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_t, 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.

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

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

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

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

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

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

$$p = \frac{7}{8} \gamma_w a_c \sqrt{Hy} \tag{B-4}$$

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

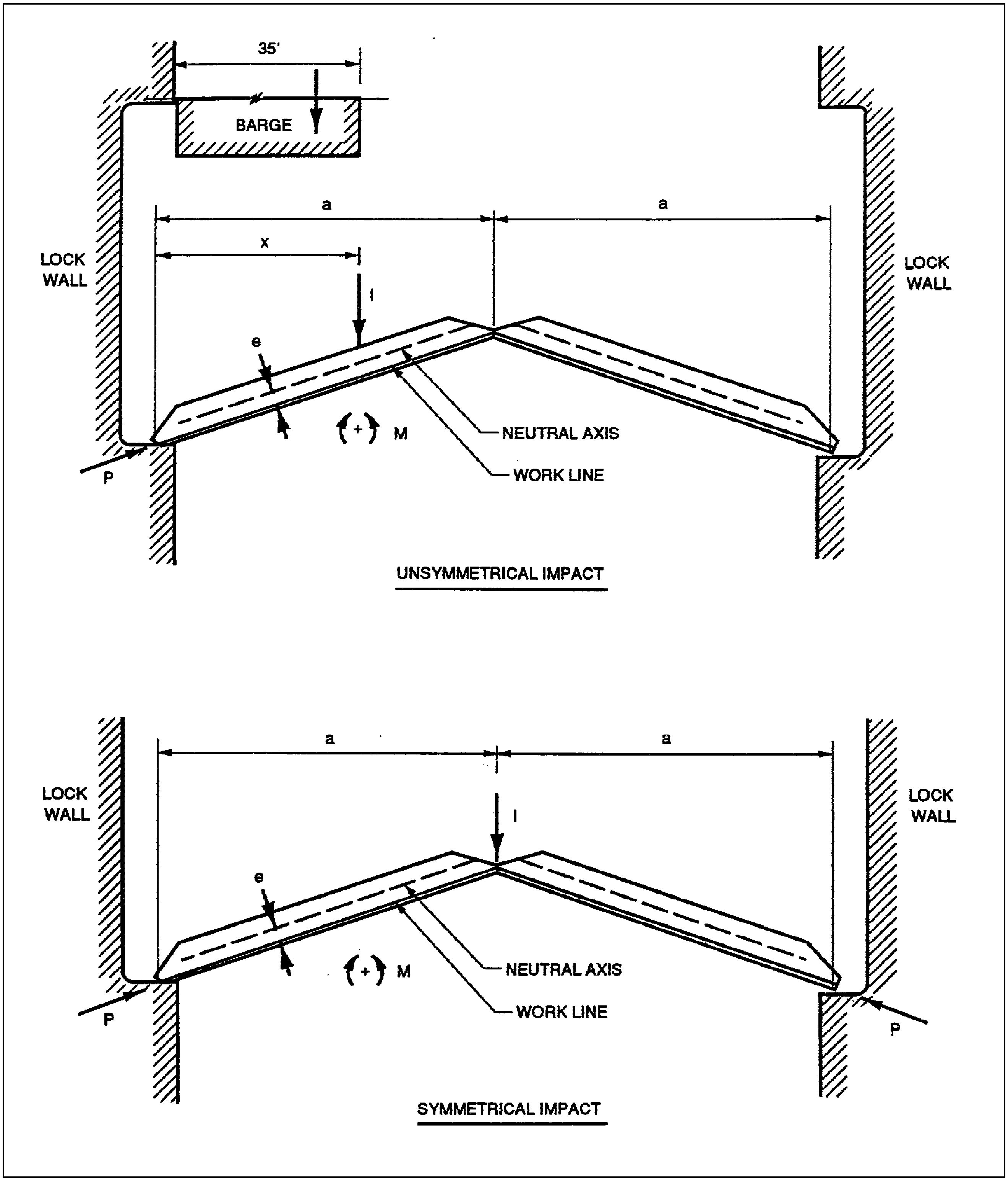

Figure B-1. Point load impact for miter gate girders

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

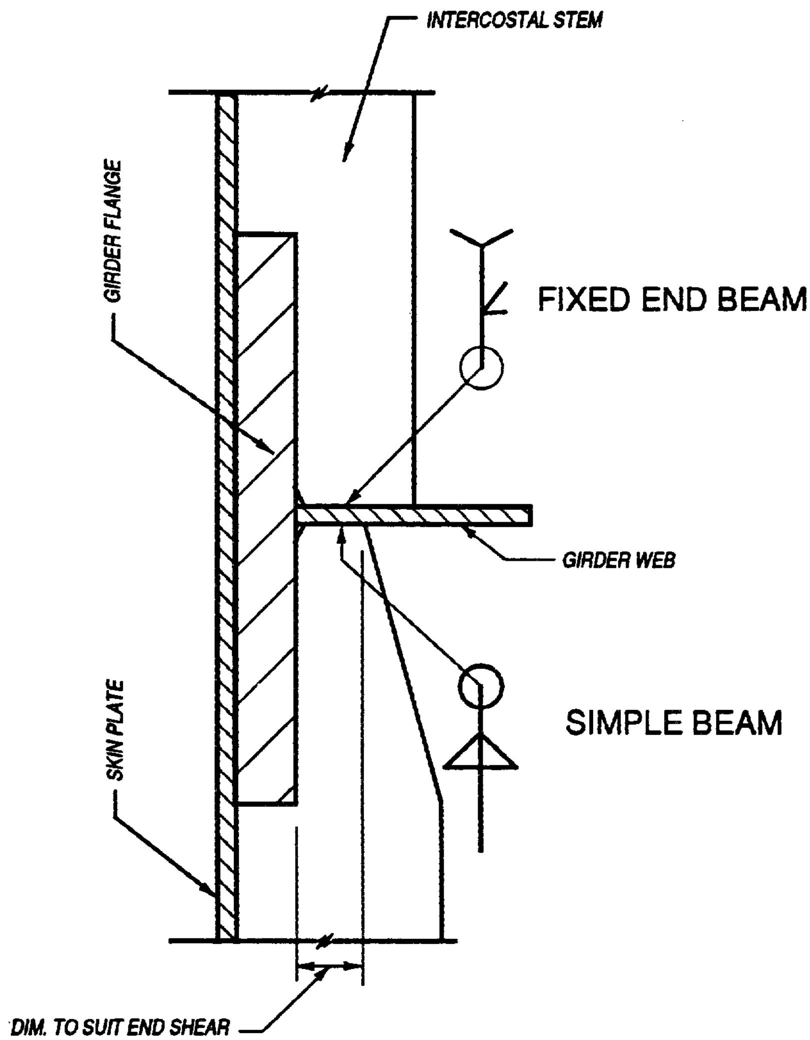

Figure B-2. Assumptions for intercostal end connections

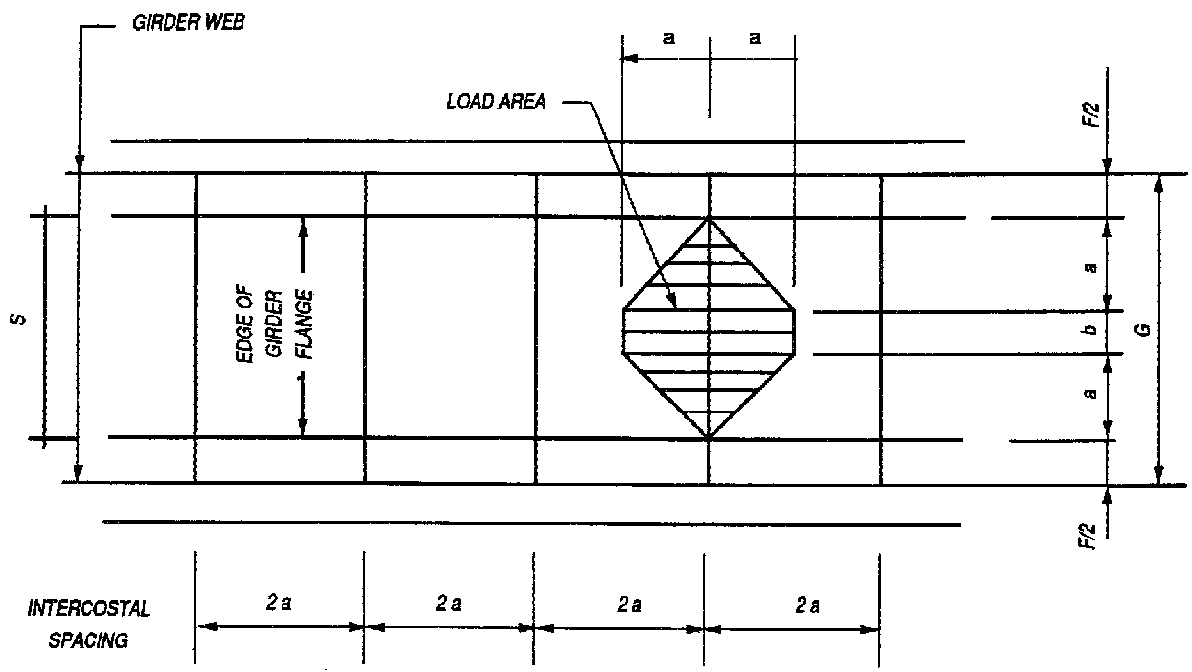

Figure B-3. Nomenclature and assumed load area for intercostal design

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 leafs, 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

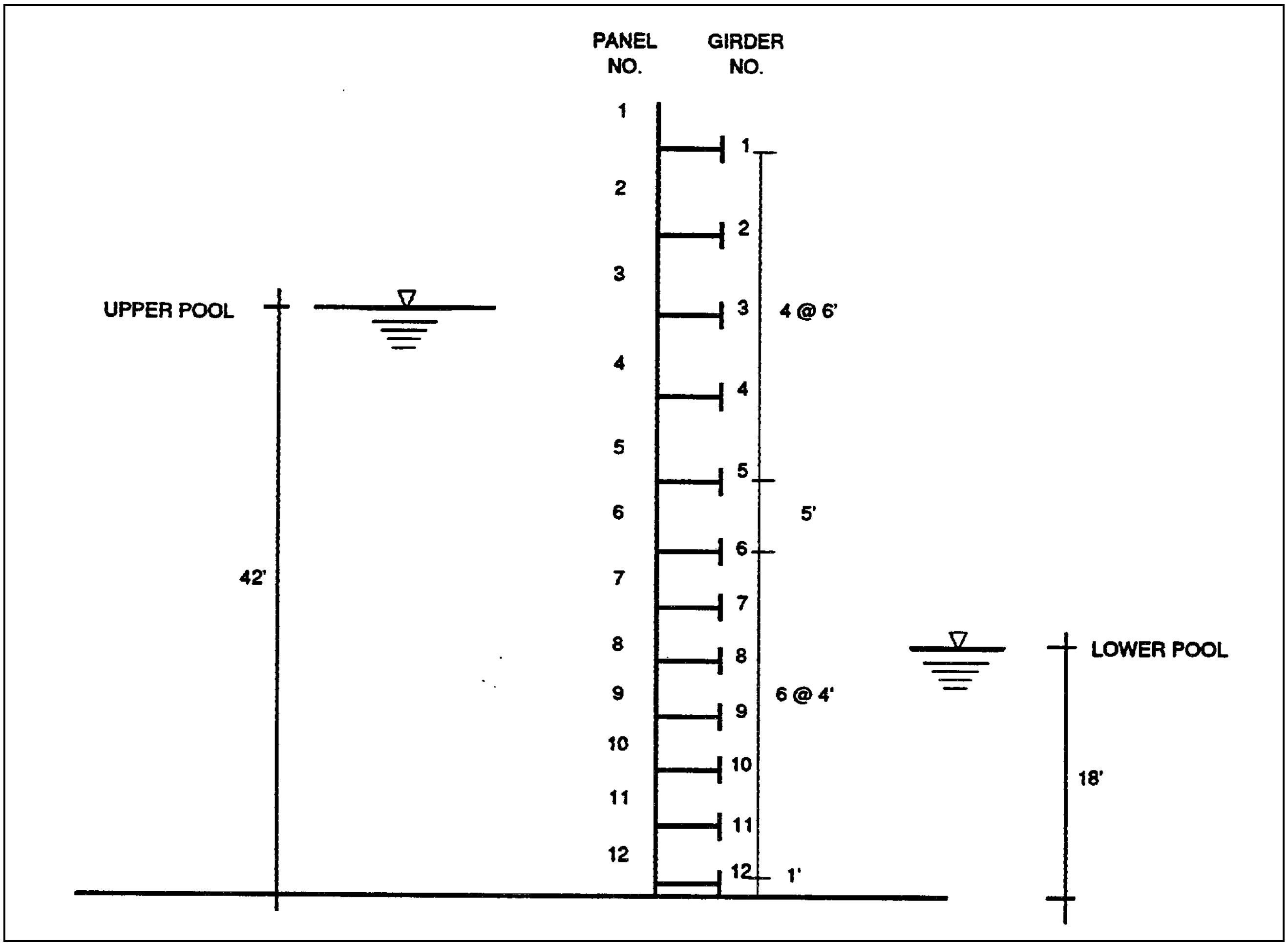

Figure B-4. Vertical cross section for example miter gate

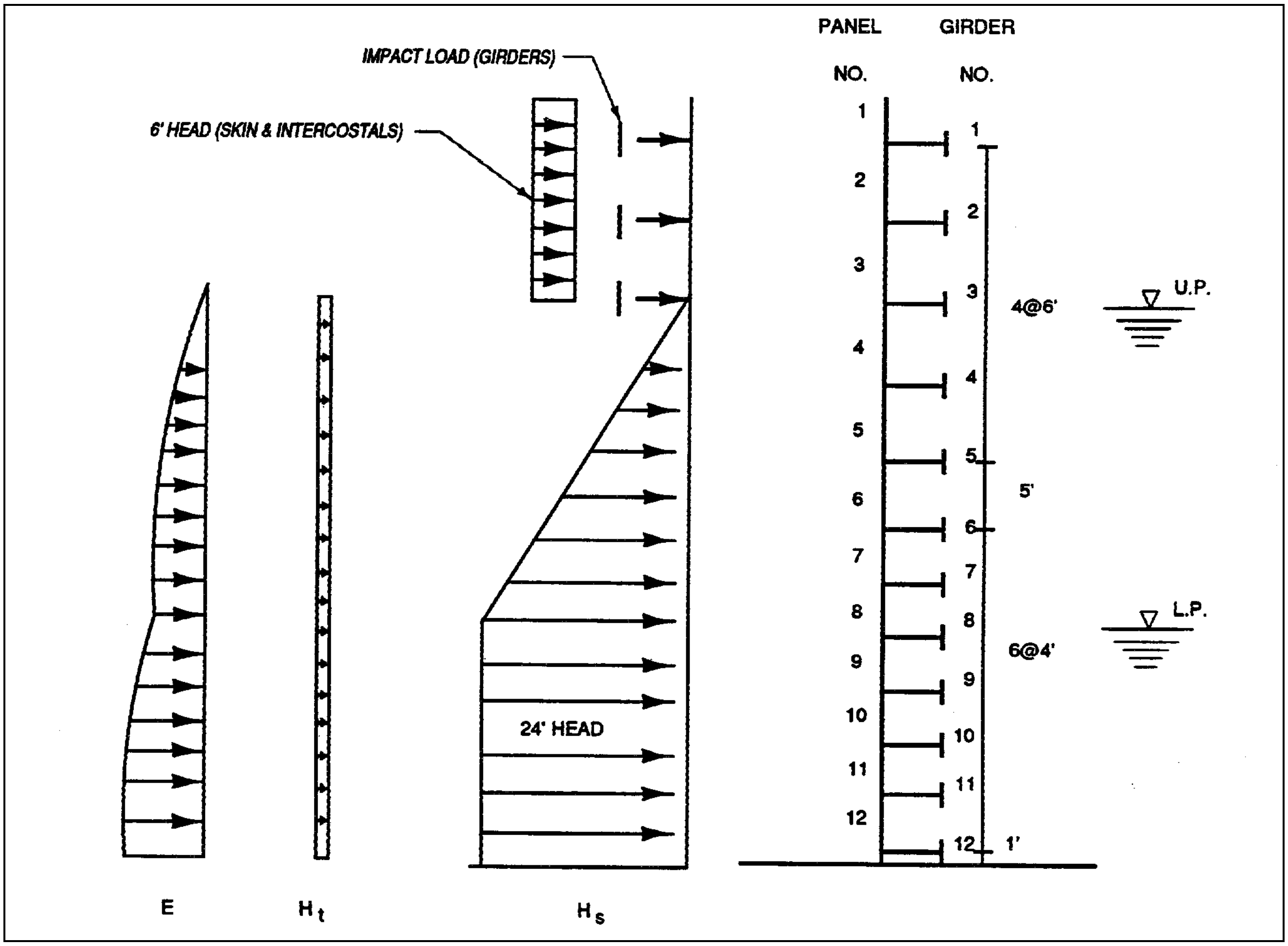

Figure B-5. Example miter gate loading

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

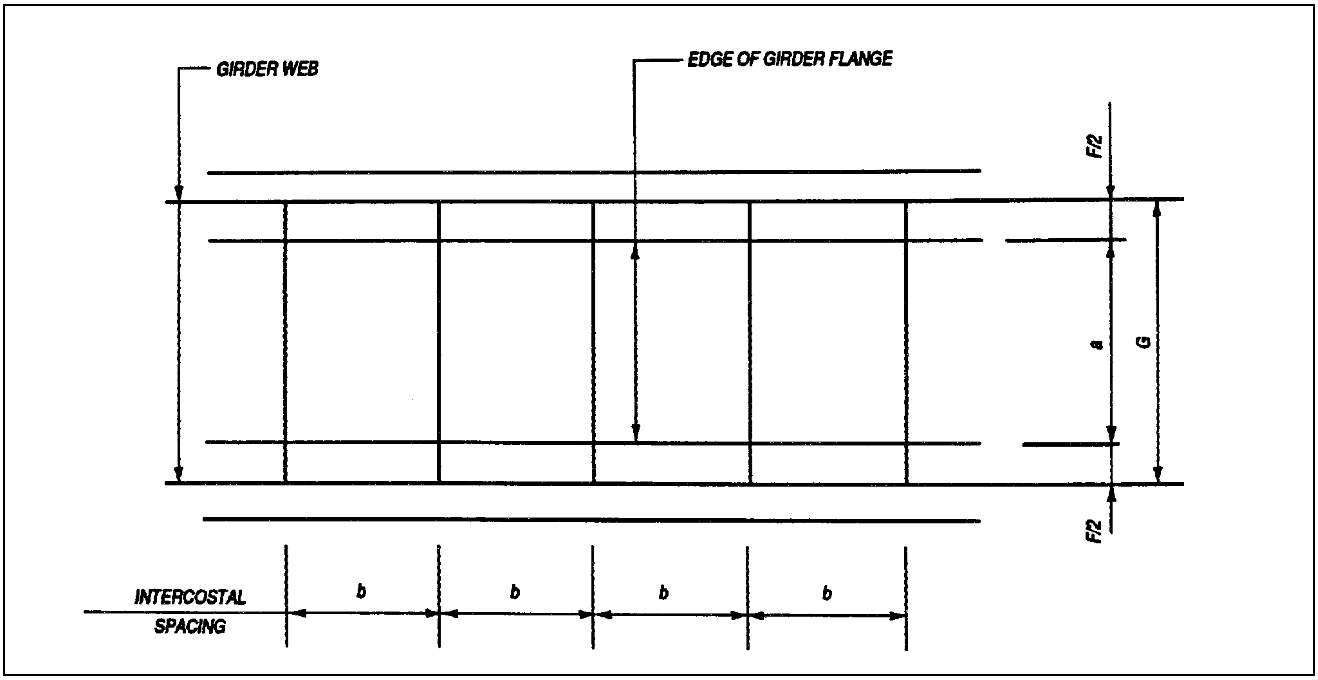

Figure B-6. Nomenclature for skin plate design

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.

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

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

$$\delta = \frac{0.0284(0.0104)(32)^4}{\left[1 + 1.056\left(\frac{32}{42}\right)^5\right] 29{,}000\left(\frac{1}{2}\right)^3}$$

$$= 0.067\text{in.} < 0.4t \quad \text{(Acceptable)}$$

(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

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

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.

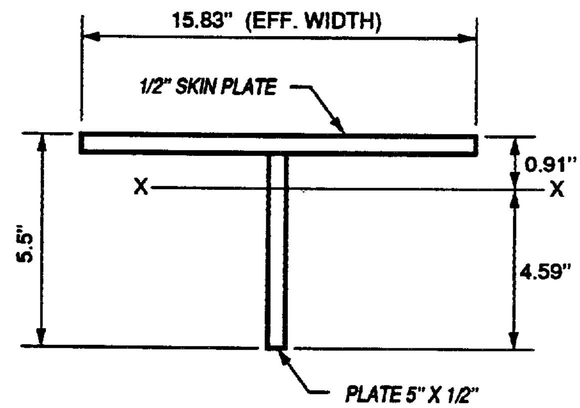

Figure B-7. Sample intercostal section

(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:

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

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

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

(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_{nt} = 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 in.^{4}

r_x = 21.81 in.

r_y = 4.43 in.

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

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

Z_x = 1,407.27 in.^{3}

y_c = 20.31 in.

A_g = 73.77 in.^{2}

where

I_x = moment of inertia about the x axis

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

S_{x1} = maximum section modulus

S_{x2} = minimum section modulus

Z_x = plastic modulus

y_c = distance from outside face of upstream flange to neutral axis

A_g = 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.

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

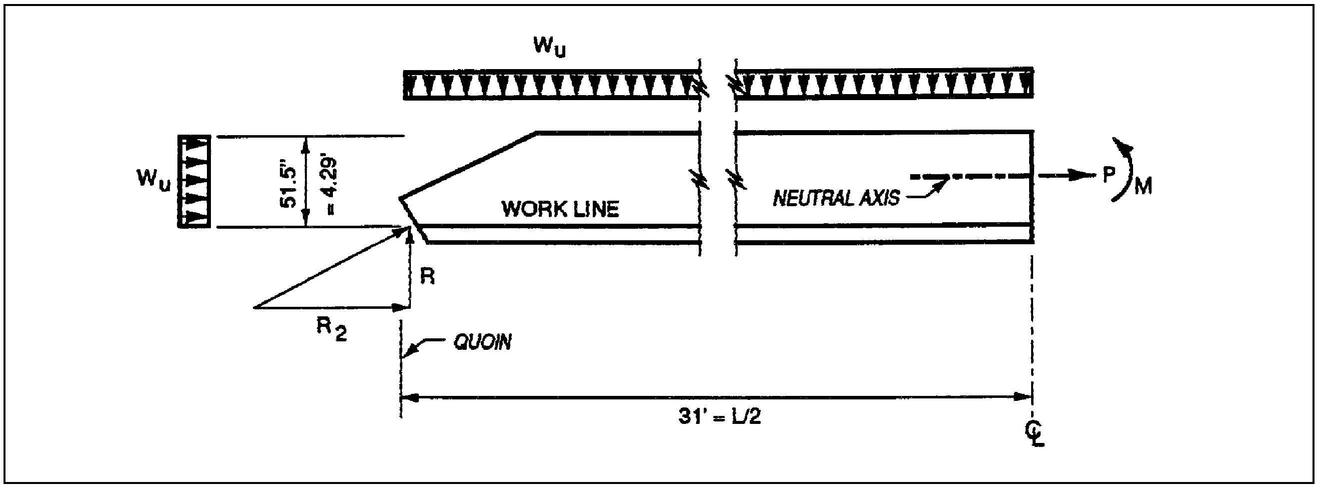

Figure B-8. Girder hydrostatic loading and reactions

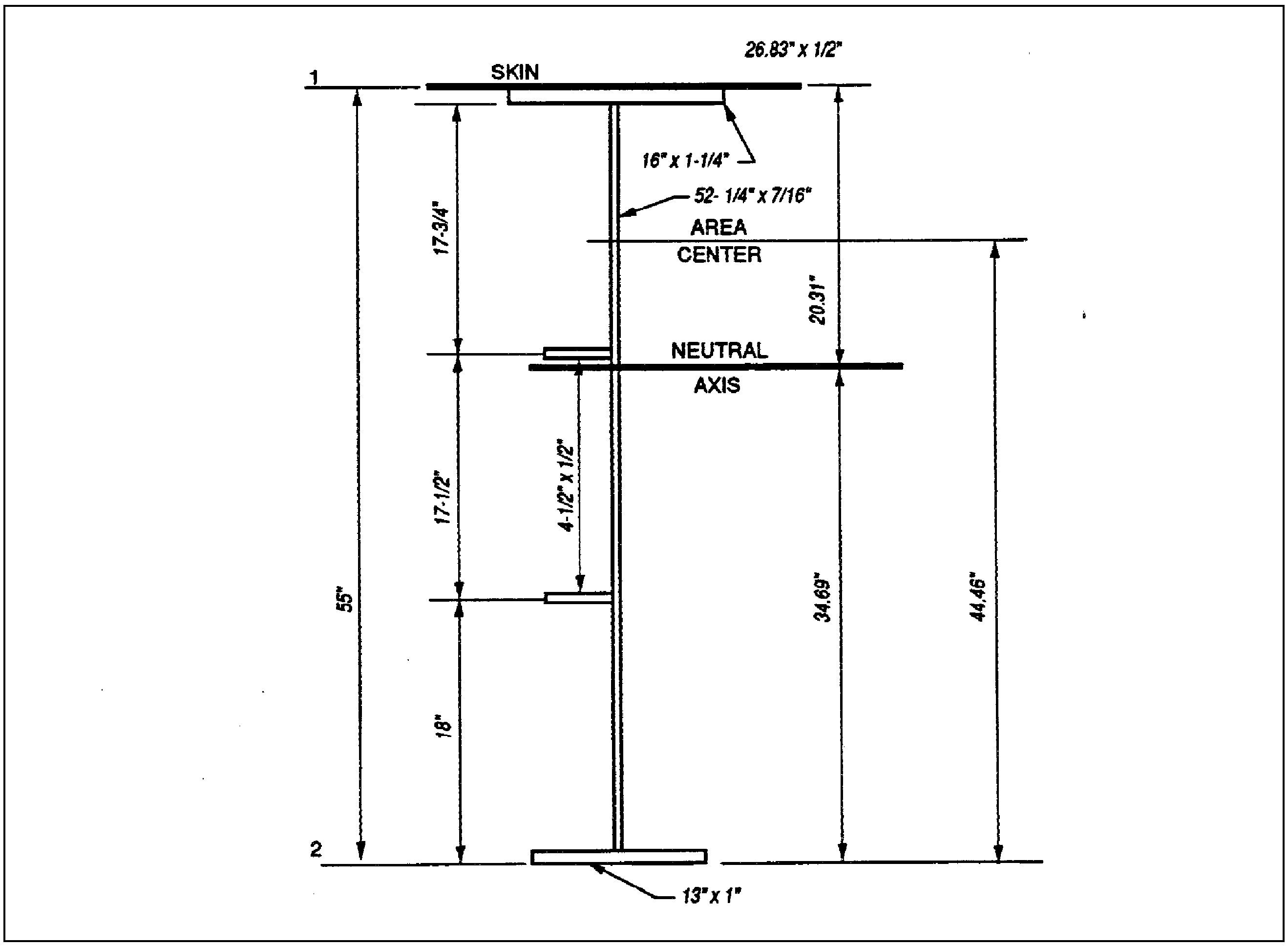

Figure B-9. Sample girder cross section

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)

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

where

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

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

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

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

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

$$\alpha \phi V_n = 0.9(0.9)(519.7) = 420.9 > 270 \qquad \text{(Acceptable)}$$

(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).

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

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

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

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

$$F_{cr} = \left(0.658^{\lambda_c^2}\right)F_y = (0.658^{0.147})36 = 33.85 \text{ ksi} \qquad \text{(AISC E2-2)}$$

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

Given P_u = 847 kips, ϕ_c = 0.85, and α = 0.9

$$\frac{P_u}{\alpha \phi_c P_n} = \frac{847}{0.9(0.85)(2{,}497)} = 0.44 > 0.2;$$

use Eq. AISC H1-1a

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

$$M_{uy} = 0$$

$$M_{ux} = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(AISC H1-2)}$$

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

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

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

$$M_{lt} = 0$$

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

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:

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

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.

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

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).

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

(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

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

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

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

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

$$Tz(H_t)_u = 1.0(\pm 130{,}971) = \pm 130{,}971 \text{ kip-ft}^2$$

Since $Tz(Q)_u$ is greater than $Tz(H_t)_u$, 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 $\phi_t = 0.9$

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

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

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

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

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:

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

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

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

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

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

$Q_p' = 229{,}629$ kip-ft and $Q_n' = 210{,}418$ kip-ft

$Q_o' = 0$ (Conservative assumption)

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

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

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

Live load gate closing deflection:

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

Let $D_p = 7.0$ in. and $D_n = -5.5$ in.

$Q_p' D_p + Q_n' D_n = 37{,}509$ kip-ft$^2 \approx Tz(D)_u$ (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:

$$f_p = \frac{R_p E (D_p - \delta_c)}{L}$$
$$= 26.4 \text{ ksi} < 29.16 \text{ ksi} \qquad \text{(Acceptable)}$$

For the negative diagonal on gate opening:

$$f_n = \frac{R_n E (D_n - \delta_o)}{L}$$
$$= 23.0 \text{ ksi} < 29.16 \text{ ksi} \qquad \text{(Acceptable)}$$

The minimum tensile stresses will occur as follows:

For the positive diagonal on gate opening:

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

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

For the negative diagonal on gate closing:

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

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

(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).

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

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

(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.

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

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

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

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 ksi < 13 ksi (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 in^2.

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

(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.

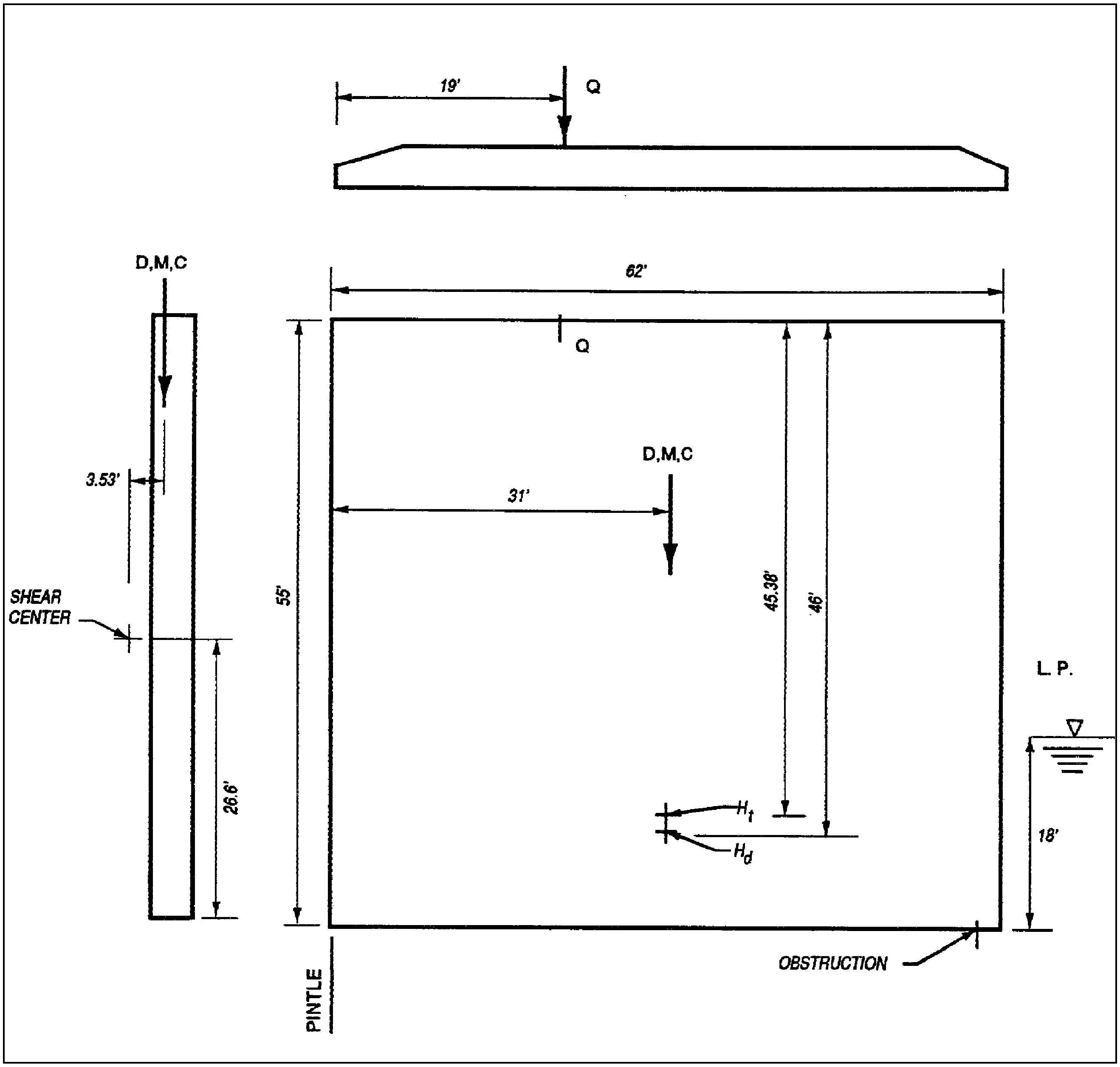

Figure B-10. Example miter leaf torsion loads