



Design of Hydraulic Steel Structures

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	Engineering and Design DESIGN OF HYDRAULIC STEEL STRUCTURES	
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Change 1

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**Engineering and Design
DESIGN OF HYDRAULIC STEEL
STRUCTURES**

1. This Change 1 to EM-1110-2-2105, 31 March 1993, updates Appendix H.
2. Substitute the attached pages as shown below:

Remove page	Insert page
ii	ii
A-1 and A-2	A-1 and A-2
H-1	H-1

3. File this change sheet in front of the publication for reference purposes.

FOR THE COMMANDER:



WILLIAM D. BROWN
Colonel, Corps of Engineers
Chief of Staff

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Engineering and Design
DESIGN OF HYDRAULIC STEEL
STRUCTURES

1. Purpose. This manual prescribes guidance for designing hydraulic steel structures (HSS) by load and resistance factor design (LRFD) and guidance for fracture control. Allowable stress design (ASD) guidance is provided as an alternative design procedure or for those structure types where LRFD criteria have yet to be developed.

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

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Chapter 1 Introduction

1-1. Purpose

This manual prescribes guidance for (a) designing hydraulic steel structures (HSS) by load and resistance factor design (LRFD) and (b) fracture control. Allowable stress design (ASD) guidance is provided as an alternative design procedure or for those structure types where LRFD criteria have yet to be developed.

1-2. Applicability

This manual applies to HQUSACE/OCE 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

a. Types of HSS. Typical HSS are lock gates, tainter gates, tainter valves, 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.

b. Types of steels. Structural grade steels used for design of HSS are as referred to in CW-05502 and American Institute of Steel Construction (AISC) (1986, 1989). High-strength structural steels may be considered where economy, simplicity of detail, or greater safety of design may result from their use. Instability, local buckling, and deflection of members shall be checked regardless of the type of steel used to fabricate the structure. However, these design limit states will generally be more critical for structures fabricated from high-strength steel.

c. Design policy. Previously, in accordance with EM 1110-1-2101, ASD criteria were specified for design of all HSS. LRFD is now the preferred method of design

and should be used for those structure types for which LRFD guidance is provided (see Appendixes B through I). For HSS where LRFD has been developed, ASD may be used as an alternative design method only with prior approval of CECW-ED. Chapter 4 includes ASD criteria which are required for those HSS where LRFD has not yet been developed. For design of a structure, LRFD and ASD methods shall not be combined; however, use of LRFD and ASD methods for the design of separate structures on large construction projects is allowed.

d. Structures other than HSS. Designs for aluminum, timber, and masonry structures, service bridges and highway structures, building construction, cold-formed steel construction, railroad bridges and other railroad structures, and open-web steel joist construction shall conform to the respective industry standards and are not included in this manual.

1-5. Commentary on Paragraph 1-4, Background

Historically, the ASD method has yielded safe and reliable structures; however, the method does not recognize differing variability of different load effects (live load, dead load) and resistances (i.e. bending capacity, shear capacity, fracture, etc.). For this reason, LRFD is the preferred method of design. In the ASD method, an elastic analysis is performed for the structure of interest and the computed stress is compared with an allowable stress. The allowable stress is the yield stress, buckling stress, etc., divided by a single factor of safety (FS). In order to obtain structures with a more uniform reliability and to achieve economy, a limit states design (LSD) approach such as LRFD has been adopted by most specification writing committees. The Load and Resistance Factor Design (LRFD) approach (an LSD approach) recognizes that the loads applied to a structure and resistances of structural members are random quantities. The LRFD method has two main advantages over the ASD method. First, in a limit state analysis, one does not have to assume linearity between load and force, or force and stress. Second, multiple load factors can be used to reflect the degree of uncertainty for different loads (dead, live), while application of multiple resistance factors reflects differing uncertainties in a particular resistance (bending capacity, shear capacity, etc.). Due to these advantages of LRFD, more uniform reliability is attained in the design process and in many cases a more economical structure results.

Chapter 2 General Considerations

2-1. Limit States

All possible modes of failure should 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, and unstable crack extension leading to failure of a member. The first two failure modes (general yielding and buckling) are addressed by LRFD and ASD principles while the third failure mode (fatigue) and the fourth (brittle fracture) can be addressed using fatigue and fracture mechanics principles.

2-2. Corrosion

a. Introduction. Painting is the primary method of preventing corrosion. It may be supplemented with cathodic protection in severe environments or when other design considerations so dictate. Design considerations for reducing corrosion problems include:

(1) In certain cases, very severe environments may warrant an additional thickness added to critical structural members.

(2) In general, welded connections are more resistant to corrosion than bolted connections.

(3) Intermittent welds are more susceptible to corrosion than are continuous welds.

CW-09940, CW-16643, and EM 1110-2-3400 provide guidance for preventing corrosion.

b. Requirements. The structural engineer shall consider corrosion effects throughout the design process. Items to consider when designing the HSS include:

(1) Detail the members as much as possible so there is access for a sandblasting hose (2-ft minimum bend).

(2) Make provisions for sand to escape where member connections form open-ended chambers.

(3) Try to avoid lap joints but where used, seal weld the joint.

(4) Grind slag, weld splatter, or any other deposits off the steel.

(5) Where dissimilar metals are used select the proper material as recommended by Kumar and Odeh (1989), avoid large cathode-to-anode area ratios, use isolators, and paint both surfaces.

2-3. Dynamic Loading

HSS are often subjected to unpredictable dynamic loading due to hydraulic flow. Where dynamic loading is known to exist, but the loading function is not defined, ASD requires an effective increase in the design factor of safety. This increase is to account for unknown dynamic effects. For the LRFD method such loads are accounted for by assigning a higher load factor. The designer should provide proper detailing and structural layout to minimize dynamic loading and cavitation. For example, proper arrangement of seal details minimizes vibration.

2-4. Inspection and Maintenance

HSS are often difficult to inspect and maintain due to poor access, particularly at submerged locations. Inspections should be performed in close contact with the inspected part; however, this is not always possible since HSS include submerged components which require dewatering for inspection. Where structures are difficult to inspect and maintain, guidance is provided in paragraph 3-4 for LRFD and paragraph 4-4 for ASD.

2-5. Deviations from Prescribed Design

Where special conditions exist, proposed modifications to the load and resistance factors or allowable stresses specified herein shall be submitted to CECW-ED for approval prior to completing feasibility phase work.

2-6. Commentary on Paragraph 2-2, Corrosion

a. Introduction.

(1) Paint systems specified in CW-09940 and EM 1110-2-3400 provide a high degree of protection. For underwater HSS requiring a higher degree of protection, cathodic protection (impressed current or galvanic systems) may be used to supplement the paint system. Impressed current systems for lock gates are often damaged and become inoperative if not carefully maintained; galvanic systems require less maintenance. However,

both systems require regular maintenance. If cathodic protection is included as part of the corrosion protection system, it is imperative that a long-term maintenance plan be developed, particularly for impressed current systems.

(2) General corrosion occurs uniformly over a large metallic surface. Specifying a uniform increase in design thickness is one means to protect a structure from this type of corrosion damage. However, the total structural cost is increased and the increase in member resistance to tension, compression, and bending effects is not uniform. The primary concern with corrosion damage in HSS is the occurrence of concentration cell corrosion, pitting corrosion, or galvanic corrosion.

(3) Concentration cell corrosion occurs at small local areas on metal surfaces which are in contact with water. Concentration cells can result from any number of differences in the environment, but the two most common are metal ion cells and oxygen cells. Either localized corrosion cell causes large tubercles of corrosion products to grow above the surface, generating a weak area in the steel member. Keeping the structure well painted and clean from mud deposits prevents this type of corrosion.

(4) Pitting corrosion is a form of extremely localized attack which results in small-diameter holes (in relation to their depth) to appear in the metal. This may be initiated by a material defect in the steel or a chip in the protective coating. Pitting corrosion is highly unpredictable since there is no means to identify where defects may occur. Regular inspection and maintenance practices can reduce the possibility of pitting corrosion.

(5) Galvanic corrosion is generally a result of current generated when two dissimilar metals are in contact and the two metals are in water.

b. Requirements.

(1) Kumar and Odeh (1989) recommend HSS be dry-blast cleaned to a grade approaching white metal grade for surface preparation prior to painting. Therefore, designers should detail the structure to allow sufficient room for the hose. Extra large drain holes located in areas where the sand may be trapped may be appropriate.

(2) Most HSS consist of welded construction. Using welded connections in lieu of bolted connections is advantageous when considering concentration cell corrosion. Areas on a surface in contact with an electrolyte having a high oxygen content are cathodic relative to those areas

where less oxygen is present. Localized areas where small volumes of stagnant solution may exist include sharp corners, spot welds, lap joints, and fasteners. Using butt welds instead of bolts; seal-welding lap joints; using continuous welds; and grinding weld splatter, slag, or any other deposits off the steel help to prevent concentration cell corrosion.

(3) Where dissimilar metals are used (generally carbon steel and stainless steel), the relative areas of each metal exposed are very important because the total amount of current that flows in the cell is dependent on the total area of both metals exposed. If the anode (carbon steel) is large with respect to the cathode (stainless steel), the current is distributed over a large area and the effect at each point will be slight. Conversely, if the cathode-to-anode ratio is large, the current becomes concentrated and severe corrosion can occur. If the carbon steel is painted and there is a small defect in the coating or it becomes damaged, then the relative areas have a large cathode-to-anode area and rapid corrosion can occur. Therefore, it is best to paint both surfaces. If the stainless steel coating has defects or damage, the current will not significantly increase even if the carbon steel has metal exposed. If the distance between the cathode and anode is large, resistance in the circuit will be sufficient to eliminate the galvanic corrosion problem.

2-7. Commentary on Paragraph 2-3, Dynamic Loading

a. Dynamic loading that may occur in HSS is unpredictable in the sense that the dynamic forcing function is unknown. Unpredictable vibrations may be caused by imperfections in the operating machinery and guide slots, hydraulic flow, and load fluctuation due to passing ice. If the forcing function is known, a dynamic analysis can be used for design. At present, it is not feasible to define the load due to the many factors that affect such loadings and therefore special attention must be given to structure details. For example, supporting members of seals should maintain adequate stiffness to limit flexing which results in leakage and flow-induced vibration. The supporting members and arrangement of the bottom seal on a tainter gate can significantly affect its vibration due to flow conditions. Some of the structure types that have experienced vibration due to dynamic loading include tainter valves, vertical lift control gates, tainter gates, and miter gates.

b. Cavitation is also a concern where dynamic hydraulic loading occurs. Cavitation damage is a result of

unpredictable dynamic fluid action which causes extreme local negative pressures resulting in pitting and erosion of the surface. As for vibration, proper structure details and

good construction practices prevent cavitation from occurring.

Chapter 3 Load and Resistance Factor Design

3-1. General

This chapter is intended to give a brief synopsis of LRFD methodology and to provide general guidance on LRFD for HSS. Appendixes B through I provide specific guidance and examples for different types of HSS. HSS designed by the LRFD method shall conform to guidance contained in AISC (1986), except as specified herein, and to the engineer manuals referenced in Appendixes B through I.

3-2. Design Basis

LRFD is a method of proportioning structures such that no applicable limit state is exceeded when the structure is subjected to all appropriate design load combinations. The basic safety check in LRFD may be expressed mathematically as

$$\sum \gamma_i Q_{ni} \leq \alpha \phi R_n \quad (2-1)$$

where

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

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

α = reliability factor (see paragraph 3-4)

ϕ = resistance factor that reflects the uncertainty in the resistance for the particular limit state and, in a relative sense, the consequence of attaining the limit state.

R_n = nominal resistance

The expression $\sum \gamma_i Q_{ni}$ is the *required strength* and the product $\alpha \phi R_n$ is the *design strength*. Load factors and load combinations for specific structure types are listed in the appropriate appendix.

3-3. Strength Requirements

Strength limit states are related to safety and load-carrying capacity (i.e., the limit states of plastic moment and buckling). Formulas giving the load combinations for

determining the required strength for buildings are given in American Society of Civil Engineers (ASCE) (1990) and AISC (1986). Similar load combinations pertaining to specific HSS are specified in Appendixes B through I. 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 limit state shall be considered. Elastic analysis is permitted unconditionally by this manual. Plastic analysis is permitted only with the approval of CECW-ED, and is subject to restrictions of paragraph A5.1 of AISC (1986).

3-4. Reliability Factors for HSS

For LRFD of HSS, resistance factors of AISC (1986) are multiplied by a reliability factor α . The reliability factor α shall be 0.9 except for the following structures where α shall be 0.85:

a. 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 which are normally submerged.

b. For those HSS in brackish water or seawater.

3-5. Serviceability Requirements

Serviceability is a state of acceptable performance in which the function of an HSS, its maintainability, durability, and operability are preserved under service or operating conditions. Serviceability should be maintained for the expected life of the project (typically 50 years for navigation and local flood protection projects and 100 years for other projects). The overall structure and the individual members, connections, and connectors shall be checked for serviceability. Limiting values of structural behavior (maximum deflections, vibrations, etc.) to ensure serviceability shall be chosen with due regard to the intended function of the structure. Serviceability may normally be checked using unfactored loads. The following limit states shall be considered in design for serviceability:

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

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

c. Structural components shall be designed to tolerate corrosion or shall be protected against corrosion that may impair serviceability or operability of the structure during its design life. Closure provisions shall be made as required to maintain the structure.

3-6. Fatigue and Fracture Control

a. *Fatigue requirements.* Fatigue design shall be in accordance with the provisions of Appendix K in AISC (1986) or AISC (1989) except as specified herein. The number and frequency of load cycles is a function of the HSS purpose and its environment. Determination of the total number of loading cycles shall consider known load fluctuations such as those due to operating cycles and fluctuations of hydraulic head. For certain HSS, vibration may result in unknown load magnitudes and number of cycles; therefore, a quantitative fatigue analysis is not possible. However, for HSS where vibration may produce significant cycles of stress, the choice of details shall be such to minimize susceptible fatigue damage (i.e., details with high fatigue resistance should be used where possible).

Welding processes induce significant residual stresses, and welded members may include high tensile residual stress in the welded region. Therefore, welded members which include any computed stress variation, whether it is tension or compression, shall be checked for fatigue. Deviation from this conservative assumption requires the approval of CECW-ED.

b. *Fracture control requirements.* For fracture-critical members (FCM) and/or components, the designer shall enforce controls on fabrication and inspection procedures to minimize initial defects and residual stresses, designate the appropriate temperature zone (see Table 3.1, Note 1), and specify the related minimum Charpy V-notch (CVN) fracture toughness. FCMs shall be defined as "members and their associated connections subjected to tensile stresses whose failure would cause the structure to be inoperable." Fracture critical members shall be identified by the designer (minimum requirements are given in Appendixes B through I). Minimum allowable CVN values shall be as given in Table 3.1. Tests to determine material CVN values shall be performed in accordance with the requirements of the American Association of State Highway and Transportation Officials (AASHTO) (1978). For construction of FCMs, fabricators, welding inspectors, and nondestructive examination personnel shall be certified

according to AASHTO (1978). Designers are referred to American Welding Society (AWS) (1990) and AASHTO (1978) for guidance on developing adequate quality control and fabrication procedures that will minimize initial defects.

3-7. Commentary on Paragraph 3-2, Design Basis

Load factors and load combinations for structural steel design are based upon 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 (1986). For HSS, the load and resistance factors are governed by items discussed in paragraph 3-8 (commentary of paragraph 3-4). 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.

3-8. Commentary on Paragraph 3-4, Reliability Factors for HSS

Reliability factors are applied to AISC (1986) resistance factors for HSS design. This is to reflect a higher level of uncertainty (compared to building design) due to more aggressive environments in which HSS are placed. Historically, HSS have been designed using a higher factor of safety than that used for building design to account for the unpredictable nature of various items. The variables which 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 (50 to 100 years). For these reasons, reliability factors are applied to the resistance factors specified by AISC (1986) to effectively increase the factor of safety.

3-9. Commentary on Paragraph 3-6, Fatigue and Fracture Control

Fatigue damage and brittle fractures in HSS are rare but as structure designs, fabrication, and construction become more complex, the probability of brittle fracture increases. Welded construction, with its emphasis on monolithic structural members, increases the need to add fracture criteria to strength and buckling criteria when designing a structure. Various HSS have failed due to fatigue and brittle fracture. Many of the cracking problems that have occurred in HSS originate from poor weld details or poor fabrication. For control of fatigue and fracture, consideration must be given to the following parameters: (a) stress range, detailing, and the number and frequency of load cycles to control fatigue and (b) geometry, toughness, and stress levels to control fracture.

a. Fatigue requirements.

(1) Fatigue is the process of formation and growth of a crack due to repeated fluctuating loads. The designer cannot control the number and frequency of load cycles since this is a function of the operational requirements of the HSS. However, design options include selection of larger members to control the stress range and choice of details with low stress concentrations which have a high fatigue life.

(2) Significant vibration may occur in certain HSS due to hydraulic flow, imperfect seals, movable supports and operating machinery, and impact of passing ice or debris which may occur during a single operating cycle. For these situations, the magnitude of load and the number of load cycles are unknown. Unless predictions for load magnitude and frequency may be made using probabilistic methods, a quantitative fatigue analysis is not possible. However, the possibility of fatigue damage can be controlled by considering the design options given in the previous paragraph.

(3) AISC (1986, 1989) do not require any fatigue check for members with a calculated repetitive stress variation from zero to compression, since crack propagation will not occur in the absence of tensile stress. However, whether a stress variation is tensile or compressive, paragraph 3-6a does require a fatigue check for welded members. This is due to the possible presence of large residual tensile stresses caused by welding processes. For example, if a residual tensile stress of 25 ksi exists, a calculated stress variation from zero to -10 ksi would actually be a variation from 25 ksi to 15 ksi, which could cause fatigue cracking. Tensile residual stresses for

welded members are near the yield stress in most cases. The consideration of residual tensile stress is a conservative assumption for fatigue design. It is not currently a uniform practice in the United States; however, it is common in Europe. The assumption is currently favored by many welding specialists.

b. Fracture control requirements.

(1) Fracture is the sudden growth of a crack which may cause failure of a component. Fracture behavior is governed mainly by nominal stress level, material toughness, and geometry of the existing crack or flaw. The fracture control requirements specified herein are based on imposing material toughness requirements and limiting geometry of initial flaws for FCMs, the most critical structural components. Fracture toughness criteria are supplemented with welding and inspection requirements to form a complete fracture control plan. The toughness is controlled by imposing minimum CVN requirements per Table 3-1 and the geometry of initial flaws is controlled by imposing strict fabrication and inspection requirements. Project specifications should require qualification of fabricators and welding inspectors according to AASHTO (1978), to assure that FCMs and their components are in compliance with the requirements specified in paragraph 3-6.

(2) Table 3-1 values are the same as those required by AASHTO (1978) for steel bridges. The basic requirement used in the development of Table 3-1 was to ensure elastic-plastic behavior (i.e. prevent brittle fracture) under service loading at the minimum operating temperature. CVN tests were carried out under service load rates to determine the minimum CVN requirements to assure elastic-plastic behavior for various service temperatures (AASHTO 1978).

(3) Material toughness is affected by load rate, yield strength, service temperature, component thickness, and type of detail. Each of these effects was considered in the development of Table 3-1, and all but load rate are explicitly accounted for in Table 3-1. The following discussion is included to provide a brief explanation of toughness requirements for the various categories of Table 3-1. A more complete discussion is provided in AASHTO (1978) and Barsom and Rolfe (1987).

(a) Load rate. The effect of load rate was considered in the determination of required test temperatures. A consistent temperature shift exists between CVN values obtained for specimens subject to a given load rate (less than impact load rate) and those obtained for impact

Table 3-1
Fracture Toughness Requirements for Fracture Critical Members

Welded or Mechanically Fastened	Grade σ_{ys} (ksi)	Thickness (in.)	Zone 1 (ft-lb at °F)	Zone 2 (ft-lb at °F)	Zone 3 (ft-lb at °F)
Welded	36	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 4.0$	25 at 70	25 at 40	25 at -10
Welded	50	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 2.0$	25 at 70	25 at 40	25 at -10
		$2.0 < t \leq 4.0$	30 at 70	30 at 40	30 at -10
Welded	70	$t \leq 1.5$	30 at 20	30 at 20	30 at -10
		$1.5 < t \leq 2.5$	30 at 20	30 at 20	30 at -30
		$2.5 < t \leq 4.0$	35 at 20	35 at 20	35 at -30
Welded	100	$t \leq 2.5$	35 at 0	35 at 0	35 at -30
		$2.5 < t \leq 4.0$	45 at 0	45 at 0	Not allowed
Mechanically Fastened	36	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 4.0$	25 at 70	25 at 40	25 at -10
Mechanically Fastened	50	$t \leq 1.5$	25 at 70	25 at 40	25 at 10
		$1.5 < t \leq 4.0$	25 at 70	25 at 40	25 at -10
Mechanically Fastened	70	$t \leq 1.5$	30 at 20	30 at 20	30 at -10
		$1.5 < t \leq 4.0$	30 at 20	30 at 20	30 at -30
Mechanically Fastened	100	$t \leq 4.0$	35 at 0	35 at 0	35 at -30

NOTE:

1. Zone 1 minimum service temperature is 0°F and above; Zone 2 minimum service temperature is from -1°F to -30°F; and Zone 3 minimum service temperature is from -31° to -60°F.
2. Charpy impact tests are required on each end of each piece tested for Zone 3.

specimens. The CVN value for a specimen tested under a service load rate at service temperature is equivalent to the CVN impact value for a specimen tested at a temperature which is a constant magnitude greater (temperature shift) than the service temperature. For example (see Table 3-1), for welded 36-ksi components of thickness less than 1.5 in. which are subject to bridge service load rates and minimum service temperature, ductile behavior is assured if CVN impact values are at least 25 ft-lb for tests conducted at 70°F higher than the minimum service temperature. The temperature shift is dependent on service load rate. The temperature shift comparing static and impact load rates is maximum and as load rate increases, the temperature shift decreases. Adoption of bridge criteria for HSS is generally conservative since loading rates on bridges are likely higher than those which occur on most HSS.

(b) *Yield strength.* The more stringent requirements for steels of higher yield strengths are identified by higher CVN requirements and lower test temperatures. The higher CVN requirements for increased yield strengths are due to the fact that the design stress is generally higher which will result in more elastic stored energy. In order to attain the same degree of safety as in the lower yield steels, the CVN requirement is also increased. The reduced test temperatures are based primarily on the fact that the temperature shift between toughness under service load and impact load decreases with increasing yield strength; thus, lower CVN impact test temperatures are specified to reflect the decrease in temperature shift.

(c) *Service temperature.* The expected service temperature for a structure is a critical factor in determining toughness requirements since most steels exhibit a

transition from ductile to brittle behavior at a certain temperature. As temperature decreases, toughness and ductility decrease. Therefore, for lower minimum service temperatures, CVN specimens must be tested at lower temperatures to ensure that the steel has adequate toughness.

(d) *Component thickness.* For thick plates under tensile loading, through-thickness stresses at a crack tip are large due to the through-thickness constraint. This results in a triaxial stress state which reduces the apparent ductility of the steel by decreasing the shear stresses. Because yielding is restricted, the constraint ahead of the notch is increased resulting in reduced toughness. In order to assure ductile behavior, the CVN requirements of Table 3-1 are increased for increasing thickness.

(e) *Detail.* Welded details require more conservative CVN values than mechanically fastened details for certain thicknesses and service temperatures. The heat input due to welding can reduce toughness properties in the heat affected zone (HAZ). The HAZ is the area of unmelted parent material adjacent to the weld, which is sufficiently heated by the welding that its metallurgical properties are affected. This area may be of special importance in thick members since these usually have lower toughness and are subject to greater heat input during welding. Unfortunately, stress concentrations often overlap the HAZ of welds, thus combining the adverse effects of high stress and low toughness.

Chapter 4 Allowable Stress Design

4-1. General

HSS designed by the ASD method shall conform to specifications contained in AISC (1989), except as specified herein, and to the engineer manuals referenced in Appendixes B through I.

4-2. 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

$$f(\sum Q_i) \leq F_{\text{allow}} \quad (3-1)$$

where

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

F_{allow} = allowable stress (yield stress, buckling stress, shear, net section tension, bearing strength, etc. divided by a factor of safety).

4-3. Load and Stress Requirements

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

Group I	
Dead load	Buoyancy load
Live load (serviceway)	Hydrostatic load
Thermal stress load	Operating equipment load
Ice loads (static)	
Group II	
Impact (vessel, debris, ice)	Water hammer
Wind loads	Ice loads (transient)
Wave loads	Operational basis earthquake (OBE)

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

(2) When the loading includes Group II loads acting alone or in combination with Group I loads, allowable stresses may be increased 1/3 above the values otherwise provided. However, the section thus provided shall not be less than that required for Group I loads when designed with the normal allowable stresses.

b. Stresses. It is considered necessary to reduce the allowable stresses given in AISC (1989) for HSS design (see commentary for paragraph 4-4 (paragraph 4-8)). Allowable stresses for three main types of HSS are specified in paragraph 4-4. Examples of each HSS type are discussed in the Commentary. If a structure has characteristics of more than one type, the lesser allowable stress is required.

4-4. HSS Types: Modifications for Allowable Stresses.

a. 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 shall be 0.75 times that allowed by ASIC (1989).

b. Type B. HSS which are normally hydraulically loaded and are not subjected to unknown dynamic loading. For Type B HSS, the allowable stress shall be 0.83 times that allowed by AISC (1989).

c. Type C. HSS which are used for maintenance and are not considered emergency closures. For Type C HSS, the allowable stress shall be 1.1 times that allowed by AISC (1989). These allowable stresses are the maximum allowable values and may not be further increased due to Group II loading.

4-5. Serviceability Requirements

Guidance in paragraph 3-5 is applicable.

4-6. Fatigue and Fracture Control

Guidance in paragraph 3-6 is applicable.

4-7. Commentary on Paragraph 4-3, Load and Stress Requirements

a. ASD guidance for HSS considers Groups I and II loading, and Types A, B, and C stresses. The loading groups determine which conditions must stay within the modified AISC allowable stresses and which loading conditions are permitted a 1/3 increase in allowable stress. 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 which is used in building design.

b. Group I loads include those loads which are relatively constant for a significant time period, and Group II loads are those which vary with time. The 1/3 increase in allowable stress for structures subject to Group II loads acting alone or in combination with Group I loads is to account for the improbability of the simultaneous occurrence of maximum lifetime loads. 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. If ice is acting dynamically on the structure due to wind or flowing water, it is considered a Group II load.

4-8. Commentary on Paragraph 4-4, HSS Types: Modifications for Allowable Stresses

a. In general, it is considered that HSS are subjected to more extreme environments and are subject to less predictable loads than are buildings. Variables listed in paragraph 3-8 (commentary of paragraph 3-4) are among the causes of this additional uncertainty. Therefore, an increase in the design factor of safety over that used for building design is considered necessary for HSS design.

b. 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.

c. Type A includes those structures which are subject to unpredictable dynamic loading, or those which 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), tainter and vertical lift crest gates used for regulation and subject to unknown dynamic hydraulic forces, and lock valves (normally submerged and difficult to maintain).

d. 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 tainter crest gates, vertical lift crest gates, power intake gates designed for top of power pool, lock gates (miter gates, lift gates, and sector gates), and floodwall closures.

e. Type C structures include temporary closure items which 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 applied to AISC (1989) allowable stresses reflects a 1/3 increase of the Type B allowable stresses. This increase is considered appropriate due to the fact that such structures are used on a temporary basis under essentially constant loading.

Chapter 5 Connections and Details

5-1. General

Connections consist of connecting elements (e.g., stiffeners, gusset plates, angles, brackets) and connectors (bolts, welds, or for older HSS, rivets). Connection design shall conform to the specifications contained in AISC (1986, 1989) and AWS (1990) except as specified herein. Critical connections should be fully detailed by the design engineer. Connections which are considered noncritical may be detailed by the fabricator; however, the designer shall clearly define the requirements of the noncritical connection. Any deviation from details originally specified by the design engineer shall be reviewed and approved by the design engineer. Details that will result in safe economical fabrication methods shall be used. Special critical connections for specific structure types are discussed in the appropriate appendixes.

5-2. 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 must consider stress concentrations, eccentricities, field splices, imposed restraints (fixity), and fatigue resistance. Following is a discussion of these design considerations.

a. Stress concentrations. Avoid abrupt transitions in thickness or width, sharp corners, notches, and other stress raising conditions.

b. Eccentricities. Effects of eccentricity of fastener groups and intersecting members shall be accounted for in the design of connections (see Chapter J of AISC (1986, 1989)).

c. Splices. Shipping restrictions require large HSS to be delivered in sections, which makes field splicing necessary to form the completed structure. 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 or design forces.

d. Restraints. Connections between intersecting members are usually designed to be rigid (original angle

between connected members remains fixed) or simple (pinned). If the design assumed a pinned connection, the as-built connection should provide for members to rotate relative to each other to accommodate simple beam end rotation (to accomplish this, inelastic deformation is permitted).

e. Fatigue. Connections shall be designed to minimize the possibility of fatigue damage by using proper detailing practices (see AISC (1984, 1986, 1989) and AASHTO (1978)), and limiting the stress range in accordance with Appendix K of AISC (1986, 1989). Corrosion-fatigue shall be controlled with a well designed and maintained corrosion protection system.

5-3. Bolted Connections

Fully tensioned high-strength bolts shall be used for all HSS structural applications. For nonstructural applications, use of A307 bolts or snug-tight high-strength bolts is allowed, provided requirements of AISC (1986, 1989) are followed. Bolts shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. AISC (1984, 1986, 1989) and Kulak, Fisher, and Struik (1987) are useful aids to designing bolted connections.

5-4. Welded Connections

Most HSS are constructed using welded connections. AISC (1984, 1986, 1989) and AWS (1990) are useful aids to selecting the connection details. Welding requirements of AISC (1986, 1989) and AWS (1990) shall be followed. Thick plate weldments shall be designed considering heat requirements (see Section 4 of AWS (1990)), toughness requirements, and geometric requirements (see Section A3 of AISC (1986, 1989) for toughness and geometric requirements). Intersecting and overlapping welds should be avoided. Intermittent welds should be avoided for dynamically loaded members and members subject to corrosion. Through-thickness welds should have backing bars removed and should be ground smooth. The designer shall review and approve the contractor's proposed welding processes and shop drawings.

5-5. Commentary on Paragraph 5-1, General

Connections for HSS are usually in a more severe environment than connections for buildings. HSS connections may be exposed to weather, fresh or salt water, flowing water, and, for many HSS, impacts. AISC (1986 or 1989) can be used as guidance but should be

supplemented with AASHTO (1989) since many HSS members have more in common with bridges (sizes, types of connections, and loads) than with steel building frames. Connection details must be consistent with the assumptions used in the design analysis of the structure and must be capable of transferring the required forces between connected members. 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. Most HSS use welded or bolted connections; however, many older structures have riveted connections.

5-6. Commentary on Paragraph 5-2, Design Considerations

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. These details are critical for fatigue resistance. AWS (1990) shows geometries for welded connections that minimize stress concentrations at transitions between members of different thicknesses or widths.

b. Eccentricities.

(1) 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 gravity axes; if not, provision shall be made for bending and shearing stresses due to the eccentricity.

(2) The designer has the option of selecting a concentric connection or, in some cases, an eccentric connection. A concentric connection is detailed so that the gravity axes 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 an additional moment in the connection. However, in some cases a concentric connection may be undesirable because it can require poorly shaped elements such as long gusset plates with a limited buckling capacity that is difficult to assess.

(3) An eccentric connection may be detailed to simplify the design of gusset plates. For example, a member may be located such that its line of force passes through the corner of the gusset plate. However, the lines of action of the force in the intersecting members usually do not pass through the same point. The axial force acting eccentrically will produce a moment in the connection which must be distributed among the connected members based on their relative stiffness. See AISC (1984) for illustrated examples.

5-7. Commentary on Paragraph 5-3, Bolted Connections

In the past many HSS have used riveted connections; however, the use of rivets has largely been replaced by use of high strength bolts. Per AISC (1986, 1989), full tightening is required for cyclic loads, for bolts in over-size 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.

5-8. Commentary on Paragraph 5-4, Welded Connections

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 which 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 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.

Appendix A References

A-1. Required Publications

EM 1110-1-2101

Working Stresses for Structural Design

EM 1110-2-2400

Structural Design of Spillways and Outlet Works

* EM 1110-2-2502

Retaining and Flood Walls

EM 1110-2-2602

Planning and Design of Navigation Lock Walls and Appurtenances *

EM 1110-2-2701

Vertical Lift Crest Gates

EM 1110-2-2702

Design of Spillway Tainter Gates

EM 1110-2-2703

Lock Gates and Operating Equipment

EM 1110-2-2705

Structural Design of Closure Structures for Local Flood Protection Projects

EM 1110-2-2901

Tunnels and Shaft in Rock

EM 1110-2-2902

Conduits, Culverts, and Pipes

EM 1110-2-3001

Planning and Design of Hydroelectric Power Plants

EM 1110-2-3104

Structural Design of Pumping Stations

EM 1110-2-3400

Painting: New Construction and Maintenance

CE-1507.01

Tractor Gates-Broome Type

CE-1602

Dam Gantry Cranes

CW-05502

Miscellaneous Metal Materials, Standard Articles and Shop Fabrication Items

CW-09940

Painting: Hydraulic Structures and Appurtenant Works

CW-16643

Cathodic Protection Systems (Impressed Current) for Lock Miter Gates

American Association of State Highway & Transportation Officials (AASHTO) 1978

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Galambos, Ellingwood, MacGregor, and Cornell 1982

Galambos, T. V., Ellingwood, B., MacGregor, J. G., and Cornell, C. A. 1982 (May). "Probability Based Load Criteria: Assessment of Current Design Practice," ASCE Journal of the Structural Division, Vol 108, No. ST5.

Kulak, Fisher, and Struik 1987

Kulak, G.L., Fisher, J.W., and Struik, H.A., 1987. "Guide

to Design Criteria for Bolted and Riveted Joints," Second Edition, John Wiley & Sons, Inc., New York

Kumar and Odeh 1989

Kumar, A., and Odeh, A. A. 1989. "Mechanical Properties and Corrosion Behavior of Stainless Steels for Locks, Dams, and Hydroelectric Plant Applications," Technical Report REMR-EM-6. Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

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Appendix B Load and Resistance Factor Design Criteria for Miter Gates

B-1. Introduction

a. Purpose. This appendix provides guidance for design of miter gates by the load and resistance factor design (LRFD) method. Load-carrying members (including but not limited to: skin plates, intercostals, girders, diagonals, vertical diaphragms, and anchorage systems) shall be designed in accordance with the criteria contained in this appendix and Chapters 1, 2, 3, and 5. Miter gate layout, selection of materials, and assumed member loading shall follow guidance specified in EM 1110-2-2703 unless otherwise stated herein. Mechanical and electrical items shall be designed in accordance with Chapter 4 and guidance specified in EM 1110-2-2703.

b. References. Required references are listed in Appendix A.

c. Background. ASCE (1990) and AISC (1986) specify load factors and load combinations for buildings; however, for miter gates, unique loads and load combinations exist. The load factors and load combinations specified in paragraph B-2a pertain specifically to miter gates. Development of the load factors included consideration of the respective load variability, definition, and likeness to those loads specified in ASCE (1990) and AISC (1986). Some loads I , H_s , and E (discussed in paragraph B-2b) are difficult to predict and are highly variable, yet are assigned a load factor of 1.0. This is not what might be expected for such unpredictable loads. The load factor 1.0 for barge impact and temporal hydraulic loads was chosen, in part, on the basis that these loads are specified based on historical experience and are assigned extreme values. It is not realistic to use load factors other than 1.0 for such arbitrarily designated loads. The 1.0 load factor for earthquake loading was chosen to remain consistent with what will be presented in the revision to ASCE (1990) and the 2nd edition of AISC (1986).

B-2. Load and Resistance Factor Design

a. Strength requirements. Miter gates shall have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in the following load combinations. The most unfavorable effect may occur when one or more of the loads in a particular load combination is equal to zero.

$$1.4H_s + 1.0I \quad (B-1a)$$

$$1.4H_s + 1.0H_t \quad (B-1b)$$

$$1.2D + 1.6(C+M) + 1.0H_t \quad (B-2a)$$

$$1.2D + 1.6(C+M) + 1.2Q \quad (B-2b)$$

$$1.2H_s + 1.0E \quad (B-3)$$

The nominal loads are defined as follows:

D = dead load

Q = maximum operating equipment load

E = earthquake load

I = barge impact load

H_s = hydrostatic load

H_t = temporal hydraulic load

C = ice load

M = mud load

b. Load considerations. Loads due to thermal stresses need not be considered. Serviceway loads are not included in the above combinations due to their low magnitude, and they are counteracted by buoyancy of the structure. Walkways are not HSS and should be designed in accordance with the requirements in AISC (1986).

(1) Hydraulic loads. The temporal hydraulic load H_t shall be equal to 1.25 ft of head as specified in paragraph 3-9 of EM 1110-2-2703. The hydrostatic load H_s shall be determined based on site-specific conditions for upper and lower pool elevations. The predictability of maximum hydrostatic load justifies using a relatively low load factor which reflects the low level of uncertainty in the loading. The 1.4 load factor in Equations B-1a and B-1b is relatively low, yet considering the reduction in resistance due to the resistance factor ϕ and the reliability factor α , it provides an adequate overall factor of safety.

(2) Gravity loads. Loads D , C , and M shall be determined based on site-specific conditions. Ice loads C are considered as gravity loads; ice acting as lateral loads are not considered in the load combinations (see paragraph B-2c).

(3) Operating loads. The load Q shall be the maximum load which can be exerted by the operating machinery (obtained from the mechanical engineer that designed the machinery). The inertial resistance of water while a leaf is operated is the hydrodynamic load H_d . Effects of H_d are included in paragraph B-2f. This load will control fatigue design and shall be equal to 30 pounds per square ft (psf) or 45 psf based on requirements given in Chapter 3 of EM 1110-2-2703. H_d never controls the strength design when compared with H_t or Q and is not included in the load combinations.

(4) Barge impact load. The barge impact load I shall be specified as a point load as shown in Figure B-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 (unsymmetric 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 impact load I shall be equal to 250 kips for unsymmetric loading and 400 kips for symmetric loading.

(5) Earthquake load. Design loads shall be determined based on an operational basis earthquake (OBE) defined as that earthquake having a 50 percent chance of being exceeded in 100 years. This translates to a probability of annual exceedance of 0.0069, or approximately a 145-year mean recurrence interval. The earthquake load E shall be based on inertial hydrodynamic effects of water moving with the structure. Inertial hydrodynamic loads shall be determined based on Westergaard's equation

$$p = \frac{7}{8} \gamma_w a_c \sqrt{Hy} \quad (\text{B-4})$$

where

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

γ_w = unit weight of water

a_c = maximum acceleration of the supporting lock wall due to the OBE (expressed as a fraction of gravitational acceleration g)

H = pool depth

y = distance below the pool surface

The lock wall shall be assumed rigid in determination of a_c , and the assumed direction of a_c shall be parallel to the lock centerline. The inertial forces resulting from the

mass due to structural weight D , ice C , and mud M are insignificant compared to the effect of p and need not be considered.

c. Load cases. The following load cases shall be considered with the appropriate loading combinations:

(1) Case 1: Mitered condition. Loads include hydrostatic loads due to upper and lower pools, and barge impact or temporal hydraulic loads (Equations B-1a and B-1b). Although not included in Equations B-1a and B-1b, loads C , D , and M act when the gate is in the mitered position. However, in the mitered position their effects will not control the member sizes and these loads are accounted for in load case 2 where they may control. Lateral ice loads, as discussed in the commentary of paragraph 4-3 (paragraph 4-7) are not considered in Equations B-1a and B-1b. It would be appropriate to include such a load in place of I as specified by Equation B1-a. However, design for a lateral ice load of 5 kips per ft (as specified by EM 1110-2-2702) with a load factor of 1.0 will not control when compared to design required by I .

(a) Above pool. Equation B1-a is applicable to the girders located above pool (upper pool elevation for the upper gate and lower pool for the lower gate) where barge impact may occur. The skin plate and intercostals need not be designed for barge impact. For design of skin plate and intercostals located above pool, a minimum hydrostatic head of 6 ft shall be assumed.

(b) Below pool. The upper gate shall be designed assuming the lock is dewatered. Loads include hydrostatic loads due to upper pool only (Equation B-1b; $H_t = 0$). The lower gate shall be designed considering normal upper and lower pool elevations including temporal hydraulic loads H_t . H_t is applicable only to the submerged part of the gate.

(2) Case 2: Gate torsion. Loads include gravity loads (C , M , and D), and operating equipment load Q or temporal hydraulic load H_t (Equations B-2a and B-2b). In this condition there are no differential hydrostatic loads.

(a) Temporal condition. Equation B-2a shall be applied to consider gate leaf torsion with the temporal hydraulic load acting on the submerged part of leaf (the temporal hydraulic load may act in either direction).

(b) Submerged obstruction. Equation B-2b shall be applied to consider leaf torsion which may be caused by a

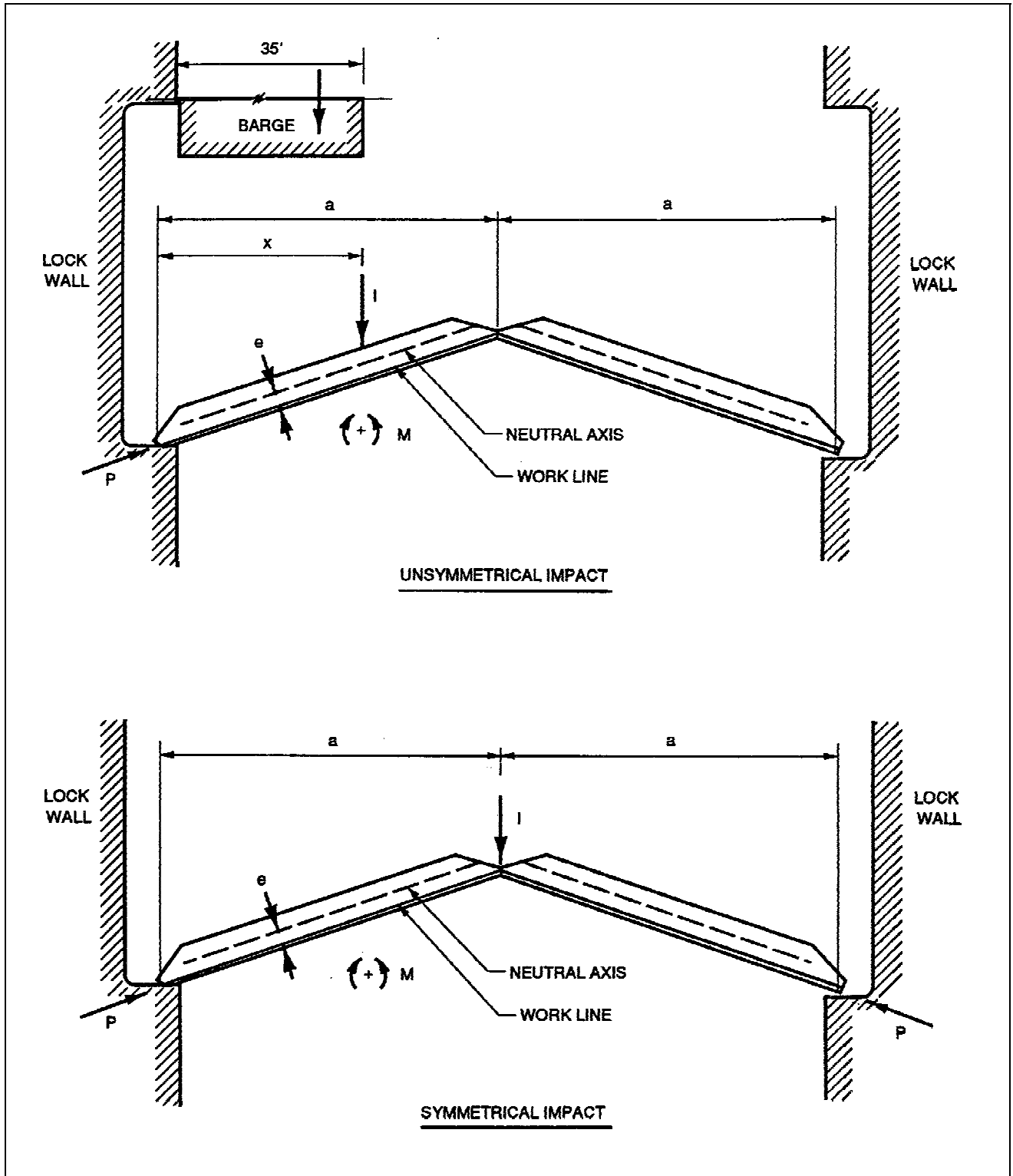


Figure B-1. Point load impact for miter gate girders

submerged obstruction. For this case, it is assumed that the bottom of the leaf is held stationary by a submerged obstruction while Q is applied causing the gate leaf to twist.

(3) Case 3: Earthquake. Equation B-3 shall be applied assuming that the gate is mitered, and hydrostatic loads due to upper and lower pools are acting. The earthquake acceleration shall be applied in the direction parallel to the lock centerline. Elastic structural analysis shall be performed with no allowance for ductility.

d. Design for individual members. The following is a brief description of design assumptions, appropriate LRFD formulas, and load cases for the design of individual gate members. These items are further discussed in the design examples of paragraph B-4 and EM 1110-2-2703.

(1) Skin plate.

(a) Skin plates shall be sized such that the maximum calculated stress is less than the yield limit state of $\alpha\phi_b F_y$ where α is defined in paragraph 3-4 and ϕ_b is defined in AISC (1986). Stresses shall be determined on the basis of small deflection thin plate theory using load cases 1 and 3 of paragraph B-2c. Small deflections are assured by limiting deflections per paragraph B-2e (deflections are small and significant membrane stresses do not develop). The minimum size for the skin plate located above the pool level shall be determined using an assumed hydrostatic head of 6 ft.

(b) The skin plate is designed assuming that each panel acts as a rectangular fixed plate. In accordance with paragraph 2-1c(1) of EM 1110-2-2703, the edges of the skin plate panels are assumed to be fixed at the centerline of the intercostals or diaphragms and the edge of girder flanges. For rectangular fixed plates subject to uniform loading, the maximum stress occurs at the centerline of the long edge. The combined interaction of transverse stress due to intercostal or girder bending (Von Mises criteria shown in EM 1110-2-2703) need not be considered.

(2) Intercostals.

(a) Intercostals shall be flat bars or plates sized such that the maximum calculated moment is less than the nominal bending strength of $\alpha\phi_b M_n$. Intercostals may be designed as simple or fixed end beams (EM 1110-2-2703 specifies fixed end) supported at the centerline of girder webs. The end connections shall be fabricated to match the design assumptions as closely as possible. In most

cases, the ends of the intercostals are welded (Figure B-2 illustrates possible details that may be used). Load cases 1 and 3 of paragraph B-2c shall be investigated to determine the maximum load effect. The assumed loading distribution for intercostals is the trapezoidal distribution shown in EM 1110-2-2703 and Figure B-3. The minimum size for intercostals located above the pool level shall be determined using an assumed hydrostatic head of 6 ft.

(b) An effective portion of the skin plate is assumed to act as the intercostal flange. The effective width of skin plate is determined assuming the skin plate to be an unstiffened noncompact member (i.e., $\lambda_r = 95\sqrt{F_y}$). The distance between cross sections braced against twist or lateral displacement of the compression flange has a controlling influence on the member strength. For the design of a simple beam intercostal the compression flange is supported continuously by the skin plate. See paragraph 2-1c(2) of EM 1110-2-2703 for additional discussion.

(3) Girders.

(a) Horizontal girders are assumed to act as singly symmetric prismatic members subjected to axial force and flexure about their major axis. Girders shall be designed as beam-columns in accordance with AISC (1986). The criteria for action about the major axis specified in paragraphs 2-1d(6) and (7) of EM 1110-2-2703 shall be revised as follows. For determination of column action buckling strength about the major axis, each girder shall have an effective length equal to the distance from the quoin block to the miter block. The ends shall be assumed pinned; the values of K and C_m shall be 1.0. Load cases 1 and 3 of paragraph B-2c shall be investigated for all girders to determine the maximum load effect. Additionally, load case 2 shall be investigated for girders which resist diagonal loads.

(b) An effective portion of the skin plate is assumed to act with the upstream flange. The effective width of skin plate adjacent to each edge of the upstream girder flange shall be based on a width-to-thickness ratio consistent with design assumptions (i.e., assumption of compact or noncompact flange). Upstream girder flanges are braced continuously by the skin plate. Downstream flanges are braced by vertical diaphragms which resist lateral displacement and twist of the cross section.

(c) Webs shall be designed using requirements for uniformly compressed stiffened elements. The use of slenderness parameters for webs in combined flexural and

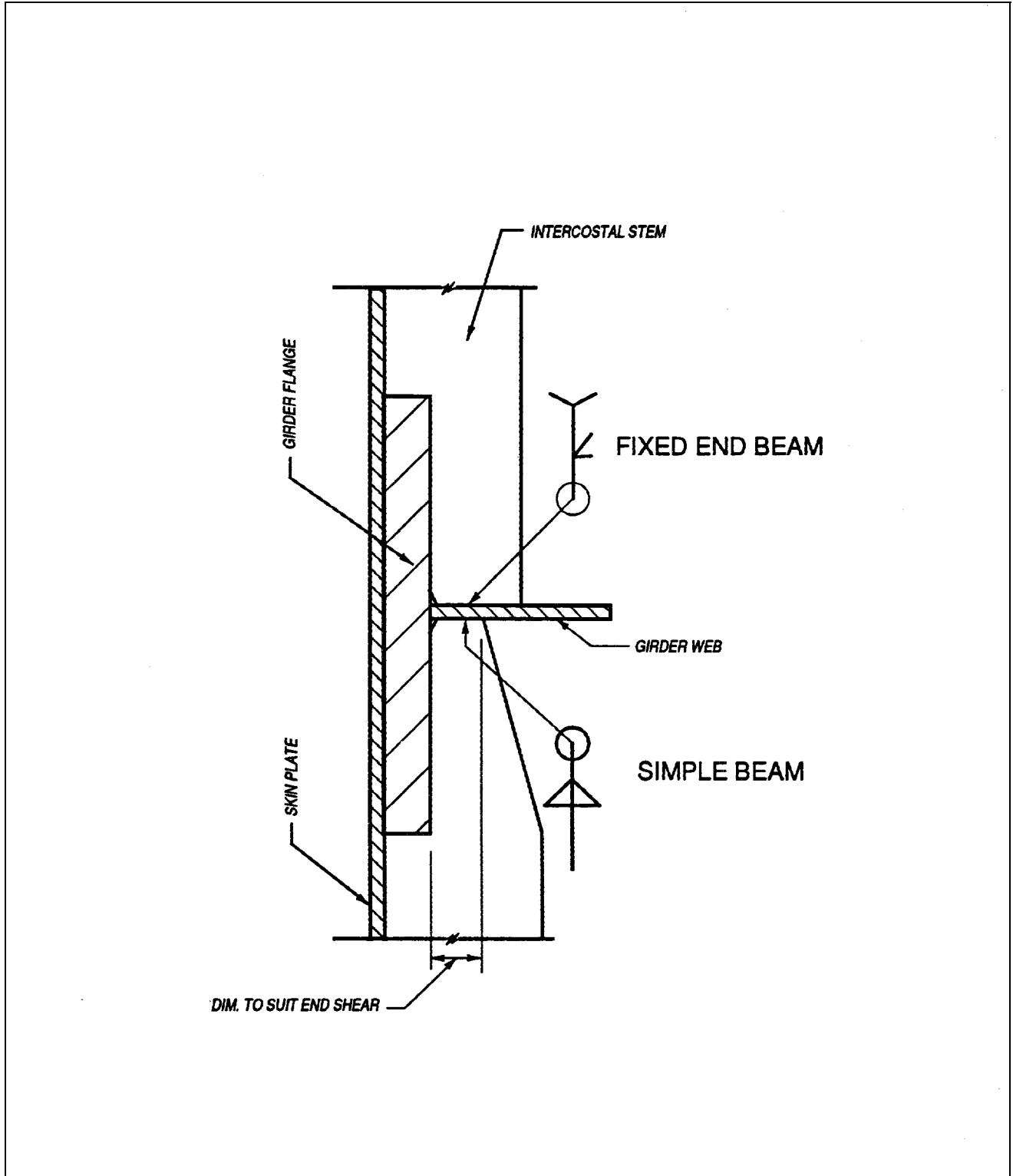


Figure B-2. Assumptions for intercostal end connections

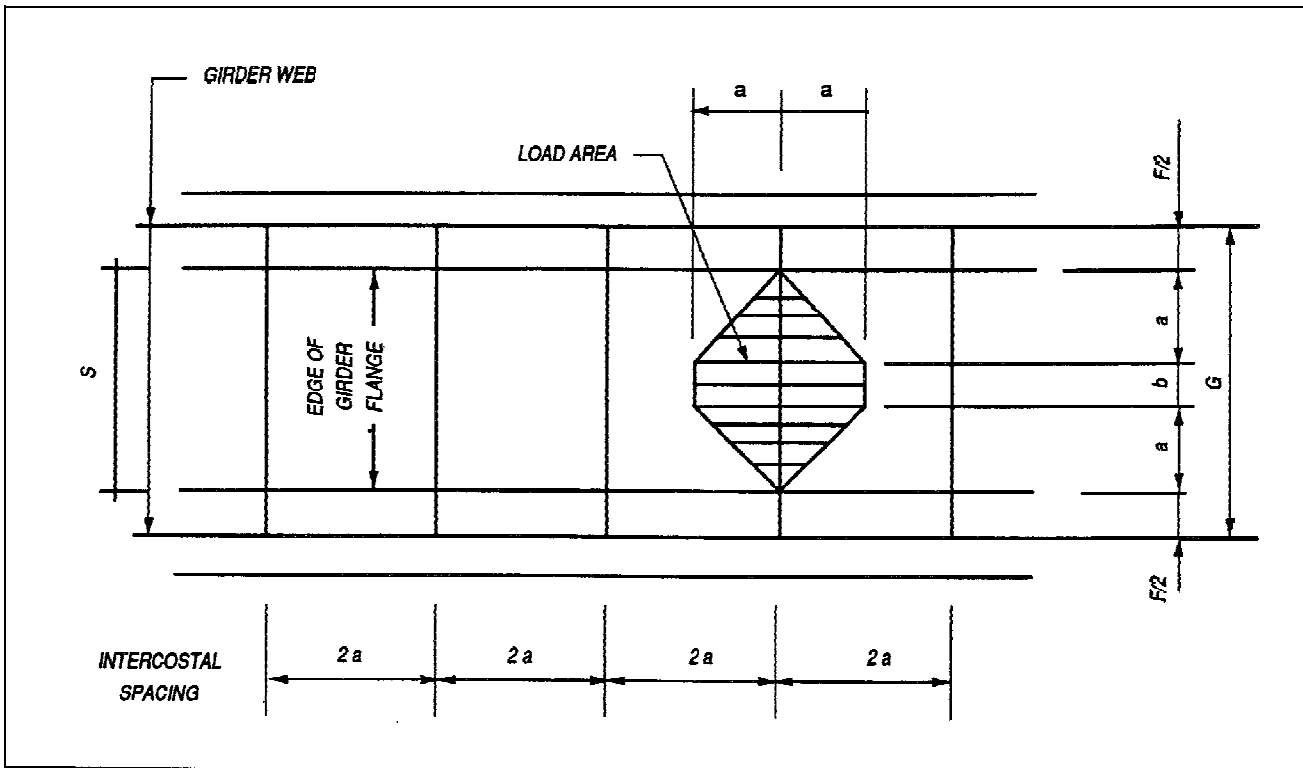


Figure B-3. Nomenclature and assumed load area for intercostal design

axial compression in Table B5-1 of AISC (1986) should be avoided since these criteria were developed for rolled shape beam-columns and may not apply for deep girder sections.

(4) Diagonals. Diagonals shall be designed as tension members considering the limit states of yielding in the gross section or fracture in the net section. The design assumptions shall be based on procedures presented in Chapter 3 of EM 1110-2-2703. Load case 2 of paragraph B-2c is applicable.

(5) Vertical diaphragms. Vertical diaphragms resisting diagonal loads shall be designed using the same load case as used for the diagonals design. See paragraph 2-1c(3) of EM 1110-2-2703 for additional discussion.

(6) Anchorage systems. The anchorage systems supporting miter gate leafs are discussed in paragraph 2-1g(2) of EM 1110-2-2703. These criteria require components of the system to be designed as individual units with the resultant force applied to the units being a combination of the strut force and the dead weight of the leaf, increased 10 percent for impact. These loading criteria should be

used with load case 2 of paragraph B-2c.

e. Serviceability requirements. Miter gates shall be designed for an expected life of 50 years. Limiting values of structural behavior to ensure serviceability (e.g., maximum deflections, vibration considerations, details for ease of maintenance, etc.) shall be chosen with due regard to assure the gate functions for its design life. Normally, serviceability can be checked using unfactored loads. As a minimum, the following guidance shall be followed.

(1) The overall structure and the individual members, connections, and connectors shall be checked for serviceability. This shall be verified by testing during erection as specified in paragraph 2-3q of EM 1110-2-2703.

(2) Gate leaf deflection (twist) shall be limited to a value which is less than 50 percent of the miter bearing block width.

(3) The skin plate deflection shall be limited to 0.4 times the plate thickness.

(4) Vibration of the seals, equipment, or movable supports shall not impair the operability of the gate.

(5) Structural components shall be designed to tolerate corrosion or be protected against corrosion that may impair the serviceability or operability of the structure. Plates shall be used for girder web stiffeners and intercostals (instead of more efficient rolled sections) to make it easier to apply the paint system.

f. Fatigue. Members and their connections subjected to repeated variation of load shall be designed for fatigue. The total number of loading cycles shall be determined based on changes in load due to lock operation. The range of stresses due to unfactored loads shall be equal to or less than the allowable stress variation given in appendix K of AISC (1986). The following conditions shall be considered for fatigue analysis.

(1) Skin plates, intercostals, and girders. Stress variation shall be determined based on variation in hydrostatic load H_s assuming the gate is in the mitered position and the hydrostatic load is due to upper and lower pools.

(2) Diagonals, vertical diaphragms, strut arm and connection, hinge and anchorage arms. These elements shall be evaluated based on variation of stress due to hydrodynamic load H_d acting as the gate operates.

g. Fracture. Requirements of paragraph 3-6 shall be applied to fracture critical members (FCM). The designer shall determine which members are fracture critical for the specific miter gate in question. Typically, strut arms and connections, anchorage arms, and diagonals are considered to be FCM. Project specifications shall address the topics which are discussed in the commentary of paragraph 3-6c (paragraph 3-9).

B-3. Connections and Details

Chapter 5 provides general guidance for connection design. Connection details shall be consistent with the design assumptions. For example, Figure B-2 illustrates the details required for consistency in design of intercostals for the assumptions of simple and fixed connections. Paragraphs 1-5a(6) and 1-5a(7) of EM 1110-2-2703 discuss the use of bolts, welds, and fabrication of gate leaves, and paragraph 2-1j(3) includes a discussion on diagonal connections.

B-4. Design Examples

a. General. To illustrate LRFD principles for the design of a miter gate, example calculations are provided in paragraph B-4b. These calculations are provided to demonstrate LRFD principles; they do not provide a comprehensive design for the entire gate. Examples are limited to the design of the skin plate, an intercostal, a horizontal girder, and the diagonals for a horizontally framed miter gate. AISC (1986) equation numbers are identified by "AISC" followed by the appropriate equation number.

b. Design examples for a horizontally framed miter gate. Examples for a horizontally framed downstream miter gate that spans a 110-ft-wide lock chamber are included. Each leaf is 55 ft high and is required to span 62 ft. A vertical cross section of the leaf is shown in Figure B-4. All material is assumed to be ASTM A36 steel. The distributions of unfactored loads H_s , H_t , and E are shown in Figure B-5, and the load magnitudes for girders and panels are listed in Tables B-1 and B-2, respectively. The kips per square foot (ksf) values for H_s are determined by the hydrostatic head and those for E are calculated by Westergaard's equation for the corresponding depths. The k/ft values for girders are determined using the ksf loads distributed over a tributary area between panel center points. Earthquake loading E is determined based on requirements of paragraph B-2b(5) assuming a maximum lock wall acceleration of 0.1 g ($a_c = 0.1$). Examples for the skin plate, intercostal, and girder are for members located at the lower part of the gate leaf where the critical loading occurs.

(1) Skin plate design example. Traditionally, the skin plate is designed as a plate fixed at the centerline of the intercostals and the edges of girder flanges. Nomenclature for skin plate design is shown in Figure B-6. The design loading includes hydrostatic H_s , temporal hydraulic H_t , and earthquake E loads. Uniform pressure loads are assumed to act over the panel surface with a magnitude equal to that of the pressure acting at the center of the panel. Per paragraph B-2d(1), the minimum size (for panels at the top of the gate) shall be determined based on a 6-ft minimum hydrostatic head. For panels 9-12 (see Figure B-4) horizontal girders are spaced 4 ft apart and intercostals are spaced on 32-in. centers. With 6-in.-wide girder flanges (conservative approximation) the plate

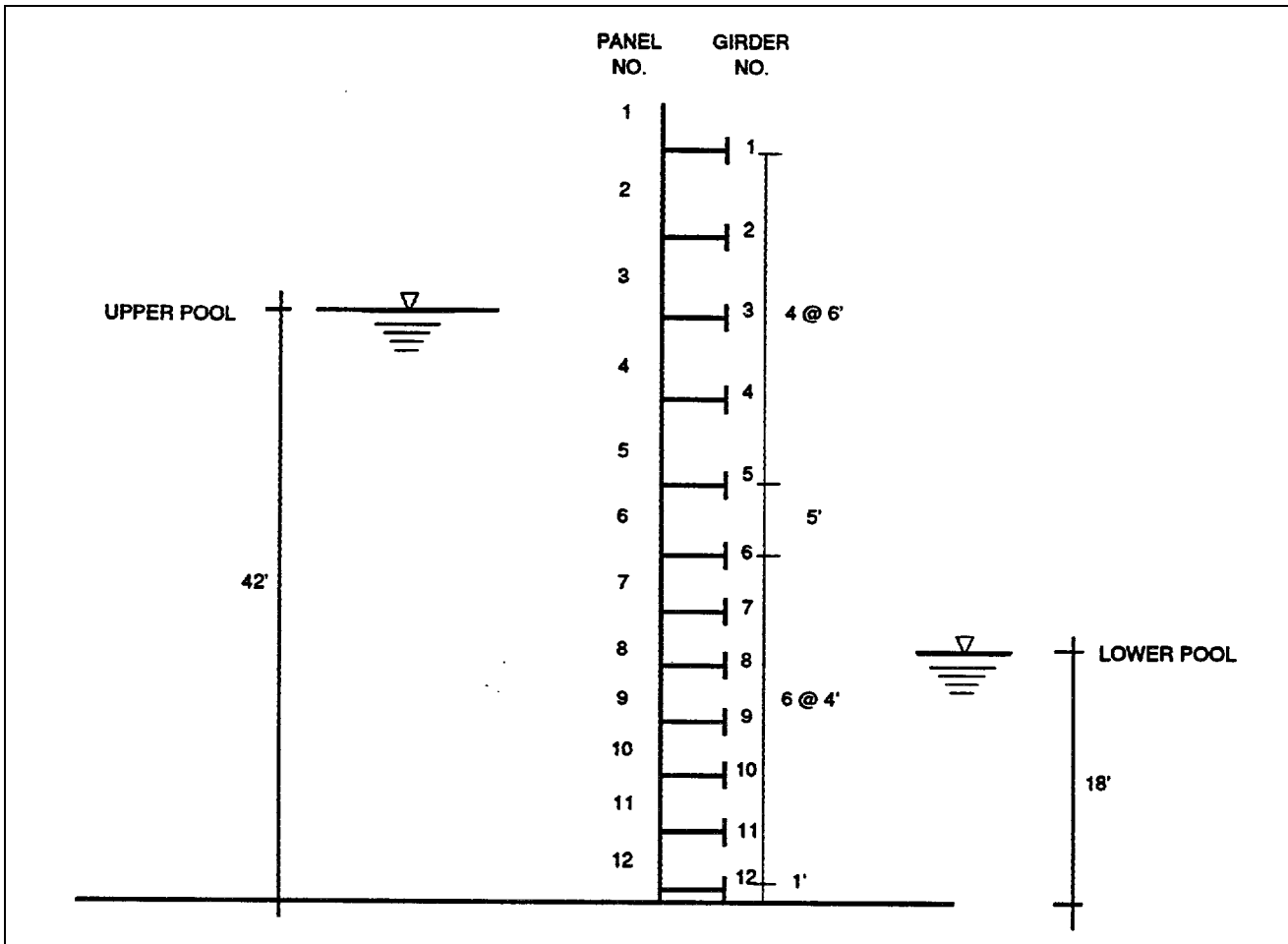


Figure B-4. Vertical cross section for example miter gate

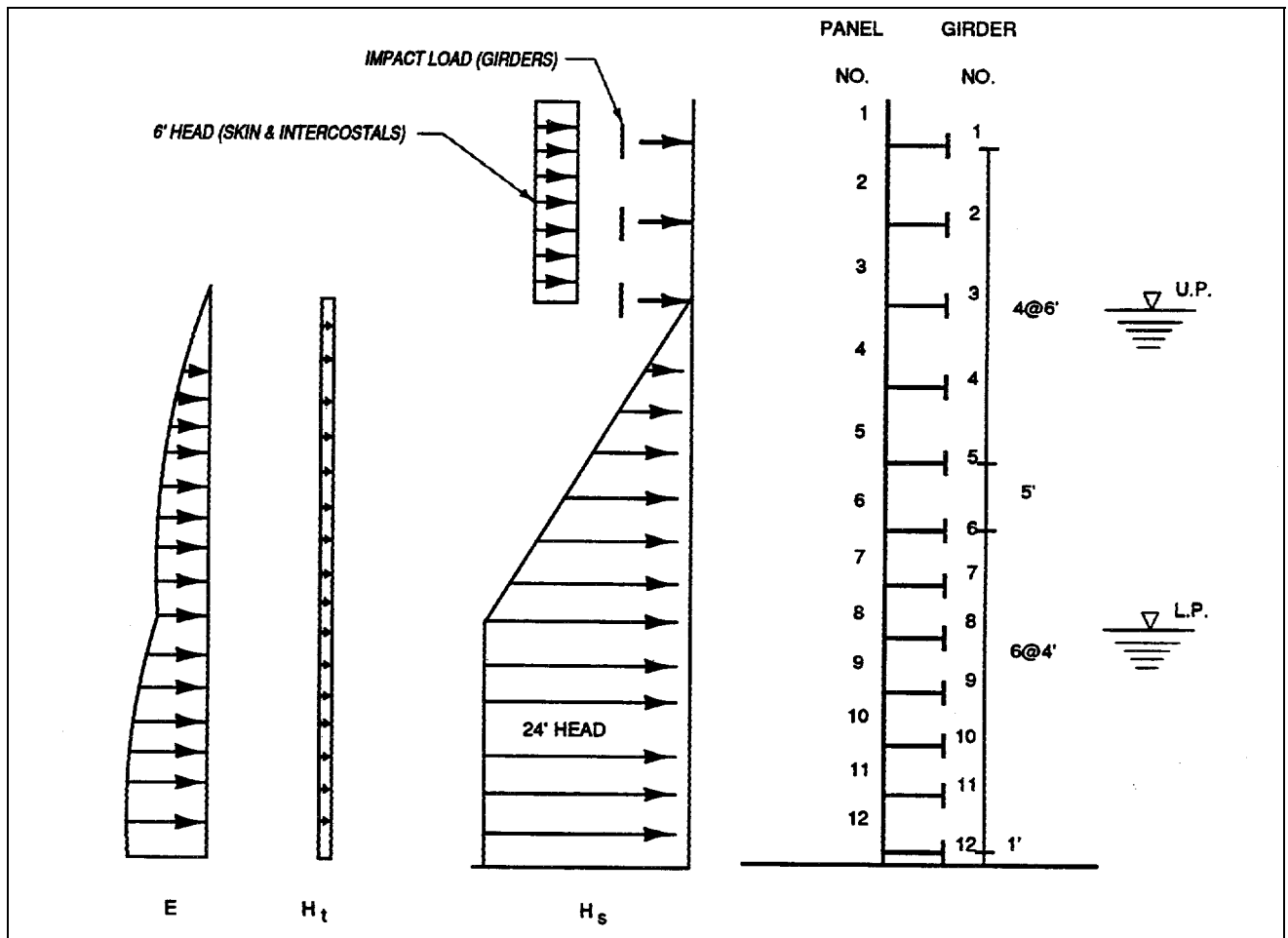


Figure B-5. Example miter gate loading

Table B-1
Girder Loads

Girder No.	H_s (ksf)	H_s (k/ft)	H_t (k/ft)	E (ksf)	E (k/ft)	$1.4H_s+H_t$ (k/ft)	$1.2H_s+E$ (k/ft)
1	0.000	0.00	0.00	0.000	0.000	0.00	0.00
2	0.000	0.00	0.00	0.000	0.000	0.00	0.00
3	0.000	0.28	0.23	0.000	0.065	0.63	0.40
4	0.374	2.24	0.47	0.087	0.522	3.61	3.22
5	0.749	4.12	0.43	0.123	0.674	6.19	5.62
6	1.061	4.77	0.35	0.146	0.657	7.03	6.39
7	1.310	5.24	0.31	0.162	0.649	7.65	6.94
8	1.498	6.00	0.31	0.200	0.800	8.71	7.99
9	1.498	6.00	0.31	0.242	0.969	8.71	8.16
10	1.498	6.00	0.31	0.273	1.091	8.71	8.38
11	1.498	6.00	0.31	0.299	1.195	8.71	8.39
12	1.498	4.49	0.23	0.322	0.960	6.53	6.35

Table B-2
Skin Plate and Intercostal Loads

Panel No.	H_s (ksf)	H_t (ksf)	E (ksf)	$1.4H_s+H_t$ (ksf)	$1.2H_s+E$ (ksf)
1	0.374	0.000	0.000	0.524	0.449
2	0.374	0.000	0.000	0.524	0.449
3	0.374	0.000	0.000	0.524	0.449
4	0.374	0.078	0.043	0.602	0.492
5	0.563	0.078	0.105	0.866	0.780
6	0.906	0.078	0.134	1.346	1.221
7	1.187	0.078	0.154	1.740	1.578
8	1.437	0.078	0.181	2.090	1.906
9	1.498	0.078	0.221	2.174	2.018
10	1.498	0.078	0.258	2.174	2.054
11	1.498	0.078	0.286	2.174	2.082
12	1.498	0.078	0.310	2.174	2.107

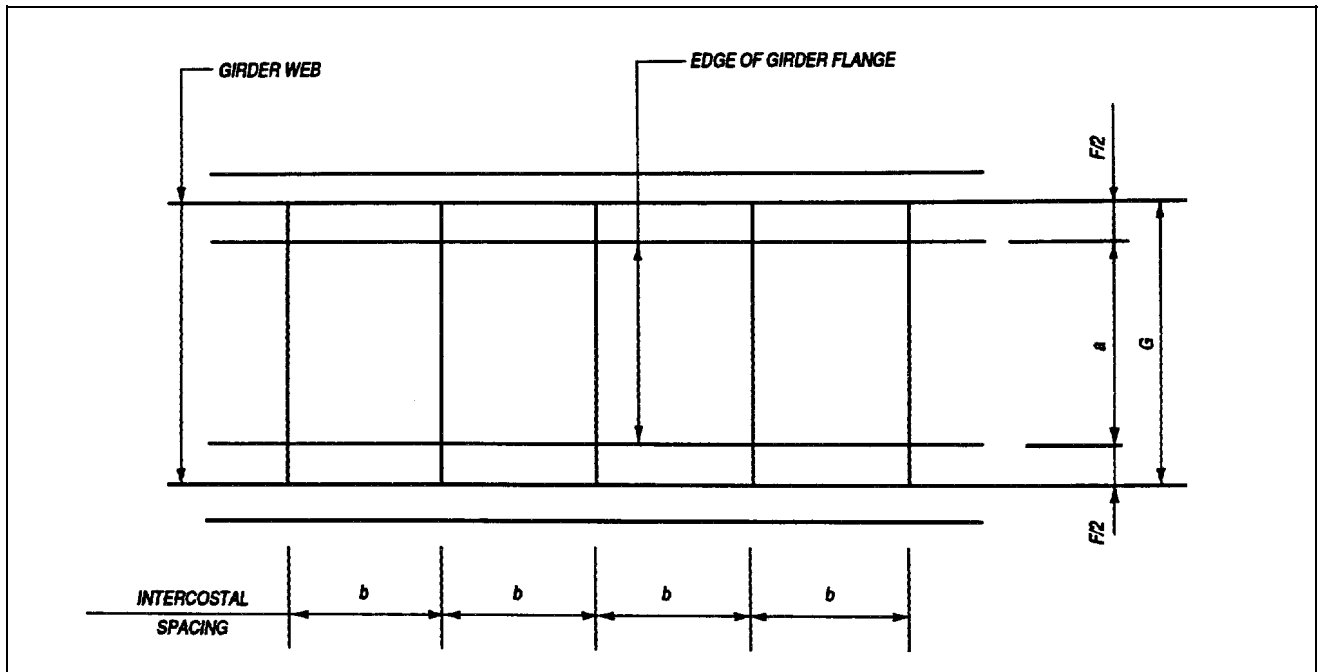


Figure B-6. Nomenclature for skin plate design

dimensions are $a = 42$ in. by $b = 32$ in. Equation B-1b is the critical load combination which yields a factored uniformly distributed load of $W_u = 2.174$ ksf = 0.0151 ksi.

(a) Required thickness based yield limit state. For a rectangular fixed plate with a uniform loading W and a limiting stress F_{lim} , the required minimum skin plate thickness t_{min} is calculated using Equation B-5.

$$t_{min} = \sqrt{\frac{0.5 W b^2}{F_{lim} \left[1 + 0.623 \left(\frac{b}{a} \right)^6 \right]}} \quad (B-5)$$

Based on yield limit state for plate bending, $F_{lim} = \alpha \phi_b F_y$. With $W = W_u$, $\alpha = 0.9$, and $\phi_b = 0.9$ the required thickness is

$$t_{min} = \sqrt{\frac{0.5(0.0151)(32)^2}{29.16 \left[1 + 0.623 \left(\frac{32}{42} \right)^6 \right]}} = 0.486 \text{ in.}$$

Therefore, select a 1/2-in.-thick plate.

(b) Deflection check. Per paragraph B-2e, the maximum deflection δ under service loading (unfactored H_s) is limited to $0.4t$. For a rectangular plate fixed on all edges,

$$\delta = \frac{0.0284 W b^4}{\left[1 + 1.056 \left(\frac{b}{a} \right)^5 \right] E t^3}$$

With $W = 1.498$ ksf = 0.0104 ksi and $E = 29,000$ ksi, the deflection δ is

$$\begin{aligned} \delta &= \frac{0.0284(0.0104)(32)^4}{\left[1 + 1.056 \left(\frac{32}{42} \right)^5 \right] 29,000 \left(\frac{1}{2} \right)^3} \\ &= 0.067 \text{ in.} < 0.4t \quad (\text{Acceptable}) \end{aligned}$$

(c) Fatigue considerations. The skin plate will be checked for fatigue considering cyclic bending stresses along its welded edge. The welds which attach the skin plate to girder flanges and intercostals are typically located on the downstream side of the skin plate. Plate bending stresses due to hydrostatic loading act in compression on the downstream face of the skin plate. Although the stress range due to plate bending at the welds is always in compression, it is likely that residual tensile stresses due to welding will exist. Therefore, the

stress range will vary from an initial positive value and fatigue is a concern. The condition illustrated in example 7 of Appendix K, AISC (1986) is assumed. It is assumed that the water in the lock chamber will be cycled between 100,000 and 500,000 times. For stress category C and loading condition 2, the allowable stress range is $F_r = 21$ ksi. The fatigue stress range will be controlled by the unfactored hydrostatic load H_s . For this case $W = 0.0104$ ksi, and F_{lim} of Equation B-5 is F_r .

$$t_{min} = \sqrt{\frac{0.5(0.0104)(32)^2}{21 \left[1 + 0.623 \left(\frac{32}{42} \right)^6 \right]}} = 0.475 \text{ in.}$$

Therefore, a 1/2-in.-thick plate is adequate.

(2) Intercostal design example. Intercostals may be designed assuming either fixed or pinned ends. However, the designer must ensure that end connections are detailed consistent with the assumption (see Figure B-2). The assumed loading for intercostals consists of a uniform pressure acting on the load area shown in Figure B-3 (nomenclature for this example is also included). This example pertains to the design of miter gate intercostals located on panels 9 through 12 (see Figure B-4) which are spaced at 32 in. on center and span 4 ft. The ends of the intercostals are assumed pinned and the load is applied as an assumed trapezoidal distribution as shown in Figure B-3. Assuming a 6-in.-wide girder flange (conservative assumption), $F = 6$ in., $S = 42$ in., $G = 48$ in., $a = 16$ in., and $b = 10$ in. For this case, the critical load combination is determined by Equation B-1b; $W_u = 0.0151$ ksi. The required factored moment capacity for the example intercostal subject to the trapezoidal load distribution is $M_u = 104.7$ kip-in.

(a) Intercostal design. The effective width of skin plate acting as the intercostal flange shall be determined by treating the skin plate as an unstiffened noncompact element under compression (see paragraph B-2d(2)). The limiting width-to-thickness ratio to satisfy noncompact requirements is

$$\lambda = \frac{b}{2t_f} \leq \lambda_r = 95/\sqrt{F_y} \quad (\text{AISC Table B5.1})$$

The effective width b of a 1/2-in.-thick skin plate is then

$$b = \frac{2t_f(95)}{\sqrt{F_y}} = \frac{2(1/2)(95)}{\sqrt{36}} = 15.83 \text{ in.}$$

The chosen intercostal section shown in Figure B-7 is a tee section composed of a 5-in. by 1/2-in. stem and 15.83-

in. by 1/2-in. effective skin plate flange. Per Table B5.1 of AISC (1986), the stem satisfies noncompact requirements.

$$\frac{d}{t} = \frac{5}{1/2} = 10.0 < \frac{127}{\sqrt{F_y}} = 21.2 \quad (\text{Acceptable})$$

In accordance with Equations F1-15 and F1-16 of AISC (1986), the nominal strength M_n equals M_y ; $\lambda < \lambda_r$ and the compression flange has continuous lateral support ($L_b = 0$). The chosen section has an area $A = 10.4$ in.², a moment of inertia $I_x = 19.7$ in.⁴, a minimum section modulus $S_x = 4.3$ in.³, and a yield moment of $M_y = 154.8$ kip-in. The design strength is

$$\alpha \phi M_n = (0.9)(0.9) 154.8 = 125.4 \text{ kip-in.}$$

which exceeds the required $M_u = 104.7$ kip-in. Therefore, a 5-in. by 1/2-in. stem is acceptable.

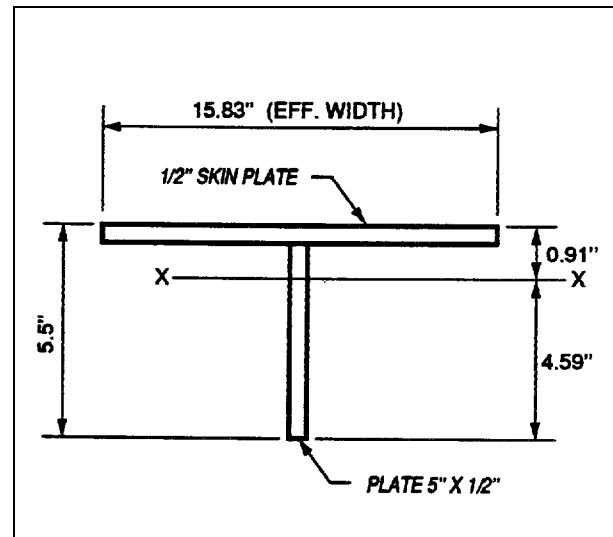


Figure B-7. Sample intercostal section

(b) Fatigue considerations. The fatigue stress range will be controlled by the unfactored load due to the hydrostatic load H_s . For this case $W = 0.0104$ ksi, and $M = 72.1$ kip-in. The extreme fiber of the tee stem is a category A detail. Per Appendix K of AISC (1986), the allowable stress range for a category A detail in load condition 2 is $F_r = 37$ ksi $> F_y = 36$ ksi and fatigue will not control. The intersection of the stem and the skin plate is a category B detail. Per Appendix K of AISC (1986), the allowable stress range is $F_r = 29$ ksi for a category B detail in load condition 2. The stress on the

extreme fiber of the skin plate due to $M = 72.1$ kip-in. is -3.3 ksi. The stress range (considering the presence of tensile residual stress per paragraph 3-6.a) is $f_r = 3.3$ ksi $< F_r = 29$ ksi.

(3) Girder design example. This example applies to the design of the required cross section at center span of the critical horizontal girder (girders 9-11 of Figure B-4) for the miter gate leaf. The required leaf span from the quoin block to miter block is 62 ft (744 in.), and framing details require that the girder depth be maintained at 55 in. Hydrostatic loading and reactions are shown in Figure B-8. The girder is subject to reverse bending; however, at the center span the upstream flange is in compression. The upstream girder flange is laterally braced continuously along its length by the skin plate. The downstream flange of the girder is braced against lateral displacement and twist of the cross section by intermediate diaphragms every 128 in. Transverse web stiffeners are placed at 64-in. intervals.

(a) Width-thickness ratios. For this example, the member is proportioned with the following width-thickness ratios to satisfy compact section requirements in order to avoid local buckling:

$$\text{For girder flanges, } \lambda_p \leq 65/\sqrt{F_y} = 65/6 = 10.83$$

Per paragraph B-2d(3), girder webs shall be proportioned using requirements of uniformly compressed stiffened elements. This ensures compact sections for flexural behavior.

$$\lambda_r \leq 253/\sqrt{F_y} = 253/6 = 42.2$$

(b) Design loading. For this girder, the controlling load combination is given by Equation B-1b. Based on Equation B-1b, the factored uniformly distributed load $W_u = 8.71$ kips/ft or 0.726 kips/in. This loading produces an axial compressive resultant force of $P_u = 847$ kips and a moment at center span of $M_m = 24,757$ kip-in., such that the direction of the moment produces compression in the upstream girder flange. The maximum shear is $V_u = 270$ kips.

(c) Chosen cross section. After several iterations, the sample girder cross section shown in Figure B-9 was selected. This section is composed of 13-in. by 1-in. downstream flange, 52-1/4-in. by 7/16-in. web with 4-1/2-in. by 1/2-in. longitudinal stiffeners located as shown, and a 16-in. by 1-1/4-in. upstream flange. The effective width of the skin plate adjacent to each edge of the upstream

girder flange is based on a $65\sqrt{F_y}$ width-to-thickness ratio as required to satisfy compact section requirements of AISC (1986). Based on this geometry, the girder has the following cross-sectional properties;

$$I_x = 35,097.1 \text{ in.}^4$$

$$r_x = 21.81 \text{ in.}$$

$$r_y = 4.43 \text{ in.}$$

$$S_{x1} = 1,727.69 \text{ in.}^3$$

$$S_{x2} = 1,011.86 \text{ in.}^3$$

$$Z_x = 1,407.27 \text{ in.}^3$$

$$y_c = 20.31 \text{ in.}$$

$$A_g = 73.77 \text{ in.}^2$$

where

$$I_x = \text{moment of inertia about the } x \text{ axis}$$

r_x and r_y = radius of gyration about the x and y axes, respectively

$$S_{x1} = \text{maximum section modulus}$$

$$S_{x2} = \text{minimum section modulus}$$

$$Z_x = \text{plastic modulus}$$

$$y_c = \text{distance from outside face of upstream flange to neutral axis}$$

$$A_g = \text{gross area.}$$

(d) Compact section check. The following calculations show that the section is compact. With two lines of longitudinal stiffeners located as shown, the maximum clear distance of the web is $d = 17.5$ in. The width-thickness ratio for the web is acceptable.

$$\lambda = \frac{d}{t} = \frac{17.5}{7/16} = 40.0 < \lambda_r$$

The upstream flange is compact. For the upstream flange, the thickness including the skin plate is 1.75 in.

$$\lambda = \frac{b}{2t} = \frac{16}{2(1.75)} = 4.57 \text{ in.} < \lambda_p$$

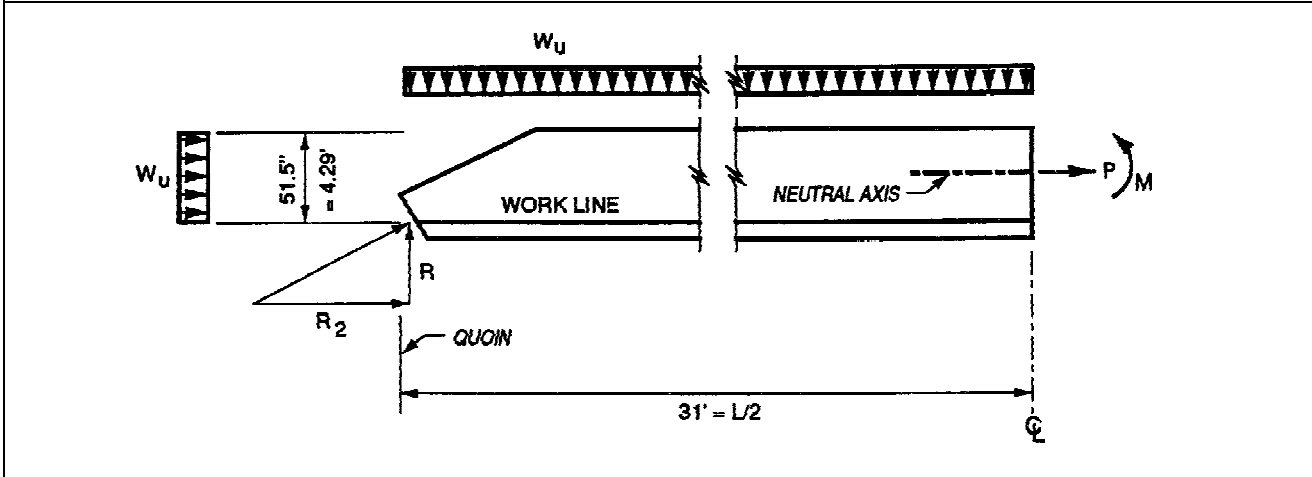


Figure B-8. Girder hydrostatic loading and reactions

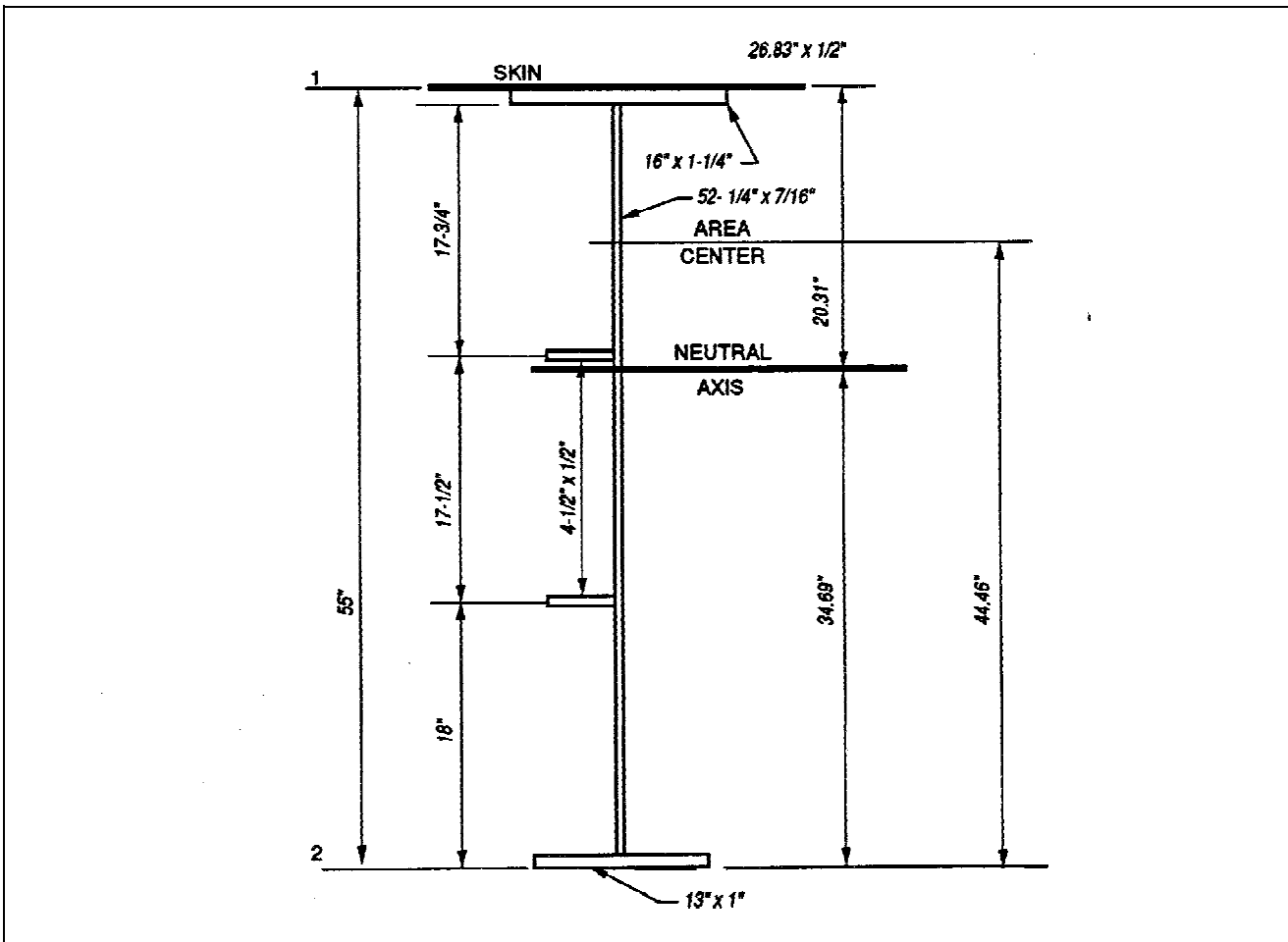


Figure B-9. Sample girder cross section

The downstream flange is compact.

$$\lambda = \frac{b}{2t} = \frac{13}{2(1.0)} = 6.5 \text{ in.} < \lambda_p$$

(e) Web shear. The girder web will be checked for the maximum shear $V_u = 270$ kips. Per Section F2 of AISC (1986)

$$\text{for } \frac{h}{t_w} < 187\sqrt{k/F_{yw}}, \quad V_n = 0.6F_{yw}A_w \quad (\text{AISC F2-1})$$

where

$$k = 5 + \frac{5}{(a/h)^2} \quad (\text{AISC F2-4})$$

unless a/h exceeds 3.0 or $[260/(ht_w)]^2$, in which case $k = 5$. With $a = 64$ in. (transverse stiffener spacing), and $h = 17.5$ in. (web maximum clear depth),

$$\frac{a}{h} = \frac{64}{17.5} = 3.7 > 3.0; \quad k = 5$$

$$\frac{h}{t_w} = \frac{17.5}{(7/16)} = 40.0 < 187\sqrt{5/36} = 69.7$$

$$V_n = 0.6(36)(24.06) = 519.7 \text{ kips}$$

$$\begin{aligned} \alpha \phi V_n &= 0.9(0.9)(519.7) \quad (\text{Acceptable}) \\ &= 420.9 > 270 \end{aligned}$$

(f) Combined forces. The horizontal girder is considered a singly symmetric prismatic member subjected to axial force and flexure about its major axis. This category of design is discussed in Chapter H of AISC (1986) and the section is checked by the following calculations. Column action is based on requirements of Chapter E of AISC (1986). Per paragraph B-2d(3), $K_x = 1.0$, $C_m = 1.0$ and $l_x = 744$ in. (strong axis; distance between quoin and miter blocks). Per EM 1110-2-2703 $K_y = 0.65$ and $l_y = 128$ in. (weak axis; distance between intermediate diaphragms).

$$\frac{Kl_x}{r_x} = \frac{1.0(744)}{21.81} = 34.11 \quad (\text{controls})$$

$$\frac{Kl_y}{r_y} = \frac{0.65(128)}{4.43} = 18.8$$

$$P_n = A_g F_{cr} \quad (\text{AISC E2-1})$$

$$\begin{aligned} \lambda_{cx} &= \frac{Kl}{r\pi} \sqrt{\frac{F_y}{E}} = \frac{34.11}{\pi} \sqrt{\frac{36}{29,000}} \\ &= 0.383 \end{aligned} \quad (\text{AISC E2-4})$$

$$\begin{aligned} F_{cr} &= (0.658^{\lambda_c^2}) F_y = (0.658^{0.147}) 36 \\ &= 33.85 \text{ ksi} \end{aligned} \quad (\text{AISC E2-2})$$

$$P_n = (73.77)33.85 = 2,497 \text{ kips}$$

Given $P_u = 847$ kips, $\phi_c = 0.85$, and $\alpha = 0.9$

$$\begin{aligned} \frac{P_u}{\alpha \phi_c P_n} &= \frac{847}{0.9(0.85)(2,497)} = 0.44 > 0.2; \\ &\text{use Eq. AISC H1-1a} \end{aligned}$$

$$\frac{P_u}{\phi_c P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \quad (\text{AISC H1-1a})$$

$$M_{uy} = 0$$

$$M_{ux} = B_1 M_{nt} + B_2 M_{1t} \quad (\text{AISC H1-2})$$

$$B_1 = \frac{C_m}{(1 - P_u/P_e)} \geq 1.0 \quad (\text{AISC H1-3})$$

$$P_e = \frac{A_g F_y}{\lambda_{cx}^2} = \frac{73.77(36)}{(0.383)^2} = 18,104$$

$$B_1 = \frac{1.0}{1.0 - 847/18,104} = 1.05$$

$$M_{1t} = 0$$

$$\begin{aligned} M_{ux} &= B_1 M_{nt} = (1.05)24,757 \\ &= 25,995 \text{ kip-in.} \end{aligned}$$

For compact sections, with the beam compression flange laterally supported continuously, $M_n = M_p$.

$$M_n = M_p = F_y Z = 36(1,407.27) = 50,662 \text{ kip-in.}$$

Substitution into AISC H1-1a:

$$0.44 + \frac{8}{9} \left(\frac{25,995}{0.9(0.9)(50,662)} \right) = 1.0 \leq 1.0 \text{ (Acceptable)}$$

At the midspan location, the chosen section is adequate for combined forces. The cross section consists of the following elements:

Upstream flange	16 in. by 1-1/4 in.
Downstream flange	13 in. by 1 in.
Skin plate	1/2 in.
Web	52-1/4 in. by 7/16 in.

(2 longitudinal stiffeners 4-1/2 in. by 1/2 in.)

(g) Fatigue considerations. At the location of a transverse stiffener or intermediate diaphragm, the girder is a category C detail. Per Appendix K of AISC (1986), the allowable stress range for a category C detail under load condition 2 is $F_r = 21$ ksi. The compression flange is subject to larger stress variations under hydrostatic loading and will be checked for fatigue due to the probable tensile residual stress that exists as a result of welding. For the unfactored load due to hydrostatic load H_s , $W = 6$ kips/ft, $P = 584$ kips, and $M = 17,054$ kip-in. at the midspan of the girder.

$$f_a = \frac{P}{A_g} = \frac{-584}{73.77} = -7.9 \text{ ksi}$$

$$f_b = \frac{M}{S_1} = \frac{-17,054}{1,727.69} = -9.9 \text{ ksi}$$

The stress on the extreme fiber of the upstream flange is

$$f = f_a + f_b = -17.8 \text{ ksi}$$

The stress range (considering tensile weld residual stress) is $f_r = 17.8$ ksi < $F_r = 21$ ksi; acceptable. For locations at the termination of a welded cover plate, a category E detail should be assumed.

(h) Design for barge impact. For girder number 3, the controlling load combination is Equation B-1a. The

previously chosen section will be checked for unsymmetric and symmetric barge impact. Due to hydrostatic loading H_s , the uniformly distributed load W is 0.28 kips/ft.

For unsymmetric impact, the axial force P and flexural moment M (at the location of impact) are

$$P = \frac{(4x+a)I}{\sqrt{10}a}$$

$$M = \frac{I(ax-x^2)}{a} - Pe$$

and for symmetric impact (P and M are constant along the girder length)

$$P = \frac{5I}{\sqrt{10}}$$

$$M = -Pe$$

where x , a , and e are defined in Figure B-1.

For unsymmetric impact, $I = 250$ kips. With a girder span of 62 ft, $a = 58.8$ ft (705.6 in.) and assuming a barge width of 35 ft, $x = 38.8$ ft (465.6 in.). The eccentricity between the girder work line and the neutral axis is $e = 31.2$ in. The impact girder resultant forces at the point of impact are

$$P = \frac{(4(465.6) + 705.6)250}{705.6\sqrt{10}} = 288 \text{ kips}$$

$$M = \frac{250(705.6(465.6) - (465.6)^2)}{705.6} - 288(31.2) = 30,606 \text{ kip-in.}$$

For the distributed loading $W = 0.28$ kips/ft, $P = 27.3$ kips and at the location of impact, $M = 632.2$ kip-in. By Equation B-1a:

$$P_u = 1.4(27.3) + 288 = 326 \text{ kips}$$

$$M_u = 1.4(632.2) + 30,606 = 31,491 \text{ kip-in.}$$

With $P_n = 2,497$ kips,

$$\frac{P_u}{\alpha \phi_c P_n} = \frac{326}{0.9(0.85)(2,497)} = 0.17 < 0.2$$

Therefore, in accordance with Section H1 of AISC (1986), Equation H1-1b applies.

$$\frac{P_u}{2\phi_c P_n} + \left(\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} \right) \leq 1.0 \quad (\text{AISC H1-1b})$$

Substitution of the appropriate values into Equation AISC H1-1b shows that the section is acceptable for this case (unsymmetric impact).

$$\frac{0.17}{2} + \left(\frac{31,491}{0.9(0.9)(50,662)} \right) = 0.85 \leq 1.0 \quad (\text{Acceptable})$$

For symmetric impact, $I = 400$ kips.

$$P = \frac{5(400)}{\sqrt{10}} = 632 \text{ kips}$$

$$M = -632(31.2) = -19,718 \text{ kip-in.}$$

For the distributed loading $W = 0.28$ kips/ft, $P = 27.3$ kips and at center span of the girder, $M = 798$ kip-in. By Equation B-1a:

$$P_u = 1.4(27.3) + 632 = 670 \text{ kips}$$

$$M_u = 1.4(798) - 19,718 = -18,601 \text{ kip-in.}$$

With $P_n = 2,497$ kips,

$$\frac{P_u}{\alpha\phi_c P_n} = \frac{670}{0.9(0.85)(2,497)} = 0.35 > 0.2$$

Therefore, in accordance with Section H1 of AISC (1986) Equation H1-1a applies. Substitution of the appropriate values into Equation AISC H1-1a shows that the section is acceptable for this case (symmetric impact).

$$0.35 + \left(\frac{18,601}{0.9(0.9)(50,662)} \right) = 0.8 \leq 1.0 \quad (\text{Acceptable})$$

(i) Commentary. For this example, a compact section was chosen. Noncompact sections are allowed and may be more economical in some cases. Per AISC (1986), steel sections are classified as either compact, noncompact, or slender element sections. Compact sections are capable of developing a fully plastic stress distribution prior to element local buckling. Noncompact sections are proportioned such that compression elements can develop yield stress prior to local buckling. In slender element sections, local buckling will occur prior to initial yielding. Appropriate appendixes of AISC (1986) include requirements for the design of members controlled by local buckling. The above example considered only the

required section at midspan, and the section should be checked for the appropriate design loading at the girder ends. Longitudinal web stiffeners are placed on only one side of the web. Compared to the case of placing stiffeners on both sides of the web, this requires slightly larger stiffener plates. However, placing stiffeners on only one side of the web is more attractive due to the cost savings in fabrication and detailing. Furthermore, the adverse effects due to welding of additional stiffeners, such as residual stress, reduced toughness in the heat-affected zone, and through-thickness tension of the web, are avoided.

(4) Diagonal design example. This example pertains to the design of miter gate diagonal members utilizing ASTM A36 steel. General guidance for diagonal design is contained in EM 1110-2-2703. Diagonal design will be controlled by Equation B-2a or B-2b. Equation B-2a represents the case where the gate is subject to temporal hydraulic loading. Equation B-2b represents the case where a submerged obstruction constrains gate leaf motion while the maximum operating force Q is applied. For this particular example, Q is limited by a pressure relief valve engaged during gate motion and is equal to 125 kips. Plan and elevation views for the gate leaf, illustrating the torsional loads, are shown in Figure B-10. The length of each diagonal is $L = 831.6$ in. The unfactored loads, the distance from the pintle to the applied load z , the moment arm of the applied load with respect to the center of moments (located at the operating strut elevation), and corresponding load torque areas Tz for this case are estimated as shown in Table B-3. For loads Q , H_t , and H_d , a positive value for Tz is for the case of gate opening and a negative value is for the case of gate closing. To avoid confusion of nomenclature, the diagonal elasticity constant (denoted as Q by EM 1110-2-2703) is represented as Q' in the following calculations.

Table B-3
Gate Torsion Load

Load	Force (kips)	Moment Arm (ft)	z (ft)	Tz (kip-ft ²)
D	286.1	3.53	31.0	-31,308
$C + M$	130.0	3.53	31.0	-14,226
Q	125.0	55.00	19.0	±130,625
H_t	93.1	45.38	31.0	±130,971
H_d	33.5	46.00	31.0	±47,771

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The factored loads for Equations B-2a and b are as follows:

$$T_z(D)_u = 1.2(-31,308) = -37,570 \text{ kip-ft}^2$$

$$T_z(C+M)_u = 1.6(-14,226) = -22,762 \text{ kip-ft}^2$$

$$T_z(Q)_u = 1.2(+130,625.0) = +156,750 \text{ kip-ft}^2$$

$$T_z(H)_u = 1.0(+130,971) = +130,971 \text{ kip-ft}^2$$

Since $T_z(Q)_u$ is greater than $T_z(H)_u$, Equation B-2b will control.

(a) Design. The design strength for tension members $\alpha\phi_t P_n$ is the lower of the following:

Case a. For yielding in the gross section, $\alpha = 0.9$
and $\phi_t = 0.9$

$$P_n = F_y A_g \quad (\text{AISC D1-1})$$

$$\alpha\phi_t F_y = 0.9(0.9)(36) = 29.16 \text{ ksi}$$

Case b. For fracture in the net section, $\alpha = 0.9$
and $\phi_t = 0.75$

$$P_n = F_u A_e = F_u (U A_g) \quad (\text{AISC D1-2})$$

The end connections are welded to gusset plates with a total weld length greater than two times the bar width. Therefore, $U = 1.0$ and the effective area A_e is the same as the gross area A_g (Section B3 of AISC (1986)).

$$\alpha\phi_t F_u = 0.9(0.75)(58) = 39.15 \text{ ksi}$$

Case a controls and the limiting tensile stress is 29.16 ksi. Per equations of EM 1110-2-2703, the following is obtained:

$$A' = 30 \text{ in.}^2, R_o = \pm 0.11$$

$$A_p = 22 \text{ in.}^2 \text{ (chosen area of positive diagonal)}$$

$$A_n = 19 \text{ in.}^2 \text{ (chosen area of negative diagonal)}$$

$$R_p = \left(\frac{30}{22 + 30} \right) 0.11 = 0.0635$$

$$R_n = \left(\frac{-30}{19 + 30} \right) 0.11 = -0.0673$$

$$Q'_p = 229,629 \text{ kip-ft and } Q'_n = 210,418 \text{ kip-ft}$$

$$Q'_o = 0 \text{ (Conservative assumption)}$$

$$\Sigma Q' = 440,047 \text{ kip-ft}$$

Live load gate opening deflection (critical case is when $C + M = 0$):

$$\delta_o = \frac{T_z(Q)_u}{\Sigma Q'} = 4.27 \text{ in.}$$

Live load gate closing deflection:

$$\delta_c = \frac{T_z(C+M)_u - T_z(Q)_u}{\Sigma Q'} = -4.9 \text{ in.}$$

Let $D_p = 7.0 \text{ in.}$ and $D_n = -5.5 \text{ in.}$

$$Q'_p D_p + Q'_n D_n = 37,509 \text{ kip-ft}^2 \approx T_z(D)_u \text{ (Acceptable)}$$

The stress in the diagonals must remain between the tensile limiting stress of 29.16 ksi and the minimum stress of 1.0 ksi (diagonals must always remain in tension). The maximum tensile stresses will occur as follows:

For the positive diagonal on gate closing:

$$f_p = \frac{R_p E (D_p - \delta_c)}{L}$$

$$= 26.4 \text{ ksi} < 29.16 \text{ ksi} \quad (\text{Acceptable})$$

For the negative diagonal on gate opening:

$$f_n = \frac{R_n E (D_n - \delta_o)}{L}$$

$$= 23.0 \text{ ksi} < 29.16 \text{ ksi} \quad (\text{Acceptable})$$

The minimum tensile stresses will occur as follows:

For the positive diagonal on gate opening:

$$f_p = \frac{R_p E(D_p - \delta_o)}{L}$$

$$= 6.0 \text{ ksi} > 1.0 \text{ ksi} \quad \text{Acceptable}$$

For the negative diagonal on gate closing:

$$f_n = \frac{R_n E(D_n - \delta_c)}{L}$$

$$= 1.4 \text{ ksi} > 1.0 \text{ ksi} \quad \text{Acceptable}$$

(b) Deflection serviceability check. Per paragraph B-2e, the maximum deflection during operation shall not exceed 4 in. (1/2 contact block width). The controlling load combination is Equation B-2b with unfactored loads. The maximum deflection will occur as Q acts with C and M (gate closing).

$$\delta = \frac{Tz(Q) + Tz(C + M)}{\Sigma Q'}$$

$$= 3.95 \text{ in.} < 4.0 \text{ in.} \quad \text{(Acceptable)}$$

(c) Fatigue considerations. The welded connection at the end of each diagonal is considered a category E detail. From appendix K of AISC (1986), the allowable stress range for load condition 2 is $F_r = 13$ ksi. For each operation of the miter gate, the stress range is calculated considering the absolute difference in opening and closing deflection. This deflection is based on the assumed hydrodynamic load H_d of 30 psf acting on the submerged portion of the leaf during gate operation.

$$\delta = \frac{+Tz(H_d)}{\Sigma Q'}$$

$$\delta_o = \frac{+47,771(12)}{440,047} = 1.3 \text{ in.}$$

$$\delta_c = \frac{-47,771(12)}{440,047} = -1.3 \text{ in.}$$

The maximum stress is in the negative diagonal ($|R_n| > R_p$). Therefore the stress range is

$$f_r = \frac{R_n E(\delta_c - \delta_o)}{L} = \frac{-0.0675(29,000)(-2.6)}{831.6}$$

$$= 6.1 \text{ ksi}$$

$$6.1 \text{ ksi} < 13 \text{ ksi} \text{ (Acceptable)}$$

Based on the above calculations, $A_p = 22$ in. and $A_n = 19$ in. are adequate and the following sizes are chosen:

Positive diagonal: Select two 7-1/2-in. by 1-1/2-in. members; Area = 22.5 in².

Negative diagonal: Select two 6-1/2-in. by 1-1/2-in. members; Area = 19.5 in².

(d) Fracture control considerations. The diagonals are fracture critical members; therefore it is necessary to ensure that the material has adequate toughness as specified by paragraph 3-6b. Assuming a minimum service temperature of -10° F (Zone 2) the material specifications should require a CVN toughness of 25 ft-lb tested at 40° F for welded 36-ksi steel 1.5 in. thick.

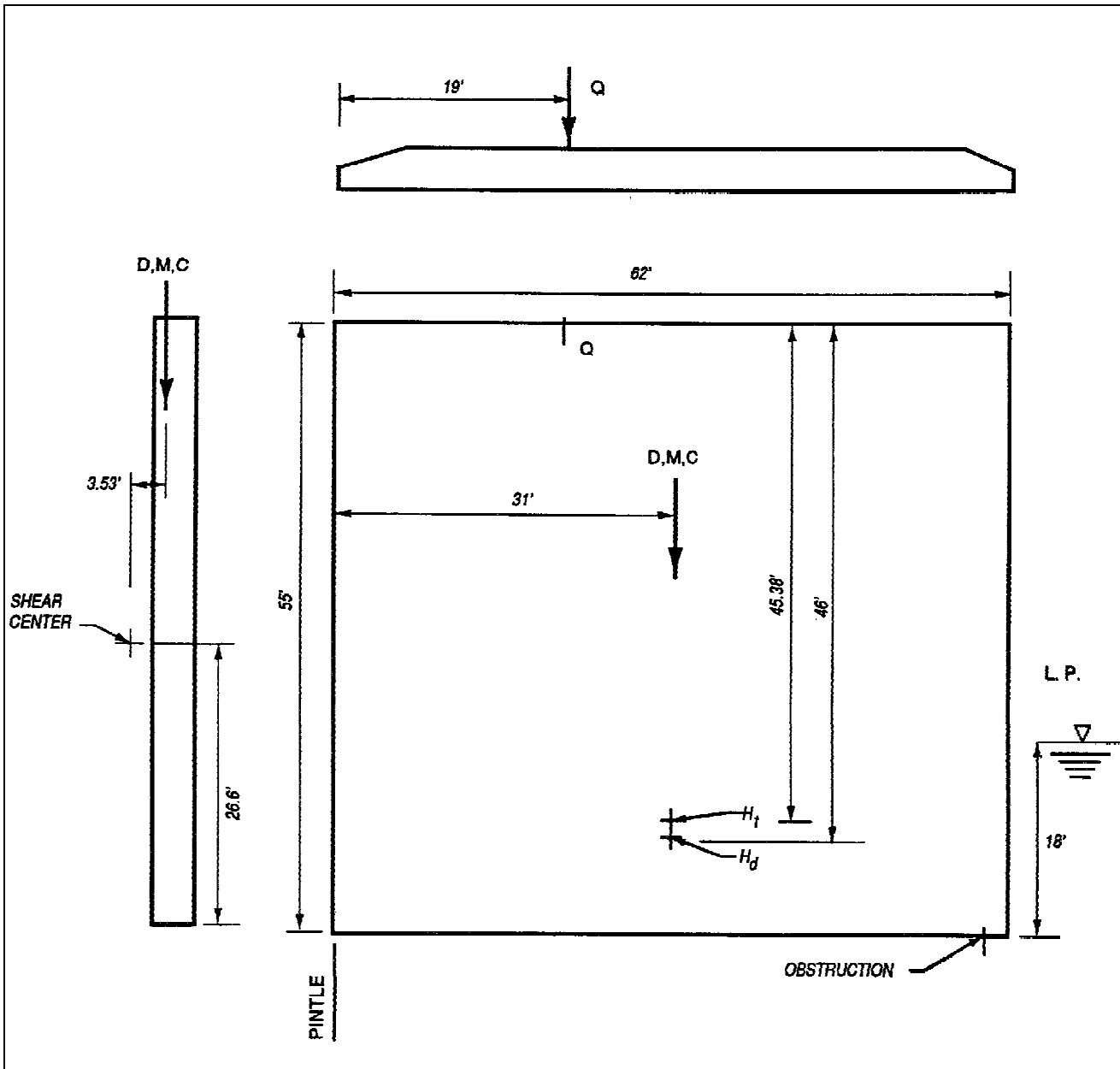


Figure B-10. Example miter leaf torsion loads

Appendix C

Tainter Gates

C-1. Introduction

Allowable stress design (ASD) criteria shall be used until load and resistance factor design (LRFD) criteria have been developed.

C-2. Reference

EM 1110-2-2702

Design of Spillway Tainter Gates

Appendix D

Tainter Valves

load and resistance factor design (LRFD) criteria have been developed.

D-1. Introduction

Allowable stress design (ASD) criteria shall be used until

Appendix E

Bulkheads and Stoplogs

load and resistance factor design (LRFD) criteria have been developed.

E-1. Introduction

Allowable stress design (ASD) criteria shall be used until

Appendix F
Vertical Lift Gates (Lock and Crest)

CE-1507.01
Tractor Gates-Broome Type

F-1. Introduction

CE-1602
Dam Gantry Cranes

Allowable stress design (ASD) criteria shall be used until load and resistance factor design (LRFD) criteria have been developed.

F-2. References

EM 1110-2-2701
Vertical Lift Crest Gates

Appendix G Hydroelectric and Pumping Plants

G-1. Introduction

Allowable stress design (ASD) criteria shall be used until load and resistance factor design (LRFD) criteria have been developed.

G-2. References

EM 1110-2-3001

Planning and Design of Hydroelectric Power Plants

EM 1110-2-3104

Structural Design of Pumping Stations

* Appendix H Load and Resistance Factor Design Criteria for Local Flood Protection Project Closure Gates

H-1. Introduction

a. Purpose. This appendix provides structural design guidance for gate closure openings in levees and floodwalls for inland local flood protection projects (LFPP) using the load and resistance factor design method. Load carrying members (including but not limited to: skin plates, intercostals, girders, diagonals, and vertical diaphragms) shall be designed in accordance with the criteria contained in this appendix, as well as criteria in Chapters 1, 2, 3, and 5 where applicable. Gate layout, selection of materials, and assumed member loading shall follow guidance in EM 1110-2-2705 unless otherwise stated herein. Mechanical and electrical items shall be designed in accordance with Chapter 4 and guidance specified in EM 1110-2-2705.

b. Background.

(1) Types of LFPP closure structures. Closure structures for openings in levee and floodwall systems of LFPPs are usually either stoplog or gate type closures. The guidance presented in this appendix is limited to swing, miter, rolling, and trolley gates since these are the most commonly used LFPP gate types. Many gate details are shown in EM 1110-2-2705.

(2) Load combinations and load factors. ASCE (1990) and AISC (1986) specify load factors and load combinations for buildings; however, for LFPP closures, unique loads and load combinations exist. The load factors and load combinations specified in paragraph H-2a pertain specifically to LFPP closure gates. Development of the load factors considered variability, definition, and likeness to those loads specified in ASCE (1990) and AISC (1986).

(3) A discussion of the need for using reliability factor α for HSS design is given in paragraph 3-8. LFPP gates are considered hydraulic steel structures; however, the environment they function in is not as severe as other

HSS and is more like building environments than the typical HSS environment. Therefore, the reliability factor for LFPP gates shall be 1.0 unless the structural engineer is aware of extenuating circumstances that require a more conservative design.

H-2. Load and Resistance Factor Design

a. Strength requirements. LFPP gates shall have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in the following load combinations. The most unfavorable effect may occur when one or more of the loads in a particular load combination are equal to zero.

$$1.4 H_s + 1.2 D \quad (\text{H-1})$$

$$1.2 D + 1.3 (Q \text{ or } W) \quad (\text{H-2})$$

The nominal loads are defined as follows:

H_s = hydrostatic load

D = dead load

Q = maximum operating load (draw bar pull or operating equipment capacity)

W = wind load

b. Load considerations. Loads due to thermal effects need not be considered. Earthquake loads need not be considered for the gate members since the probability of an earthquake occurring when the gate is closed is very remote. However, the seismic design for the concrete monolith supporting the open gate must include the inertial effects. This appendix addresses only inland structures; therefore, other special loading conditions may be necessary for gate closures in hurricane flood protection projects. For example, gates could be subjected to large wave loads during closing operations before the main hurricane storm strikes.

(1) Hydraulic loads. Hydrostatic load H_s shall be determined based on water to the top of the gate on the unprotected side and at or below the gate sill elevation on the protected side.

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* (2) Gravity loads. Dead load¹ D shall be determined based on site-specific conditions. Ice and mud loads are generally not considered in LFPP gate load combinations.

(3) Wind loads. Wind load W shall consist of a wind load of 15 psf when the gate is opening or closing (gate operating). When the gate is in the latched closed or open positions (gate stationary), a wind load as specified in EM 1110-2-2502 shall be used.

(4) Operating loads. Load Q shall be the maximum load that can be exerted by the operator (stalling torque of motorized winch, capacity of manually operated winch, etc. This value should be obtained from the mechanical engineer that designed the machinery or other equipment).

c. Load cases. The following load cases shall be considered with the appropriate loading combinations:

(1) Case 1: Gate stationary.

(a) Closed under hydrostatic loads. (Eq. H-1 with $D = 0$ would apply.)

(b) Open or closed, no hydrostatic load. (Eq. H-2, $Q = 0$ would apply.)

(2) Case 2: Gate operating.

(a) Hinge gates. This loading condition results in the maximum torsion load. Hinge gates shall be designed for a uniform wind load of 15 psf (refer to paragraph 9b(4) of EM 1110-2-2705) and an equal and opposite reaction load at the operator (usually a draw bar) connection. (Eq. H-2 with $Q = 0$ would apply.)

(b) Wheel gates. The maximum load for determining the localized stresses for designing the operator connection (usually a winch plate) shall be the limiting load for the operator. (Eq. H-2 with $W = 0$ would apply.)

d. Design for individual members. The following paragraphs include a brief description of design assumptions, appropriate LRFD formulas, and load cases for the

design of individual gate members. Further design guidance is presented in EM 1110-2-2705.

(1) Skin plate.

(a) Skin plates shall be sized such that the maximum calculated stress is less than the yield limit state of $\alpha\phi F_y$ where α is 1 as per paragraph H-1b(3), and ϕ is defined in AISC (1986). Skin plates shall be designed for hydrostatic loading only. Stresses shall be determined on the basis of small deflection thin plate theory using load case 1(a) of paragraph H-2c. 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 1/4 in.

(b) With requirements of paragraph H-2d(1)(a) above, design per paragraph B-2d(1)(b).

(2) Intercostals.

(a) Intercostals shall be sized so the maximum calculated moment is less than the nominal bending strength of $\alpha\phi_b M_n$ where α is 1 as per paragraph H-1b(3) and ϕ_b is defined in AISC (1986). Intercostals shall be designed for hydrostatic loading only. They may be flat bars or plates, tee sections, or angle sections. Load case 1(a) or paragraph H-2c shall be investigated to determine the maximum load effect.

(b) With requirements of paragraph H-2d(2)(a) above, design per the applicable requirements of paragraph B-2d(2).

(3) Girders. Horizontal girders for LFPP miter gates shall be designed in accordance with the principles discussed in paragraph B-2d(3).² 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. For all types of LFPP gates, load case 1(a) of paragraph H-2c

¹ Miter gate design includes ice and mud in the dead load. LFPP gates usually do not consider these loads because of their environment. If the designer has an unusual situation and wants to consider these loads, he should refer to paragraph B-2b(2).

² Paragraph B-2d(3) is written with emphasis on built-up sections. LFPP gates generally use rolled sections.

*

* shall be investigated to determine the maximum load effect. The maximum design wind loading acting against the exposed gate surface is insignificant when compared with the hydrostatic loading.

(4) Diagonals. Diagonals shall be designed in accordance with the principles discussed in paragraph B-2d(4). They shall be designed to resist gate torsion to the dead load as well as the torsion resulting from closing or opening the gate against a 15-psf uniform wind loading. Load case 2(a) of paragraph H-2c is applicable.

(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 those diaphragms in line with wheels or trolley hangers. They shall include axial and bending due to the forces from the wheels or trolley hangers. Load case 1(a) is applicable. The minimum thickness of any diaphragm element shall be 1/4 in.

(6) Stabilizing systems. The hinges, wheels, trolleys, latching devices, closing links, gate tie-down assemblies, gate hooks, or other stabilizing systems for the various types of LFPP gates are shown in the plates in EM 1110-2-2705. 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 load case 1(a). For example, the gates hooks for the L-frame rolling gate would use this load case. Other gate components that are designed to resist dead, operating, or wind load (depending on the component's function) shall be designed using load case 2.

e. Serviceability requirements. Serviceability requirements shall be as specified in paragraph 3-5. LFPP gates shall be designed for an expected life of 50 years. 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. Fatigue. Fatigue is not a concern for LFPP closure structures due to the small number of stress cycles over the life of the structure.

g. Fracture. Requirements of paragraph 3-6 shall be applied to fracture critical members (FCM). The designer shall determine which members are fracture critical for the specific gate in question in accordance with paragraph 3-6b. Typically, hinges, wheels, trolley gate hangers, and their associated connections are considered to be FCM.

H-3. Connections and Details

Chapter 5 provides general guidance for connection design. Connection details shall be consistent with the design assumptions. For example, Figure B-2 illustrates the details required for consistency in intercostal design for simple or fixed connections. Paragraphs 1-5a(6) and 1-5a(7) of EM 1110-2-2703 discuss the use of bolts, welds, and fabrication of miter gate leaves, and paragraph 2-1j(3) includes a discussion of diagonal connections. Although EM 1110-2-2703 is written for lock gates, the guidance is generally applicable for LFPP gates.

H-4. Design Example

Specific LFPP gate design examples are not included in this appendix. Appendices B and C demonstrate LRFD principles for the design of miter and tainter gate elements, respectively. The calculations are provided to demonstrate LRFD principles; they do not provide comprehensive design for entire gates. Examples in the two appendices are limited to the design of skin plates, intercostals, and girders. Also, design of the diagonals for a horizontally framed miter gate is shown in Appendix B. These examples should be used for guidance when designing similar structural members for LFPP gates.

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Appendix I

Miscellaneous Hydraulic Steel Structures

I-1. Introduction

Miscellaneous HSS include lock wall accessories, outlet works gates, penstocks, and sector gates. Allowable stress design (ASD) criteria shall be used until load and resistance factor design (LRFD) criteria have been developed.

I-2. References

EM 1110-2-2400

Structural Design of Spilways and Outlet Works

EM 1110-2-2901

Tunnels and Shaft in Rock

EM 1110-2-2902

Conduits, Culverts, and Pipes