | 1.3 | 0 | 0 |
| Extreme | Gate Open | 4 | 1.2 | 1.2 | 0 | 1.0 | 1.2 | 0 | 0 | 0 |
| Extreme | Gate Closed | 5 | 1.2 | 1.2 | 1.2 | 0 | 0 | 0 | 1.0(1) | 1.0(1) |
| Fatigue I | Finite Life | 4a | 0 | 0 | 1.0 | 1.0 | 0 | 1.0 | 0 | 0 |
| Fatigue II | Infinite Life | 4b | 0 | 0 | 1.0 | 1.0 | 0 | 1.0 | 0 | 0 |
| Notes: (1) Select one at a time | | | | | | | | | | |

E.3.5. Load and Resistance Factor Design. Appendix C includes design requirements using LRFD for lock lift gates.

E.4. Spillway Crest Lift Gates.

E.4.1. General. Similar to navigation lock gates, spillway crest gates use a horizontal framing system. As with navigation lock gates, vertical framing systems are not structurally efficient and require special framing to accommodate roller guides for hoisting operations. Hence, vertical framing is not recommended for new vertical lift gates, except for replacement in kind. Most spillway crest gates use a fixed-wheel end support system. Tractor and slide gates have been used for spillway crest gates. However, their use is not very common for this type of application. Much of the information regarding framing, loads, and load types for spillway crest gates is the same as that for navigation gates described previously. Only information unique to spillway crest gates is provided in this section.

E.4.2. Framing Systems. Horizontal girder framing is the most common type of framing system used for spillway crest gates. They may be framed with plate girders or rolled shapes. This type of framing system is described above.

E.4.3. Spillway Crest Lift Gate Loads. Loads. Chapter 3 describes loads for all gates. Loads that are applicable to spillway crest lift gate design include self-weight loads, gravity loads, hydrostatic and hydrodynamic loads, operating loads, environmental loads, impact loads, and earthquake loads.

- Self-weight. Dead load, D.

- Gravity loads. Gravity loads include mud weight (M), and ice weight (C), and shall be determined based on site-specific conditions.

- Hydrostatic loads. Hydrostatic load, Hs, consists of hydrostatic pressure on the gate considering both upper and lower pools.

    - 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 head differential between headwater and the sill bearing at the spillway crest (Figures E-9 and E-10). In addition, Hs acts as uplift on the bottom of the gate when passing flows through the spillway. The net uplift shall be determined from combined effects of downpull forces R.

    - For multiple-section gates, consideration must be given to water passing over the top of the sections of the gate because the gate can be split to allow flow at various sections. For each section, Hs represents hydrostatic head differential between headwater and the bottom of each section, with the bottom section at the sill, bearing at the spillway crest (Figures E-11 and E-12). These gates may be used as a single-section gate. For this condition, Hs will act as uplift on the bottom of the gate when flow is through the spillway. The net uplift shall be determined from combined effects of downpull forces R.

    - 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. Hs represents hydrostatic head differential between headwater and the bottom of both sections, with the bottom section at the sill, bearing at the spillway crest (Figures E-13 and E-14). The net uplift shall be determined from combined effects of downpull forces R.

- Hydrodynamic loads. Hydrodynamic, Hd, loads consist of overtopping of submersible gate leaves, similar to that shown in Figure E-15, and downpull, R (also known as downdrag). 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. Factors that affect the amount of downpull include the location of the gate seals (upstream or downstream), gate seal friction, upward thrust component due to energy head acting on the bottom of the gate, shape of the bottom of the gate, and flow over the top 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 U.S. Army Corps of Engineers Hydraulic Design Criteria (HDC) Sheets 320-2 to 320-2/3, Sagar and Tullis (1979), and Sagar (1977a, 1977b, 1977c).

- Operating loads. Under normal operating conditions, operating loads are treated as reactions to all opposing forces including D, G, Hd, and friction. In the case of gate binding, the operating load, Q, shall be the maximum load that can be exerted by the operating machinery (obtained from the mechanical Engineer that designed the machinery). See Section 3.2.3.2.4 for further discussion on operational loads.

- Impact. Impact load, IM, represents debris or ice loads and shall be taken as prescribed in Section 3.2.3.5.

- Environmental Loads, EV. Environmental loads consist of wind, W. Wind loads shall be based on a 300-year return period per ASCE 7. Wind load shall be applied 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.

- Earthquake design loads E. See Section 3.2.3.6 for earthquake loading.

E.4.4. Load cases. General loads and loading combinations for gates are described in Section 3.3 Lift gates shall be designed for the Strength and Extreme Limit States for each of the following load cases and corresponding load combinations. Table E-2 lists load factors for spillway crest lift gate loads The Serviceability Limit State is addressed in Section 3.1.3.

- Case 1: Strength Limit State I, Gate Closed. Loads consist of self-weight and gravity loads plus unusual hydrostatic and unusual hydrodynamic loading (overtopping) or impact, whichever produces the maximum effects:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} + (\gamma_{Hd2}H_{d2} \text{ or } \gamma_{IM2}IM_2) \quad \text{(E-8)}$$

- Case 2: Strength Limit State II, Gate Open. Loads consist of self-weight and gravity loads plus unusual hydrostatic and unusual hydrodynamic loading (downdrag) or wind, whichever produces the maximum effects:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + (\gamma_{Hd}R \text{ or } \gamma_{EV}EV) \quad \text{(E-9)}$$

- Case 3: Extreme Limit State, Gate Closed. Loads consist of usual hydrostatic head differential with gate subjected to self-weight, usual gravity loads, and earthquake or impact, whichever produces the maximum effects:

$$\gamma_{D2}D_2 + \gamma_{G1}G_2 + \gamma_{Hs1}H_{s1} + \gamma_E E \tag{E-10}$$

Table E-2. Load Factors for Spillway Crest Lift Gate Loads.

| Load Cases | | | Loads/Load Factors | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Limit State | Description | Case | γD | γG | γHs | γHd | γQ | γEV | γIM | γEQ |
| Strength I | Gate Closed | 1 | 1.2 | 1.6 | 1.4 | 1.4(1) | 0 | 1.6(1) | 1.6(1) | 0 |
| Strength II | Gate Open | 2 | 1.2 | 1.6 | 0 | 1.0(1) | 0 | 1.3(1) | 0 | 0 |
| Extreme | Gate Closed | 3 | 1.2 | 1.2 | 1.4 | 0 | 0 | 0 | 0 | 1.0 |
| Notes: (1) Select one at a time | | | | | | | | | | |

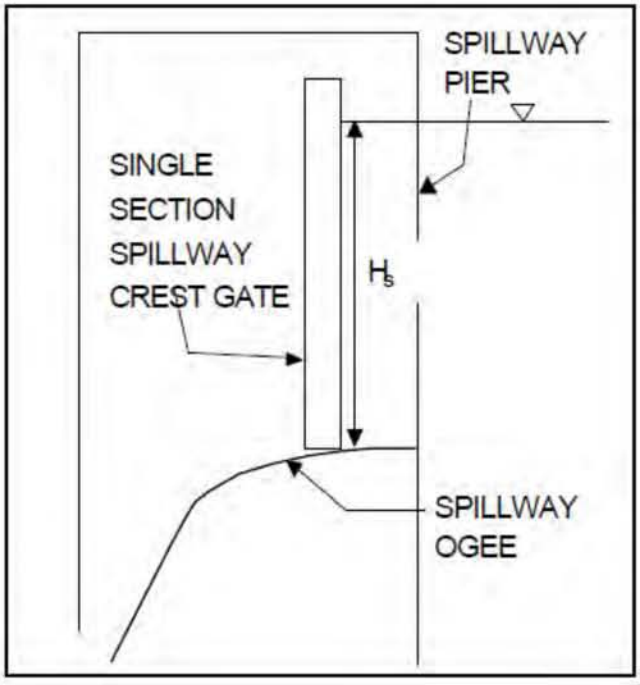


Figure E-9. Single-Section Spillway Crest Gate.

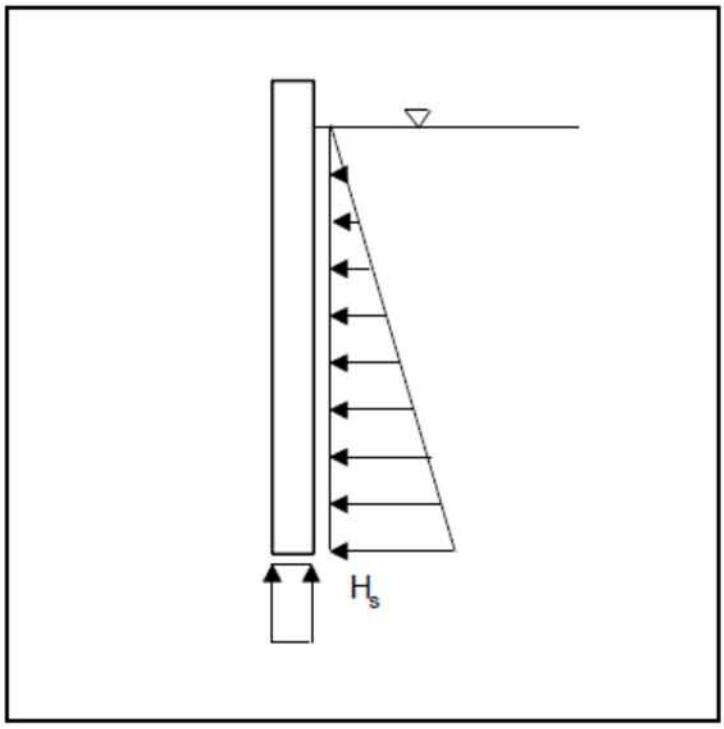


Figure E-10. Single-Section Spillway Crest Gate, Hydrostatic Loading Diagram.

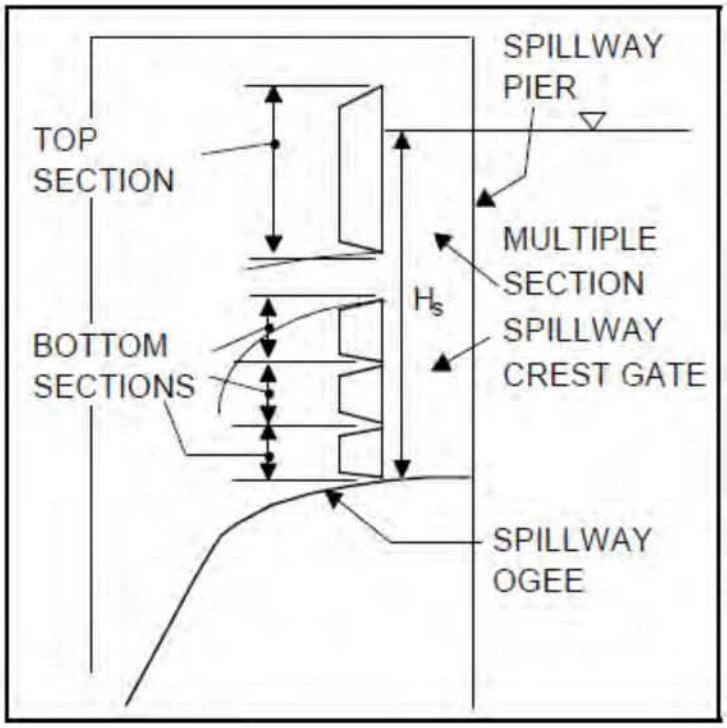


Figure E-11. Multiple-Section Spillway Crest Gate.

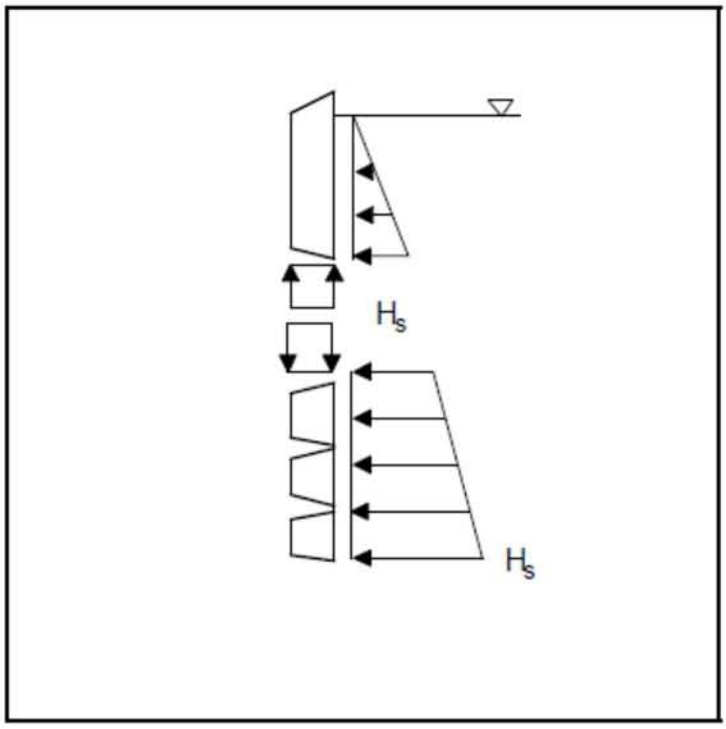


Figure E-12. Multiple-Section Spillway Crest Gate, Hydrostatic Loading Diagram, Top And Bottom Sections Split.

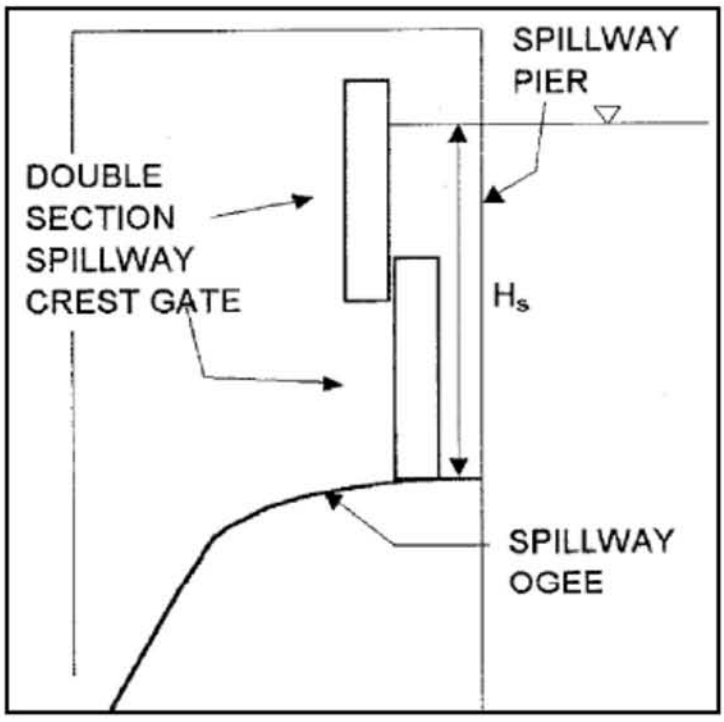


Figure E-13. Double-Section Spillway Crest Gate.

E.5. Outlet Lift Gates.

E.5.1. General. Outlet gates often use structural configurations similar to those of spillway crest lift gates. End support systems might be fixed wheels, tractors, or slides.

E.5.2. Framing Systems. Horizontal girder framing with plate girders or rolled shapes is the most common type of framing system for outlet gates. The main difference in framing compared with that of spillway crest gates and navigation lock gates is that 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.

E.5.3. Outlet Lift Gate Loads. Chapter 3 describes loads for all gates. Loads that are applicable to spillway crest lift gate design include self-weight loads, gravity loads, hydrostatic and hydrodynamic loads, operating loads, environmental loads, impact loads, and earthquake loads.

- Self-weight. Dead load, D.

- Gravity loads. Gravity loads include mud weight (M), and ice weight (C), and shall be determined based on site-specific conditions.

- Hydrostatic loads. Hydrostatic load, Hs, consists of hydrostatic pressure on the gate considering both upper and lower pools. Figures E-14 and E-15 show loading diagrams for hydrostatic loading of an outlet gate with a downstream seal with an upstream skin plate. Figures E-16 and E-17 show loading diagrams for hydrostatic loading of an outlet gate with an upstream seal with an upstream skin plate.

- Hydrodynamic loads. Hydrodynamic, Hd, loads includes of water hammer and downpull, R. Hydrodynamic forces from flow either under or over the top of this type of gate are accounted for in downpull forces. 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.

- Operating loads. Operating load, Q, shall be the maximum load that can be exerted by the operating machinery (obtained from the mechanical Engineer that designed the machinery). See Section 3.2.3.2.4 for further discussion on operational loads.

- Environmental Loads, EV. Environmental loads consist of wind, W. Wind loads shall be based on a 300-year return period per ASCE 7. Wind load shall be applied 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.

- Earthquake design loads E. See Section 3.2.3.6 for earthquake loading.

E.5.4. Load cases. General loads and loading combinations for gates are described in Section 3.3. Lift gates shall be designed for the Strength and Extreme Limit States for each of the following load cases and corresponding load combinations. Table E-3 lists load factors for Lift Gates. The Serviceability Limit State is addressed in Section 3.1.3.

- Case 1: Strength Limit State I, Gate Closed. Loads consist of self-weight and gravity loads plus unusual hydrostatic loading:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} \quad \text{(E-11)}$$

- Case 2: Strength Limit State II, Gate Open. Loads consist of self-weight and gravity loads plus unusual hydrostatic and unusual hydrodynamic loading (Downdrag, R):

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} + \gamma_{Hd2}R \quad \text{(E-12)}$$

- Case 3: Extreme Limit State, Gate Open. Loads consist of unusual hydrostatic head differential with gate subjected to self-weight, usual gravity loads, and water hammer:

$$\gamma_{D2}D_2 + \gamma_{G2}G_2 + \gamma_{Hs2}H_{s2} + \gamma_{Hs3}H_{s3} \quad \text{(E-13)}$$

- Case 4: Extreme Limit State, Gate Closed. Loads consist of usual hydrostatic head differential with gate subjected to self-weight, usual gravity loads, and earthquake:

$$\gamma_{D2}D_2 + \gamma_{G1}G_2 + \gamma_{Hs1}H_{s1} + \gamma_E E \quad \text{(E-14)}$$

Table E-3. Load Factors for Lift Gates.

| Load Cases | | | Loads/Load Factors | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Limit State** | **Description** | **Case** | **γD** | **γG** | **γHs** | **γHd** | **γQ** | **γEV** | **γIM** | **γEQ** |
| Strength I | Gate Closed | 1 | 1.2 | 1.6 | 1.4 | 0 | 0 | 0 | 0 | 0 |
| Strength II | Gate Open | 2 | 1.2 | 1.6 | 1.4 | 1.4 | 0 | 0 | 0 | 0 |
| Extreme | Gate Open | 3 | 1.2 | 1.6 | 1.4 | 1.0 | 0 | 0 | 0 | 0 |
| Extreme | Gate Closed | 4 | 1.2 | 1.2 | 1.4 | 0 | 0 | 0 | 0 | 1.0 |

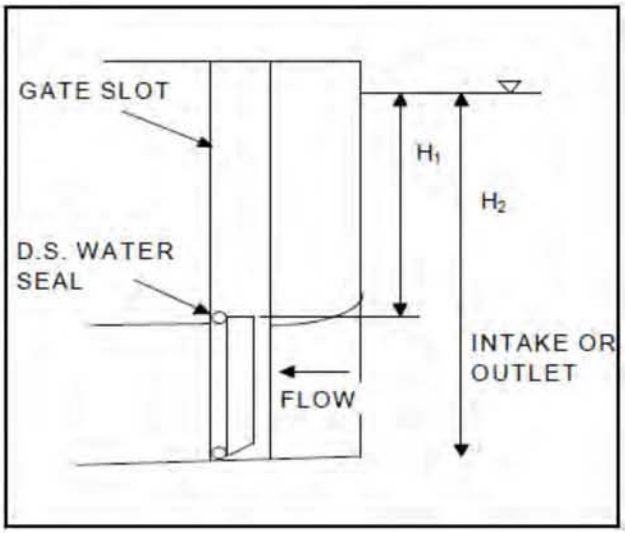


Figure E-14. Outlet Gate with Downstream Seal with an Upstream Skin Plate.

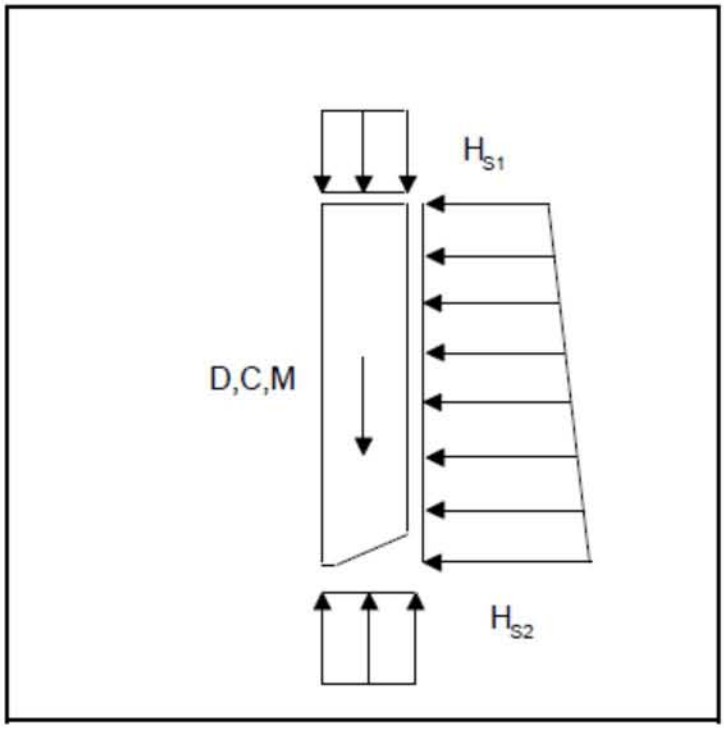


Figure E-15. Outlet Gate, Hydrostatic Loading, Downstream Seal with an Upstream Skin Plate.

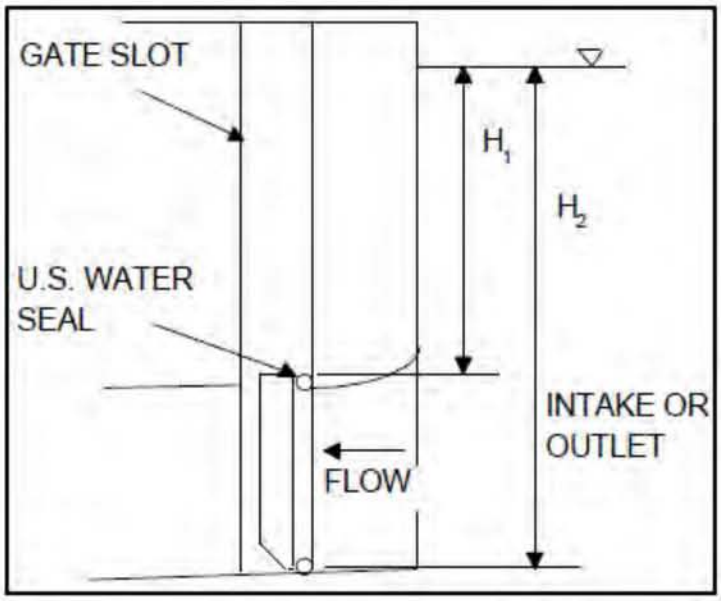


Figure E-16. Outlet Gate with Upstream Seal with an Upstream Skin Plate.

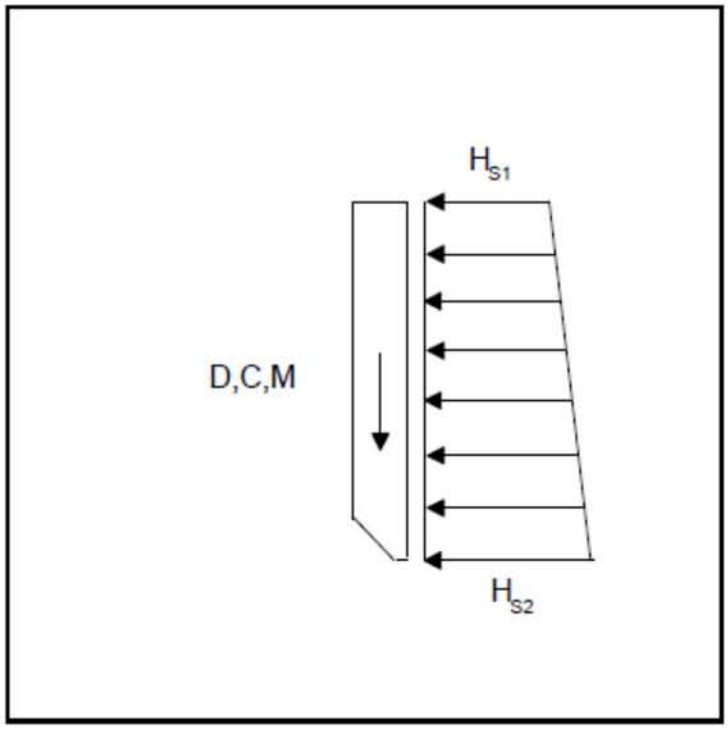


Figure E-17. Outlet Gate, Hydrostatic Loading, Upstream Seal with an Upstream Skin Plate.

E.6. Design Analysis and Detail Requirements. Basic gate analysis methods and good detailing practice are discussed in Section E.3.5.

E.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. Proper detailing of fracture critical connections to the tension tie is important to the service life of the gate.

E.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. One type provides bearing directly from the rollers as a series of point loads or from wheels as a single point load. 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, as with navigation lock gates.

E.7. Operating Equipment. Hoisting equipment is usually wire ropes, roller chains, or hydraulic cylinders. EM 1110-2-2610 are provided guidelines for operating equipment.

E.7.1. Dogging Devices. Dogging devices (dogs) are usually mounted on grillages in recesses in the piers opposite the gate end posts. They pivot to permit retraction for clearance of the gate and are operated with push rods. Two or more dogs at each end of the gate slot may be required. The number and location of the dogs 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. Another type of dogging device 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 manual 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 and bulkheads 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.

E.7.2. Lifting Beams Lifting beams are normally provided for outlet gates and maintenance bulkheads. Because these gates are normally stored in a submerged condition, the lifting beam provides a latching and unlatching mechanism to lift the gate from the slot. Design guidance for lifting beams can be found in EM 1110-2-4205.

E.8. Corrosion Control.

Chapters 5-7 discuss guidelines for corrosion control. Cathodic protection systems are discussed in EM 1110-2-2704.

E.9. Maintenance Considerations. See Chapter 6 for maintenance and inspection considerations.

E.10. Serviceability Requirements. Chapter 3 provides serviceability requirements for gates.

E.11. Fatigue and Fracture Control. Chapter 4 provides gate design requirements related to fatigue and facture control. Generally, spillway and outlet gates are not operated as frequently as navigation lock gates. Hence, the fatigue may not play a significant role in the design of the gate.

E.12. Material Selection. Material selection guidelines for gates are provided in Section 2.3. Material information for wheels and axles is provided in EM 1110-2-2610.

E.13. Weldments. Chapter 6 provides fabrication requirements for weldments.

E.14. Wheel Design Details. See EM 1110-2-2610 for roller wheel and track design requirements.

E.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. The clearance in the upstream direction is usually from ¼ to ⅜ 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. 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.

E.14.2. Seals. Because tractor roller gates are often used in high-head applications, additional considerations are required for seals. 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.

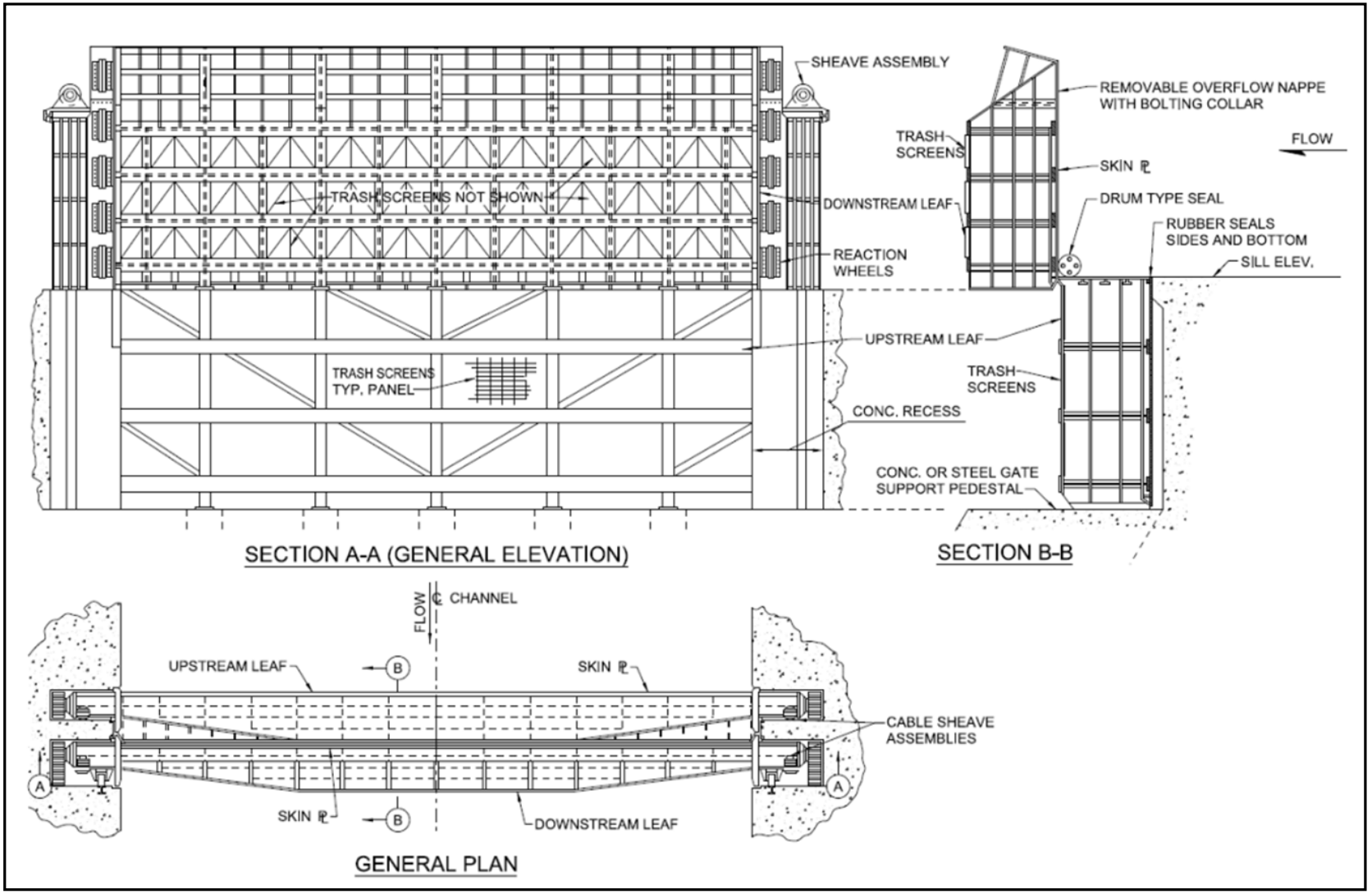


Plate E-1. Vertical Lift Gates Double-Leaf General Plan and Elevation.

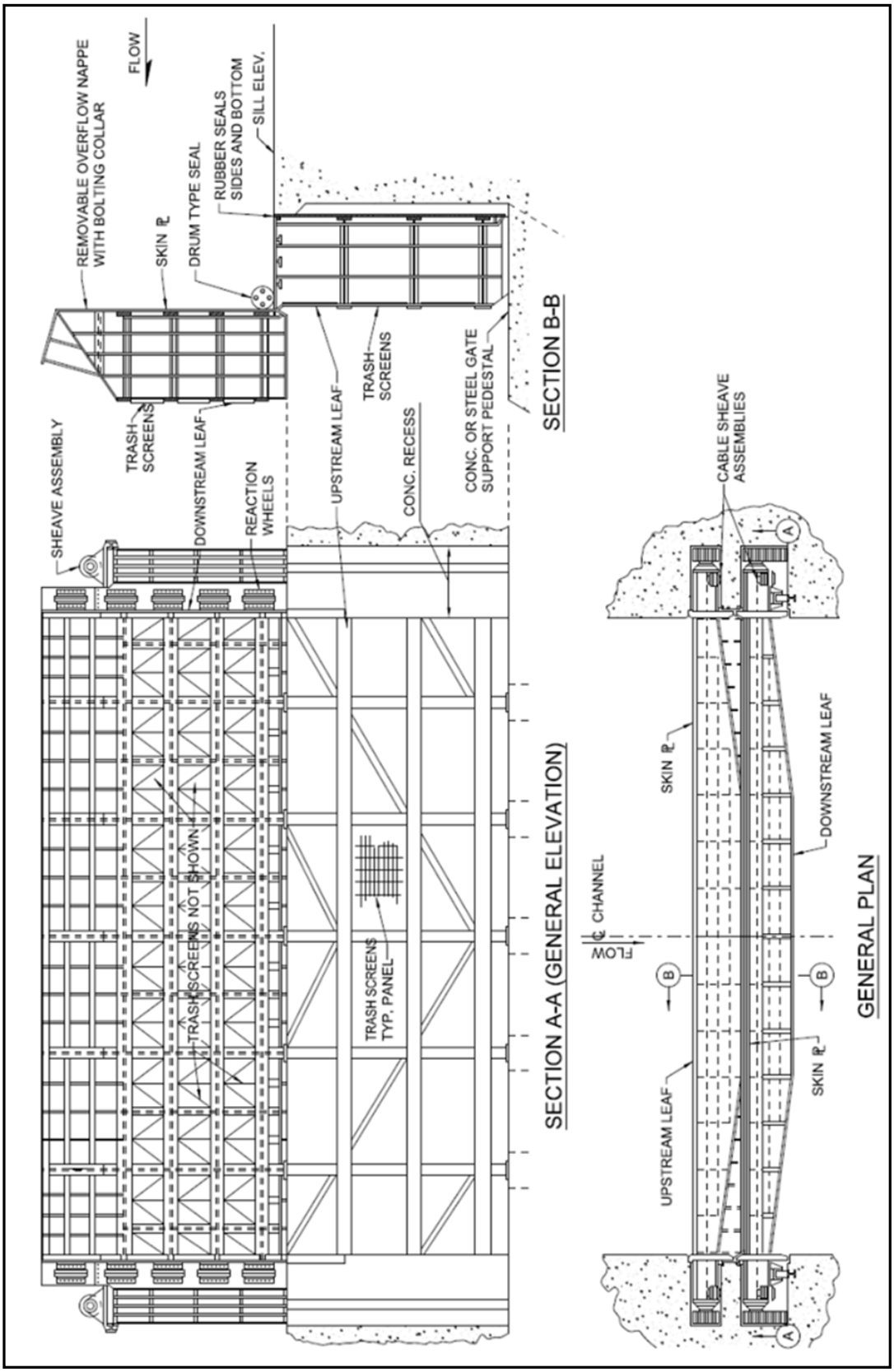


Plate E-2. Vertical Lift Gates Single-with Towers General Elevation and Typical Details.

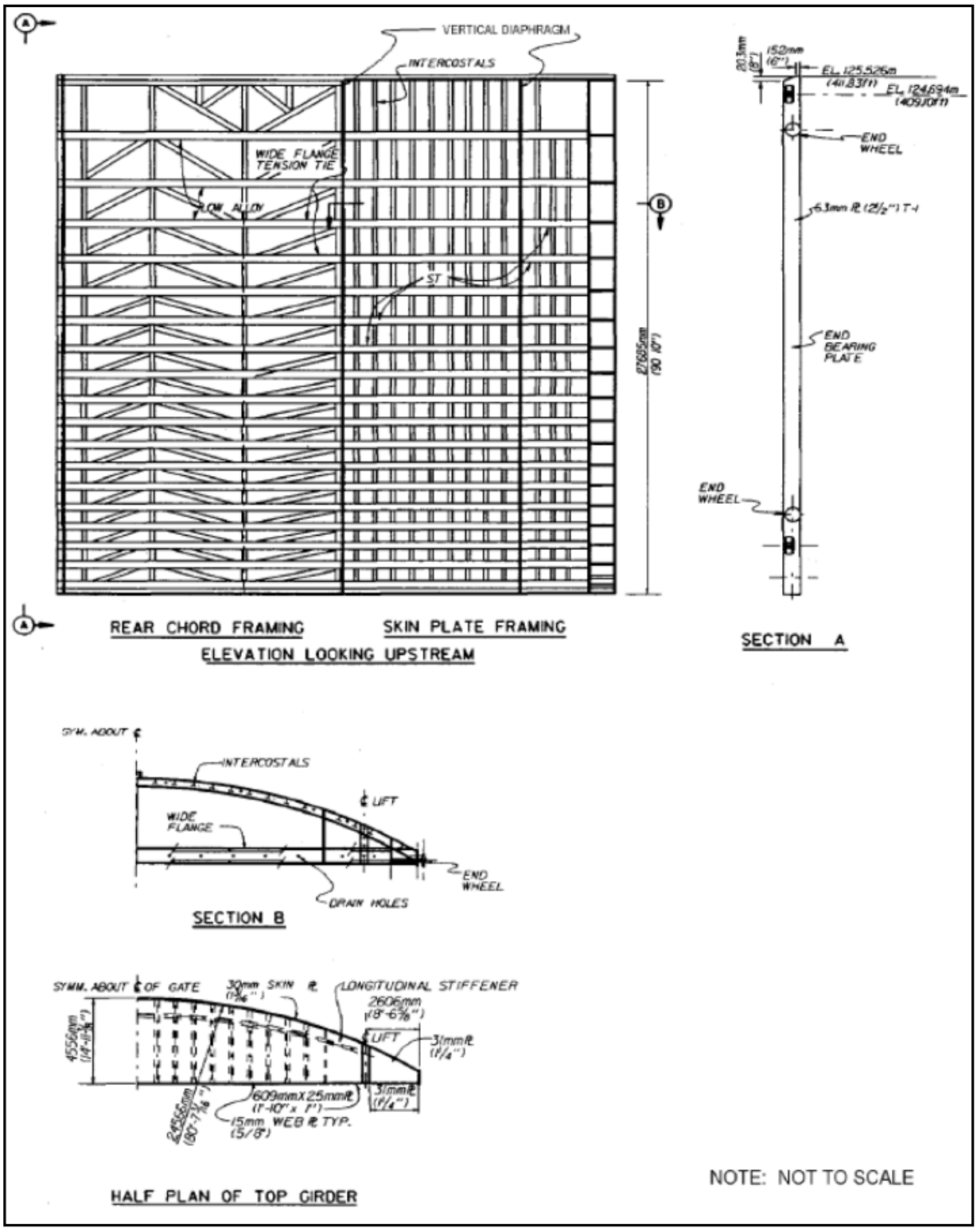


Plate E-3. Ice Harbor Lock and Dam Navigation Lock (Old Downstream Lift Gate), Snake River, Oregon, Idaho, Washington.

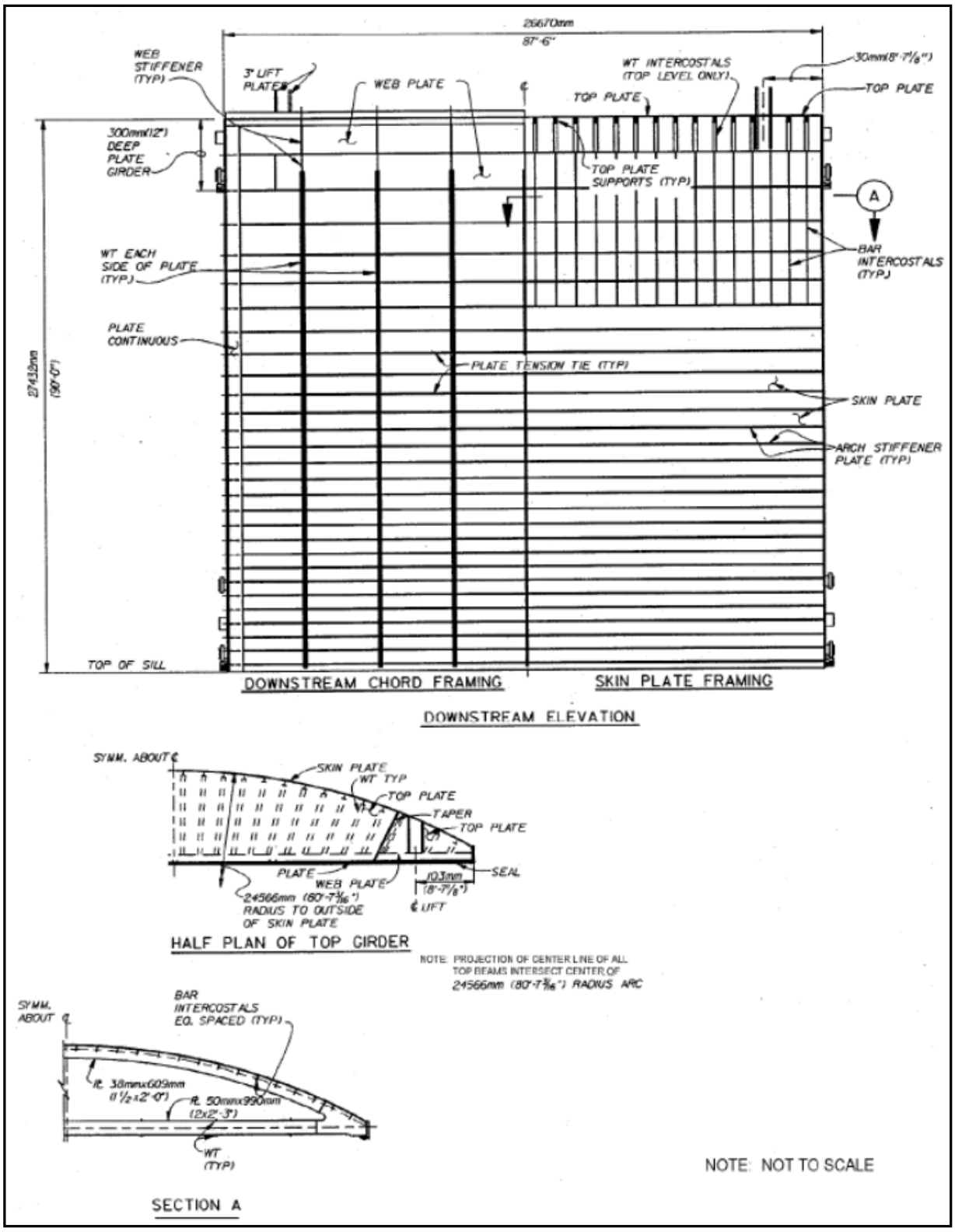


Plate E-4. Ice Harbor Lock and Dam Navigation Lock Replacement Downstream Lift Gate, Snake River, Oregon, Idaho, Washington.

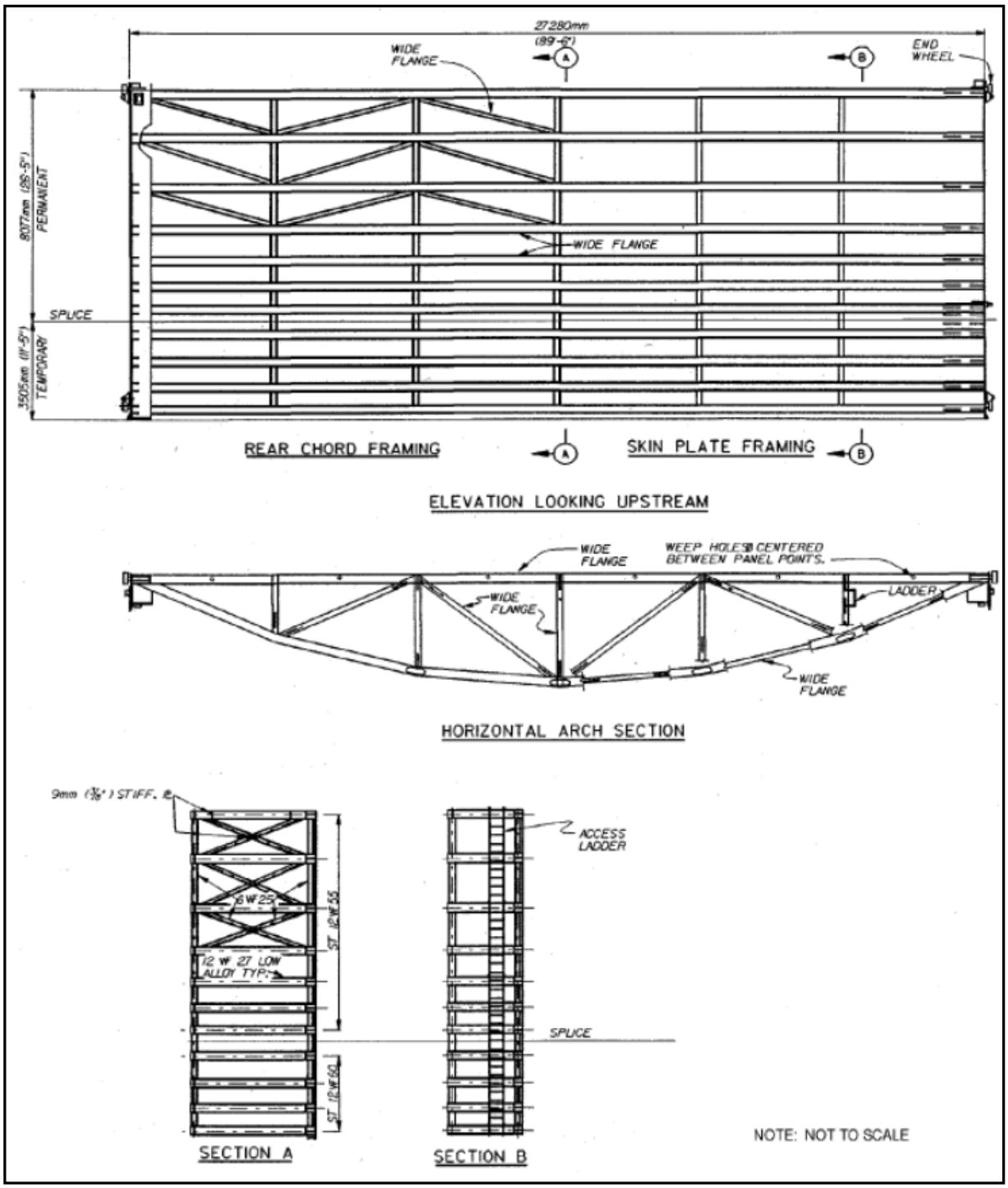


Plate E-5. John Day Lock and Dam Navigation Lock Upstream Lift Gate, Columbia River, Oregon and Washington.

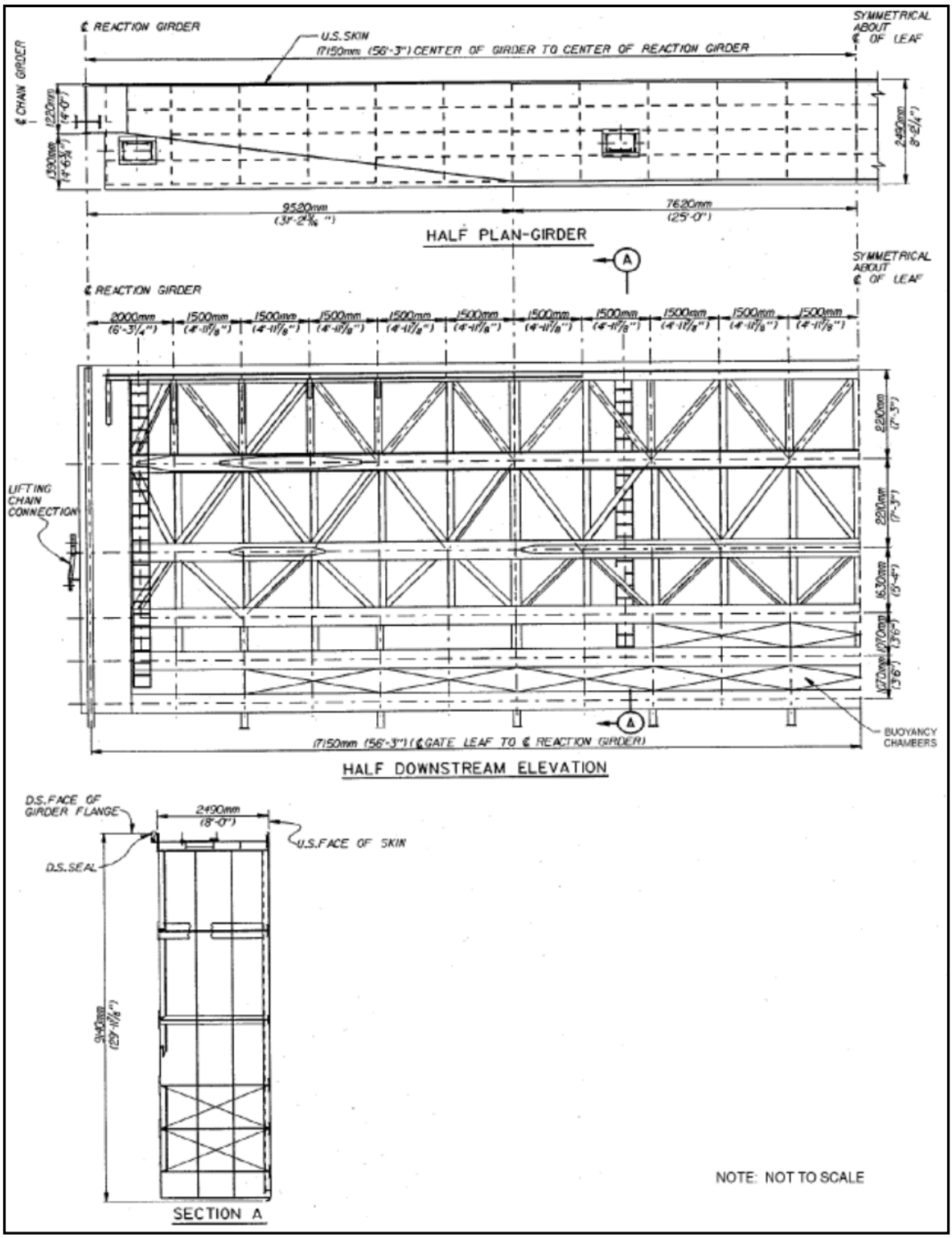


Plate E-6. Old Lift Gate Upstream Leaf, Upper Mississippi River Basin, Mississippi River, Granite City, IL, Lock No. 27, Main Lock.

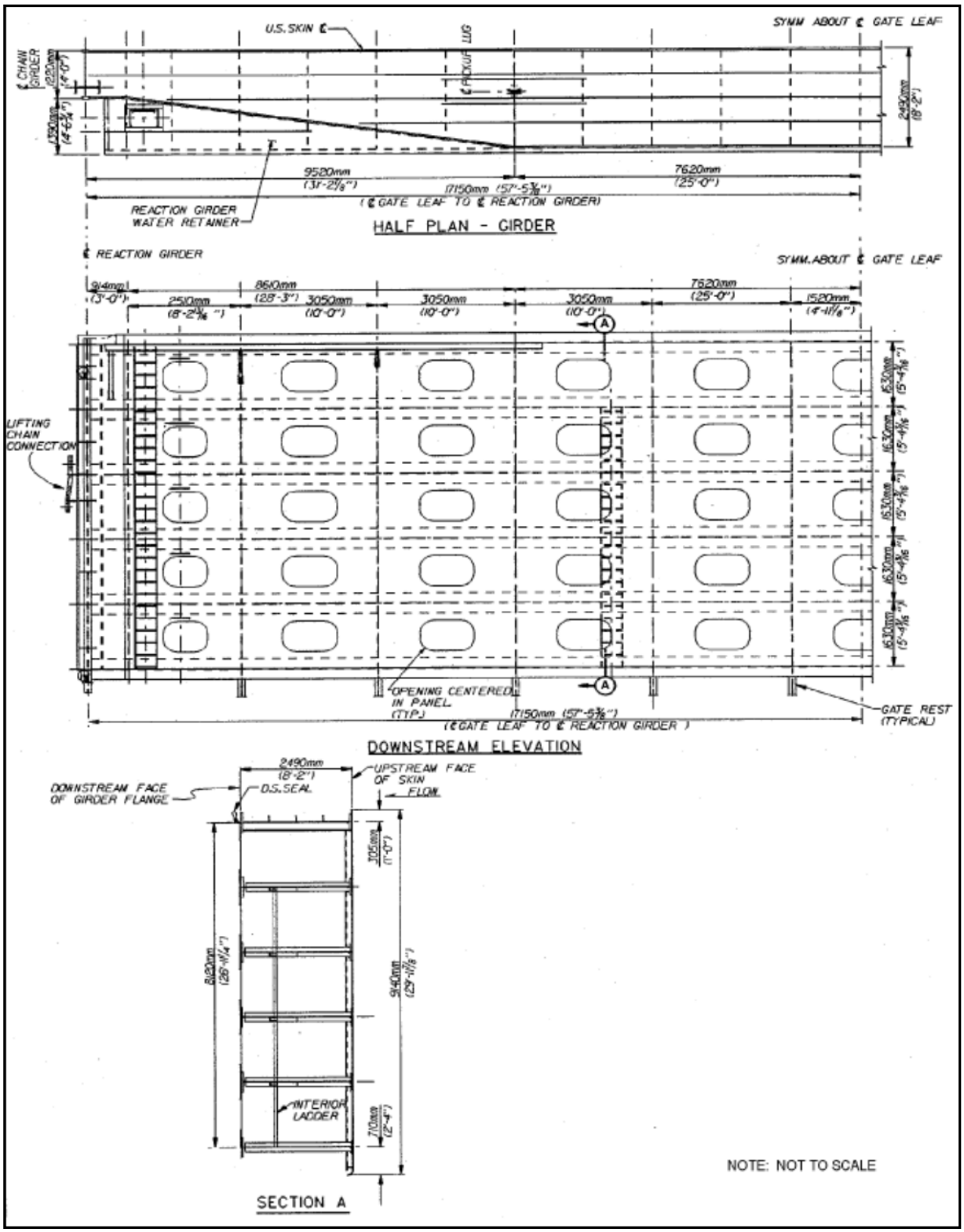


Plate E-7. Replacement Lift Gate Upstream Gate, Upper Mississippi River Basin, Mississippi River, Granite City, IL, Lock No. 27, Main Lock.

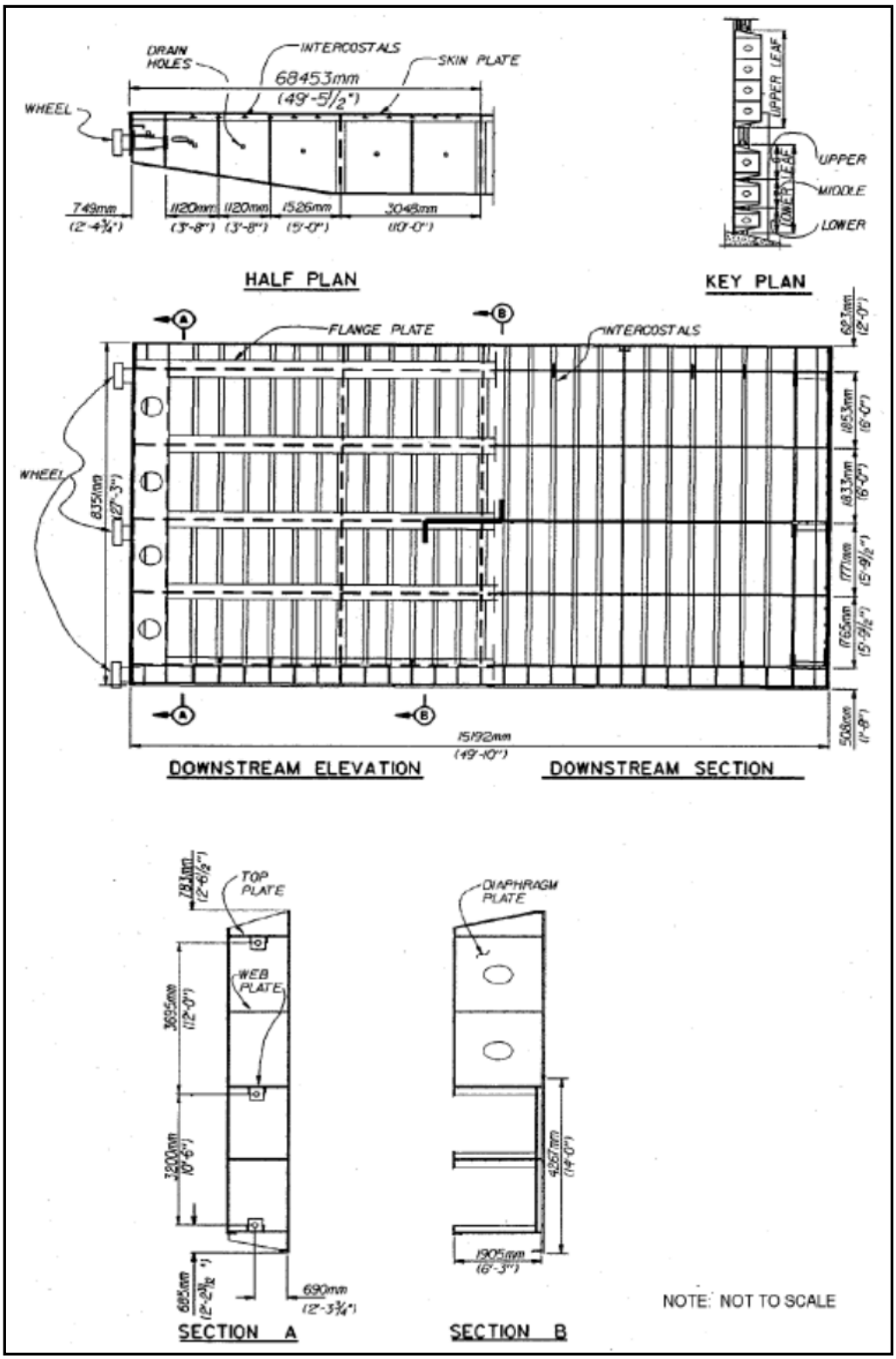


Plate E-8. Spillway Crest Gate Upper Leaf, McNary Lock and Dam, Columbia River, Oregon, Idaho, and Washington.

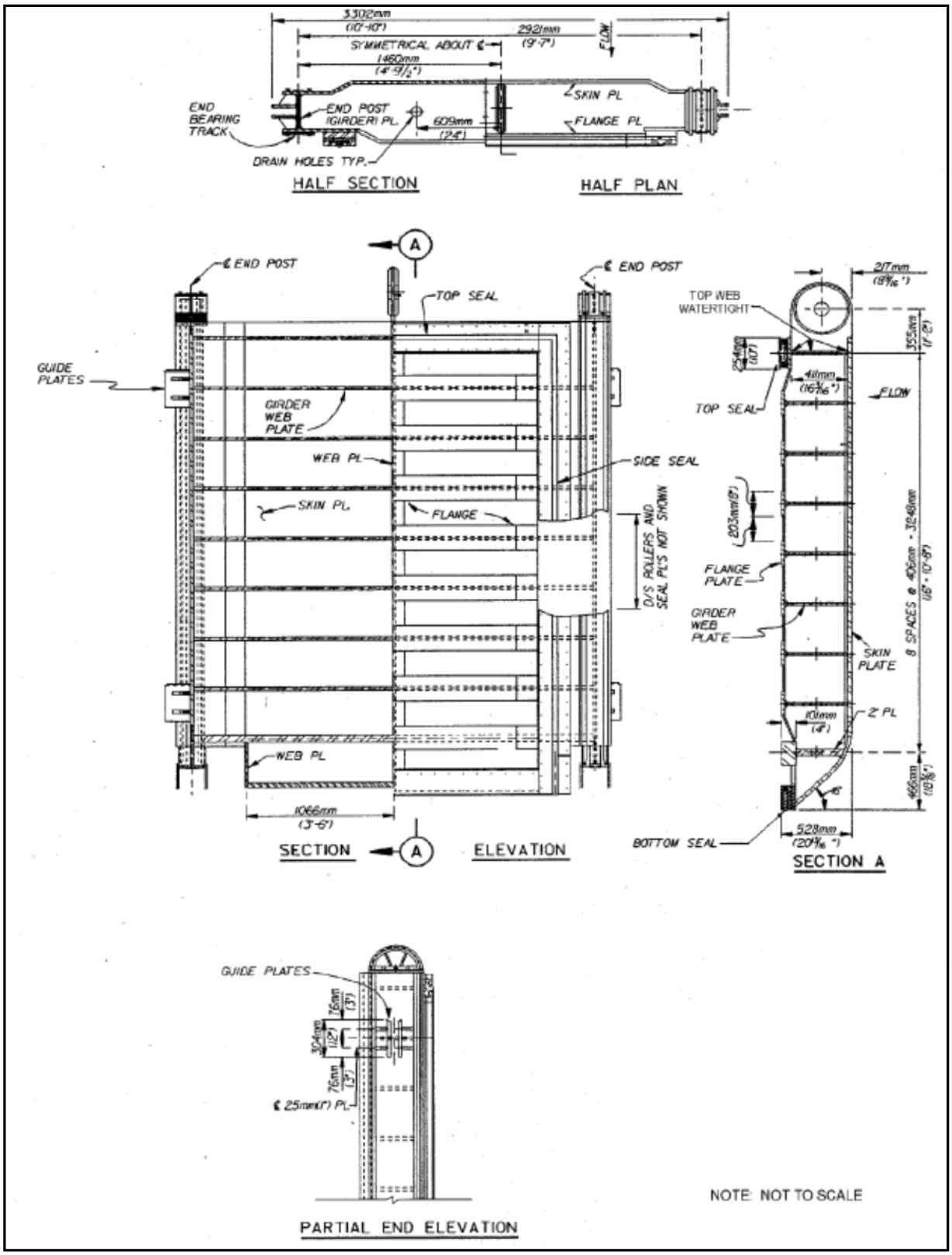


Plate E-9. Emergency Closure Gates, Mud Mountain Dam, White River, Washington.

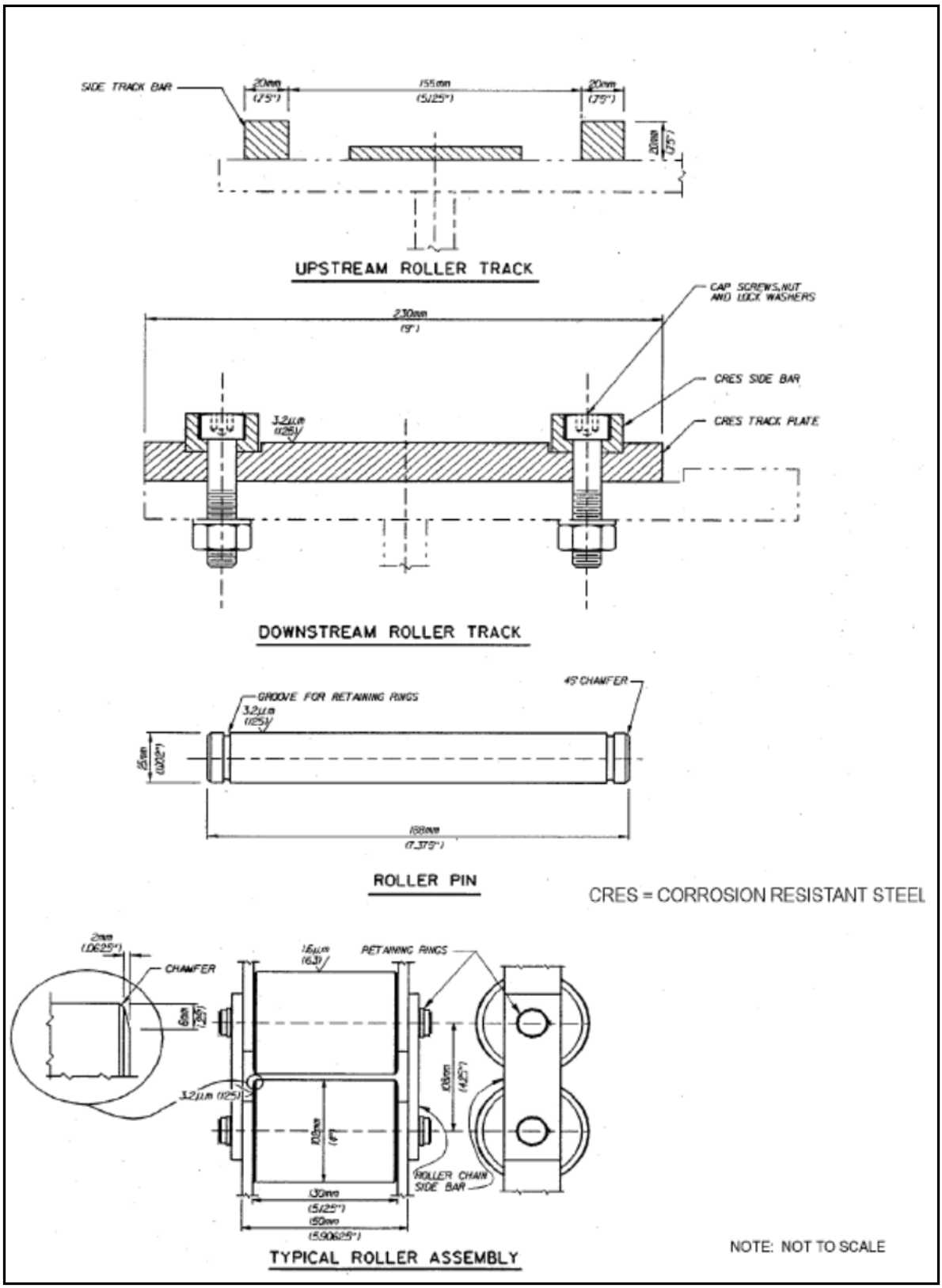


Plate E-10. Tractor Gate Chain and Roller Track Details.

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# APPENDIX F

## Closure Gates

F.1. Introduction. While the need for closures should be avoided when possible by using ramps or pedestrian overpasses, closure structures are often required at openings in levee and floodwall systems for traffic to pass through levee and floodwall systems of local flood protection projects (LFPPs) at elevations below the level of protection provided by the project. These openings might be necessary for normal road, railroad, or pedestrian traffic. This appendix provides guidance for selection and design of closure gates for such openings. The appendix discusses advantages of various gate configurations, provides some typical details, and identifies design requirements.

F.2. Design.

F.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.

F.2.2. General Design Requirements.

F.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, but closing the gates too early can unnecessarily inconvenience traffic that normally passes through the gate. 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.

F.2.2.2. Function. 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 and railroads shall comply with the requirements of the AASHTO. 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 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.

F.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.

F.2.2.4. Aesthetics. Aesthetics is usually not a major consideration for flood closures. However, EM 1110-2-38 provides guidance for aligning flood control channels, landscaping along channels, and the aesthetic treatment of channel linings. This EM might provide some insights on aesthetic treatments, where necessary. For a few LFPP projects, an open view of the waterway has been an aesthetic require-

ment. These projects incorporate closures with bottom hinged walls that can be stored in the lowered position and raised for protection during flood periods.

F.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 that the appropriate design provisions are incorporated to ensure safe operation. General safety provisions include providing railings on the top of gates and adjacent walls for public protection and providing ladders for access by operations personnel. Additional safety features could include warning signs and barriers that prevent access by unauthorized persons. Compliance with appropriate traffic safety standards is also necessary.

F.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.

F.2.2.7. Construction. Construction of closure gates sills can interfere with normal traffic. 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.

F.2.2.8. Cost. The costs of closure structures vary according to the closure type and opening size. First cost is always a consideration for all projects.

F.2.2.9. 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 retractable bottom sill to accommodate uneven sill surfaces.

F.3. Selection of Closure Types.

F.3.1. Stoplogs. 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 do not have the same strength. Note that design of aluminum closures is not addressed in this manual. 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 posts are required (Figure F-1). Seals are not normally attached to the stoplogs. However, plastic sheeting, sandbags, or other available means can be used to reduce leakage through the stoplog closure. Secures storage facilities must be provided for the stoplogs, removable posts, and accessories. Advantages and disadvantages of stoplog closures are:

- Advantages:
    - Fabrication methods are simple and economical.

- Initial cost is usually less than for other gated closures.
- Easy stoplog placement for narrow and low openings.

- Disadvantages:
  - 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, and time to allow cleaning of the post sockets during installation.
  - Because of the longer installation time, accurate long-range weather forecasting is needed to provide that time.

F.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 F-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 F-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.

- Advantages:
  - No special skills or equipment are required for operation except when removable intermediate support posts are used with double-leaf gates.
  - Can be closed quickly except when removable intermediate support posts are used with double-leaf gates.

- Disadvantages:
  - 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.
  - Difficult to operate during high winds.

F.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 F-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. 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. Appendix C includes provisions for the design of hemispherical 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. 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 during strong winds.

- Advantages:
    - Suitable for large openings.
    - Closure can be made quickly without the use of skilled personnel.
    - A storage building is not required.
    - A center support is not required.
    - Weighs less than other types of gates designed for large openings.
- Disadvantages:
    - Hinges require complex shop fabrication with machine work.
    - Requires right-of-way area for operating.
    - Support structure is larger and more expensive than for other gate types.
    - Difficult to operate during high winds.

F.3.4. Rolling Gates – General. Rolling gates are constructed similar to swing gates. The gates are supported by wheels that roll on tracks embedded in the sill across the closure opening and the storage area. The gates are sometimes operated by a cable attached to a motorized winch. However, the cable could 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. The following sections identify a few other advantages and disadvantages for specific types of rolling gates. Some general advantages and disadvantages of rolling gates are:

- Advantages:
    - Adaptable to wide openings.
    - Closure can be made quickly without the use of skilled personnel.
    - A storage building is not required.
    - Requires small storage space.
- Disadvantages:
    - Unless wheel assemblies are designed to accommodate the lateral bottom girder deflection, jacks must be provided to lift the wheel assemblies from the tracks when the gate is in the closed position.
    - Requires level storage area immediately adjacent to the closure opening.

F.3.5. Rolling Gates – Two Lines of Wheels. Rolling gates can be stabilized with two lines of wheels. Figure F-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 on wind speed.

F.3.6. 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.

F.3.7. 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 F-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. Some general advantages and disadvantages of these type of rolling gates are:

- Advantages:
    - Can be fabricated in sections to simplify handling and storage.
- Disadvantages:
    - Requires level track surface.
    - Requires wide sill to accommodate the installation of tracks and hook anchorages.

F.3.8. 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 F-7). The gates are opened and closed by a winch arrangement similar to that used for rolling gates. Some general advantages and disadvantages of trolley gates are:

- Advantages:
    - Is practical for wide openings.
    - Closure can be made quickly without the use of skilled personnel.
    - Can obtain a good seal against irregular sill surfaces.
    - Requires small storage space.
    - Is suitable for railroad closures because required vertical clearances for railroads are fixed.
- Disadvantages:
    - Slope of the ground adjacent to the closure opening must allow adequate clearance to open the gate.
    - 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.

F.4. Structural Design. Structural design of closure gates shall comply with the strength and serviceability requirements defined in the main body of this EM, and those described in the following paragraphs:

F.4.1. Design Loads. The following loads shall be considered in the design of closure structures

- Self-weight, D.
- Gravity, G2, Unusual Load. 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.
- Hydrostatic, Hs2, Unusual Load. May be conservatively taken as water to the top of the gate with no tail water.
- Hydrodynamic, Hd2, Unusual Load. Waves or other hydrodynamic loads may be neglected unless the Engineer has reason to believe they will exist.
- Operating Machinery, Q2, Unusual Load. Conservatively use the maximum machinery load that can be applied to the gate. Consult with the Mechanical Engineer for the project to determine this load.

- Impact, IM3, Extreme Load. May be neglected unless the Engineer has reason to believe this load may exist.

- Earthquake, not considered for the design of closure gates, but should be considered for gate support columns and walls.

- Environmental, W, Wind, See ASCE for recurrence interval, in lieu of site-specific data, use 15 psf during operation and up to 50 psf when not in operation (fully closed or fully open).

F.4.2. Design Load Cases. The following load cases shall be evaluated using the load factors listed in Table F-1.

- Case 1: Strength I, Gate not operating:

$$\Sigma\gamma_i Q_{nt} = \gamma_{D2} D + \gamma_{Hs2} H_{s2} \quad \text{(F-1)}$$

- Case 2: Strength I, Gate not operating, Gate subjected to the upper level Wind pressure of up to 50 psf:

$$\Sigma\gamma_i Q_{nt} = \gamma_{D2} D + \gamma_{EV2} W_2 \quad \text{(F-2)}$$

- Case 3: Strength II, Gate operating, Hinged gate subjected to Dead and Wind (lower level of 15 psf), operating load is treated as a reaction:

$$\Sigma\gamma_i Q_{nt} = \gamma_{D2} D + \gamma_{EV2} W_1 \quad \text{(F-3)}$$

- Case 4: Strength II, Gate operating, Wheeled gate subjected to Dead and Operating load:

$$\Sigma\gamma_i Q_{nt} = \gamma_{D2} D + \gamma_{EV2} Q_2 \quad \text{(F-4)}$$

Table F-1. Load Factors for Closure Gates.

| Load Cases | | Loads/Load Factors | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Limit State** | **Description** | **γD** | **γG** | **γHs** | **γHd** | **γQ** | **γEV** | **γIM** | **γEQ** |
| Strength I | Gate Closed | 0 | 0 | 1.4 | 0 | 0 | 0 | 0 | 0 |
| Strength I | Gate Closed | 1.2 | 0 | 0 | 0 | 0 | 1.3 | 0 | 0 |
| Strength II | Gate Open | 1.2 | 0 | 0 | 0 | 0 | 1.3 | 0 | 0 |
| Strength II | Gate Open | 0 | 0 | 0 | 0 | 1.3 | 0 | 0 | 0 |

F.4.3. Design for Individual Members. The following paragraphs include a brief description of design assumptions and design considerations.

F.4.3.1. Skin plate. Skin plates shall be sized such that the maximum calculated stress is less than the yield limit state of $\alpha\phi F_y$. Skin plates shall be designed for hydrostatic loading only. Stresses shall be determined based on small deflection thin plate theory and by using Equation F-1. Deflection shall be limited to 0.4 of the plate thickness to prevent the development of significant membrane stresses. More than one thickness of plate may be desirable for taller gates. The minimum plate thickness shall be ¼ in. Appendix C provides additional guidance on skin plate design.

F.4.3.2. Intercostals. Intercostals shall be sized so the maximum calculated moment is less than the nominal bending strength of $\alpha\phi_b M_n$. Intercostals shall be designed using Equation F-1. They may be flat bars or plates, tee sections, or angle sections. Appendix C provides additional guidance on intercostal design.

F.4.3.3. Girders. Horizontal girders for swing gates that support components of the diagonal loads shall be designed for flexure due to hydrostatic loading plus flexure and axial load induced by dead load in the diagonals. Rolling and trolley gate girders shall be designed for flexure due to hydrostatic loading only. Appendix C provides additional guidance on girder design.

F.4.3.4. Diagonals. Diagonals shall be designed to resist gate torsion due to dead load and operation using Equation F-3. Appendix C provides additional guidance on diagonal design.

F.4.3.5. Vertical diaphragms. Vertical diaphragms for hinge gates shall be designed to resist diagonal loads as well as flexure loads. Vertical diaphragms for wheel gates shall 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 shall be ¼ in.

F.4.3.6. 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 shall be designed as individual units. The force applied to the units may be from hydrostatic, dead, operating, or wind or a combination of these loads. Components of the gate being used to stabilize the gate in the closed position with hydrostatic load shall be designed using Equation F-1. Other gate components shall be designed to resist dead, operating, or wind load as applicable.

F.4.3.7. Serviceability requirements. Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, details for ease of maintenance, details for ease of operation, ensuring the gate is not damaged in the latched open position, etc.) shall be chosen so that the closure functions properly throughout its design life.

F.5. Gate Operating Equipment. 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.

F.6. Corrosion Protection. Guidelines for corrosion control are discussed in the main body of this manual.

Figure F-1. Stoplog Closure Structure with Center Post.

Figure F-2. Swing-Gate Closure Structure.

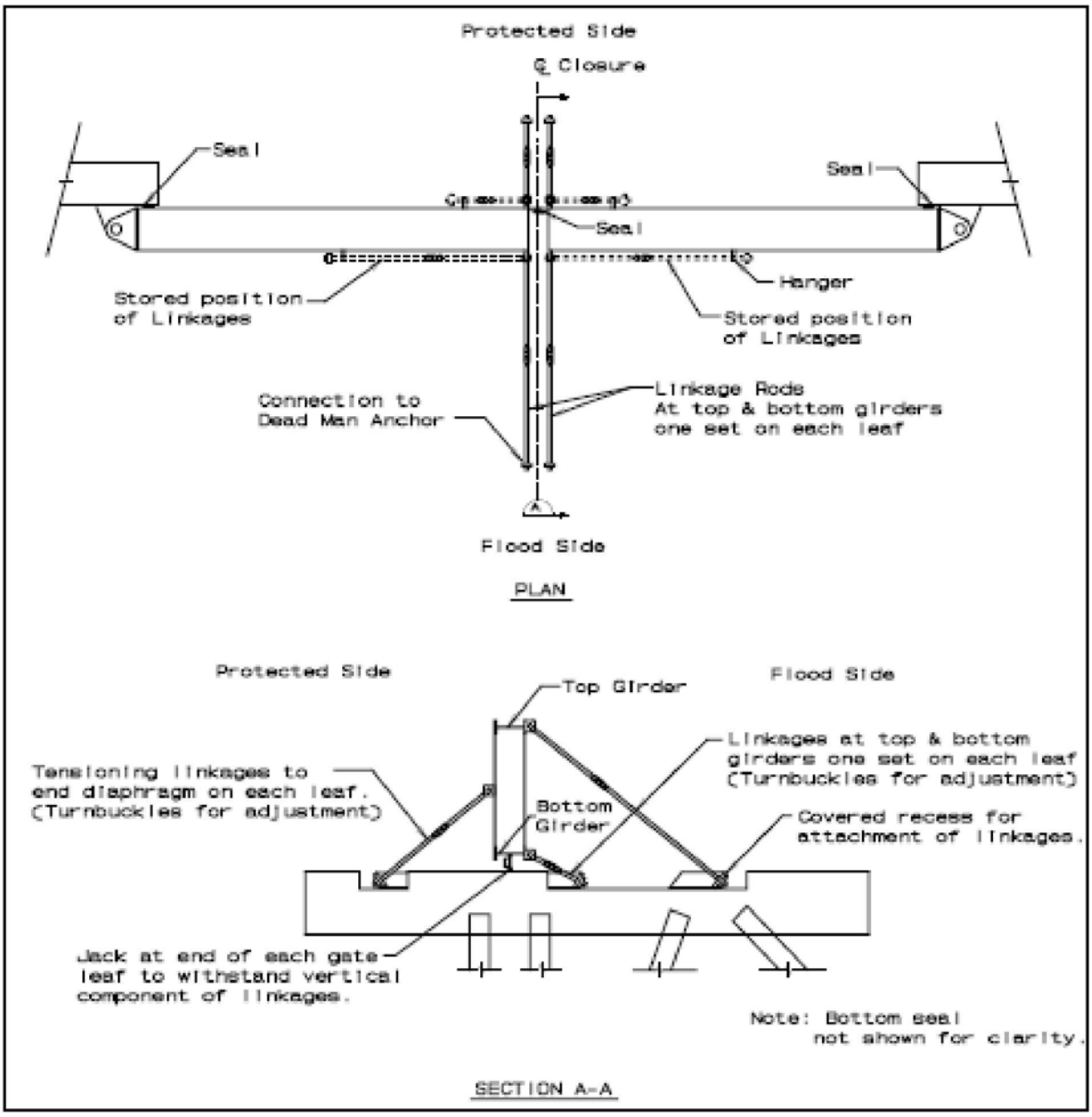


Figure F-3. Tieback Linkage for Double-Leaf Swing Gate.

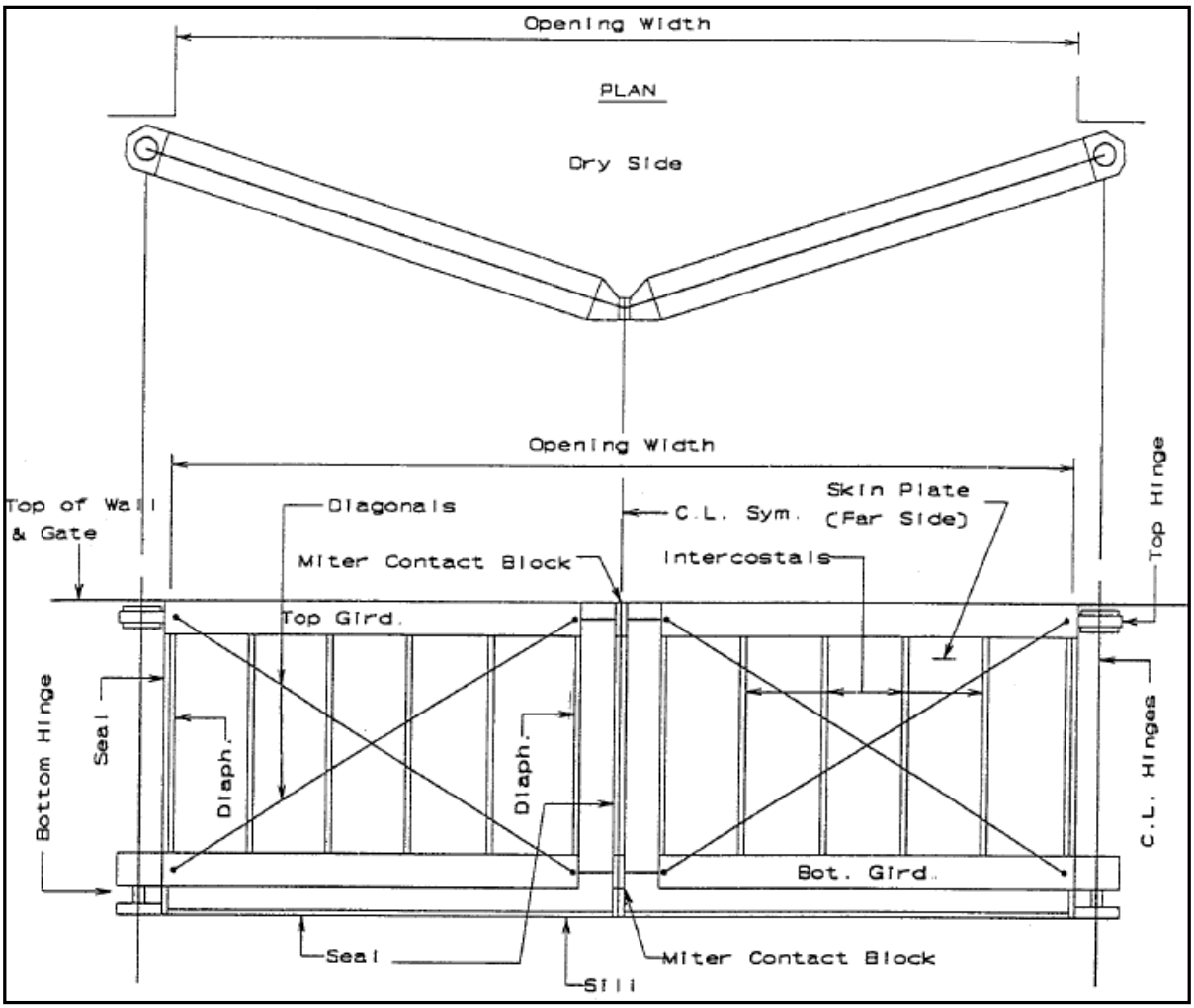


Figure F-4. Miter Gate Closure Structure.

Figure F-5. Rolling Gate Closure Structure.

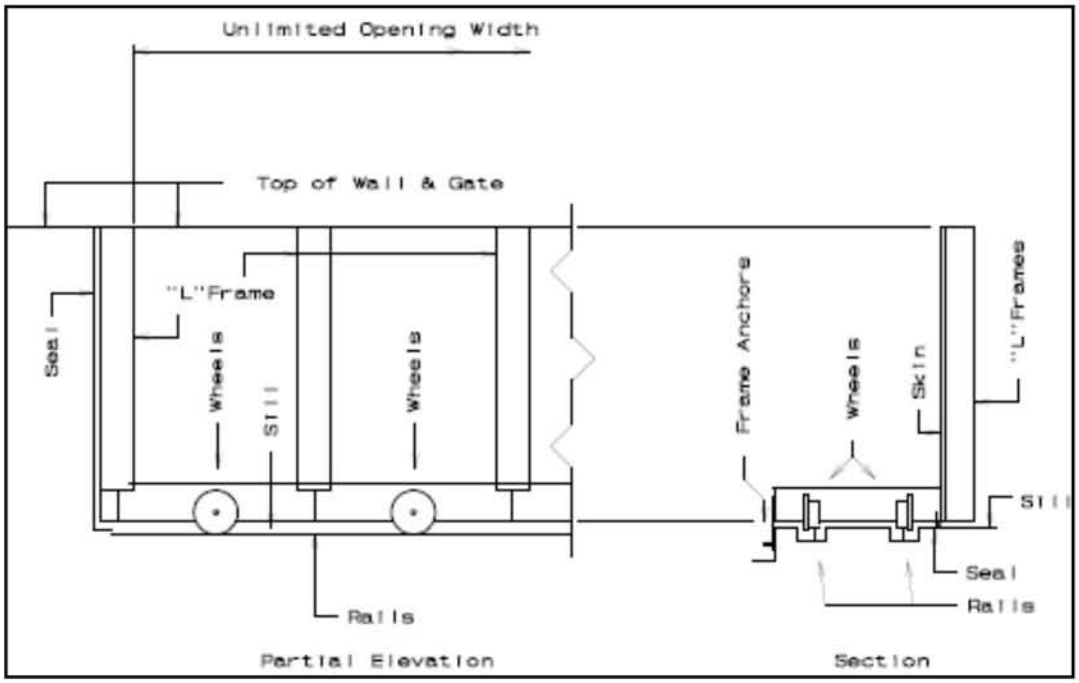


Figure F-6. Rolling Gate Stabilized by L-Frame and Hooks.

Figure F-7. Trolley Gate Closure Structure.

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# APPENDIX G

## Bulkheads and Stoplogs

G.1. General. This appendix provides guidance for the design of bulkheads. There is no specific number of 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 bulkheads might be used for maintenance in a large number of gate bays. If sizes are compatible, it might be possible to use the same bulkheads for the lock and for the dam. It might even be possible to share a bulkhead among several projects in a region. The correct number of bulkheads depends on what makes economic sense for each project. Appendix B includes examples of bulkheads and stoplogs.

G.2. Bulkhead Types.

G.2.1. One-Piece Bulkheads. For narrower openings, such as some spillway crests or hydropower discharges, a single full-height bulkhead is common. It might consist of horizontal girders, with a skinplate between girders. For these narrow openings, the span of the beams is small enough so that the beams are rolled shapes, or possibly fabricated I-shapes. Since a one-piece bulkhead is similar to a vertical lift gate, much of the information contained in Appendix E should be applicable to this type of bulkhead. Figure B-5 shows a one-piece bulkhead.

G.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 trusses with a skinplate between the trusses. Figure B-6 shows a stackable bulkhead section.

G.2.3. Poiree Dams.

G.2.3.1. Some older projects use poiree dams to permit maintenance dewatering. This type of bulkhead might be used for wider openings and shallower water depths, typically downstream of the lower lock gate. 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, panels span between the frames or beams span between the frames, with panels or beams spanning vertically between the beams. Figure B-7 shows one version of a poiree dam frame and Figure B-8 shows an installed poiree dam is shown.

G.2.3.2. This configuration can be dangerous, because the frames are attached to the concrete using embedded steel anchor points. These attachments are underwater and cannot be inspected adequately as FCMs. Because of the safety problems this causes, poiree dams should generally not be used for new projects unless a safe, inspectable method can be found to anchor the frames. In fact, several older projects with poiree dams have been retrofitted with new wall slots and sill plates to permit use of stackable unit bulkheads.

G.2.4. Floating Bulkheads.

G.2.4.1. 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. For such projects, one solution was a floating bulkhead.

G.2.4.2. Floating bulkheads are similar to one-piece bulkheads, but with a skinplate 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 tightly against the face of the dam. After use, the bulkhead can be floated into position at the next gate bay for additional maintenance.

G.2.4.3. 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. Figure B-9 shows an example of a floating bulkhead.

G.2.5. Emergency Bulkheads.

G.2.5.1. This term refers to the intended use of a bulkhead rather than to its configuration. Maintenance bulkheads are usually placed in still water with no differential head. Emergency bulkheads are intended for use when there is an unexpected problem with another gate. The bulkhead is installed to prevent excess release of water, which could cause flooding or loss of pool. This might require the emergency bulkhead to be lowered into position in rapidly flowing water. This can cause significant vertical and horizontal forces on the gate while it is being positioned.

G.2.5.2. Design of emergency bulkheads will be slightly different because of these hydrodynamic forces. The lower lip of the gate must be configured to minimize vertical forces. This is done in consultation with the Hydraulic Engineer. The horizontal forces can cause friction while the bulkhead is moving, thus wheels might be used instead of sliding bearings. 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. Appendix E includes the design of emergency bulkheads configured as a lift gate.

G.3. Bulkhead Design.

G.3.1. Structural Design.

G.3.1.1. Bulkheads are typically designed similar to lift gates, assuming simple spans between supports. Skinplate, intercostals and beam design are all similar to several other gate types. For taller bulkheads, beam spacing and skinplate thickness can be adjusted along the height of the gate to provide economy or uniform loading on each beam. Vertical diaphragms can help distribute water loads among the girders. Diaphragms can also provide additional safety by providing a means to transfer loads away from a damaged or failed girder, to be carried by adjacent girders.

G.3.1.2. Stackable unit bulkheads are also designed as simple spans, but usually have horizontal trusses instead of beams. The trusses are usually kept shallow, and the ends of the trusses are tapered to fit into wall slots. The resulting geometry can include shallow angles for the truss diagonals. With welded truss joints, shallow angles and large member sizes, this can result in significant moments in the truss members rather than the pure axial load in idealized trusses. It is important to account for these moments in member design and joint detailing.

G.3.1.3. Since the water load varies over the height of the bulkhead, using the same design for each stacked unit can be inefficient. It is possible to use different heights or member sizes for upper and lower units to make the design more efficient. However, the units must then be clearly marked to ensure they are installed in the proper position in the stack.

G.3.1.4. Each unit is usually designed to act independently, assuming no horizontal load transfer between units. Since deflections can be similar for each unit, this seems like a reasonable assumption. However, the bottom unit rests on a concrete sill, possibly with embedded steel bearing plates. As the bottom unit deflects under load, there is friction with the floor. This might reduce deflections of the bottom unit, and cause differential deflection with the unit above. While this situation has not caused significant problems with existing bulkheads, it should be kept in mind when designing the lower units and the joints between the units.

G.3.1.5. Gravity loads on the units are transferred vertically through the stack to the concrete sill. There must be matching bearing surfaces between the units to carry these loads. 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 on the other chord of the truss.

G.3.1.6. These bulkheads can be designed for use with the skinplate toward or away from the dewatered area. For structural design, the more efficient configuration is for the skinplate on the side of the water, thus on the compression chord of the truss. The skinplate then provides continuous vertical bracing along the compression chord. However, then the depth of the truss protrudes into the dewatered area and into the maintenance workspace. Therefore, these units are often designed with the skinplate on the dewatered face of the truss. This maximizes workspace while minimizing the dewatered area. This can reduce the volume of water that must be pumped out, and minimizes the net uplift pressures that the concrete structure must resist.

G.3.2. Lifting Equipment.

G.3.2.1. Bulkheads usually do not have dedicated lifting equipment at each location where they will be installed. Many dams have multiple gates along the crest, but only one or a few bulkheads to perform maintenance on those gates. The bulkhead is used in whichever gate bay requires maintenance at that time. A gantry crane often travels along the dam crest to move the bulkhead between gate bays. This crane moves, lowers, and lifts the bulkhead as required. In some cases, other types of cranes are used to place the bulkheads, sometimes even rental cranes.

G.3.2.2. Lifting beams are often required for positioning bulkheads, especially for stackable units. These units 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 attach points on each unit. The mechanisms can be activated from above the water,

thus simplifying crane hook up and release. The lifting beams are designed for the gravity loads from the unit, possible vertical friction, and a hoisting impact factor. Figure B-10 shows a typical lifting beam in use. Notice the two hooks between the lifting beam and the bulkhead unit.

G.3.3. Transportation. For bulkheads located on a project feature, with a permanent crane to place the bulkheads, the size and weight of each unit is not a significant issue. Other bulkheads must 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 even the largest units. If movement by road is necessary, this might limit the length, width and weight of bulkheads. Transportation issues must be considered early in the design process.

G.3.4. Seals. The purpose of bulkheads is to permit dewatering. Therefore, each unit requires seals around the perimeter to seal against the concrete structure or against adjacent units. Many of the seal details used for other types of gates will also function well for bulkheads. Since bulkheads are more static, seal details might even be simpler. One consideration for stackable units is that the seals between units must function even with potential differential displacement between the units.

G.4. 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 least inspection to ensure that they are still functioning properly.

G.4.2. 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 FCMs, e.g., truss tension chords on stackable units.

G.5. 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 is, the more subject it is to potential theft.