CECW-ET Engineer Manual 1110-2-2702	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-2702 1 January 2000
	Engineering and Design DESIGN OF SPILLWAY TAINTER GATES	
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EM 1110-2-2702
1 January 2000

US Army Corps
of Engineers

ENGINEERING AND DESIGN

Design of Spillway Tainter Gates

ENGINEER MANUAL

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

EM 1110-2-2702

CECW-ET

Manual
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DESIGN OF SPILLWAY TAINTER GATES

1. Purpose. This manual provides guidance for the design, fabrication, and inspection of spillway tainter gates, trunnion girder, and trunnion girder anchorage for navigation and flood control projects. Load and resistance factor design (LRFD) criteria is specified for design of steel components. Allowable stress design (ASD) criteria is provided in EM 1110-2-2105 and may be used only with prior approval of CECW-ET. Orthotropic shell, vertical framed, and stress skin-type tainter gates may be suitable in some locations but are not covered in this manual. Other types of control gates, including radial lock valves (reverse tainter valves) and sluice gates, may be referred to as tainter gates but also are not included in this manual.

2. Applicability. This manual applies to USACE commands having responsibility for Civil Works projects.

FOR THE COMMANDER:

Major General, USA
Chief of Staff

This manual supersedes EM 1110-2-2702, 1 August 1966.

	DEPARTMENT OF THE ARMY	EM 1110-2-2702
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Chapter 1
Introduction

1-1. Purpose

This manual provides guidance for the design, fabrication, and inspection of spillway tainter gates, trunnion girder, and trunnion girder anchorage for navigation and flood control projects. Load and resistance factor design (LRFD) criteria are specified for design of steel components. Allowable stress design (ASD) criteria are provided in EM 1110-2-2105 and may be used only with prior approval of CECW-ET. Orthotropic shell, vertical framed, and stress skin-type tainter gates may be suitable in some locations but are not covered in this manual. Other types of control gates, including radial lock valves (reverse tainter valves) and sluice gates, may be referred to as tainter gates but also are not included in this manual.

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This manual applies to USACE commands having responsibility for Civil Works projects.

1-3. References

References are provided in Appendix A.

1-4. Distribution

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1-5. Background

a. The previous version of this document was published in 1966, and since that time, design and fabrication standards have improved. Load and resistance factor design has been adapted by many specification writing organizations including American Institute of Steel Construction (AISC) (1994) and American Association of State Highway and Transportation Officials (AASHTO) (1994). In addition to the development and adoption of LRFD criteria, general knowledge on detailing and fabrication to improve fracture resistance of structures has advanced greatly. Most of the research and development behind current fatigue and fracture provisions of AISC, American Welding Society (AWS), and AASHTO were accomplished during the 1970's. EM 1110-2-2105 has been revised recently (1993) to include new LRFD and fracture control guidance for hydraulic steel structures.

b. Additionally, knowledge has expanded due to operational experience resulting in improved design considerations. During the late 1960s and early 1970s, many tainter gates on the Arkansas River exhibited vibration that led to fatigue failure of rib-to-girder welded connections. Study of these failures resulted in development of improved tainter gate lip and bottom seal details that minimize vibration. Tainter gates at various projects have exhibited operational problems and failures attributed to effects of trunnion friction not accounted for in original design. As a result of related studies, information regarding friction magnitude and structural detailing to withstand friction forces has been gained. Traditionally, tainter gates have been operated by lifting with wire rope or chains attached to a hoist located above the gate. More recently, hydraulic cylinders are being used to operate tainter gates due to economy, reduced maintenance, and advantages concerning operating multiple gates.

c. The intent for this publication is to update tainter gate design guidance to include the most recent and up-to-date criteria. General applications are discussed in Chapter 2. Guidance for LRFD and fracture control of structural components is provided in Chapter 3. Criteria for design of trunnion, gate anchorage, and trunnion is are given in Chapter 4 through 6. Considerations for operating equipment are discussed in Chapter 7. Chapter 8 provides general guidance on corrosion control. Appendix A includes references and Appendix B presents general design considerations and provides guidance on preparation of technical project specifications regarding fabrication and erection of tainter gates. Considerations for design to minimize operational problems are included in Appendix C. Appendix D provides data on existing tainter gates.

1-6. Mandatory Requirements

This manual provides design guidance for the protection of U.S. Army Corps of Engineers (USACE) structures. In certain cases guidance requirements, because of their criticality to project safety and performance, are considered to be mandatory as discussed in ER 1110-2-1150. In this manual, the load and resistance factors for the design requirements of paragraphs 3-4, 4-4, 5-4, and 6-4 are mandatory.

Chapter 2
Applications

2-1. General

a. *Application*. Controlled spillways include crest gates that serve as a movable damming surface allowing the spillway crest to be located below the normal operating level of a reservoir or channel. Information on the use of various crest gates and related spillway design considerations is provided in EM 1110-2-1603, EM 1110-2-1605, and EM 1110-2-2607. Tainter gates are considered to be the most economical, and usually the most suitable, type of gate for controlled spillways due to simplicity, light weight, and low hoist-capacity requirements. A tainter gate is a segment of a cylinder mounted on radial arms that rotate on trunnions anchored to the piers. Spillway flow is regulated by raising or lowering the gate to adjust the discharge under the gate. Numerous types of tainter gates exist; however, this manual includes guidance for the conventional tainter gate described in Chapter 3, paragraph 3-2. Figures 2-1 and 2-2 show photographs of actual dams with tainter gates. Figure 2-3 presents a downstream view of a typical tainter gate.

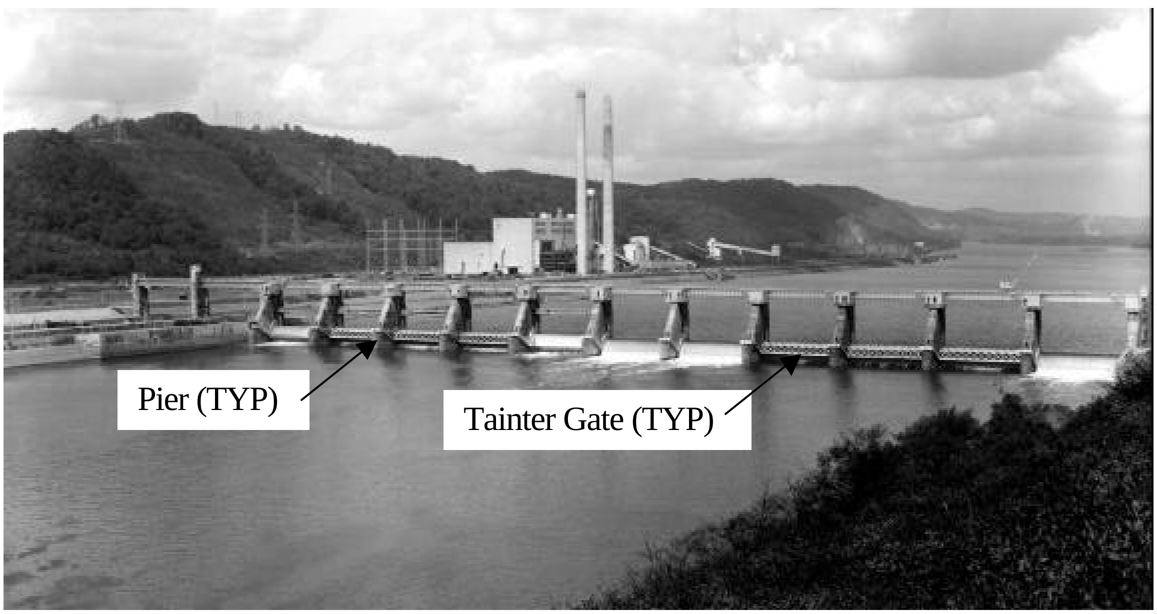


Figure 2-1. Overall view of navigation dam from downstream

b. *Tainter gate construction*. Gates are composed primarily of structural steel and are generally of welded fabrication. Structural members are typically rolled sections; however, welded built-up girders may be required for large gates. Various components of the trunnion assembly and operating equipment may be of forged or cast steel, copper alloys, or stainless steel. Based on project requirements, trunnion girders are either posttensioned concrete girders or steel girders as described in Chapter 6.

2-2. Advantages and Disadvantages of Tainter Gates vs Other Spillway Crest Gates

a. Tainter gates have several unique advantages compared to other spillway gate types (lift gates, roller gates, hinged or flap gates).

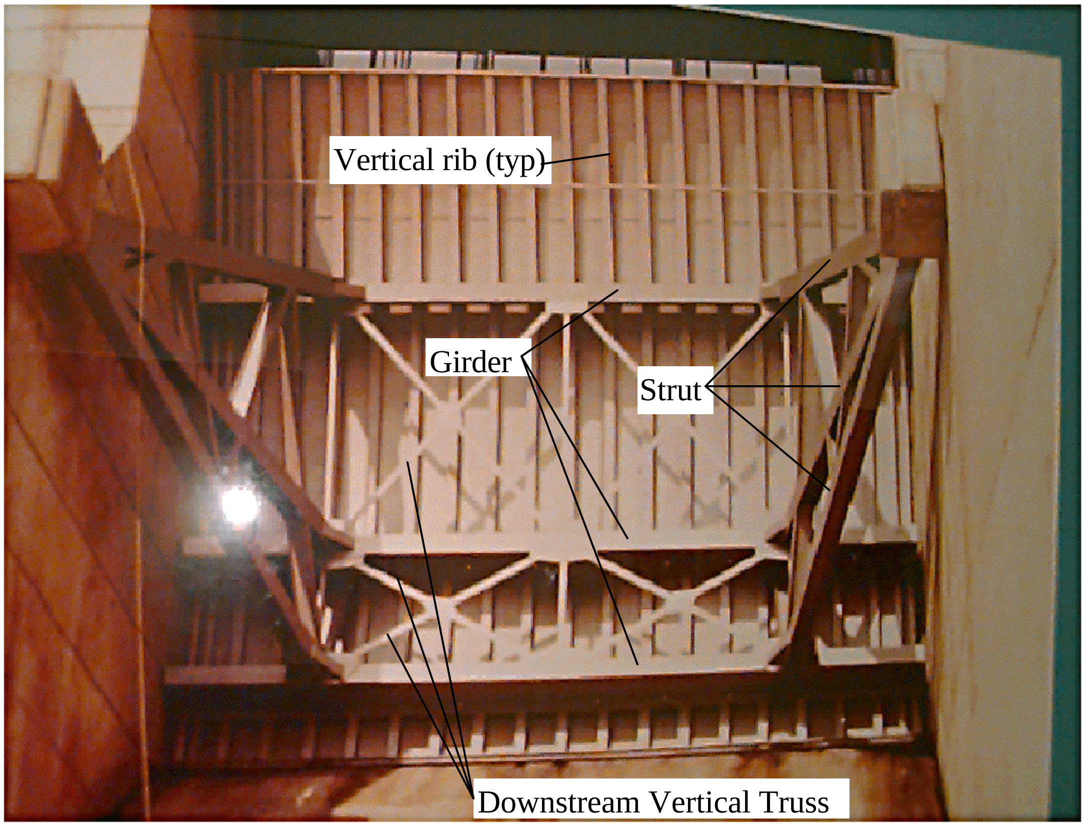


Figure 2-2. Closeup view of tainter gate from downstream

(1) The radial shape provides efficient transfer of hydrostatic loads through the trunnion.

(2) A lower hoist capacity is required.

(3) Tainter gates have a relatively fast operating speed and can be operated efficiently.

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

(5) Tainter gate geometry provides favorable hydraulic discharge characteristics.

b. Disadvantages include the following:

(1) To accommodate location of the trunnion, the pier and foundation will likely be longer in the downstream direction than would be necessary for vertical gates. The hoist arrangement may result in taller piers especially when a wire rope hoist system is used. (Gates with hydraulic cylinder hoists generally require shorter piers than gates with wire rope hoists.) Larger piers increase cost due to more required concrete and will usually result in a less favorable seismic resistance due to greater height and mass.

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

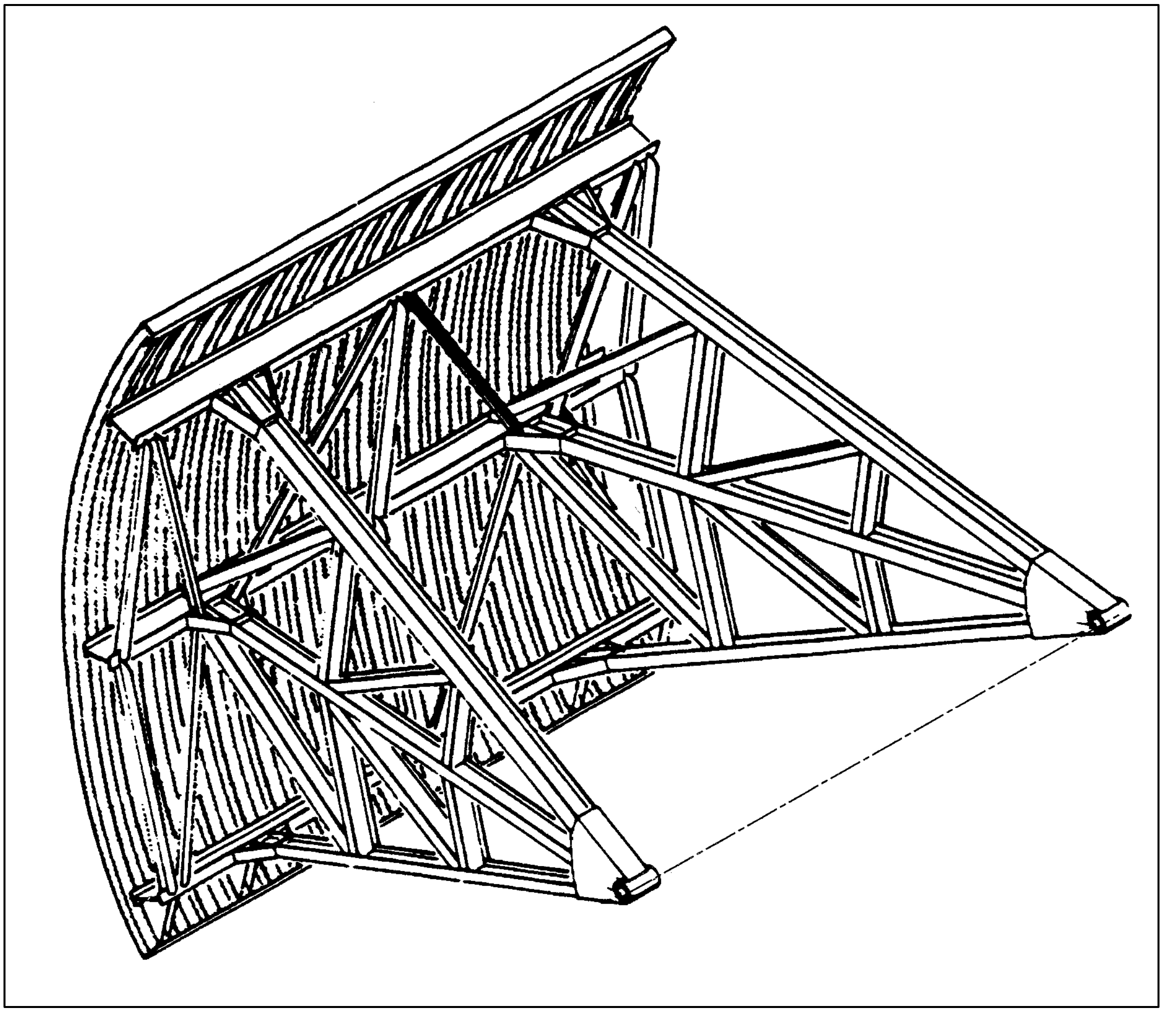


Figure 2-3. Downstream view of a typical tainter gate

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

2-3. Use on Corps of Engineers Projects

Spillway tainter gates are effectively applied for use on spillways of various projects due to favorable operating and discharge characteristics. Gates are used on flood control projects, navigation projects, hydropower projects, and multipurpose projects (i.e., flood control with hydropower). Although navigation and flood control tainter gates are structurally similar and generally have the same maximum design loads, the normal loading and function may be very different. In general, gates on navigation projects are subject to significant loading and discharge conditions most of the time, whereas gates on flood control projects are loaded significantly only during flood events. These differences may influence selection of the lifting hoist system, emphasis on detailing for resistance to possible vibration loading, and selection of a corrosion protection system.

a. Navigation projects. Navigation projects are normally built in conjunction with a lock. Navigation gates are designed to maintain a consistent pool necessary for navigation purposes, while offering minimum resistance to flood flows. Gate sills are generally placed near the channel bottom, and during normal flows, damming to the required upper navigation pool elevation is provided by tainter gates. Under normal conditions, most gates on a navigation dam are closed, while several other gates are partially open to provide discharge necessary to maintain a consistent upper lock pool. During flood events, gates are open and flood flow is not regulated. The upper pool elevation often rises significantly during flood events and the open gate must clear the water surface profile to pass accumulated drift. As a result, the trunnion elevation is often relatively high and the gate radius is often longer than gates designed for other applications. Under normal conditions, navigation gates are generally partially submerged and are significantly loaded with the upstream-downstream hydrostatic head. In addition, these gates are more likely to be subject to flow-induced vibration and cavitation. A typical cross section of a navigation dam with tainter gates is presented in Figure 2-4.

b. Flood control and hydropower projects. Flood control projects provide temporary storage of flood flow and many projects include gated spillways to provide the capability to regulate outflow. On flood control projects with gated spillways, gate sills are generally located such that the gates are dry or only partially wet under normal conditions. In general, gates are exposed to the atmosphere and are subject to slight loads, if any. Only during infrequent flood events are gates loaded significantly due to increases in pool, and during subsequent discharge hydraulic flow-related conditions exist. Trunnions are typically located at an elevation approximately one-third the height of the gate above the sill. Some unique multipurpose projects (projects that provide flood control and reservoir storage) and most hydropower projects include aspects of flood control and navigation gates. Gates on these projects are normally subject to significant hydrostatic loading on the upstream side and may be used to regulate flow on a regular basis. A typical cross section of a flood control or hydropower dam with tainter gates is presented in Figure 2-5.

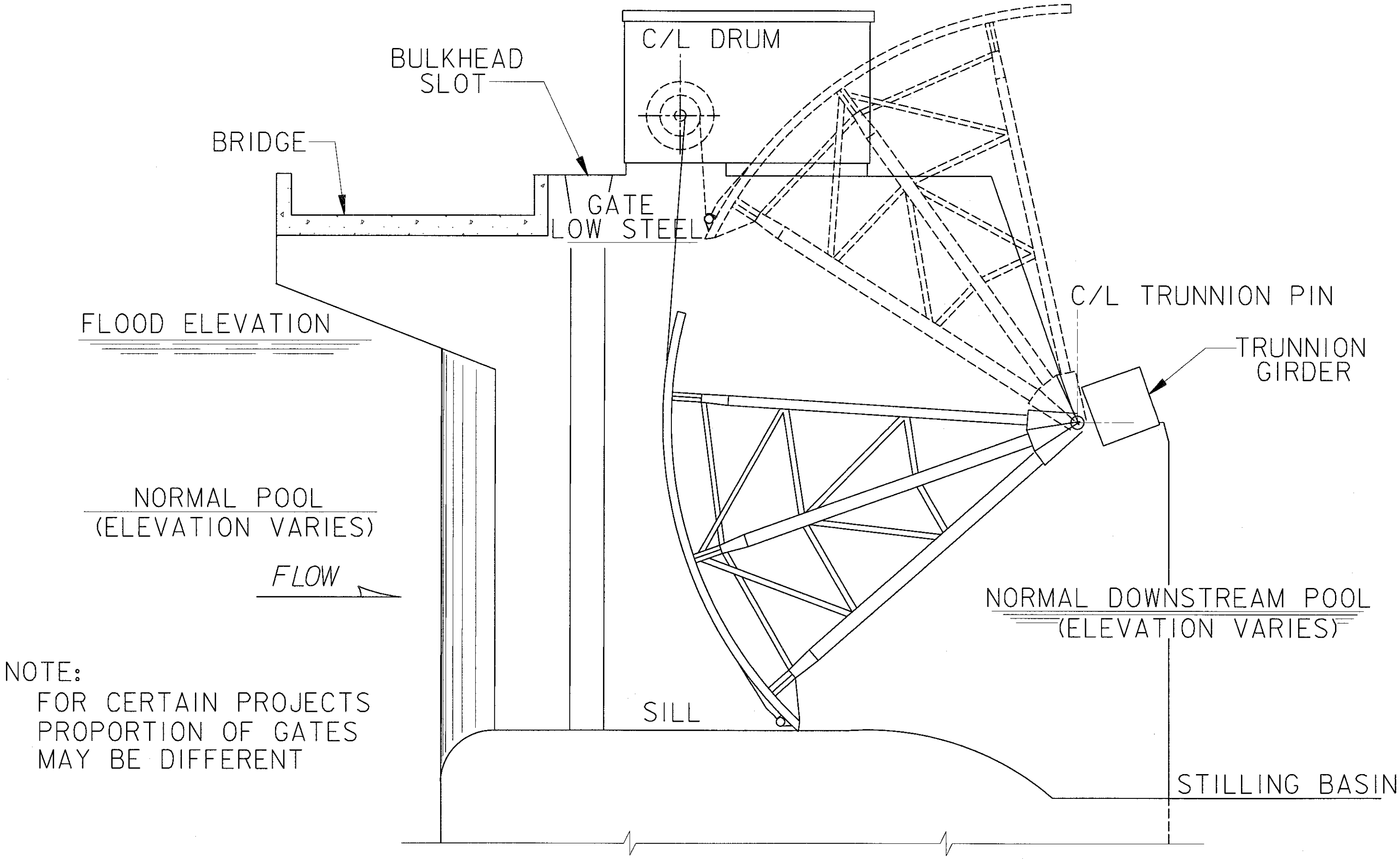


Figure 2-4. Typical navigation tainter gate

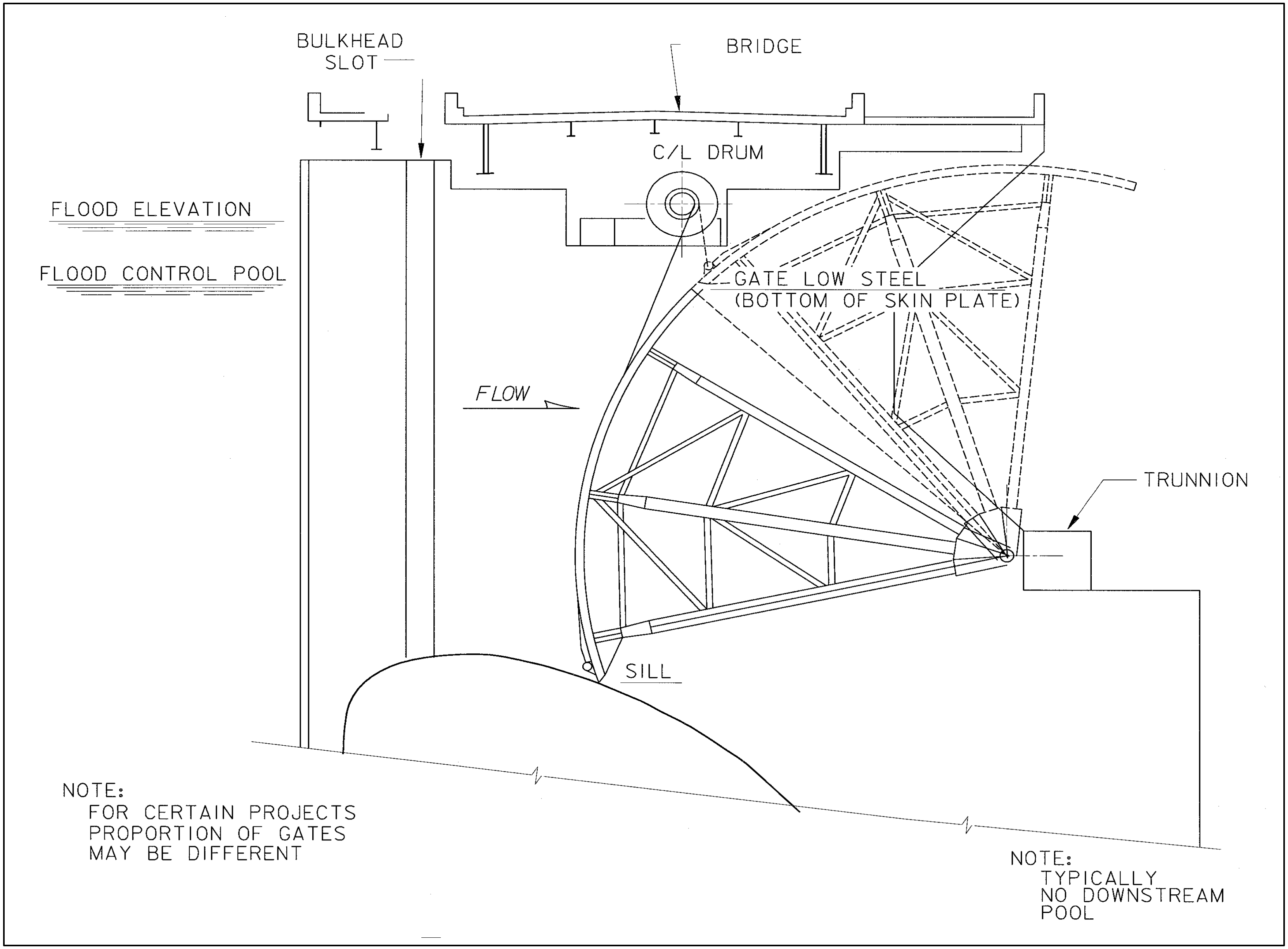


Figure 2-5. Typical flood control or hydropower tainter gate

Chapter 3
Tainter Gate Design

3-1. Introduction

This chapter presents design guidance for the Corps of Engineers standard tainter gate described herein. The configuration for the standard gate has resulted from much practical and theoretical investigation of alternatives and over 60 years of design and field experience with construction, operation, and maintenance. It is generally the simplest and most economical tainter gate configuration for most applications.

3-2. Geometry, Components, and Sizing

a. Standard Corps of Engineers tainter gate geometry and components.

(1) Primary gate components. The principal elements of a conventional tainter gate are the skin plate assembly, horizontal girders, end frames, and trunnions (Figure 3-1). The skin plate assembly, which forms a cylindrical damming surface, consists of a skin plate stiffened and supported by curved vertical ribs. Structurally, the skin plate acts compositely with the ribs (usually structural Tee sections) to form the skin plate assembly. The skin plate assembly is supported by the horizontal girders that span the gate width. The downstream edge of each rib is attached to the upstream flange of the horizontal girders. The horizontal girders are supported by the end frames. End frames consist of radial struts or strut arms and bracing members that converge at the trunnion which is anchored to the pier through the trunnion girder. The end frames may be parallel to the face of the pier (support the horizontal girders at the ends) or inclined to the face of the pier (support the horizontal girders at some distance from the end with cantilever portions at each end). The trunnion is the hinge upon which the gate rotates. The trunnion is supported by the trunnion girder which is addressed in Chapter 6.

(2) Other structural members. Structural bracing members are incorporated to resist specific loads and/or to brace compression members. Certain bracing members are significant structural members, while others can be considered secondary members.

(a) Horizontal girder lateral bracing. Cross bracing is generally placed between adjacent girders in a plane perpendicular to the girder axes, sometimes at several locations along the length of the girders. This horizontal girder lateral bracing may simply provide lateral bracing for the girders or may serve to carry vertical forces from the skin plate assembly to the end frame. Lateral bracing that is located in the same plane with the end frames is generally made up of significant structural members, while intermediate bracing located away from the end frames provides girder lateral stability and can be considered secondary members. The bracing located in the same plane with the end frames carries significant vertical forces from the skin plate assembly to the end frame and is often considered a part of the end frame (Figure 3-2).

(b) Downstream vertical truss. The downstream vertical truss consists of bracing provided between the downstream flanges of the horizontal girders. Various configurations have been used depending on the gate size and configuration as shown by Figure 3-3. For gates with more than two girders, the downstream vertical truss does not lie in a single plane. Since the horizontal girders are arranged along the arc of the skin plate assembly, the downstream girder flanges do not lie in the same plane. Therefore,

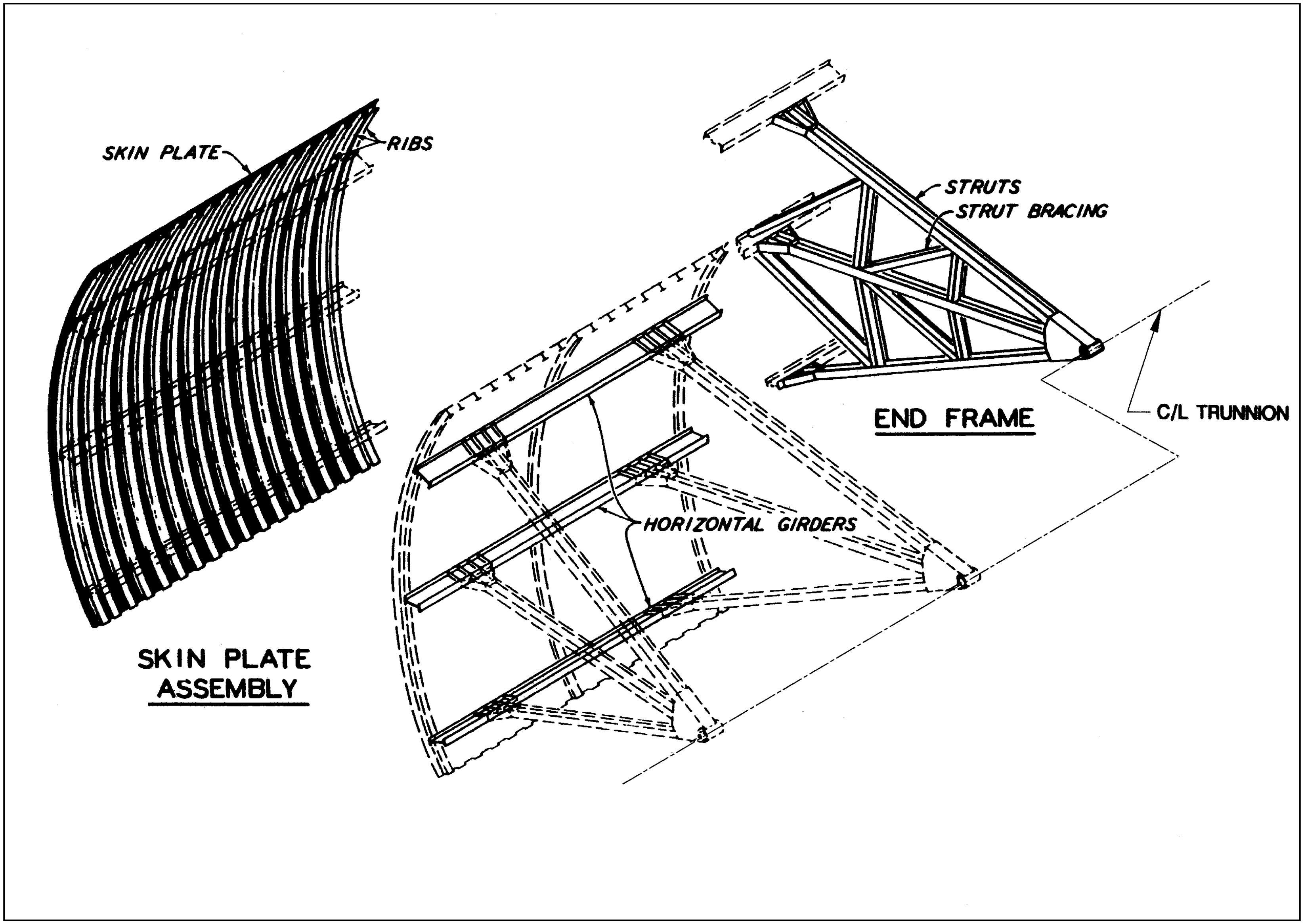


Figure 3-1. Primary tainter gate components

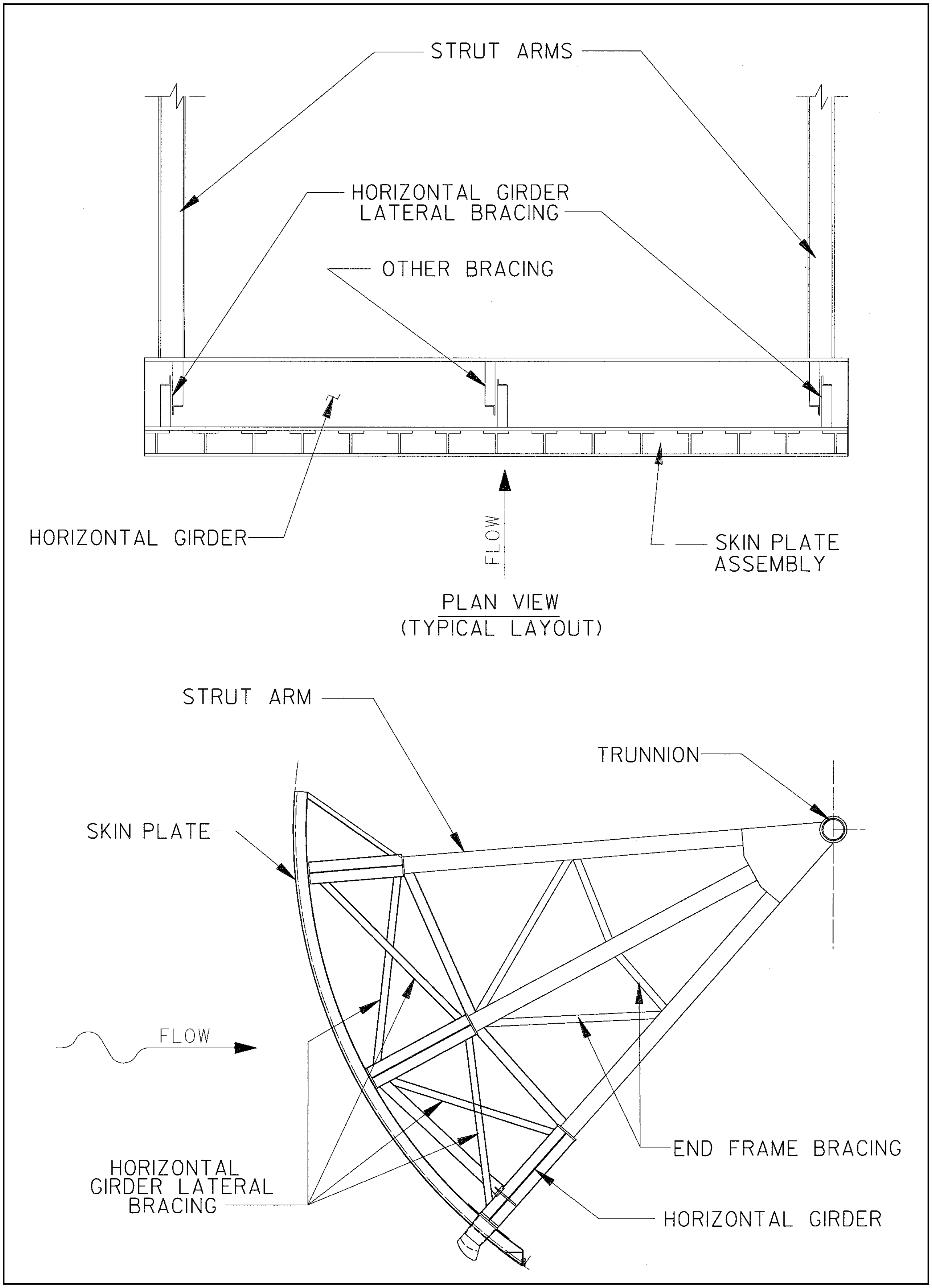


Figure 3-2. Horizontal girder lateral bracing

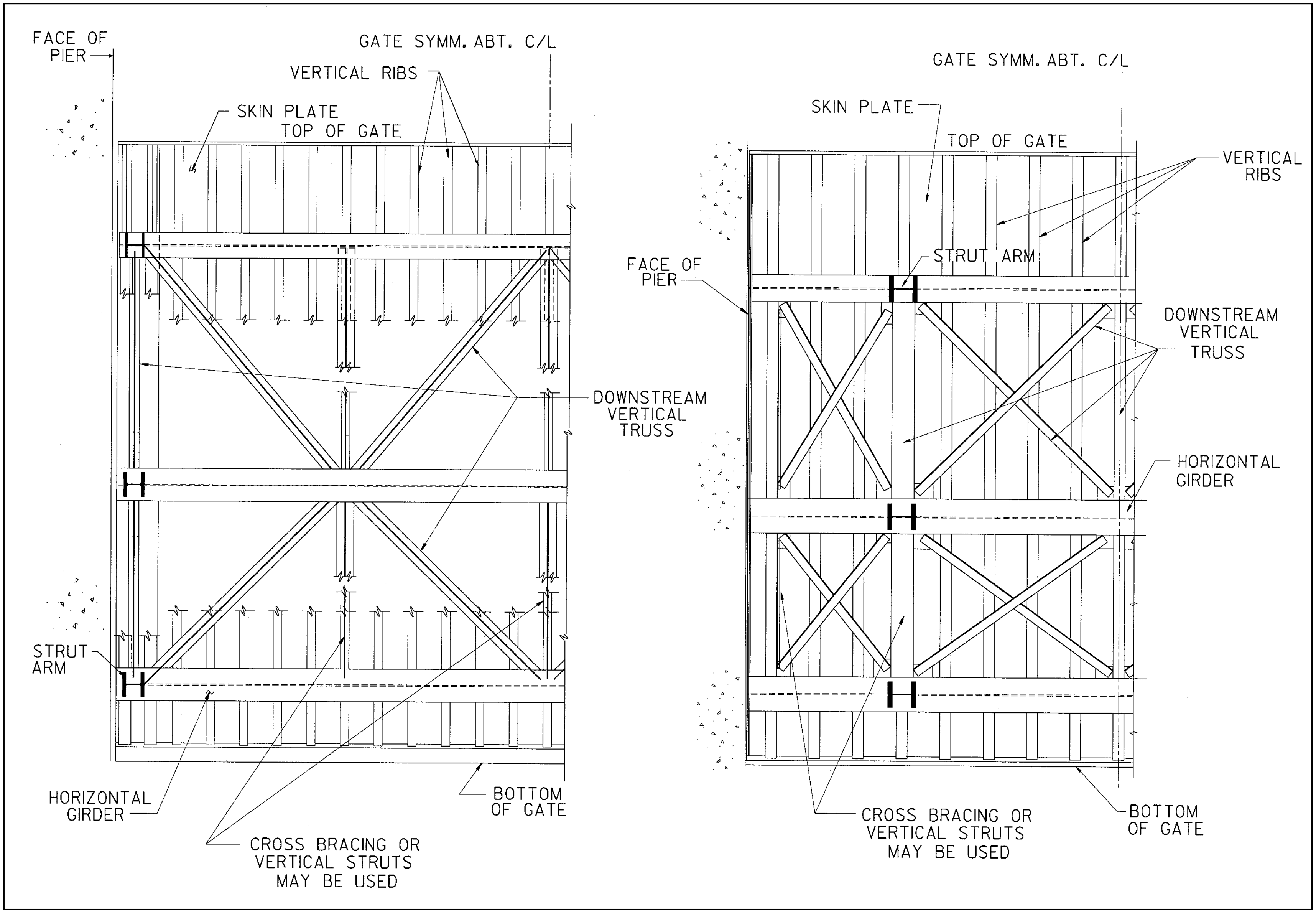


Figure 3-3. Downstream vertical truss (typical configurations)

bracing members located between one pair of adjacent horizontal girders are not in the same plane as those between the next pair. This out-of-plane geometry is commonly ignored for design purposes.

(c) End frame bracing. For the standard tainter gate configuration, bracing is provided for the end frame struts as shown by Figure 3-4. The end frame bracing members are ordinarily designed to brace the struts about the weak axis to achieve adequate slenderness ratios. As such, these members are considered secondary members. However, depending on their configuration and connection details, these bracing members may carry significant forces and act as primary members.

(d) Trunnion tie. A trunnion tie is a tension member provided on some gates with inclined strut arms that is designed to resist lateral end frame reaction loads (loads that are parallel to trunnion pin axis or perpendicular to the pier). Trunnion ties are not generally provided on gates with parallel strut arms, since the lateral reaction loads are normally negligible (paragraph 3-5.a(2)(c)). The trunnion tie extends across the gate bay from one end frame to the other and is attached to each end frame near the trunnion (Figure 3-5). The tie can be made up of a single member or multiple members depending on how it is attached to the end frames. Tubular members are often used.

(3) Gate lifting systems. Two standard lifting arrangements presently recommended for new construction are the wire rope hoist and hydraulic hoist system. The wire rope system incorporates wire ropes that wrap around the upstream side of the skin plate assembly and attach near the bottom of the skin plate as shown in Figure 3-6. The hydraulic hoist system incorporates hydraulic cylinders that attach to the downstream gate framing, usually the end frames (Figure 3-7). Hoist layout geometry is addressed in paragraph 3-2.c. Hoist loads and attachment details are addressed later in Chapter 3 and operating equipment is addressed in Chapter 7.

b. Alternative framing systems. In the past, many alternatives to the standard framing system have been designed and constructed. Each of these configurations may be suitable for certain applications and a brief description of some configurations is provided for information. The design guidance and criteria presented herein are not necessarily applicable to these gates.

(1) Vertical girders. For the standard gate configuration, fabrication at the trunnion and economy would normally limit the number of end frame strut arms to a maximum of four on each side. This in turn limits the design to four horizontal girders when each strut supports a horizontal girder. For tall gates, vertical girders have been used to simplify the end frame configuration. Curved vertical girders may be used to support several horizontal girders at each. Each vertical girder is supported by the corresponding end frame that may include two or more struts. The concept may be used with parallel or inclined end frames.

(2) Vertically framed gates. In vertically framed gates, vertical girders support ribs that are placed horizontally. With this configuration, horizontal girders and vertical ribs are eliminated. As with vertical girder gates, the vertical girders can be supported by two or more struts. This system has been used on small gates and gates with low hydrostatic head.

(3) Orthotropic gates. An alternative design approach is to design the gate as an orthotropic system. With the orthotropic approach, the skin plate, ribs, and horizontal girders are assumed to act as a stiffened shell. Typically, the ribs are framed into the horizontal girder webs. This approach can save material and gate weight, but fabrication and maintenance costs are often higher. Its use has been very limited.

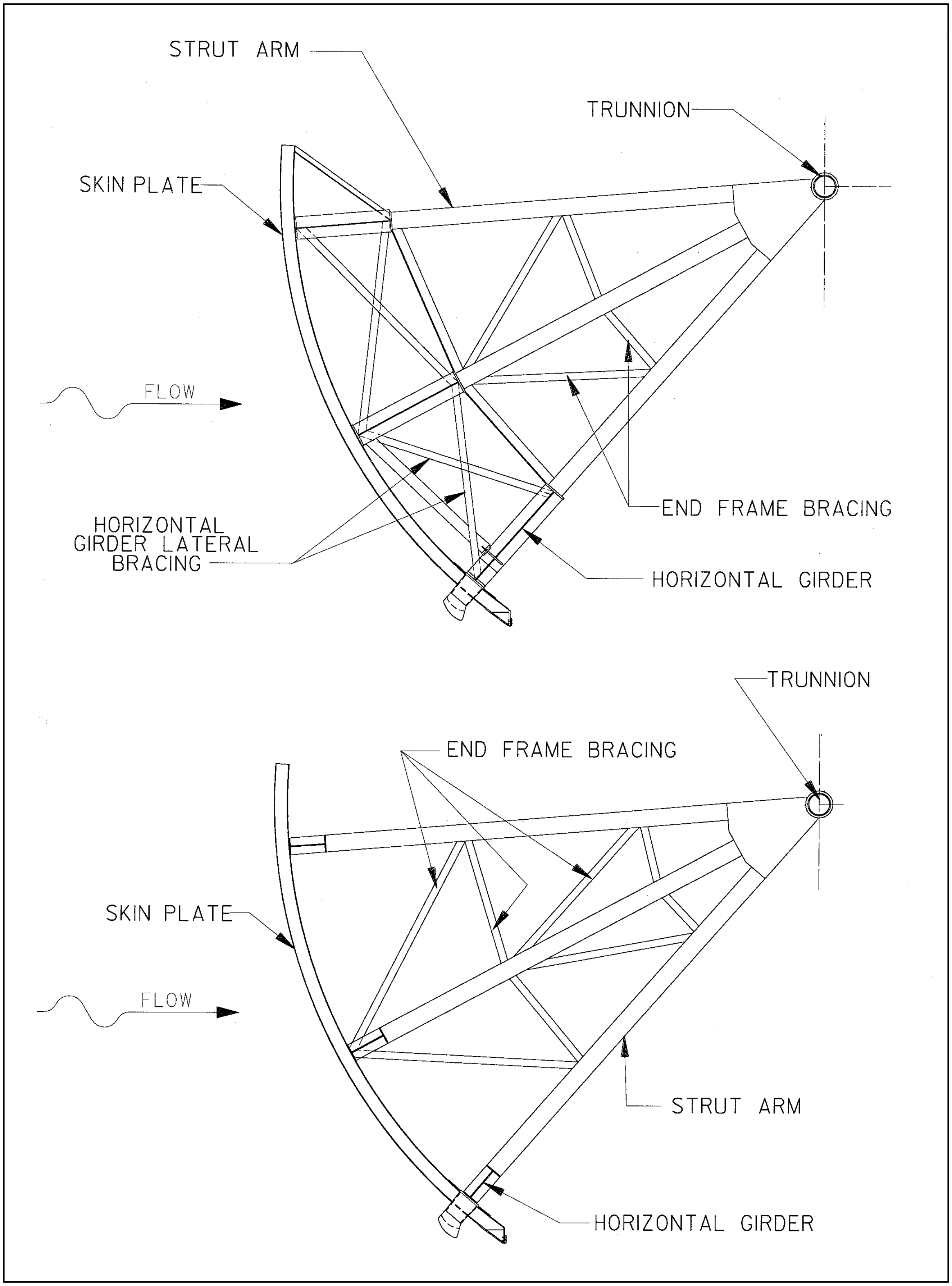


Figure 3-4. End frame bracing (typical arrangements)

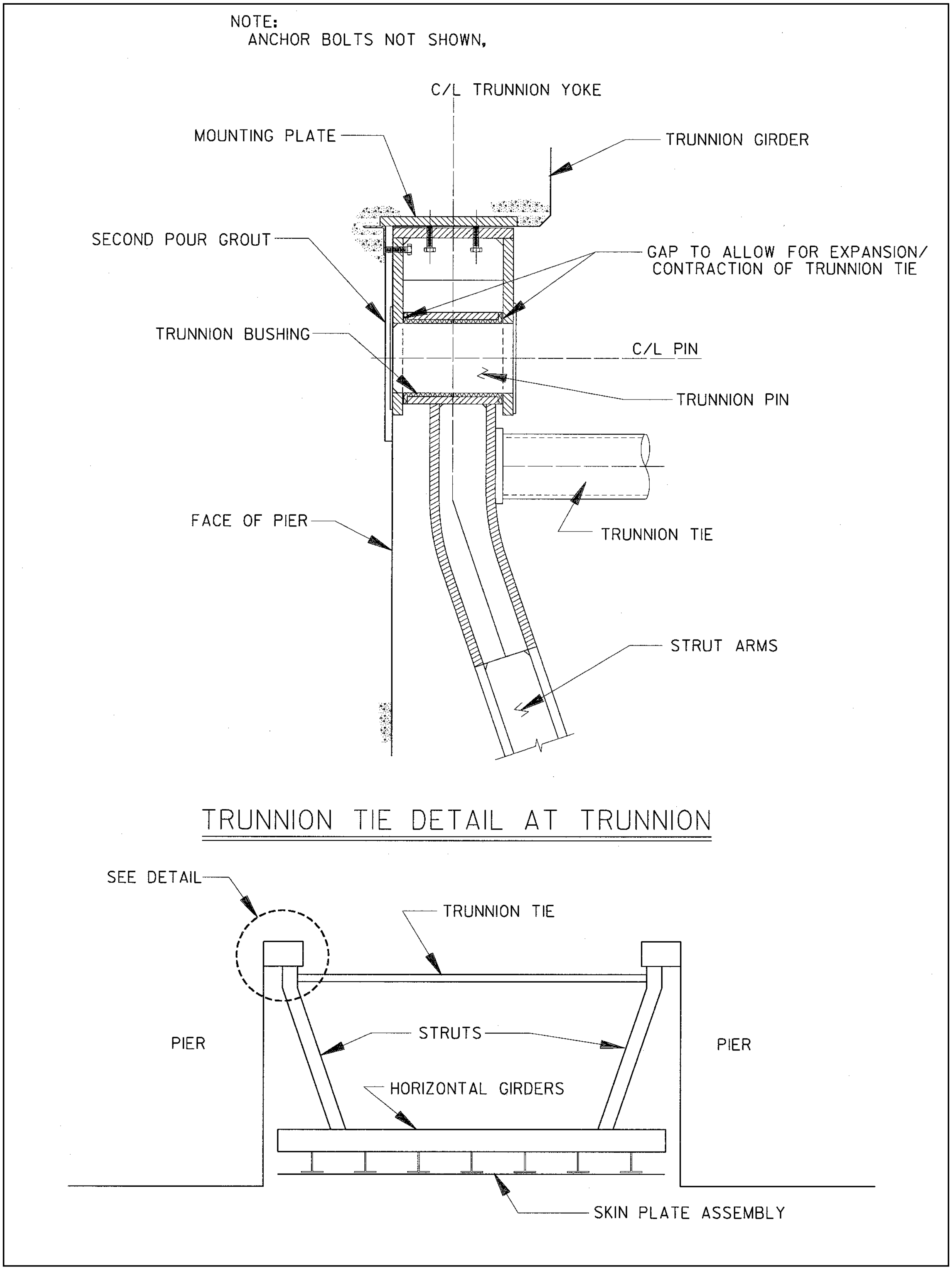


Figure 3-5. Trunnion tie

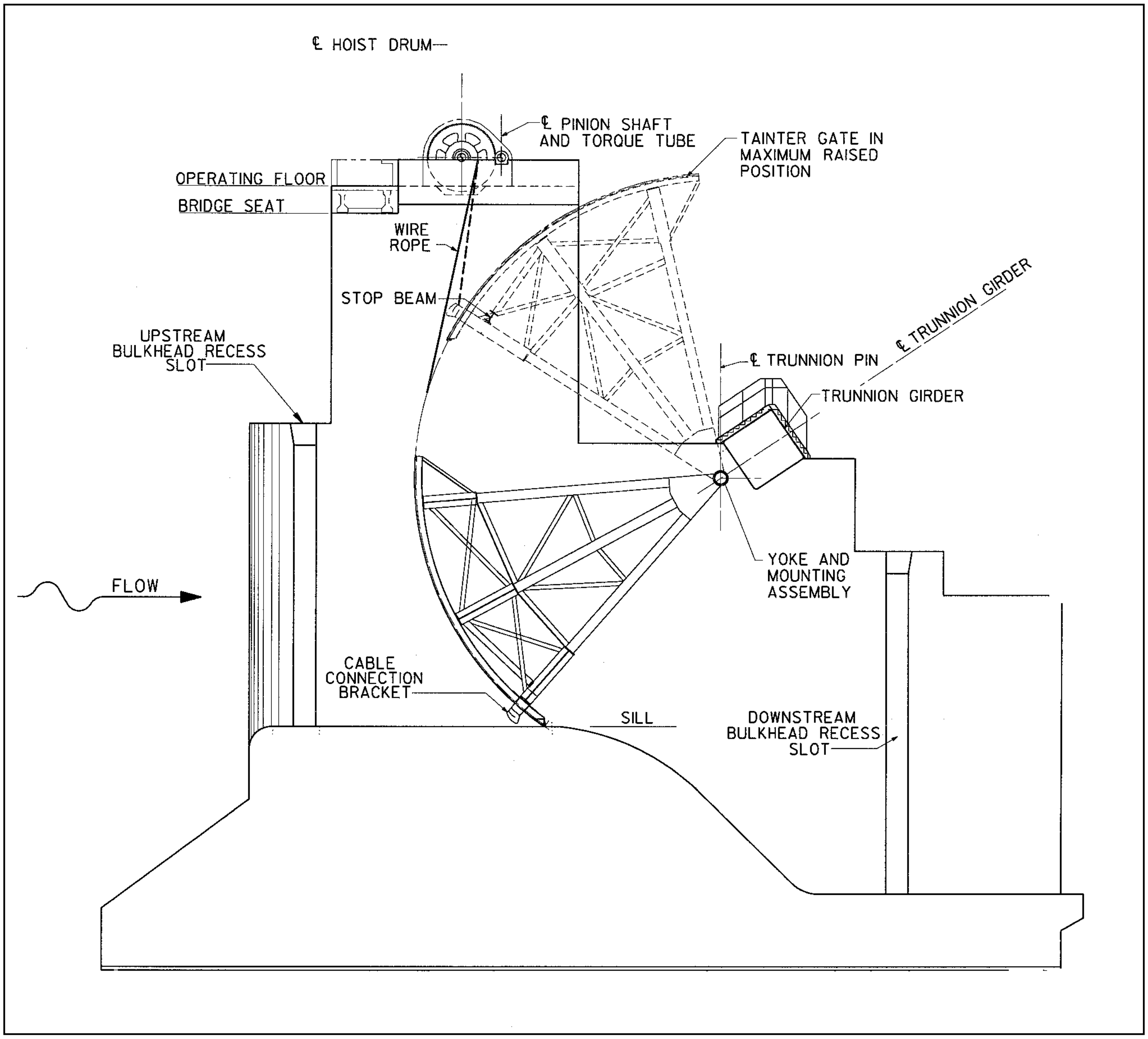


Figure 3-6. Example of wire rope hoist system

(4) Stressed skin gates. Stressed skin gates are a type of orthotropic gate in which the skin plate assembly is considered to be a shell or tubular structure spanning between trunnion arms. The skin plate is stiffened with horizontal and vertical diaphragms and intermediate stiffening members (usually horizontal tee sections parallel to the intermediate or midlevel horizontal diaphragm). As with other orthotropic gates, this type of gate can save material and gate weight, but fabrication and maintenance costs are often higher.

(5) Truss-type or space frame gates. Three-dimensional (3-D) truss or space frame gates were sometimes used in early tainter gate designs in the 1930s and 1940s. These early gates were designed as a series two-dimensional (2-D) trusses and were referred to as truss-type gates. They were typically as heavy or heavier than girder designs and fabrication and maintenance costs were very high. For this reason they were not adopted as a standard design. More recently, the use of computer designed 3-D space frame gates constructed with tubular sections has been investigated and may be practical in some situations.

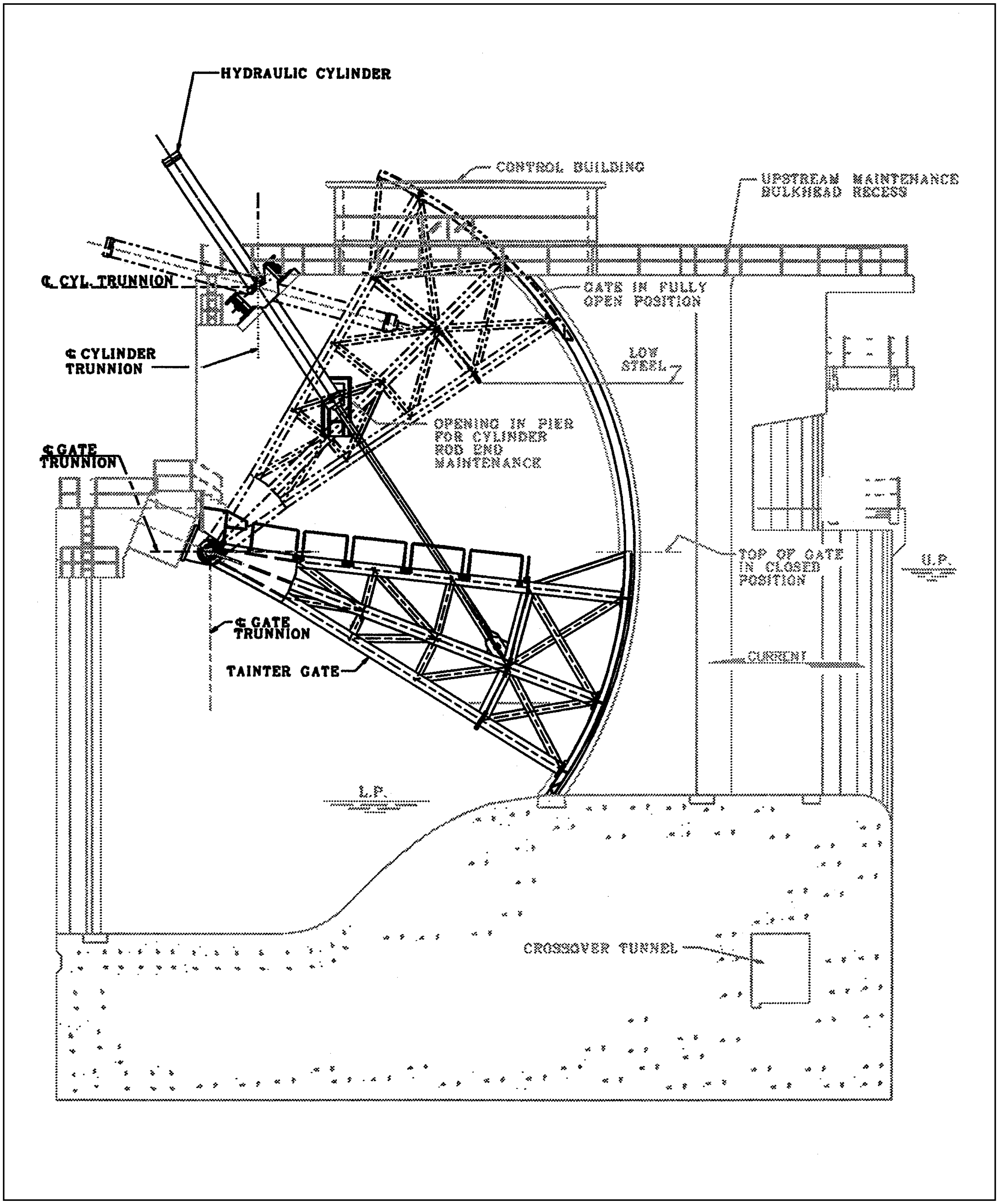


Figure 3-7. Hydraulically operated tainter gate

(6) Overflow/submersible gates. These gates may be of the standard configuration but are designed to allow water to pass over the top the gate. Deflector plates are often provided on the downstream side of the gate to allow water and debris to pass over the framing with minimized impact. Other gates have been designed to include a downstream skin plate, so the gate is completely enclosed. Vibration problems have been prevalent with this type gate.

c. General gate sizing and layout considerations. The sizing of the gates is an important early step in the design process. Gate size affects other project components, project cost, operation, and maintenance of the project. The following paragraph includes various considerations that should be taken into account while selecting a practical and economical tainter gate size. Related guidance can be found in EMs 1110-2-1603, 1110-2-1605, and 1110-2-2607. Appendix D provides pertinent data for a number of existing tainter gates. Each project is unique and the gate size and configuration should be determined based on careful study of the project as a whole. The best alternative is not necessarily a gate with the lightest gate weight-to-size ratio.

(1) Gate size. The hydraulic engineer will normally establish the limiting parameters for gate height and width. Within those limits, various height-to-width ratios should be studied to find the most suitable gate size for the project. The structural designer should coordinate closely with the hydraulic engineer in determining the basic limiting requirements for size and shape. The size, shape, and framing system of the gates should be selected to minimize the overall cost of the spillway, rather than the gate itself. Determination of gate size will also consider practical operation and maintenance considerations specific to the project.

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

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

(4) Trunnion location. It is generally desirable to locate the trunnion above the maximum flood water surface profile to avoid contact with floating ice and debris and to avoid submergence of the operating parts. However, it is sometimes practical to allow submergence for flood events, especially on navigation dams. Designs allowing submergence of 5 to 10 percent of the time are common. Gates incorporating a trunnion tie should not experience trunnion submergence. If other considerations do not control, it will usually be advantageous to locate the trunnion so that the maximum reaction is approximately horizontal to the trunnion girder (typically about one-third the height of the gate above the sill for hydrostatic loading). This will allow for simplified design and construction by allowing the trunnion posttensioned anchorage to be placed in horizontal layers.

(5) Operating equipment location. The type and position of the gate lifting equipment can have a significant effect on gate forces as the gate is moved through its range of motion. As stated previously, the two gate lifting systems recommended for new construction are the wire rope hoist system and the hydraulic hoist system.

(a) Wire rope hoist system. Generally, the most suitable layout for wire rope is one that minimizes the effects of lifting forces on the gate and lifting equipment. The three possible variations in cable layout include: 1) cable more than tangent to the skin plate, 2) cable tangent to the skin plate, and 3) cable less than tangent to the skin plate (Figure 3-8). Considering the gate and hoist system, the most ideal configuration is when the rope is pulled vertically and is tangent to the arc of the skin plate. For this condition, horizontal forces exerted to the hoist equipment are insignificant, and rope contact forces act radially on the gate. A nonvertical wire rope introduces a horizontal component of force that must be balanced by the operating equipment and associated connections. With a rope in the more-than-tangent condition, an edge reaction force exists at the top of the skin plate due to an abrupt change in rope curvature. This force affects the rope tension, trunnion reaction, and rib design forces. If the rope is in the less-than-tangent configuration, the rope force required to lift the gate increases exponentially as the direction of rope becomes further from tangent. The large lifting force affects the hoist and gate. Due to various constraints, some compromise on location of the hoist is usually required. Many gates have non-vertical wire ropes and many gates include ropes that are nontangent at or near the full, closed and/or full, opened positions.

(b) Hydraulic cylinder hoist system. Many new gate designs utilize hydraulic cylinder hoist systems because they are usually cost effective. However, these systems have some disadvantages and are not suited for all applications. Close coordination with the mechanical design engineer is required to optimize the hoist system. A hydraulic cylinder hoist system generally comprises two cylinders, one located at each side of the gate. Each cylinder pivots on a trunnion mounted on the adjacent pier, and the piston rod is attached to the gate. The cylinder magnitude of force and its orientation will change continually throughout the range of motion. In determining the optimum cylinder position, the location of the cylinder trunnion and piston rod connection to the gate are interdependent. Generally, the piston rod connection position is selected and then the cylinder trunnion position is determined to minimize effects of lifting forces. For preliminary design layouts, it is often assumed that the cylinder will be at a 45-deg angle from horizontal when the gate is closed, although optimization studies may result in a slightly different orientation. Generally, the most suitable location for the piston rod connection is on the gate end frame at or near the intersection of a bracing member and strut. It is preferable to have the piston rod connection above tailwater elevations that are consistent with the gate operating versus tailwater stage schedule; however, partial submergence may be acceptable for navigation projects. The connection location influences the gate trunnion reaction forces due to simple static equilibrium. When the connection is located upstream of the gate center of gravity, the dead load reaction at the gate trunnion will be downward while the gate is lifted off the sill. However, if the connection is downstream of the center of gravity, the reaction at the gate trunnion will act upward while the gate is lifted off the sill.

(6) Other sizing considerations. The face of gate and the stop log slots should be located far enough apart to permit the installation of maintenance scaffolding. Spillway bridge clearance is a factor in determining the gate radius and the trunnion location. Operating clearances from the bridge and the location of the hoist will usually require that the sill be placed somewhat downstream from the crest, but this distance should be as small as possible to economize on height of gate and size of pier. Additional considerations could include standardization of gate sizes on a project involving multiple spillways. The standardization of sizes could result in savings by eliminating multiple sets of bulkheads, standardizing machinery, and reducing stored replacement parts, etc.

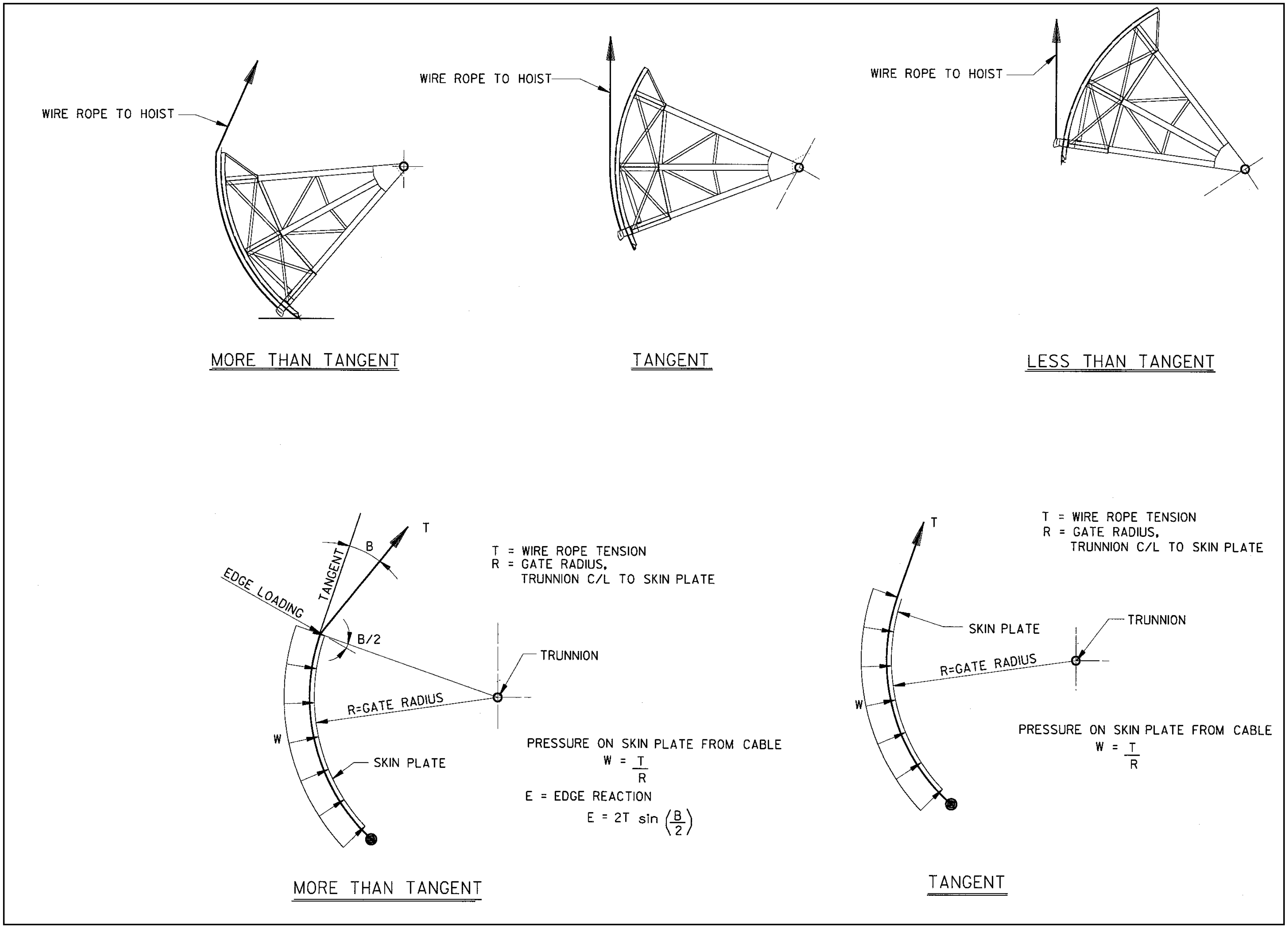


Figure 3-8. Loads due to various wire rope configurations

3-3. Material Selection

Structural members shall be structural steel. Embedded metals, including the side and bottom seal plates, should be corrosion-resistant stainless steel. Material selection for trunnion components is discussed in Chapter 4, and considerations for corrosion are discussed in Chapter 8. Table 3-1 provides a general reference for material selection of various tainter gate components, including American Society for Testing and Materials (ASTM) standards, given normal conditions. Material selection recommendations for various civil works structures including tainter gates is provided by Kumar and Odeh (1989).

Table 3-1
Tainter Gate Component Material Selection

Component	Material Selection
Skin plate, girders, trunnion girders, lifting bracket, wear plates, end frames	ASTM A36 or ASTM A572 Steel
Trunnion pin	ASTM A705, type 630 condition H 1150 steel forging[1,2] stainless steel[3] ASTM A27 or A148 cast steel
Trunnion bushing	ASTM B148 aluminum bronze ASTM B 22 manganese bronze or leaded tin bronze Self-lubricating bronze[4]
Trunnion hub	ASTM A27 cast steel ASTM A668 steel forging[2]
Trunnion yoke	ASTM A27 cast steel structural steel weldment
Seal plates and bolts	304 stainless steel
Lifting rope	308 stainless steel
J-seal keeper plate	410 stainless steel galvanized steel
Posttensioning anchorage steel	ASTM A772 Steel
Reinforcing steel	ASTM A615 grade 60 steel
Deflector plates	Ultra high molecular weight polyethylene

[1] Pin may be clad with corrosion resistant steel.
[2] If welded, carbon content not to exceed 0.35 percent.
[3] Type of stainless steel should be resistant to galling and crevice corrosion.
[4] Use with stainless steel pin.

3-4. Design Requirements

Tainter gate structural components shall be designed based on load and resistance factor design (LRFD) principles per EM 1110-2-2105. 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. (See EM 1110-2-2105 and AISC (1994) for a more complete description of LRFD.)

a. Design basis. The basic safety check in LRFD may be expressed mathematically as

$$\Sigma \gamma_i Q_{ni} \leq \alpha\phi R_n \tag{3-1}$$

where

γ_i = load factors that account for variability in the corresponding loads

Q_{ni} = nominal load effects defined herein

α = reliability factor as defined in EM 1110-2-2105

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.

For the appropriate limit states (paragraph 3-4.c), all structural components shall be sized such that the design strength $\alpha\phi R_n$ is at least equal to the required strength $\Sigma\gamma_i Q_{ni}$. The design strength shall be determined as specified in paragraph 3-4.c. The required strength must be determined by structural analysis for the appropriate load combinations specified in paragraph 3-4.b.

b. Load requirements.

(1) Loads. Loads that are applicable to tainter gate design include gravity loads, hydrostatic loads, operating loads, ice and debris loads, and earthquake loads. Reactions are not listed below or in the load cases. Reactions loads are not factored since they are determined from equilibrium with factored loads applied. As a result, reaction forces reflect the load factors of the applied loads.

(a) Gravity loads. Gravity loads include dead load or weight of the gate (D), mud weight (M), and ice weight (C), and shall be determined based on site-specific conditions.

(b) Hydrostatic loads. Hydrostatic loads consist of hydrostatic pressure on the gate considering both upper and lower pools. Three levels of hydrostatic loads are considered. The maximum hydrostatic load H_1 is defined as the maximum net hydrostatic load that will ever occur. The design hydrostatic load H_2 is the maximum net hydrostatic load considering any flood up to a 10-year event. The normal hydrostatic load H_3 is the temporal average net load from upper and lower pools, i.e., the load that exists from pool levels that are exceeded up to 50 percent of the time during the year.

(c) Gate lifting system (operating machinery) loads. Operating machinery is provided to support gates during lifting or lowering operations. Under normal operating conditions, the machinery provides forces necessary to support the gate, and for the load cases described herein, these forces are treated as reaction forces. Loads Q are machinery loads for conditions where the machinery exerts applied forces on an otherwise supported gate (paragraph 3-4.b(2)(f)). There are three levels of loads applied by the operating machinery to the gate. The hydraulic cylinder maximum downward load Q_1 is the maximum compressive downward load that a hydraulic hoist system can exert if the gate gets jammed while closing or when the gate comes to rest on the sill. The hydraulic cylinder at-rest load Q_2 is the downward load that a hydraulic cylinder exerts while the gate is at rest on the sill (due to cylinder pressure and the weight of the piston and rod). Loads Q_1 and Q_2 do not exist for wire rope hoist systems. The maximum upward operating machinery load Q_3 is the maximum upward load that can be applied by the wire rope or hydraulic hoist systems when a gate is jammed or fully opened. The gate lifting systems exert forces on specific gate members whether the forces are reactions or applied loads. For example, where the wire rope bears on the skin plate, the rope exerts a contact pressure (line load) on the skin plate. The contact pressure force is equal to the rope tension force divided by the gate radius. If the wire rope is not tangent to the skin plate, the rope will exert an additional concentrated load on the gate (Figure 3-8.). Concentrated forces that typically vary with gate position in magnitude and direction are present at the attachment points for both gate lifting systems. Operating machinery loads must be quantified in consultation with the mechanical engineer that designs the machinery. Determination of load magnitudes and suggested coordination are discussed in Chapter 7.

(d) Ice-impact load *I*. The ice-impact load is specified to account for impact of debris (timber, ice, and other foreign objects) or lateral loading due to thermal expansion of ice sheets. Additionally, this load provides the overall structure with a margin of safety against collapse under barge impact. (Barge impact is an accidental event that is not practical to design for and is not specifically considered in design). *I* is specified as a uniform distributed load of 73.0 KN/M (5.0 kips/ft) that acts in the downstream direction and is applied along the width of the gate at the upper pool elevation.

(e) Side-seal friction load F_s. Loads exist along the radius of the skin plate because of friction between the side seals and the side-seal plate when the gate is opening or closing. The friction force per unit length along the skin plate edge dF_s/dl is equal to the product of the coefficient of friction and normal force between the seal plates and the side seals. For rubber seals, a coefficient of friction (μ_s) equal to 0.5 is recommended. (Seals that have Teflon rubbing surfaces provide a lower coefficient of friction and are recommended for serviceability. However, wear of the Teflon is a concern, and applying a lower coefficient of friction for design purposes is not recommended.) The normal force on the side seal is a function of the preset force in the seal and the hydrostatic pressure on the surface of the seal. For normal tainter gate configurations, side-seal friction can be approximated by Equation 3-2 (Figure 3-9).

$$F_s = \mu_s S\, l + \mu_s \gamma_w \frac{d}{2}\left(l_1 \frac{h}{2} + h\, l_2 \right) \tag{3-2}$$

where

μ_s = coefficient of side-seal friction

l = total length of the side seal

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

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

S = force per unit length induced by presetting the seal and can be approximated as

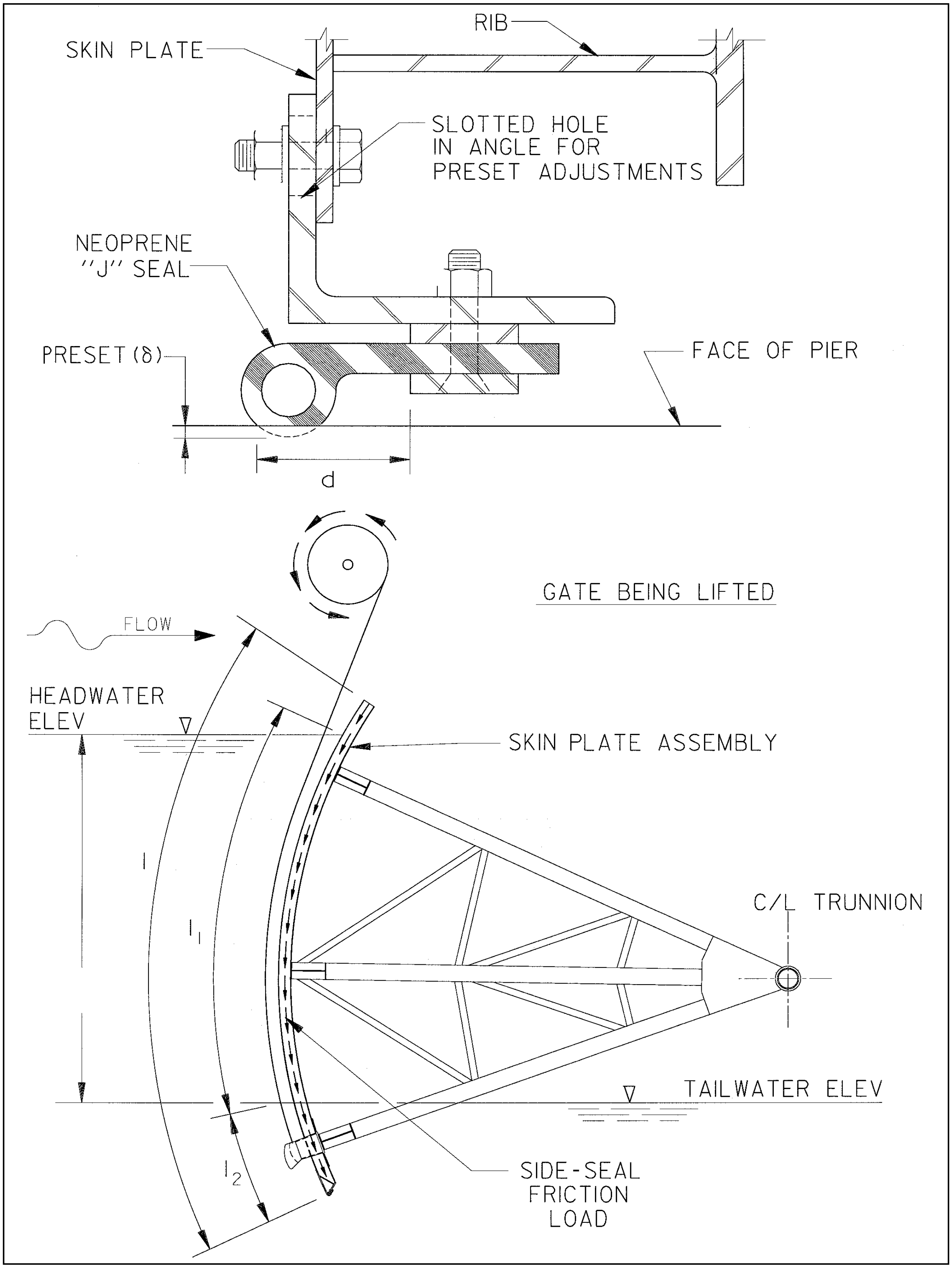


Figure 3-9. Standard side-seal arrangement and friction load

$S = \dfrac{3\delta EI}{d^3}$, where δ is the seal preset distance

γ_w = unit weight of water

d = width of the J seal exposed to upper pool hydrostatic pressure (Figure 3-9)

h = vertical distance taken from the headwater surface to the tail water surface or the bottom of the seal if there is no tailwater on the gate (Figure 3-9)

(f) Trunnion pin friction loads F_t. During opening or closing of gates, friction loads exist around the surface of the trunnion pin between the bushing and the pin and at the end of the hub between the hub bushing and side plate (yoke plate for yoke mounted pins) (Refer to Chapter 4 for description of trunnion components.). These friction loads result in a trunnion friction moment F_t about the pin that must be considered in design. The friction moment around the pin is a function of a coefficient of friction, the trunnion reaction force component R that acts normal to the surface of the pin (parallel to the pier face), and the radius of the pin. The friction moment at the end of the hub is a function of a coefficient of friction, the trunnion reaction force component R_z that acts normal to the end of the pin (normal to the pier face), and the average radius of the hub. The reaction forces R and R_z are discussed in paragraphs 3-5.a(2)(c) and 3-5.a(3). A coefficient of friction of 0.3 is an upper bound for design purposes. This is a conservative value that applies for any bushing material that may be slightly worn or improperly maintained. A realistic coefficient of friction for systems with lubricated bronze or aluminum bronze bushings is 0.1 to 0.15. The designer should ensure that detailed criteria are specified in project operations and maintenance manuals to ensure that trunnion systems are properly maintained.

(g) Earthquake design loads E. Earthquake design loads are specified based on an operational basis earthquake (OBE) with a 144-year mean recurrence interval. For gate design, the direction of earthquake acceleration is assumed to be parallel to the gate bay centerline (i.e., it is assumed that the effects of vertical acceleration and acceleration perpendicular to the gate bay are comparatively negligible). Earthquake forces include mass inertia forces and hydrodynamic forces of water on the structure. When a tainter gate is submerged, the inertial forces due to structural weight, ice, and mud are insignificant when compared with the hydrodynamic loads and can be ignored. For load case 1 (paragraph 3-4.b(2)(a)), the structure is submerged, and E shall be based on inertial hydrodynamic effects of water moving with the structure. For the structural member in question, E is determined based on the pressure that acts over the tributary area of the particular member. The hydrodynamic pressure may be estimated by Equation 3-3 (Westergaard 1931). This equation applies for water on the upstream and downstream sides of the structure.

$$p = \frac{7}{8}\gamma_w a_c \sqrt{Hy} \tag{3-3}$$

where

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

γ_w = unit weight of water

H = reservoir pool depth (to bottom of dam) on upstream or downstream side of the structure

a_c = maximum base acceleration of the dam due to the OBE (expressed as a fraction of gravitational acceleration)

For load case 5 (paragraph 3-4.b(2)(e)), water is not on the structure, and E is due to mass inertia forces of the structure, ice, and mud.

$$E = a_c W_{D,C,M} \tag{3-4}$$

where

$W_{D,C,M}$ = weight of the portion of the structure, ice, and mud that are supported by the member in question.

(h) Wave load W_A. Wave loads are site specific and should be determined in consultation with the project hydraulic engineer. Guidance on development of wave loading is provided in the Shore Protection Manual (1984).

(i) Wind load W. Wind loads are site specific and should be calculated in accordance with ASCE (1995) but not more than 2.4 KPa (50 psf). Wind loads are small when compared to hydrostatic loads and only affect gate reactions when the gate is in an open position.

(2) Load cases. Tainter gates shall be designed considering the strength requirements for each of the following load cases and corresponding load combinations. The most unfavorable effect on various gate components may occur when one or more of the loads in a particular load combination is equal to zero. Various conditions are described for the cases of gate in the closed position (load case 1), gate operating (load case 2 and load case 3), gate jammed (load case 4), and gate fully opened (load case 5). The operating machinery may include forces in each load case; however, these forces are treated as gate reactions in some cases. As a result, the load Q does not appear in some load cases (paragraph 3-4.b(2)(f)).

(a) Load case 1: Gate closed. Load combinations for this load case (Equations 3-5, 3-6, and 3-7) apply when the gate is in the closed position.

$$1.4 H_1 + 1.2D + 1.6(C + M) + 1.2 Q_2 \tag{3-5}$$

$$1.4 H_2 + 1.2D + 1.6(C + M) + [1.2 Q_1 \text{ or } (1.2Q_2 + 1.2W_A) \text{ or } (1.2Q_2 + k_I I)] \tag{3-6}$$

$$1.2 H_3 + 1.2D + 1.6(C + M) + 1.0E \tag{3-7}$$

1) Extreme pool condition. Equation 3-5 describes the condition where the maximum hydrostatic load H_1 is applied, gravity loads D, C, and M exist, and the hydraulic cylinders exert the at-rest load Q_2. (For gates with wire rope hoists, Q_2 does not exist.) It is assumed that the likelihood of the simultaneous occurrence of H_1 and W_A or I or E is negligible.

2) Operating pool condition. Equation 3-6 describes the condition for which the moderate hydrostatic load H_2 acts in combination with Q_1 or W_A or I. Gravity loads D, C, and M always exist, and it is assumed that Q_1, W_A, and I will not occur at the same time. The load Q_2 will likely always exist with W or

I. The I load factor k_I shall be equal to 1.6 when considering ice load due to thermal expansion and 1.0 when considering impact of debris.

3) Earthquake condition. Equation 3-7 describes the condition where the normal hydrostatic load H_3 acts in combination with earthquake loading E and gravity loads D, C, and M exist.

(b) Load case 2: Gate operating with two hoists. Load combinations for this load case (Equations 3-8 and 3-9) represent the condition when the gate is opening or closing with both hoists functional. Operating machinery loads Q are not listed in these load combinations, because it is assumed that the hoists are gate supports that will include reaction forces (paragraph 3-4.b(2)(f)). This load case does not include E because it is assumed that the likelihood of opening or closing the gate at the same time an earthquake occurs is negligible. Effects on members forces should be checked considering the entire operating range (any position between the sill and the upper gate stops) for gates either opening or closing.

$$1.4H_1 + 1.2D + 1.6(C+M) + 1.4F_S + 1.0F_t \tag{3-8}$$

$$1.4H_2 + 1.2D + 1.6(C+M) + 1.4F_S + 1.0F_t + (1.2W_A \text{ or } k_I I) \tag{3-9}$$

1) Extreme pool condition. Equation 3-8 (similar to Equation 3-5) describes the condition where the maximum hydrostatic load H_1 is applied, gravity loads D, C, and M exist, and friction loads F_s and F_t exist due to gate motion. It is assumed that the likelihood of the simultaneous occurrence of H_1 and W_A or I or E is negligible.

2) Operating pool condition. Equation 3-9 (similar to Equation 3-6) describes the condition for which the moderate hydrostatic load H_2 acts in combination with W_A or I. Gravity loads D, C, and M exist, and due to gate motion, friction loads F_s and F_t are applied. In the determination of H_2, the lower pool elevation will include a hydrodynamic reduction due to flow of water under the gate. The magnitude of hydrodynamic reduction should be determined in consultation with the project hydraulic engineer. As described for Equation 3-6, k_I shall be equal to 1.6 for ice and 1.0 for debris.

(c) Load case 3: Gate operating with one hoist. The load combination described by Equation 3-10 applies for the case where the gate is operated with only one hoist (subsequent to failure of the other hoist). Effects on members forces should be checked for all gate positions throughout the operating range for the gate either opening or closing.

$$1.4H_2 + 1.2D + 1.6(C+M) + 1.4F_S + 1.0F_t \tag{3-10}$$

For this case, the moderate hydrostatic load H_2 is applied, gravity loads D, C, and M exist, and friction loads F_s and F_t exist due to gate motion. In the determination of H_2, the lower pool elevation will include a hydrodynamic reduction due to flow of water under the gate (must be coordinated with the project hydraulic engineer). The effects of H_1, W_A, I, or E are not considered for this load case, since it is assumed that the likelihood of the simultaneous occurrence one of these loads and the failure of one hoist is negligible. Design considerations for this condition are provided in paragraph 3-5.e and serviceability considerations are discussed in paragraph 3-6.b(1).

(d) Load case 4: Gate jammed. The load combination of Equation 3-11 accounts for the possible condition where one hoist fails, and the gate becomes jammed between piers due to twist of the gate (such that one end frame is higher than the other). Effects on members forces should be checked with the gate jammed at positions throughout the operating range.

$$1.4\,H_2 + 1.2D + 1.6(C + M) + (1.2\,Q_3 \text{ or } 1.2\,Q_1) \tag{3-11}$$

For this case, the moderate hydrostatic load H_2 is applied in combination with the maximum machinery load Q_3 (for wire rope hoists or hydraulic hoists) or Q_1 (for hydraulic hoists), and gravity loads D, C, and M exist. It is assumed that only one hoist is functional. In the determination of H_2, the lower pool elevation will include a hydrodynamic reduction due to flow of water under the gate (must be coordinated with the project hydraulic engineer). The effects of H_1, W_A, I, or E are not considered for this load case, since it is assumed that the likelihood of the simultaneous occurrence one of these loads while the gate is jammed is negligible.

(e) Load case 5: Gate fully opened. The load combination of Equation 3-12 accounts for the condition where the gate is fully opened (raised to the gate stops) with wind, earthquake, or operating equipment loads.

$$k_D D + 1.6(C + M) + (1.3W \text{ or } 1.0E \text{ or } 1.2\,Q_3) \tag{3-12}$$

For this case, it is assumed that the gate is raised above the pool, so effects of H, W_A, and I are not included. Effects of Q_3 oppose gravity load effects, and effects of W or E may add to or oppose gravity load effects. When Q_3 is considered, or when effects W or E oppose those of gravity, C and M should be equal to 0 and the load factor k_D is equal to 0.9. When the direction of W or E is such that their effect increases gravity load effects, k_D is equal to 1.2 and C and M should be considered.

(f) Assumptions on support or loading of gate lifting systems (operating machinery). The load combinations included herein were developed for gate design based on various assumptions on loading and support conditions. (See Chapter 7 for discussion on criteria regarding load and operational requirements for operating machinery.) These assumptions must be considered in structural analysis of gate components (paragraph 3-5). For a tainter gate to be stable, rotational restraint must be provided by a suitable support. It is assumed that this support is the sill when the gate is closed (case 1), the hoists when the gate is operated (case 2 and case 3), the pier or some obstruction at the gate sides when the gate is jammed (case 4), and the hoists or the gate stops when the gate is fully opened (case 5). In load cases 2, 3, and some applications of 5, operating machinery loads Q are not included, since for these cases, the hoists are supports. The hoists provide reaction forces which are a function of all other gate loads. In load cases 1, 4, and some applications of 5, where the gate is supported by something other than the hoists, any force exerted by the hoists is an external load on the gate Q.

c. Limit states and design strength for individual members. EM 1110-2-2105 requires that strength and serviceability limit states be considered in the design of tainter gate structural components. Strength limit states include general yielding, instability, fatigue, and fracture. The design strength for each applicable limit state $\alpha\phi R_n$ is calculated as the nominal strength R_n, multiplied by a resistance factor ϕ and a reliability factor α. Except as modified herein, limit states, nominal strength R_n, and resistance factors ϕ shall be in accordance with AISC (1994). For normal conditions, α shall be equal to 0.9. For gates that are normally submerged and whose removal would disrupt the entire project, or for gates in brackish water or sea water, α shall be equal to 0.85. Fatigue and fracture design requirements are included in paragraph 3-8 and EM 1110-2-2105. Serviceability requirements for tainter gates are specified in paragraph 3-6. The following paragraphs provide specific guidance for the design of skin plate, ribs, girders, and end frame members.

(1) Skin plate. The skin plate shall be sized such that the maximum calculated stress is less than the yield limit state of $\alpha\phi_b F_y$. In determining the required strength, all load cases in paragraph 3-4.b shall be

considered; however, the ice-impact load I may be set equal to zero at the designer's option. This is frequently done, with the intent of allowing local damage to the skin plate for very infrequent loading events rather than increasing the gate weight.

(2) Rib members. Ribs shall be sized such that the maximum calculated moment M_u is less than the nominal bending strength of $\alpha\phi_b M_n$. In determining the required strength M_u, all load cases in paragraph 3-4.b shall be considered. For load cases 2, 3, and 4, the maximum effect will normally occur assuming that the gate is near the closed position.

(3) Girders. Girders shall be designed as beams or plate girders in accordance with AISC (1994). In determining the required strength, all load cases in paragraph 3-4.b shall be considered. For load cases 2, 3, and 4, the maximum effect will normally occur assuming that the gate is near the closed position.

(4) End frame. Struts and bracing members shall be designed as members under combined forces in accordance with AISC (1994). For gates not braced against side sway, struts shall be sized to avoid side sway frame buckling (lateral buckling of gate toward pier face, see paragraph 3-5.a(2)). In determining the required strength, all load cases in paragraph 3-4.b shall be considered.

(5) Secondary bracing members. The minimum axial design load in all bracing members shall be 2 percent of the total axial compression force or of the flexural compressive force in the compression flange of the corresponding braced member. Trunnion hub flange plates shall have adequate design strength to resist the required flexural and axial loads between the struts and the trunnion hub.

3-5. Analysis and Design Considerations

This paragraph includes guidance on design of tainter gate structural components (Chapters 4 through 6 provide guidance for the design of the trunnion and trunnion girder). The design and behavior of individual structural components are interrelated. The gate design should be optimized to achieve the most economical design overall, not necessarily to provide the most efficient geometry and member sections for each component. A large percentage of total gate cost is associated with the fabrication of the skin plate assembly. Therefore, the design process should be tailored to minimize the cost of the skin plate assembly considering the most normal load conditions. The required strength (design forces) and deflections for all structural components must be determined by structural analysis. Three-dimensional (3-D) analyses or more approximate (generally conservative) two-dimensional (2-D) analyses may be conducted. All analytical models must include boundary conditions that are consistent with load requirements specified in paragraph 3-4.b. Paragraph 3-5.a includes a description of acceptable 2-D models (various modeling alternatives exist), and the remainder of paragraph 3-5 provides general design considerations.

a. Two-dimensional analytical models. In the design of tainter gate structural members, it has been common practice to model the 3-D behavior with several independent 2-D models. With the 2-D approach, the overall behavior is simulated by modeling separately the behavior of the skin plate assembly (composed of the skin plate and supporting ribs), horizontal girder frames (composed of a horizontal girder and the two adjacent struts), vertical end frames (composed of struts and braces), and the vertical downstream truss. Analysis of the 2-D models is interdependent. Various loads on one model can be reactions from another (girder frame loads are obtained from the rib model reactions), and many of the same loads are applied to each model. Additionally, struts include member forces from separate models (the strong axis flexural behavior of the struts is simulated with the girder frame model, and axial and weak axis flexural forces are provided by the end frame model). An alternative for each 2-D model is described in the following subsections. In the discussion for each model, loads for all load conditions are described.

For design analysis, load combinations and associated load factors should be applied accordingly. Various loads that are applied to the models (such as the gate reaction loads including sill load R_S and the wire rope pressure load R_Q/r) are not factored, since they are a function of other factored loads.

(1) Skin plate assembly. For the 2-D approximate model, the skin plate and ribs are assumed to have zero curvature. The skin plate serves two functions. First, each unit width of skin plate is assumed to act as a continuous beam spanning the ribs in the horizontal direction (Figure 3-10). Second, the skin plate acts as the upstream flange of the ribs. The ribs, with the skin plate flange, are continuous vertical beams that are supported by the horizontal girders (Figure 3-11). A portion of the skin plate is considered to act as the upstream flange of the rib (paragraph 3-5.b(2)).

(a) Boundary conditions. Boundary conditions consist of simple supports located at each rib for the skin plate and at each girder for the rib members.

(b) Loads. Loading on the skin plate assembly consists of various combinations of factored loads and gate reaction loads. Factored loads consist of $1.2H$ or $1.4H$, $1.2\ W_A$, $1.0E$, $k_I I$, and $1.2Q_3/r$, and gate reaction loads include R_s and R_Q/r. H, W_A, E, I, and Q_3 are defined in paragraph 3-4.b, r is the gate radius, R_s is the sill reaction load at the bottom of the skin plate (load case 1 gate closed), and R_Q is the wire rope reaction load (for load cases 2 and 3 for gates with a wire rope hoist). The factored loads shall be applied in accordance with paragraph 3-4.b(2). Gate reaction loads R_S and R_Q/r are determined by equilibrium of the end frame model for each appropriate load condition (paragraph 3-5.a(3)) and are not factored since they are a function of other factored loads. For skin plate design, loads are determined based on a unit width of plate, and it is not considered necessary to include I and R_S. For the rib model, the magnitude for loads is determined based on the tributary area of the rib. For each rib, the tributary portion of R_S is resolved into a radial component that is applied as a concentrated load at the end of the rib cantilever, and a tangential component that is applied as an axial force. Although not applied directly, all loads described in paragraph 3-4.b can affect rib member forces, since the reaction forces at the sill or under the wire rope include the effects of trunnion friction F_t, hydraulic machinery loads Q_1 and Q_2, and gravity loads C, M, and D as defined in paragraph 3-4.b.

(c) Results. Analysis of the skin plate model will yield the calculated stresses and out-of-plane deflections necessary to size the skin plate in accordance with paragraphs 3-4.c(1) and 3-6. Analysis of the rib model yields the calculated moments necessary for rib design in accordance with paragraph 3-4.c and reactions are the girder loading for the girder frame model.

(2) Girder frame model. The 2-D analytical model is a single-story frame consisting of beam members that simulate the horizontal girder and two columns that represent the corresponding end frame struts (Figure 3-12). The strong axes of the struts are oriented to resist flexural forces in the plane of the frame. The model shown in Figure 3-12a applies for all load cases described in paragraph 3-4.b except the unsymmetric load cases 3 and 4. For these cases, the analytical model should include applied loads with an imposed lateral displacement to represent side sway of the frame (Figure 3-12b). For load case 3, the lateral displacement should be consistent with the expected displacement as determined by an appropriate analysis (paragraph 3-5.e). For load case 4, the maximum displacement (lateral displacement to the pier face) should be applied. For all load cases except load case 4, the frame is not braced against side sway and the analysis should be conducted accordingly (AISC (1994), Chapter C). For load case 4, the frame can be assumed to be braced with the maximum displacement applied.

(a) Boundary conditions. The supports for the structure are at the bottom of the columns or struts and should be modeled to simulate actual conditions. Where cylindrical pins are used at the trunnion, a fixed

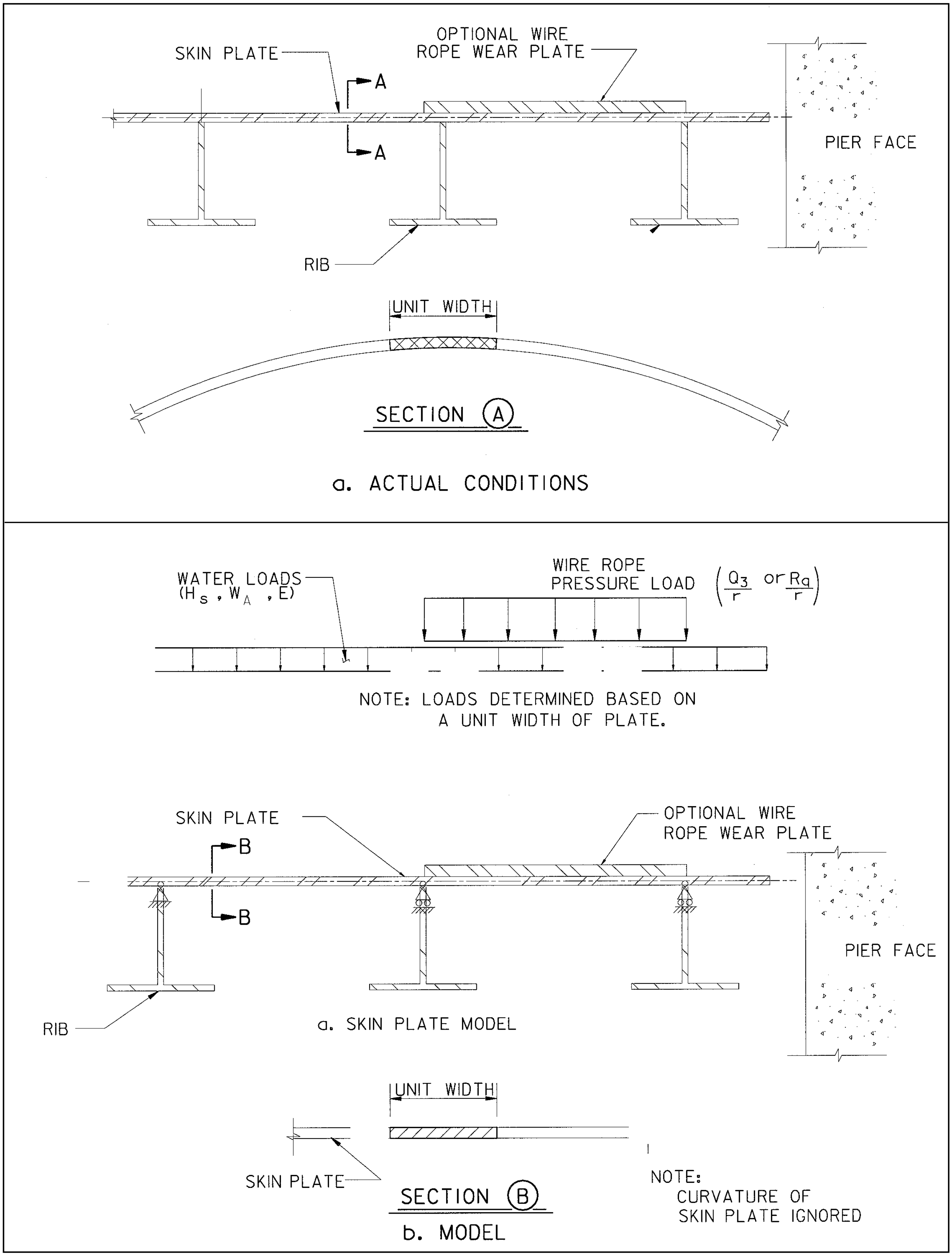


Figure 3-10. Skin plate 2-D design model

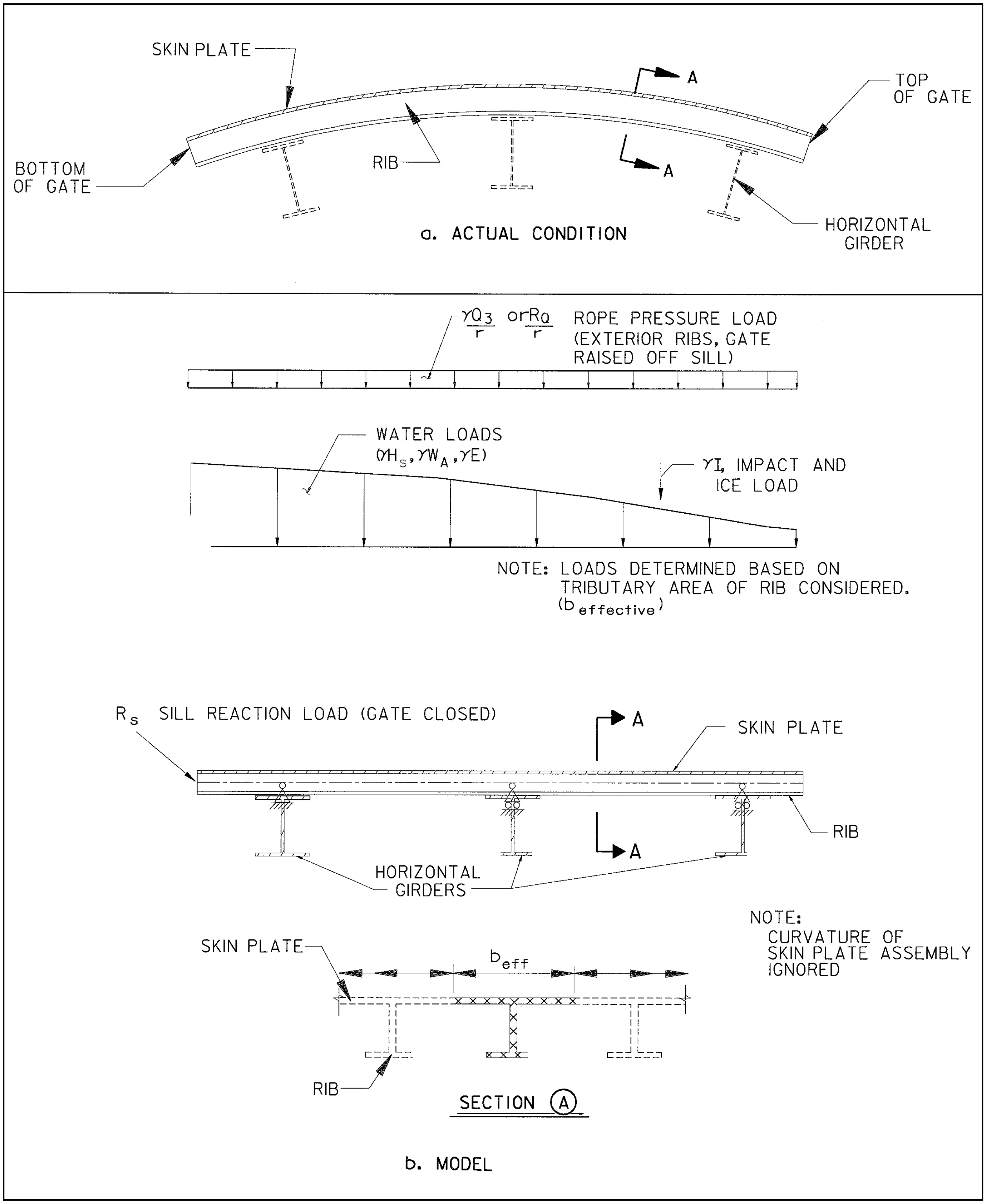


Figure 3-11. Skin plate assembly with ribs (2-D design model)

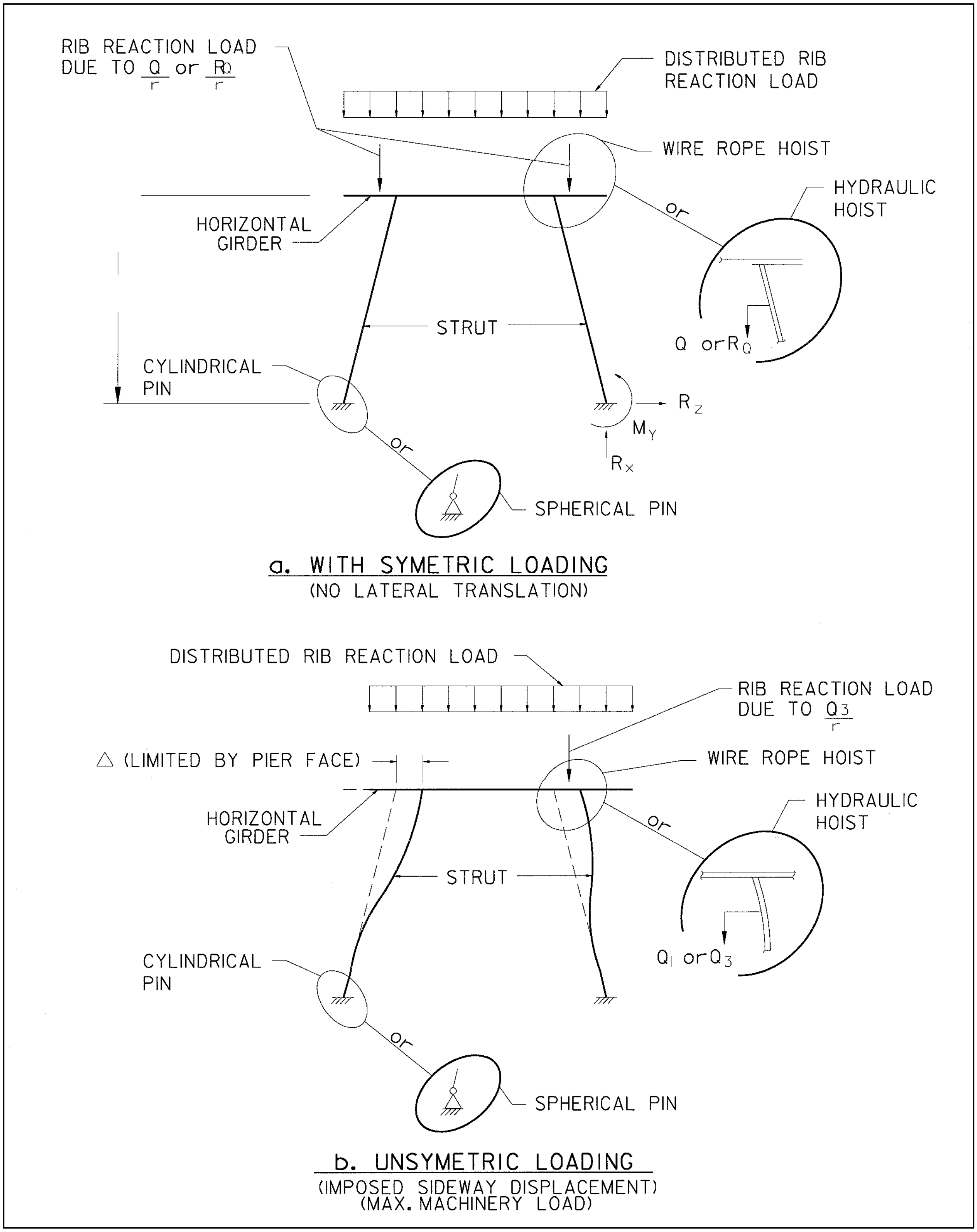


Figure 3-12. Girder model loads and boundary conditions

support should be assumed and where spherical bearings are used at the trunnion, a pinned support should be assumed.

(b) Loads. The girders support the skin plate assembly and loads to the girder are applied through the ribs. Therefore, for each load combination, girder loading is based on reaction forces from the rib model. A uniformly distributed load equivalent to the rib reactions distributed over each tributary area is applied along the length of the girder. This distributed load does not require a load factor, since the rib reactions are a function of factored loads. For gates with wire rope hoists, a concentrated load equal to the reaction for the rib that supports the wire rope should be applied to the girder at the corresponding rib location. The concentrated load is due to a distributed rib load equal to $1.2Q/r$ for load cases 4 and 5 or R_Q/r for load cases 2 and 3 (R_Q is determined by the end frame model described in paragraph 3-5.a(3)). For hydraulic hoists, the cylinder force load should be applied at the cylinder connection location (for most cases on the strut near the girder). This force is equal to $1.2Q$ for load cases 1, 4, and 5 or the cylinder force as determined by the end frame model analysis for load cases 2 and 3. All loads described in paragraph 3-4.b affect the girder frame member forces since components of each load are transferred through the ribs. It is assumed that girder lateral bracing resists girder torsional forces that are caused by gravity loads.

(c) Results. The girder frame analysis results include all design forces and deflections for the girder, flexural design forces about strong axis of the struts, and reactions that simulate lateral thrust R_z into the pier and moment at the trunnion M_y (Figure 3-12). The lateral thrust force R_z induces friction forces that are a component of trunnion friction moment F_t as discussed in paragraph 3-4.b(1)(f). The effect of R_z on F_T should be considered in the analysis of the end frame model. For gates with parallel end frames, the effect of R_z may be negligible. However, R_z is more significant for gates with inclined end frames, since R_z includes a component of the strut axial loads.

(3) End frame model. The analytical model for the end frame consists of elements to simulate struts and strut bracing, girders (webs), girder lateral bracing, and the skin plate assembly (Figure 3-13). Struts are modeled with frame elements, and bracing members are modeled with either frame or truss elements to be consistent with connection details. The elements that represent the skin plate assembly and girder webs are included in the model only to transfer loads and to maintain correct geometry. These elements should be relatively stiff compared to other elements. The girder members should be simulated by truss elements so the girder lateral bracing elements resist all forces transverse to the girder. (This will ensure that bracing is proportioned so that girder torsion is limited.) The assumed boundary conditions, loading, and model geometry are shown by Figure 3-13 for: a) gate closed (load case 1); b) gate operating with wire rope hoist (load cases 2 and 3); c) gate operating with hydraulic cylinder hoist (load cases 2 and 3); and d) gate jammed (load case 4) or raised to stops (load case 5). Boundary conditions and loading described in the following paragraphs are based on assumptions discussed in paragraph 3-4.b(2). The purpose of the end frame model is: a) to determine the sill reaction load R_S, operating machinery reaction load R_Q, and trunnion reaction R; and b) to determine end frame member design forces.

(a) Boundary conditions. For each model described by Figure 3-13, the trunnion is modeled as a pin free to rotate with no translation. For the gate-closed case, the boundary conditions for gates with wire rope or hydraulic hoists are identical as shown by Figure 3-13a. The gate is supported by the trunnion (modeled as a pin) and the sill. The sill boundary condition consists of a roller-pin free to translate tangent to the sill. For a 3-D model, the boundary condition along the sill should resist compression only (i.e., allow for deformation near the center of a gate that will likely result between the sill and gate bottom). There is no boundary condition for the hoist; a hoist force is treated as an external load. For gate-operating cases (Figures 3-13b and 3-13c), the gate is supported by the trunnion (modeled as a pin) and the hoist. (For this case, the hoist force is a reaction and not a load as discussed in paragraph 3-4.b(2)(f)). For wire

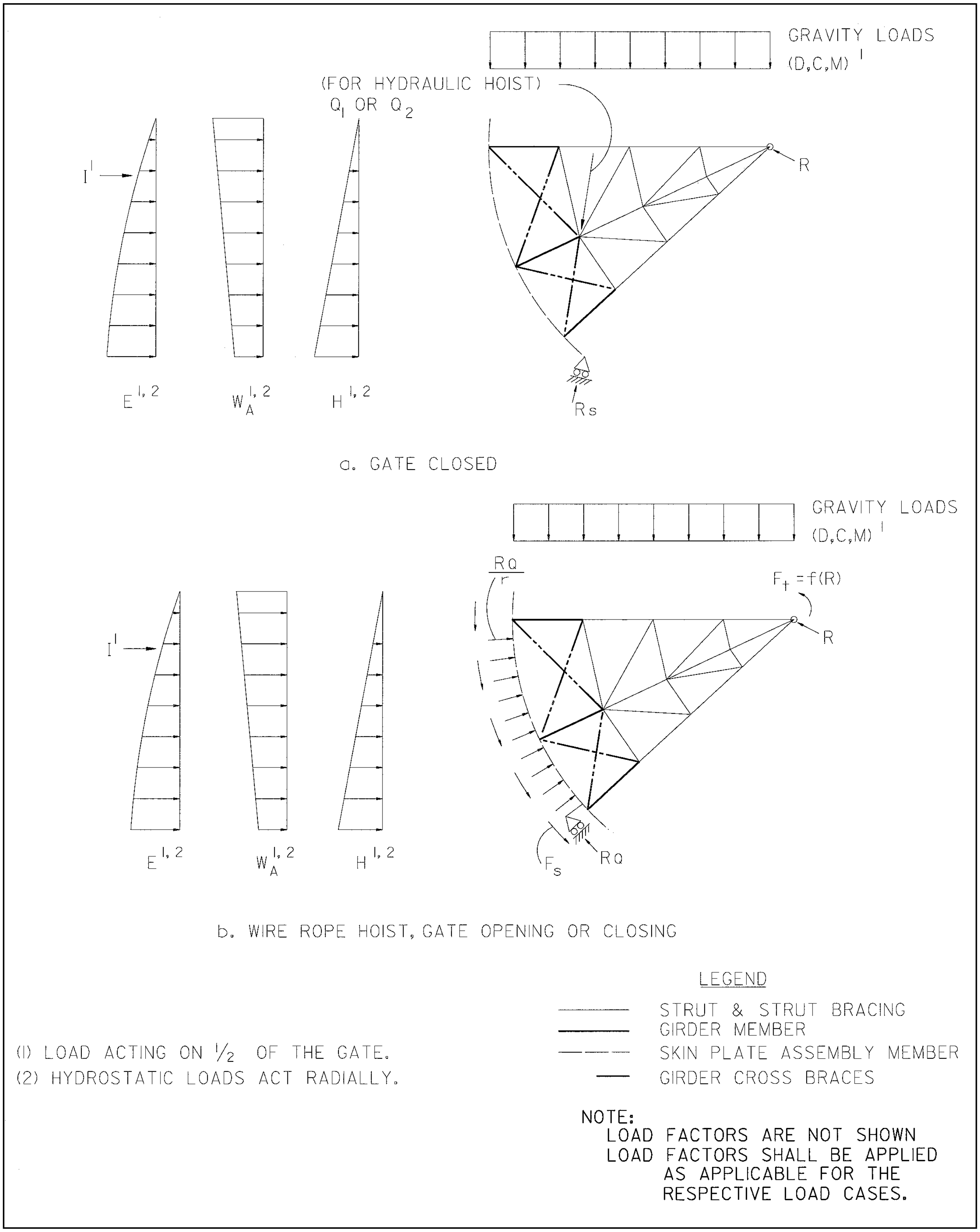
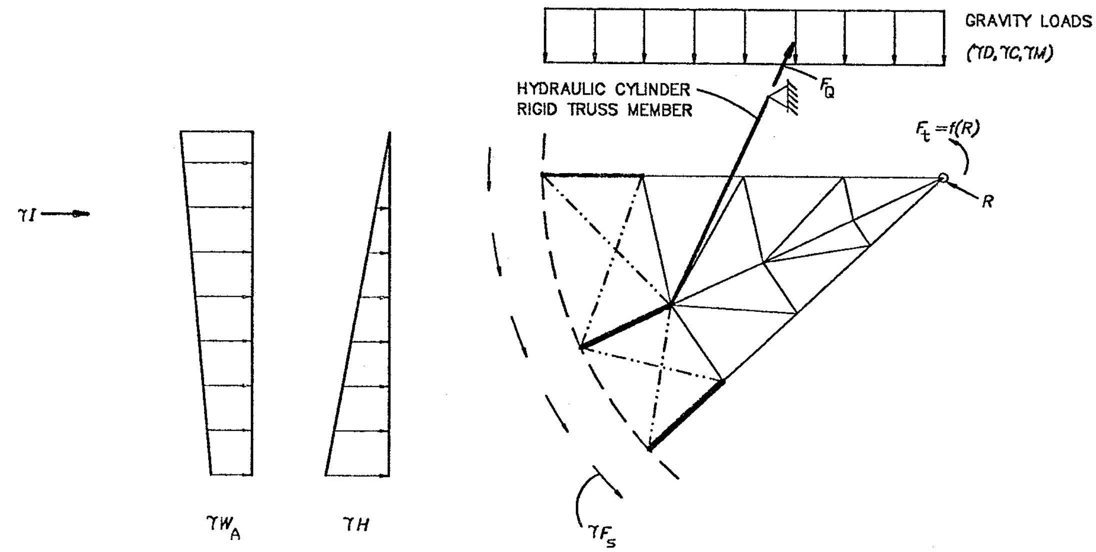
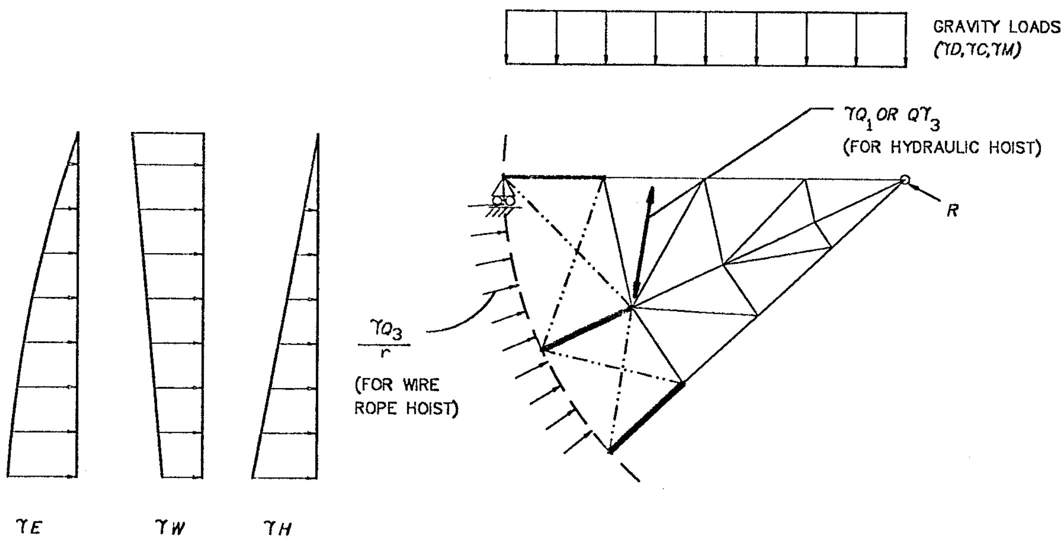


Figure 3-13. End frame 2-D model (Continued)



c. HYDRAULIC HOIST, GATE OPERATING (LOAD CASE 2 OR 3)



d. GATE JAMMED (LOAD CASE 4) OR FULLY OPEN (LOAD CASE 5)

NOTES:

(1) HYDROSTATIC LOADS ACT RADIALLY.

(2) GATE MAY BE ORIENTED ANYWHERE FROM CLOSED TO FULLY OPEN POSITION FOR THE GATE JAMMED OR GATE OPERATING CONDITION.

(3) FOR THE GATE OPERATING CASE, FRICTION LOADS F_t AND F_S ARE SHOWN FOR GATE OPENING. THESE LOADS ACT IN THE DIRECTION OPPOSITE TO THAT SHOWN FOR GATE CLOSING.

LEGEND

STRUT & STRUT BRACING
GIRDER MEMBER
SKIN PLATE ASSEMBLY MEMBER
GIRDER LATERAL BRACING

Figure 3-13. (Concluded)

rope hoists (Figure 3-13b), the boundary condition where the wire rope is attached to the skin plate assembly is modeled as a roller-pin support that is free to translate in the radial direction. For gates with a hydraulic hoist (Figure 3-13c), the cylinder is simulated by a rigid (very stiff) truss element positioned between the location of the cylinder connection to the end frame and the cylinder trunnion location. This is analogous to a roller-pin support located at the cylinder connection that is free to translate perpendicular to the hoist cylinder. (The cylinder connection to the end frame may include eccentricity out of the plane of the model depending on how the connection is detailed. If this is significant it should be accounted for in the design.) For the gate jammed or fully opened cases (Figure 3-13d), the gate is supported by the trunnion (modeled as a pin) and is restrained from rotation by the pier or some obstruction (gate jammed) or the gate stops (gate fully opened). The boundary condition to restrain rotation is a roller-pin support free to translate in the radial direction and located where the elements for the top girder and skin plate intersect or at another appropriate position along the skin plate.

(b) Loads. For each load case, the appropriate analytical model should include the combinations of factored loads as required by paragraph 3-4.b(2). With the gate closed, appropriate combinations of loads (1.2 or 1.4)H, 1.2D, 1.6(C + M), 1.2(Q_1 or Q_2), 1.2W_A, $k_I I$, and 1.0E are included as illustrated by Figure 3-13a. For wire rope systems, there is no force in the rope, so no operating machinery loads are included. For hydraulic hoist systems, the operating machinery force is treated as a load Q_1 or Q_2 since the machinery is not considered as a support. For load cases 2 and 3 (Figures 3-13b and 3-13c), loads include appropriate combinations of 1.4H, 1.2D, 1.6(C + M), 1.4F_s, 1.0F_t, 1.2W_A, and $k_I I$. Friction loads are present due to gate motion. The side-seal friction force F_s is applied along the perimeter of the skin plate, and trunnion friction moment F_t is applied at the pin support. Operating machinery loads Q (as defined in paragraph 3-4.b(1)) do not apply because the hoists serve as supports. However, for wire rope systems, the analysis must include the distributed load R_Q/r that acts radially wherever the rope contacts the skin plate. The wire rope tension is equal to the hoist reaction R_Q. Because external loads R_Q/r and the trunnion friction load F_t are a function of reaction forces R_Q and R (and the reactions are a function of external loads), a special analysis such as the iterative approach described in paragraph 3-5.a(3)(d) is necessary to determine the equilibrium state. As defined in paragraph 3-4.b(1)(f), the trunnion friction force F_t includes the effect of the end frame trunnion reaction R (parallel to the pier or abutment face) and the lateral strut reaction R_z (component perpendicular to the pier). R_z is estimated by analysis of the girder frame model. For load cases 4 and 5 (Figure 3-13d), loads include appropriate combinations of 1.4H, 1.2D, 1.6(C + M), 1.3W, 1.2Q_1 or 1.2Q_3, and 1.0E. For these cases, operating machinery loads Q are included since the hoists are not considered gate supports. For wire rope hoist systems, the analysis for load case 4 or load case 5 must include the factored distributed load 1.2Q_3/r which acts radially wherever the rope contacts the skin plate. For hydraulic hoist systems, the analysis should include the operating machinery load 1.2Q_1 or 1.2Q_3. (For load case 5, a downward load Q_1 may not be possible when the gate is fully opened, depending on gate arrangement.)

(c) Results. The end frame model provides strut weak axis flexural design forces, strut axial design forces, axial and flexural design forces for strut bracing, girder lateral bracing design forces, trunnion reaction forces, and operating equipment load requirements. The end frame model reaction forces R_s and R_Q are utilized in the other 2-D models as described in the previous sections.

(d) Iterative determination of reaction forces. For gate-operating cases in which external forces are a function of gate reactions, a special analysis such as iteration is necessary to determine forces and reactions because the reactions are a function of the external forces. Considering load case 2 or load case 3 with a wire rope hoist system, the trunnion friction moment F_t and distributed rope load R_Q/r (external loads) are a function of the trunnion reaction force R, and the hoist reaction force R_Q. A simple procedure to conduct the iteration is as follows:

1) Approximate the trunnion reaction R due to factored hydrostatic loading 1.4H and estimate the trunnion friction moment F_t as a function of R, R_z (determined from the girder frame analysis), the pin diameter, and coefficient of friction.

2) Determine the hoist reaction R_Q by equilibrium.

3) Recalculate the trunnion reaction R due to all appropriate factored loads (i.e., for load case 3, 1.4H, 1.2D, 1.6(C + M), 1.4F_s, 1.0F_t) including the reaction load R_Q/r.

4) Determine a modified trunnion friction moment F_t as a function of the modified reaction R, R_z (unchanged), pin diameter, and coefficient of friction.

5) Repeat steps 2 through 4 until the trunnion reaction R does not change significantly.

(4) Downstream vertical truss model. Bracing members that make up the downstream vertical truss are proportioned for forces that occur when the gate is supported at one end. To determine these forces accurately, a 3-D analysis is required because of the complex interaction of the skin plate assembly, end frames, and bracing members. However, various 2-D models can be used to conservatively approximate the forces. Figure 3-14 and the following paragraphs describe a recommended 2-D model. The simplified model shown in Figure 3-14b does not represent actual loading and support conditions but will provide approximate bracing forces for the simulated condition. Continuous frame elements simulate girder flanges, and the bracing members are represented by truss elements (include only axial forces). As stated in paragraph 3-2a(2)(b), out of plane geometry is ignored, and the truss members are assumed to lie in a single plane.

(a) Boundary conditions. With the analytical model described by Figure 3-14b, boundary conditions do not represent physical characteristics of the gate but provide geometric stability of the model. A roller-pin support free to translate in the horizontal direction is provided at the node where the shear load R_H is applied. This should be located at the end of the element that simulates the upper girder (near a bumper location or where the gate would contact the pier if rotation of the skin plate assembly were to occur). On the opposite corner of the model (opposite end of the lower girder), a pin support free to rotate with no translation is applied.

(b) Loads. Model loading (Figure 3-14b) consists of a horizontal shear load R_H. The magnitude of R_H as defined in Figure 3-14a is that required to maintain equilibrium with the gate subjected to vertical loads and suspended from one end. As described by the free body diagram of Figure 3-14a, the effects of factored loads 1.2D, 1.6(C+M), and 1.4F_s are included (simulates load case 3, paragraph 3-4.b(2)(c)). The trunnion friction load F_t and hydrostatic loads H, do not directly cause twisting of the gate and are not considered.

(c) Results. Truss members are to be sized to resist the calculated axial forces as determined by analysis of the downstream vertical truss model.

b. Skin plate assembly. The skin plate assembly consists of the skin plate and vertical ribs. Horizontal intercostals are not recommended since material savings realized in the design of the skin plate are offset by higher fabrication and maintenance costs. The designs of the skin plate and ribs are inter-related. The required skin plate thickness is dependent on the rib spacing (skin plate span), and the required rib size is dependent on the skin plate thickness since an effective portion of skin plate acts as a rib flange.

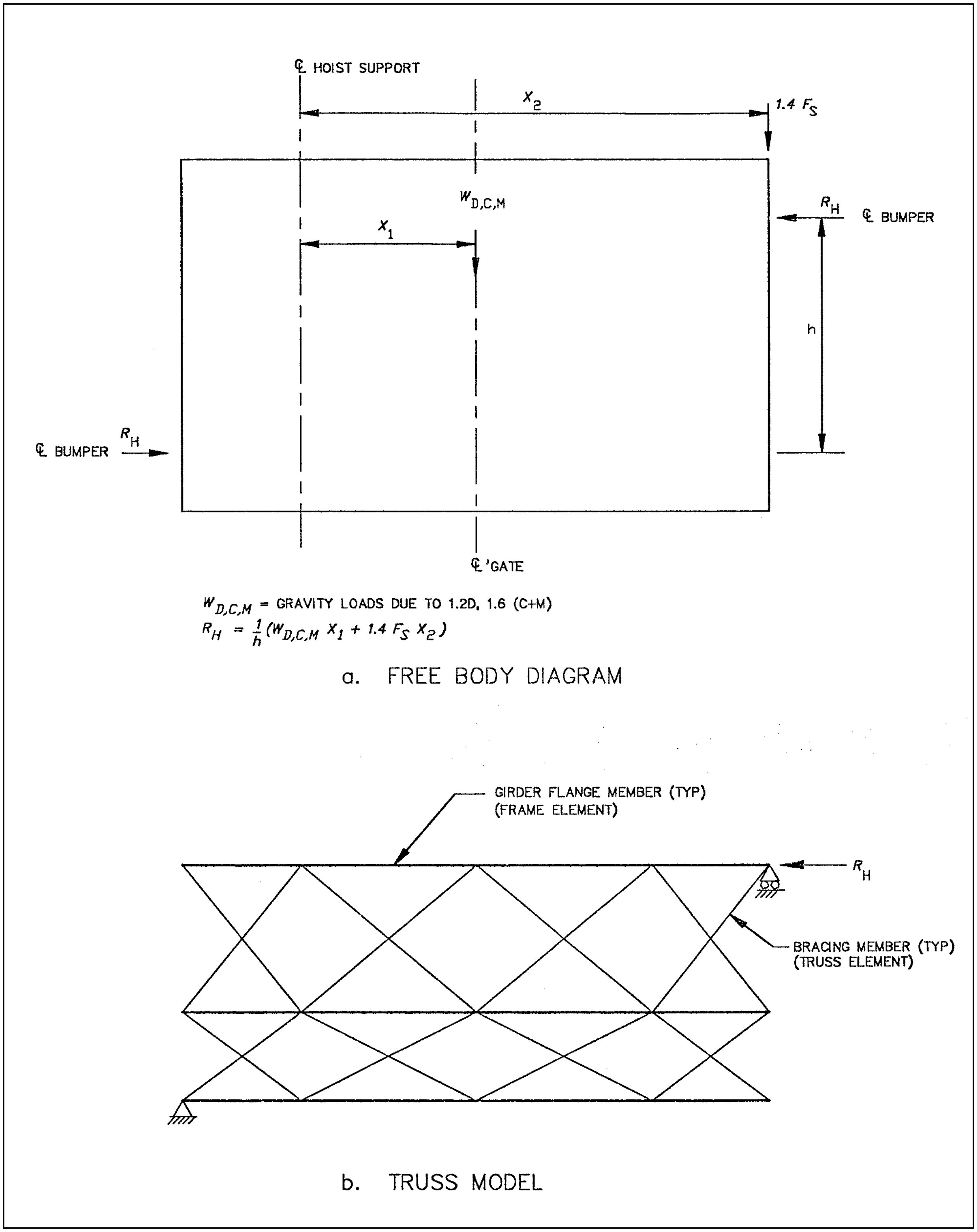


Figure 3-14. Downstream vertical truss model

(1) Skin plate design considerations. The skin plate design stress should be based on the negative moment at the supports for equally spaced interior ribs (fixed-end moment). The spacing between the exterior ribs at the ends of the gate should be adjusted such that the moment does not exceed the fixed-end moment of the interior spans. For gates with a wire rope hoist, thicker plate and/or closer rib spacing is normally required under the wire rope due to the rope pressure exerted on the plate. Because of the varying loading on the skin plate, it may be economical to vary the thickness of the plate over the height of the gate. For gates less than approximately 3m (10 ft) high, it is generally economical for the entire skin plate to be of one thickness. It is recommended to maintain a minimum thickness of 10 mm (3/8 in.), while a thickness greater than 20 mm (3/4 in.) will rarely be required for any gate.

(2) Rib design considerations. Although wide-flange or built-up sections are acceptable, structural tee sections with the web welded to the skin plate are recommended for ribs. In determining member geometric properties, an effective width of skin plate is assumed to act as the upstream flange of the vertical rib. The effective width b_e of skin plate shall be based on width-to-thickness ratios for compact or noncompact limits that are consistent with rib design assumptions. For rib sections that are considered as compact,

$$b_e = \frac{187t}{\sqrt{F_y}} \tag{3-13}$$

and for sections that are considered as noncompact,

$$b_e = \frac{255t}{\sqrt{F_y}} \tag{3-14}$$

where t is the skin plate thickness. Economical design for the ribs will be achieved by locating the horizontal girders (rib support locations) to minimize the bending moments, M_u with positive and negative moments approximately equal.

(3) Fabrication and maintenance considerations. General considerations are discussed in Appendix B.

(a) Skin plate. All skin plate splices shall be full penetration welds and smooth transitions shall be provided at splices between plates of different thickness. Corrosion is controlled by protective coating systems and maintenance, and increasing skin plate thickness to allow for corrosion is not recommended. (Chapter 8.) However, due to inevitable wear and deterioration, it is appropriate to increase the skin plate thickness along the bottom of the gate or under wire ropes for gates with wire rope hoists.

(b) Ribs. Ribs should be spaced and proportioned to provide adequate clearances required for welding and maintenance painting, even with a slight increase in steel quantity. The depth of the ribs must be sufficient to provide access for welding or bolting the rib flanges to the supporting girders. For welded construction, 200 mm (8 in.) has been considered a minimum rib depth in the past.

c. Horizontal girder. Girders provide support for the skin plate assembly and transfer all loads from the skin plate assembly to the end frames. The girders act as rib supports and are generally located to achieve an economical design for the ribs. However, the location of girders also affects the load on each girder since the rib reactions are the girder loads. The overall economy considering the effect on girder design should be considered. The end frame (strut) design affects girder forces since the struts are the girder supports.

(1) Design considerations. Horizontal girders are singly or doubly symmetric prismatic members that are designed primarily for flexure about their major axis. The distribution of flexure along the length of the girder is significantly influenced by the orientation of the end frames. Maximum girder moments will result with struts that are parallel to the pier face. The maximum girder moment is reduced (moments are redistributed) if the struts are inclined and will be minimized if the struts intersect the girder at approximately one-fifth the gate width from the pier face. With inclined struts, the limit state of lateral torsional buckling of the girder should be checked since a significant length of the downstream flange of the girder will be in compression. The unsupported length of the downstream flange is the distance between supports consisting of downstream vertical truss members and the end frame strut. (The upstream flange is laterally supported nearly continuously by the vertical ribs.) The downstream vertical truss is designed primarily to resist forces that occur when the gate is supported at only one end (paragraph 3-5.e.) and also provides lateral stability and resistance to torsional buckling for the girders. The tension flange of critical girders may be fracture critical and should be designed and fabricated accordingly (paragraph 3-8).

(2) Fabrication and maintenance considerations. Appendix B includes general guidance for inspection and welding. Girders may be rolled sections or built-up plate girders. The rib-to-girder connection, girder-to-strut connection, and bracing connections should be detailed to minimize concern for fracture (Appendix B). Drain holes with smooth edges should be provided in the girder webs at locations most appropriate for drainage. All stiffeners shall be coped. Use of a minimum number of girders will simplify fabrication and erection and facilitate maintenance. However, the overall economy considering the design of the skin plate assembly should not be compromised.

d. End frames. The end frames transfer loads from the girders and skin plate assembly to the trunnion. End frames include the struts and associated bracing and the trunnion hub flange plates. The arrangement and orientation of the end frames affects the magnitude and distribution of end frame and horizontal girder forces, trunnion fabrication, trunnion pin binding, and thrust forces into the pier. To achieve an economical design, these effects should be recognized considering all applicable load cases.

(1) Design considerations. Struts include flexure about both axes and significant axial forces. With a 3-D analysis, all forces are obtained from the same model. However, if a 2-D analysis as described in paragraph 3-5.a is utilized, the strut strong axis moments shall be determined from the girder model and the axial forces and weak axis moments shall be determined by the end frame model. End frame bracing should be spaced to achieve adequate weak axis slenderness ratios for the struts and must be designed to resist calculated forces. Bracing members may include significant flexural forces depending on member sizes, connection rigidity, and trunnion friction. Critical bracing members subject to flexural tension can be fracture critical and should be designed and fabricated accordingly (paragraph 3-8). Trunnion hub flanges shall be proportioned to resist the strut flexural, shear, and axial loads.

(2) Fabrication and maintenance considerations. Strut bracing members can be structural tee or wide flange sections; however, using wide flange sections with the same depth as the struts generally facilitates fabrication of connections. Trunnion hub flanges should be proportioned to provide a surface perpendicular to each strut with clearance provided between the ends of intersecting struts (Figure 3-15). Critical connections include the strut-to-girder connection (discussed in Appendix B) and the strut-to-trunnion hub flanges. The latter connection generally involves full-penetration butt splices involving thick plates, and welding requirements should be developed to minimize associated problems (Appendix B). The layout of end frames, parallel to the pier or inclined, is an important consideration regarding the overall design of the structure. Each configuration has unique advantages and disadvantages.

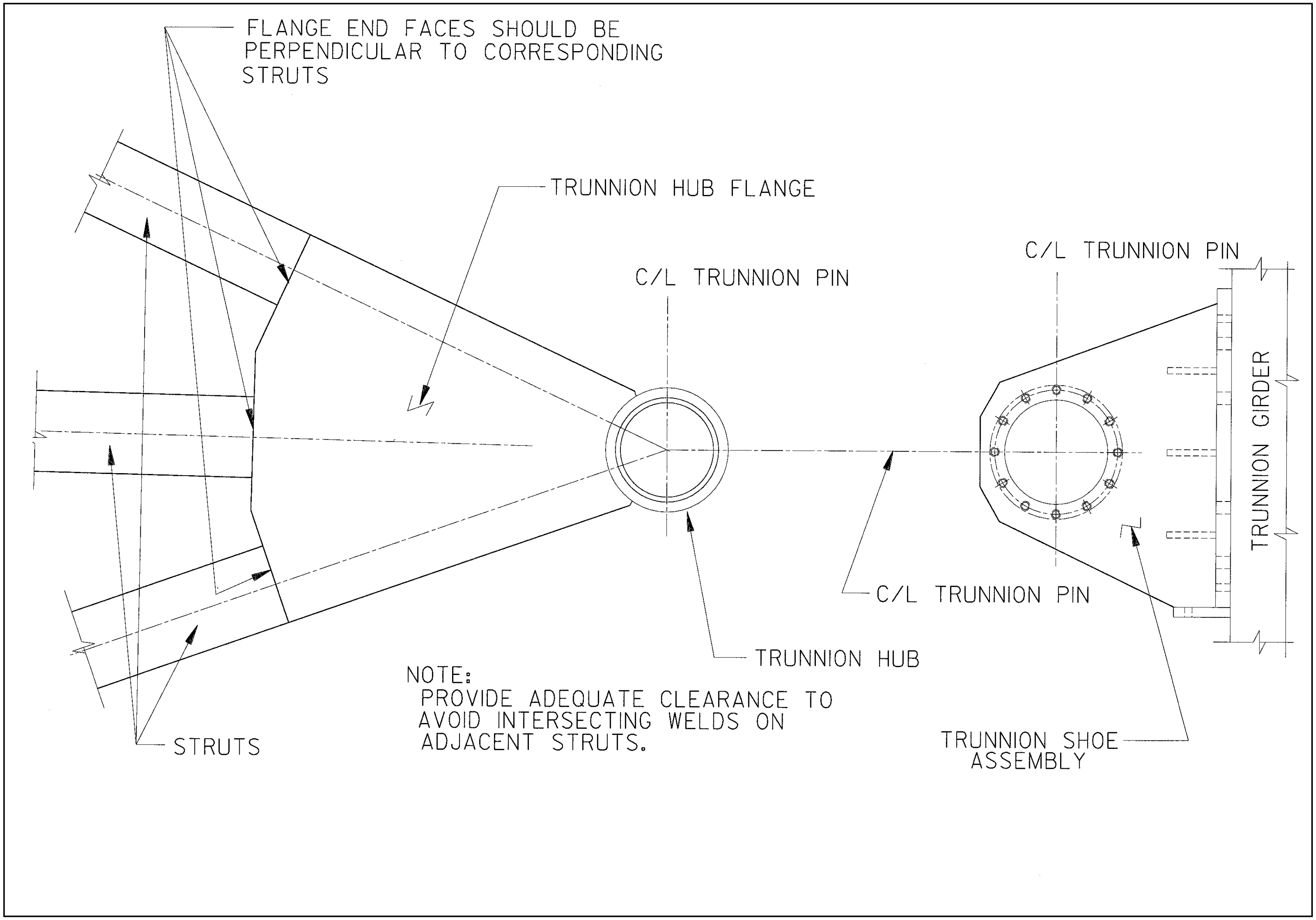


Figure 3-15. Trunnion hub flange

(a) Parallel end frames. End frames that are parallel to the pier and perpendicular to the horizontal girders are positioned such that interference to flow and debris accumulation on the struts is minimized. Fabrication and layout is relatively simple and struts that are parallel to the pier face transfer minimal lateral thrust into the piers. However, economy of the gate is sacrificed due to large flexural loads in the struts and girders, and clearance for maintenance painting between the pier and struts is limited.

(b) Inclined end frames. By inclining the end frames from the pier face, flexural forces are redistributed and the girder and strut flexural forces are reduced. The point of intersection of the strut and girder can be selected such that the girder moment at midspan is equal in magnitude to the cantilever moment as discussed in paragraph 3-5.c. With this arrangement, maximum economy of the girder design is achieved, and flexure in the end frames is minimal. A second option is to locate the intersection where there would be zero rotation in the horizontal girder. This would theoretically eliminate bending in the end frames. The side thrust component of the gate reaction introduced by inclining the end frames is transmitted directly to the pier or is resisted by a trunnion tie. Where the lateral thrust appreciably increases the pier requirements, it may be preferred to reduce the degree of inclination to achieve a more economical design. While inclined end frames are usually desirable for flood control projects, they are often not feasible for navigation dam projects where floating debris is a concern.

(c) Inclined frame layout. With inclined end frames, there are two options for layout of the struts. The struts can be positioned in a single vertical plane, or such that the girder end of each strut is an equal horizontal distance from the pier face. Placing struts in the same plane is generally recommended since fabrication is simplified and it will usually be cost effective.

1) Struts equal distance from pier. The struts can be positioned such that the connection between each strut and its corresponding horizontal girder is an equal horizontal distance from the pier face. In this case, the centerlines of the struts will form a conical surface with the apex at the trunnion. This results in complex fabrication of the strut-to-trunnion hub flange connection, since the struts are rotated with respect to one another and do not lie in one plane. However, this configuration has been widely used.

2) Struts positioned in a single plane. With struts positioned in a single vertical plane, only two struts can be at the same horizontal distance from the pier face. For gates with more than two girders, this results in differing support locations for the horizontal girders. However, fabrication of the strut-to-trunnion hub flange connection is simplified since all struts fall in a single plane. (This configuration is the only option for a vertical girder gate, since the struts and the vertical girder must fall in one plane.)

e. Downstream vertical truss. The primary structural purpose of the downstream vertical truss bracing members is to provide torsional rigidity for the condition when the gate is supported at one end. It also provides gate rigidity for resisting gravity loads with symmetric hoist support conditions, lateral bracing for the horizontal girders, and structural rigidity during field erection.

(1) Design considerations. For design purposes, bracing members should be sized to resist axial forces determined as described in paragraph 3-5.a(4). This procedure is conservative since the vertical support of the end frames and additional torsional rigidity of the skin plate assembly are ignored. Where practical, bracing should be placed in positions appropriate to provide lateral support to the girders. For gates that have a low height-to-width ratio, it may not be practical to design a bracing system that would prevent significant lateral displacements if the gate were supported on only one side. In these cases it may be necessary to provide side bumpers to limit lateral movement as described in paragraph 3-6.b(1).

(2) Fabrication and maintenance considerations. Single angles, double angles, or WT sections are commonly used for the bracing members. Connections between bracing members and the downstream girder flanges should be detailed to minimize fracture as discussed in paragraph 3-8 and Appendix B. The girder flange is subject to significant flexural tension and may be considered fracture critical in some cases. To limit accumulation of debris, deflector plates may be incorporated as described in paragraph C-2.b(4), Appendix C.

3-6. Serviceability

Tainter gates shall be designed considering the structure maintainability, durability, and operational reliability.

a. Corrosion. Gates shall be protected from corrosion by applying a protective coating system or using cathodic protection. Members shall be proportioned such that access is provided for future painting and maintenance. Chapter 8 provides guidance for corrosion protection, and paragraph 3-5 and Appendix B discuss fabrication considerations.

b. Operational reliability. Gates shall be designed such that they have a high degree of operational reliability in addition to adequate strength to resist applied loads.

(1) Sidesway and binding. Sidesway and binding shall be limited such that gate operation is not impeded. Gates may include various side bumpers or rollers (paragraph 3-7e) to limit or control side sway deflection and binding. For the condition where the gate is supported on only one side, the gate may rotate so that the gate bumpers bear on the side-seal plates. If this occurs, the normal force between the bumper and plate influences the potential for gate binding between piers due to frictional forces that occur with gate movement. A 3-D finite element analysis may be required to determine the normal forces and subsequent potential for binding. Such an analysis would be nonlinear, since the boundary condition would vary depending on whether or not the bumper touches the side-seal plate. Gap and hook support elements, which allow a specified movement to occur before developing a reaction force, may be appropriate for modeling such support conditions. If operational requirements include lifting or closing the gate when it is supported on one side only, the designer should consider possibilities of roller failure or degraded embedded metal surface conditions (due to corrosion or presence of foreign materials/growths) on the effective roller drag or frictional resistance.

(2) Ice control. Where ice may accumulate and inhibit gate operation, heaters shall be considered in the design. Various gate heaters are discussed in Appendix C.

(3) Deflections. Deflections under service loads shall not impair the serviceability or operability of the gate. Girder deflections shall be limited to avoid unwanted vibrations and leakage at the sill, and plates must be sized to avoid plate membrane behavior. The maximum girder deflection between end frames shall be limited to 1/800 times the span, and the maximum girder deflection for the cantilever portion between the end frame and pier face shall be limited to 1/300 times the cantilever length. The skin plate deflection shall be limited to 0.4 times the plate thickness.

(4) Vibration. Vibration due to flow under the gate shall be considered in the design and detailing of the tainter gate. Deflections shall be limited in accordance with paragraph 3-6.b(3). To limit vibration, the bottom lip of the tainter gate and sill should be detailed as described in paragraph 3-7.a(2) (Appendix C, paragraph C-2.b(1)).

(5) Debris. Consideration should be given to debris buildup in cases where there will be downstream submergence. Debris protection should be provided as needed on the end frames and on the downstream flanges of girders to avoid debris impact damage and binding of lodged debris (paragraph C-2.b(4)). In extreme cases, floating debris swirling behind the gate has damaged lighter members such as bracing members. To avoid damage, some gates have been fitted with downstream deflector plates to protect the framing from impact due to debris.

3-7. Design Details

a. Seals. The seals used in tainter gates follow standard details. However, there will be some differences based on operational requirements and the degree of water tightness required for the specific project.

(1) Side seals. The standard side-seal arrangement is shown in Figure 3-9. This arrangement employs standard, readily available, J-bulb seals. The seals may have a hollow bulb where increased flexibility of the bulb is desired such as low head applications. The seals are available with the rubbing surface coated with fluorocarbon (Teflon) to reduce friction. This is beneficial especially for high head gates. The seal attachment plate must have slotted bolt holes to allow for field adjustment of the seals. The seals are normally installed with a precompression against the side-seal plate which allows for construction irregularities and creates a tighter seal under low heads. The standard side-seal configuration provides for an increase in the sealing force in proportion to increased head and seals usually tend to leak under low heads rather than high heads.

(2) Bottom seals. The recommended bottom-seal configurations are shown in Figure 3-16. For most conditions, the preferred configuration is that shown by Figure 3-16a. The seal is provided by direct contact between the skin plate edge and the sill plate. The lip of the tainter gate should form a sharp edge and the downstream side of the lip should be perpendicular to the sill. It is recommended that rubber seals not be used on the gate bottom unless normal leakage can not be tolerated. If leakage is critical, a narrow rubber bar seal attached rigidly to the back side of the gate lip should be used (Figure 3-16b). An alternative to this is the configuration shown by Figure 3-16c with a rubber seal embedded in the gate sill plate.

b. Lifting attachments. Lifting attachments are often generally treated as fracture critical, non-redundant connections for design. However, redundancy actually does exist since there are two lifting attachments. The force in the attachment due to the machinery's operating at maximum stall pull (Load Case 4) normally governs the design rather than normal operating forces. The magnitude of this loading will be obtained from the mechanical engineer responsible for the machinery design and will be based on the capabilities of the lifting equipment. The wire rope attachment often must be designed with a rotating attachment to allow the cable to pull away from the skin plate as the gate approaches the full open position. Many gates also have skin plate extensions of smaller radius at the top to allow the rope to wrap over the top of the gate when fully closed. A typical wire rope attachment detail is shown in Figure 3-17. A typical hydraulic cylinder attachment detail is shown in Figure 3-18.

c. Drain holes. The designer should consider all locations where water can be trapped for all gate positions. Long-term standing water should be avoided, since it contributes to corrosion and becomes stagnant ponds of scum. Drain holes properly located for drainage should always be provided in the webs of the girders, end frames, and bracing members where applicable. The typical size is 5 cm (2 in.) in diameter. Additionally, half round holes can be provided in stiffener plates along with extra large corner copes to avoid pockets of water between stiffeners. Holes in flanges should generally be avoided.

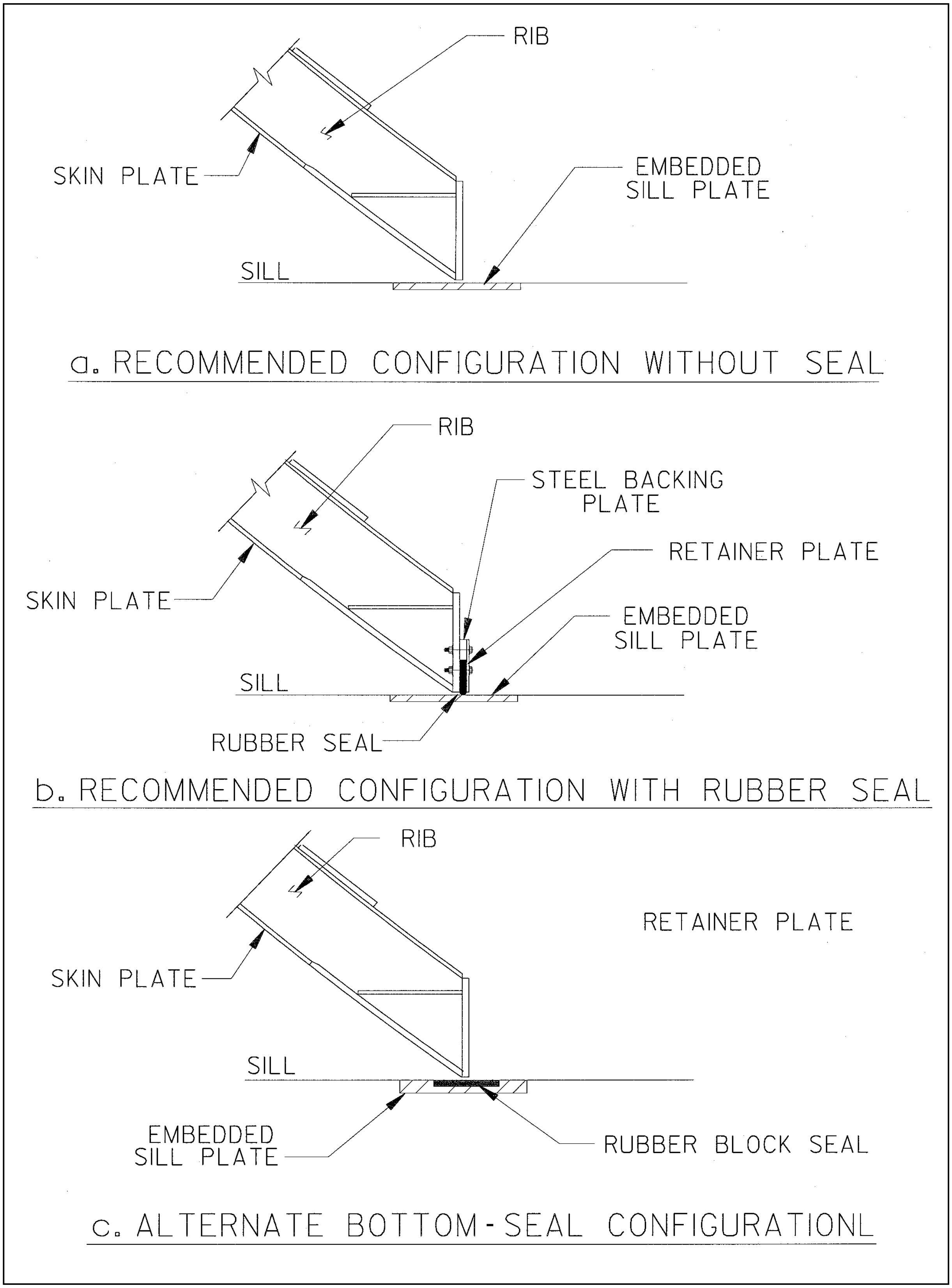


Figure 3-16. Bottom-seal configurations

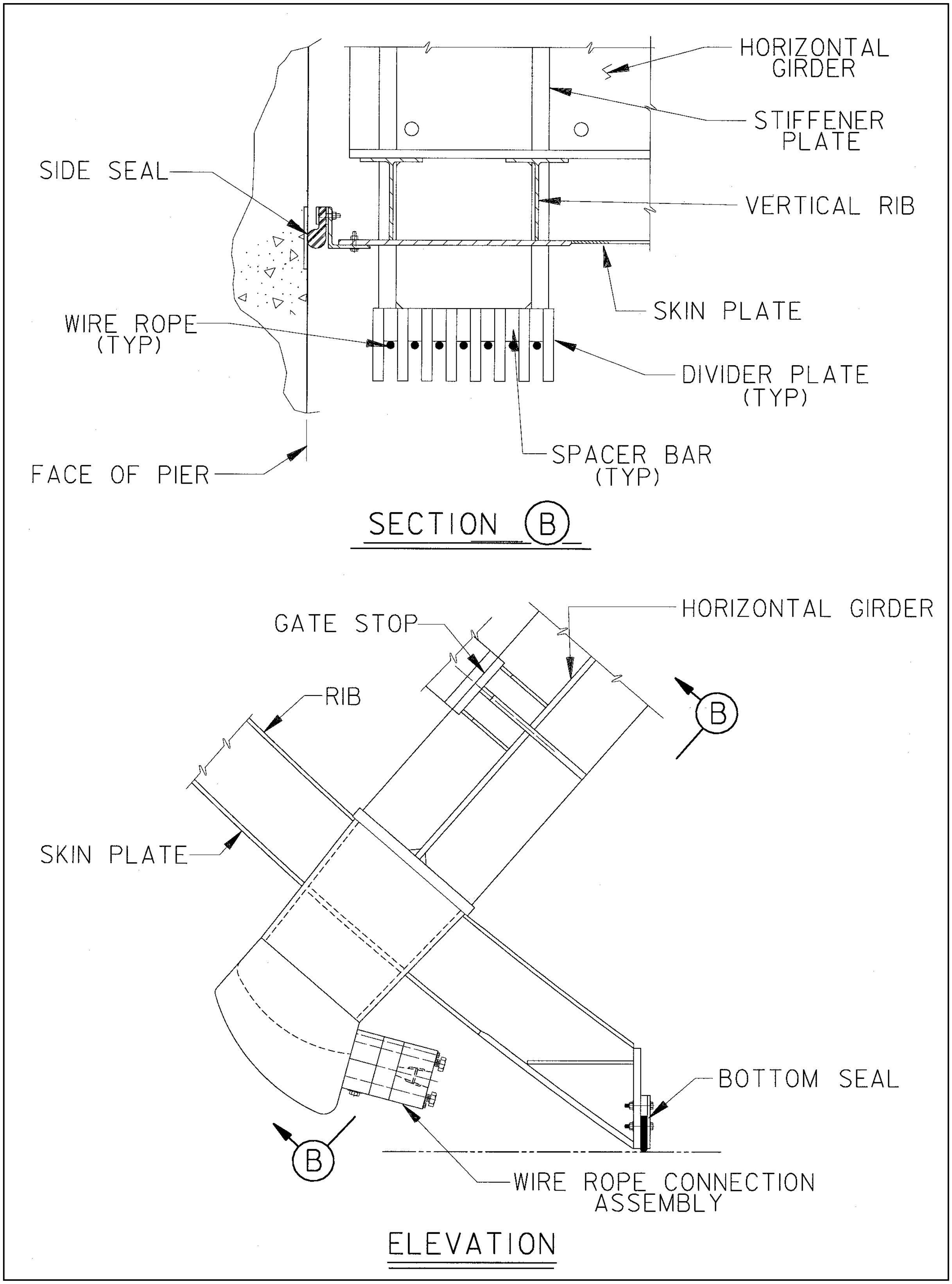


Figure 3-17. Wire rope connection bracket

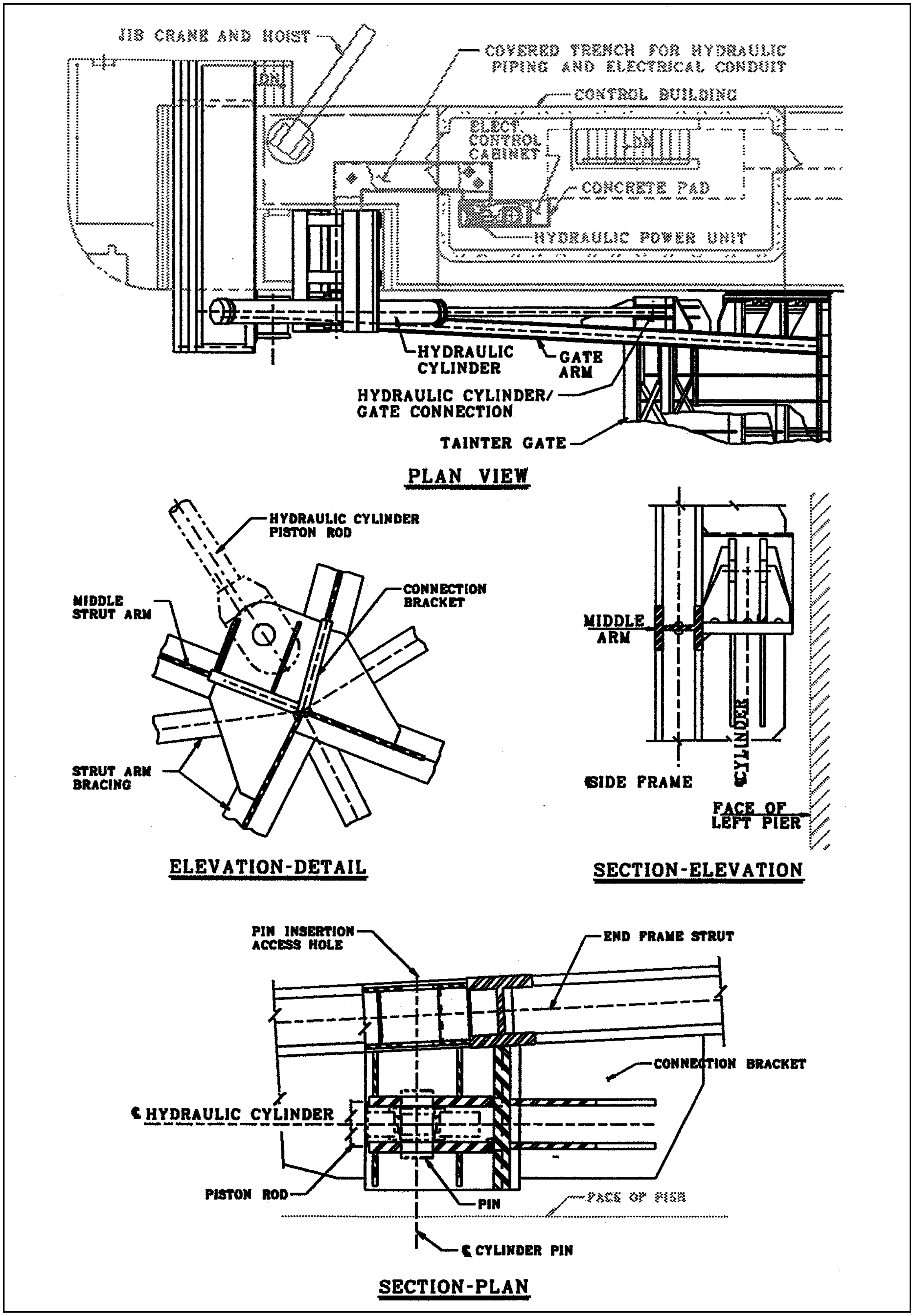


Figure 3-18. Hydraulic cylinder gate connection

d. Gate stops. Many structures are provided with gate stops which limit the gate from traveling beyond some point of opening. The stops are usually a short section of steel beam embedded and anchored into the pier which will contact a bumper on the gate if the gate travels beyond a certain position (Figure 3-19). The gate stops are not provided to physically stop the gate from opening since the machinery will be designed to stop prior to the gate contacting the gate stops. The stops are provided to keep the gate from over traveling due to wind or water loading in extreme or unusual situations. The stops are more often used with the wire rope hoist system since the ropes offer no resistance to upward movement.

e. Bumpers. Bumpers or rollers are generally located at the ends of the top and bottom horizontal girders near the upstream or downstream flanges. Bumpers are usually fitted with a bronze rubbing surface. An alternative for a bumper detail is shown in Figure 3-20. Rollers or ultra-high molecular weight plastic rubbing surfaces may be used to reduce friction when binding may impact gate operation.

f. Dogging devices. Some gates are provided with devices to temporarily support the gate in a full or partially raised position. These dogging devices will relieve the load on operating machinery and can even allow for maintenance or repair of the machinery or gate while the gate is raised.

3-8. Fracture Control

Design shall include provisions for fracture control of fracture critical members (FCM) and other critical elements that include tensile stress. Fracture control includes detailing tension connections to minimize stress concentration, specification of material properties, and enforcing prudent fabrication and inspection procedures. Guidance on preparation of project specifications is provided in Appendix B and ER 1110-2-8157. General requirements for welded connections are included in EM 1110-2-2105.

a. Fracture critical members. Fracture critical members or member components are tension members or tension components of flexural members , the failure of which would result in collapse of the structure. The design engineer shall identify all FCM on the project plans, and appropriate provisions on materials and fabrication requirements shall be included in the project specifications. These provisions shall conform to requirements specified in EM 1110-2-2105. Fracture critical members may include lifting machinery components and associated connections, tension flange of critical girders, tension flange of steel trunnion girders, and tension members of end frames.

b. Critical tension elements. Special considerations are warranted for various members or elements that are critical to structural or operational function but are not fracture critical. The engineer shall determine critical elements susceptible to fracture (i.e., strut-arm-to-girder connection) and specify any nondestructive examination requirements (other than visual inspection) of welds. Nondestructive examination is discussed in Appendix C.

c. Thick plate weldments. Appropriate fabrication requirements including weld sequence and inspection requirements shall be specified for thick plate weldments or highly constrained weldments that will include large tensile residual stresses (Appendix B). Trunnion yoke plates, trunnion bushing assembly, cable attachment brackets, steel trunnion girders, and built-up members generally include weldments with thick plates and/or high constraint. (A thick plate is generally considered to be 38 mm (1-1/2 in.) or greater in thickness.)

d. Miscellaneous considerations. In general, connections that include tensile stress should be detailed as fatigue resistant details to minimize stress concentration, even if fatigue loading is not present. Under normal operating conditions tainter gates are not subject to fatigue loading; however, fatigue loading may occur due to flow-induced vibration. Using good operating procedures and proper detailing of the gate lip as described in Appendix C can minimize vibration.

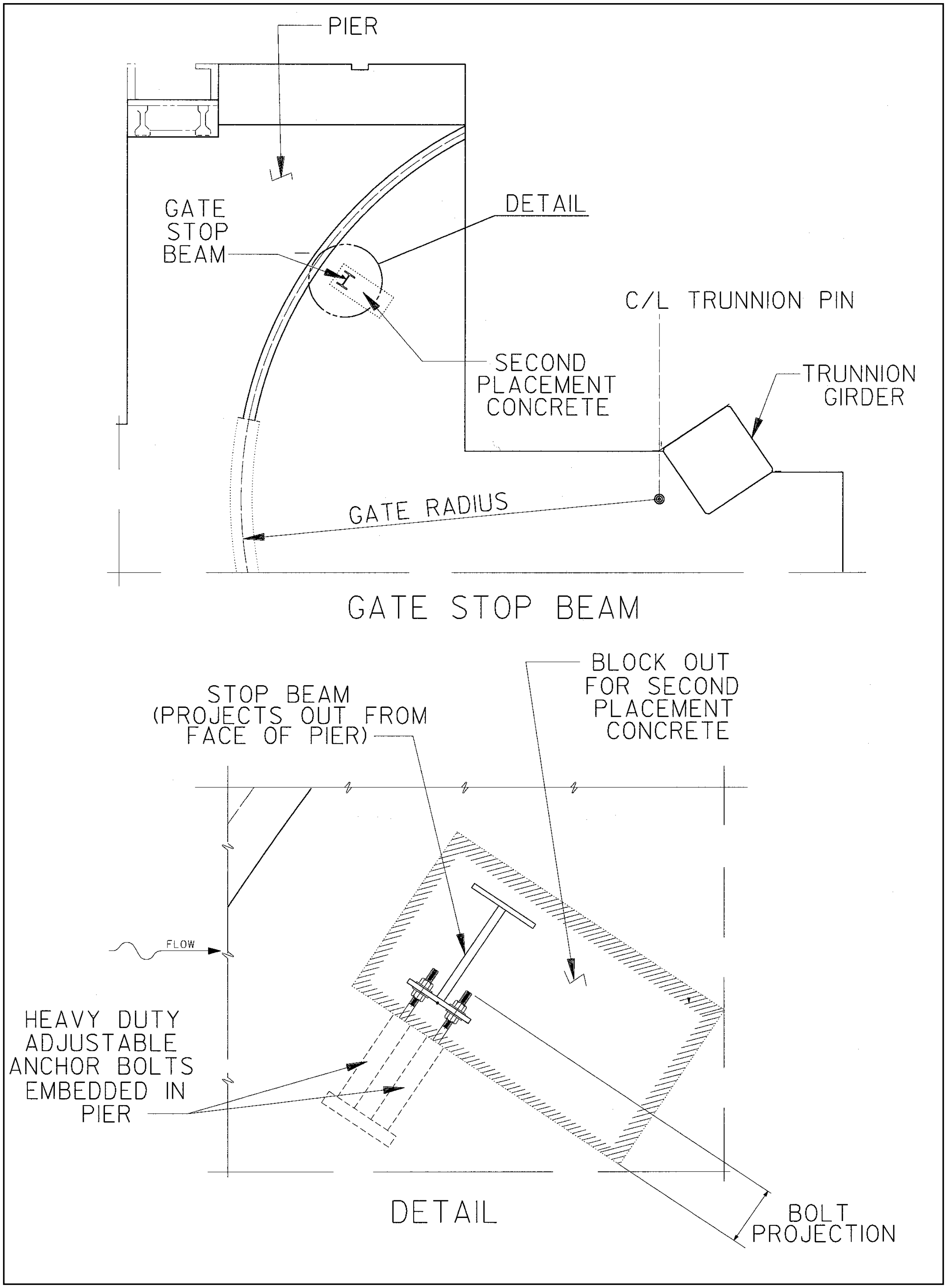


Figure 3-19. Gate stop beam

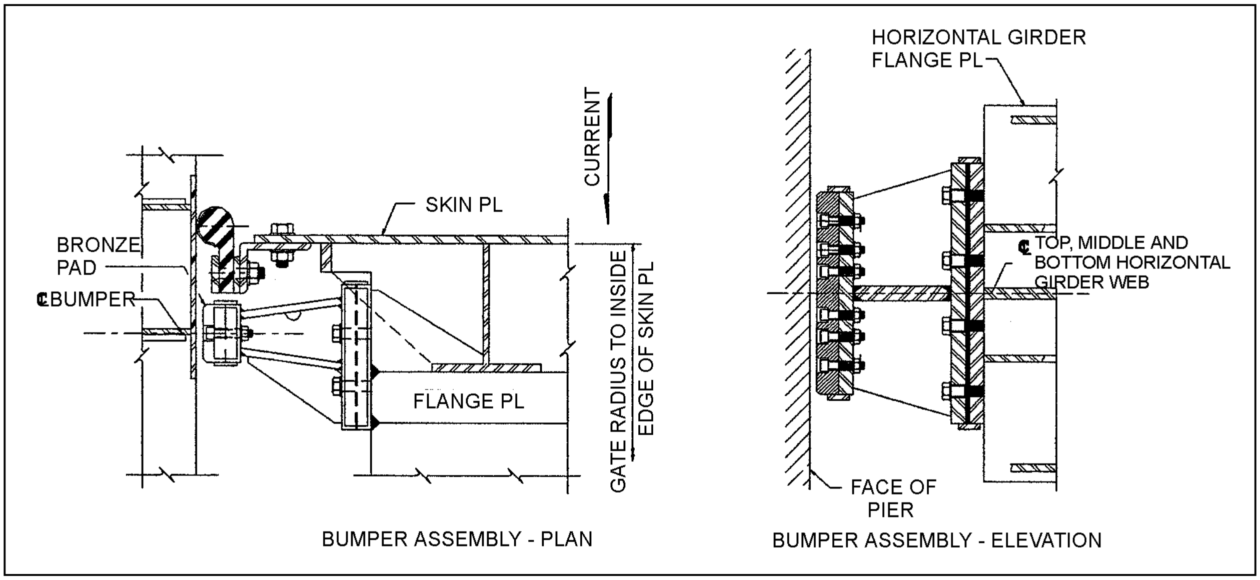


Figure 3-20. Typical gate bumper assembly

Chapter 4
Trunnion Assembly

4-1. General Description

The structural engineer shall coordinate the design of the trunnion assembly with a qualified mechanical engineer. Design of lubrication systems, tolerance and finish requirements, material selection, and determination of allowable stresses should be coordinated with a mechanical engineer. The trunnion assembly provides support for the tainter gate while allowing for rotation for operational use.

a. Conventional system. The trunnion assembly is made up of a fixed trunnion yoke that is bolted to the trunnion girder, a trunnion hub, and a trunnion pin with a bushing or bearing. Bushings or bearings are provided to minimize friction and wear during rotation of the gate about the trunnion pin. The trunnion assembly is designed to transmit gate load directly to the trunnion girder. Figure 4-1 illustrates typical details for a cylindrical bushing assembly. Spherical bearings are generally more expensive than cylindrical bearings due to their complexity. However, spherical bearings will compensate for a degree of misalignment of gate arms, construction tolerances, thermal movement, and uneven gate lifting. When compared to cylindrical bearings, spherical bearings are generally more narrow and the use of spherical bearings reduces or eliminates stresses at the edge of the bearings, produces a more uniform pressure distribution over the trunnion pin, potentially reduces trunnion pin moments and gate arm stresses due to misalignment. Spherical bearings will accommodate an angular rotation transverse to the pin centerline in the range of 6 to 10 deg depending on bearing size. A tradeoff exists with the use of spherical bearings over cylindrical bushings in that the gate arms associated with spherical bearing are usually heavier due to an increased buckling length. Figure 4-2 shows a spherical bearing configuration.

b. Other systems. Center-mounted trunnions are commonly used in combination with steel box trunnion girders. The trunnion pin is supported at the geometric centroid of the girder by plates that are oriented perpendicular to the pin centerline. The pin can bear directly on the supporting plates or within a housing tube attached to the plates. Use of the tube provides for a more accurate bore for the pin. This arrangement can significantly reduce torsion applied to the trunnion girder since load eccentricity is reduced or eliminated. A center-mounted arrangement is described in Figure 4-3.

4-2. Structural Components

Figure 4-4 describes the layout of structural components.

a. Trunnion yoke. The yoke is typically fabricated of welded structural steel and consists of two parallel plates (yoke plates) that are welded to a stiffened base plate (Figure 4-5). The yoke plates are machined to receive the trunnion pin and associated components. The assembly is bolted to the trunnion girder after final installation adjustments have been made by horizontal and vertical jackscrews. Shear bars that are welded to the base plate may be required to resist shear at the interface with the trunnion girder for loads that produce resultant forces that are not normal to the bearing surface.

b. Trunnion hub. The hub can be fabricated of cast, forged, or structural steel. Castings and forged steel are typically more costly than welded steel construction. The inside bore is machined to tolerance for proper fit with the trunnion bushing or bearing. The hub is welded to the gate arm extensions and is joined to the yoke with the trunnion pin. The hub is typically wider than the gate arm extensions to allow

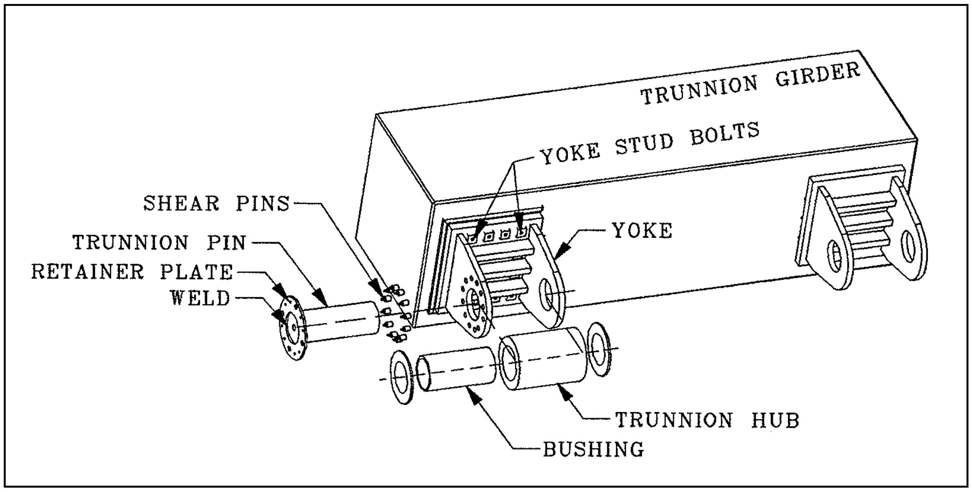


Figure 4-1. Trunnion assembly with cylindrical bushing

for a more uniform distribution of stress and to provide clearance for a welded connection. A bushing or bearing is provided between the hub and trunnion pin to reduce friction. The trunnion hubs and yokes shall be stress relieved by heat treatment and machined after fabrication welding is completed.

c. Trunnion pin. The trunnion pin transfers the gate loads from the hub to the yoke side plates. A retainer plate that is welded to the pin is fitted with shear pin to prevent the trunnion pin from rotating. The retainer plate and pin are connected to the yoke with a keeper plate.

d. Trunnion bushing. Bushings are provided between the trunnion pin and hub and between the hub and yoke plates. The bushings provide a uniform bearing surface and reduce torsional loads due to friction. The required thickness will depend on the size of the trunnion pin. However, to maintain a true shape during machining, bushings should be at least 12 mm (1/2 in.) thick. An interference fit is generally used between the hub and bushing.

e. Spherical plain bearings. Spherical plain bearings consist of an inner and outer ring and may contain intermediate sliding elements. The outer ring is fit within the trunnion hub and the inner ring is placed on the trunnion pin. The outer ring of the bearing is generally mounted inside the trunnion hub with an interference fit to prevent movement of ring seats. The inner ring may be mounted to the trunnion pin using an interference fit to prevent movement between the pin and the inner ring.

f. Anchorage. Bolts are used to attach the trunnion yoke to the trunnion girder. Consideration should be given to using partially prestressed high-strength stud bolts to minimize movement relative to the trunnion girder.

4-3. Material Selection

Material selection, surface finishes, dimensional tolerances, and allowable stresses shall be coordinated with a qualified mechanical engineer.

a. Trunnion hub. Material for trunnion hubs should be corrosion resistant, weldable, and machinable. The trunnion hub is typically machined from cast steel (ASTM A27) or forged steel (ASTM A668).

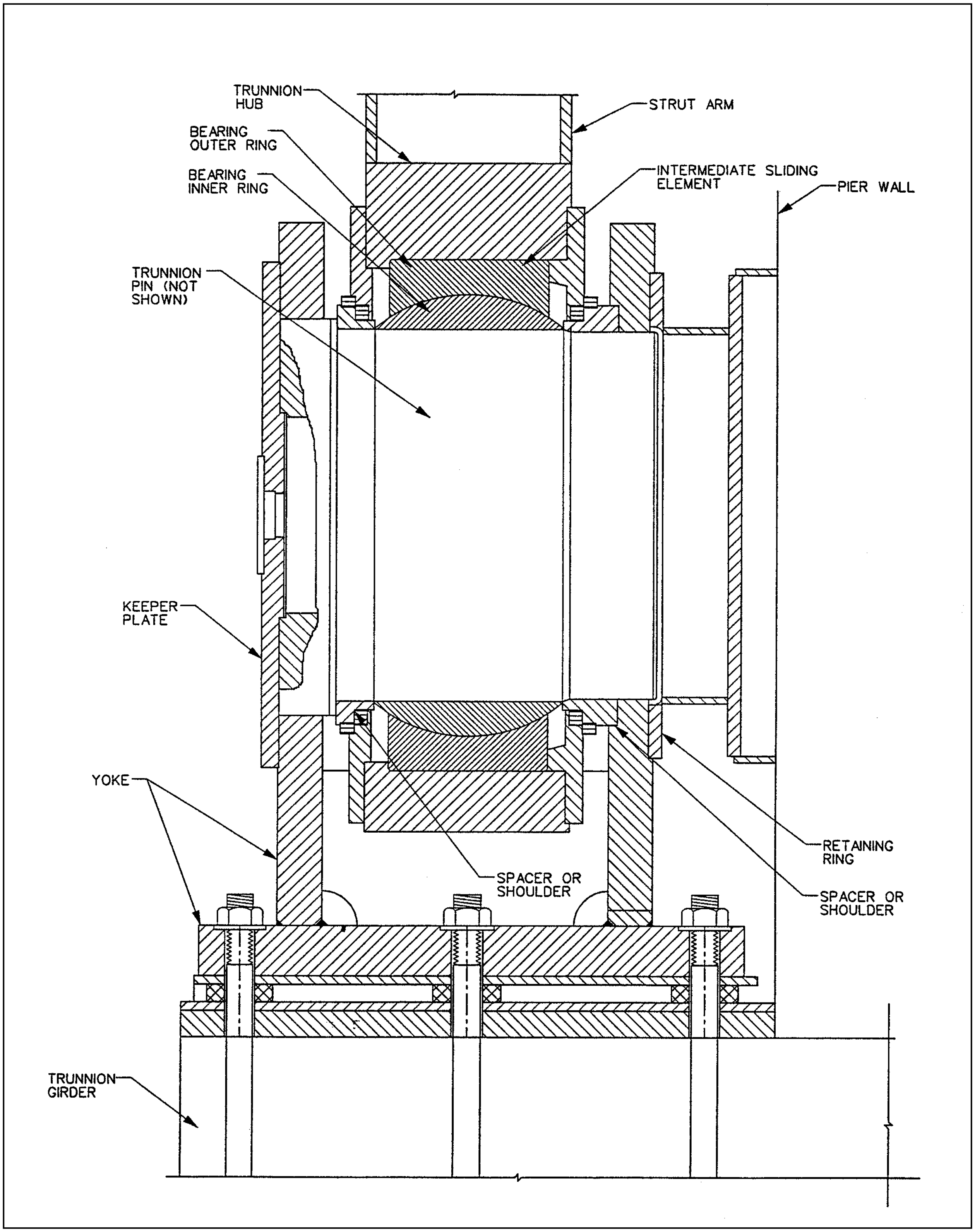


Figure 4-2. Spherical bearing

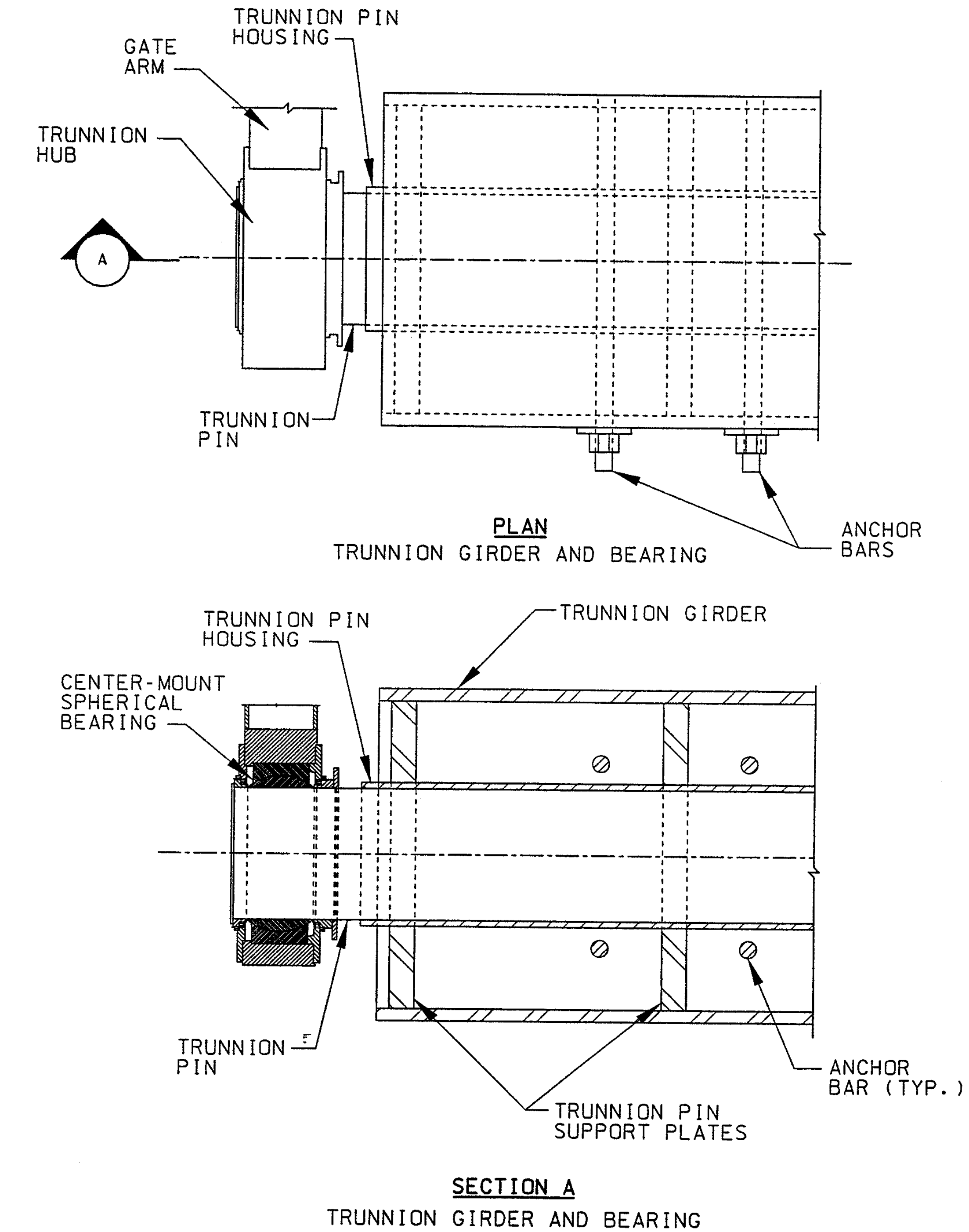


Figure 4-3. Center-pin mount bearing

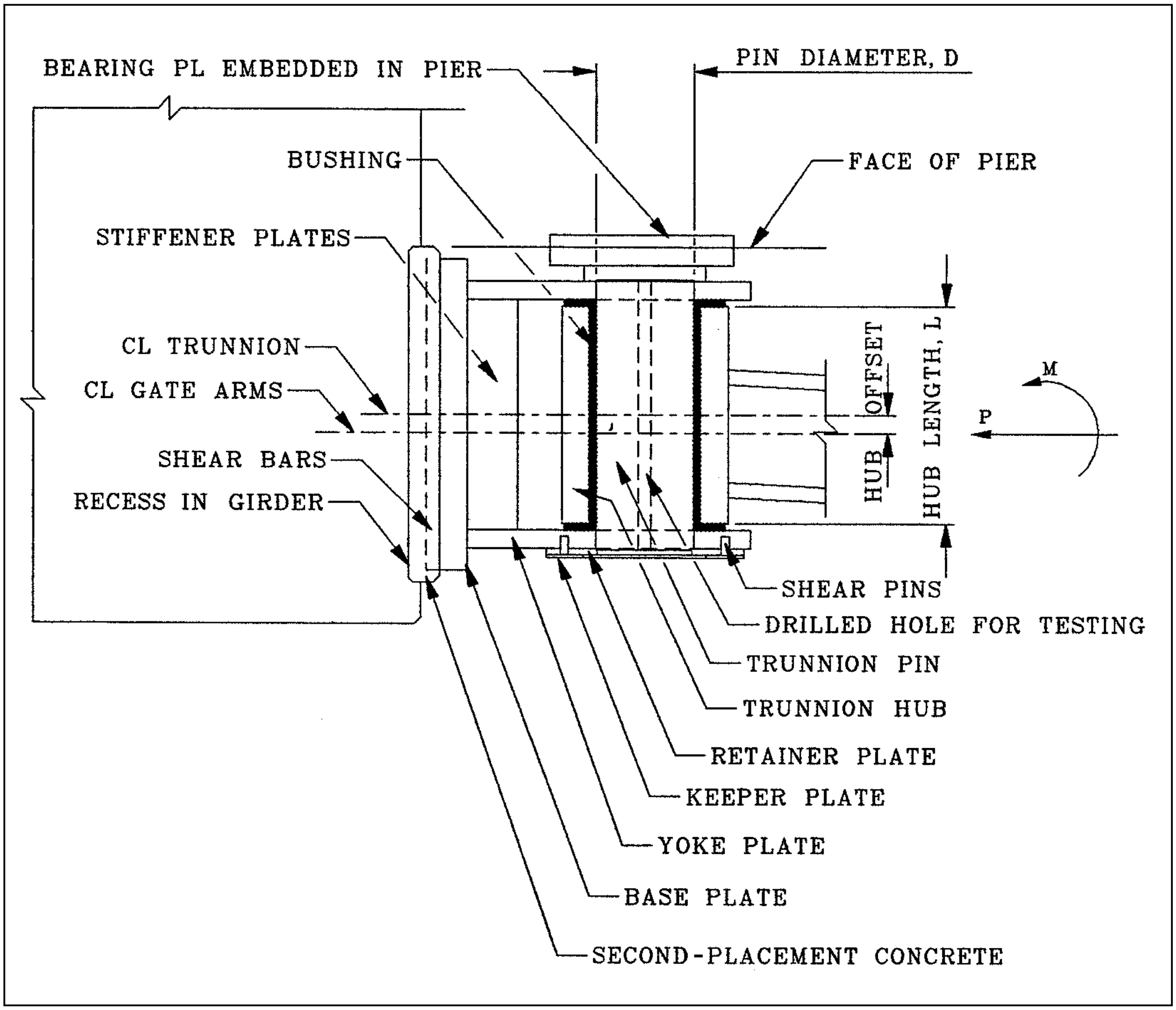


Figure 4-4. Trunnion assembly structural component layout

b. Trunnion bushing. Bushing materials are generally selected based on allowable bearing stresses, resistance to galling and coefficient of friction. Aluminum bronze (ASTM B148) is commonly used where bearing pressures do not exceed 35 MPa (5000 psi) and manganese bronze or self-lubricating bronze (ASTM B22-90a) is used for applications where bearing pressures up to 55 MPa (8000 psi) are required.

c. Spherical plain bearings. Spherical plain bearings are generally made of a high-strength carbon chromium steel treated with molybdenum disulfide. Maintenance free bearings may include a sinter-bronze composite or a poly-tetra-flouro-ethylene compound.

d. Trunnion yoke. The trunnion yokes are typically constructed of welded structural steel (ASTM A36 or A572) or cast steel (ASTM A27).

e. Stud bolts. Stud bolts, used to attach the trunnion yoke to the trunnion girder, are usually made from high-strength alloy steel conforming to ASTM A722 or ASTM A354.

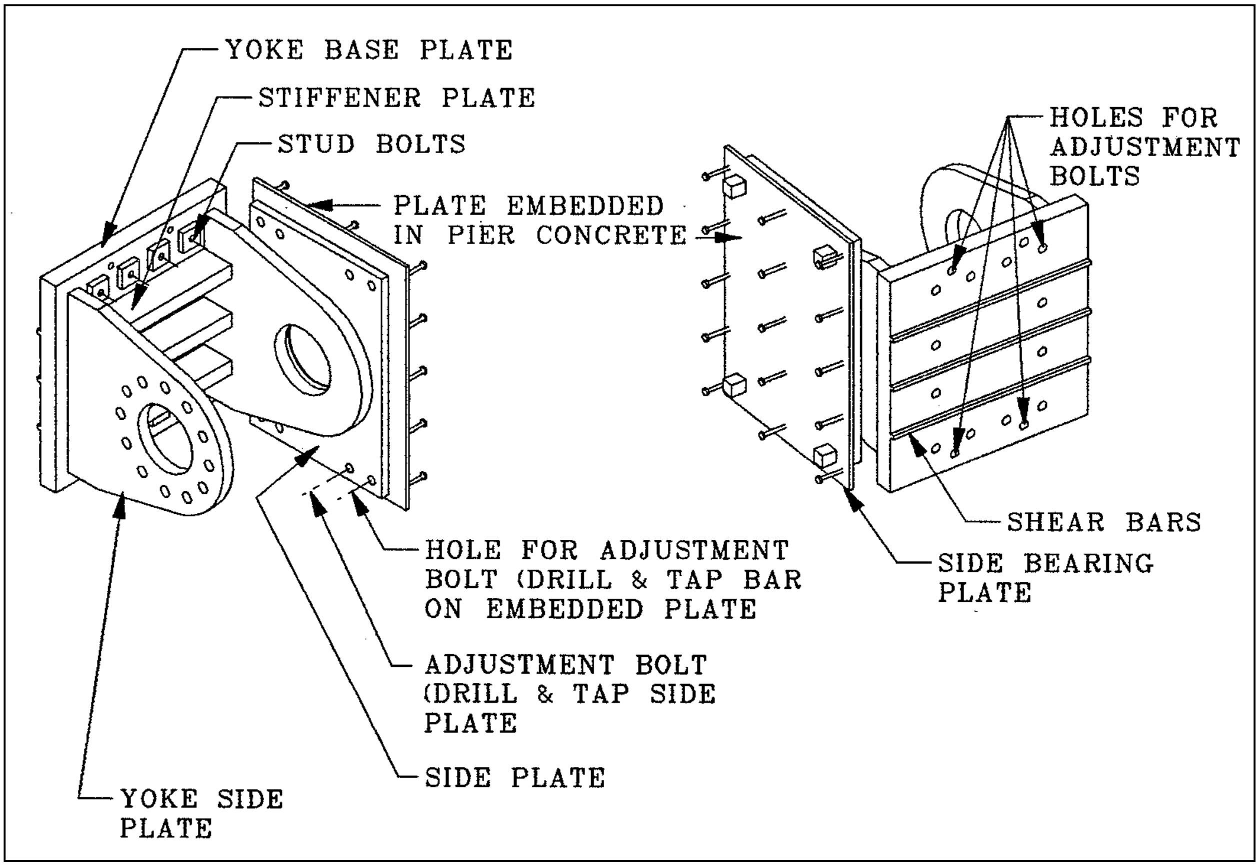


Figure 4-5. Typical trunnion yoke assembly

f. Trunnion pin. The material used for the trunnion pin must be compatible with the bushing material and be capable of high quality finishes to minimize friction. The trunnion pins shall be made from corrosion-resistant steel such as forged steel conforming to ASTM A705, Type 630, Condition H 1150. Historically, a carbon steel forging such as ASTM A668 was coated with a stainless steel weldment with subsequently machining. This practice is becoming less economical due to intensive labor costs.

g. Shear pins. Material for shear pins should corrosion resistant and machinable. The shear pins are typically machined from cast or forged steel conforming to ASTM A276.

h. Retainer and keeper plate. Material for the retainer and keeper plates should corrosion resistant, weldable, and machinable. The retainer and keeper plates can be fabricated from material conforming to ASTM A240, type 304.

4-4. Design Requirements

a. Design basis. All components of the trunnion assembly shall be designed based on allowable stress design. Maximum allowable working stresses for forgings and casting shall be limited to $0.5F_y$

where F_y is the material yield stress.

Allowable stresses and tolerances for bearings and bushings shall be established by the mechanical engineer. Serviceability requirements are specified in paragraph 4-6.

b. Load requirements. The trunnion assembly shall be designed for load combinations specified in Chapter 3 except a uniform load factor of 1.0 shall be applied to all sources of loading. Torsional loads due to trunnion pin friction shall be based on a coefficient of friction consistent with materials utilized (see paragraph 3-4.b(1)(f). The bearing stress between the yoke base plate and the trunnion girder should include both the pre-tensioning force of the anchorage stud bolts and global gate reaction forces.

4-5. Analysis and Design

Due to frictional resistance developed at the trunnion pin with gate movement, the design of the trunnion assembly affects the end frame design and the required hoist capacity (paragraphs 3-4 and 3-5). The centerline of bearing of the trunnion hub is commonly offset with respect to the centerline of the gate arms (Figure 4-4). This offset is recommended so that a uniform bearing stress distribution occurs under maximum loading (gate is nearly closed and impact and silt loads are applied). Other load conditions will produce nonuniform bearing stresses on the trunnion pin and bushing and must be investigated individually. The transfer of forces between the trunnion pin, retainer plate, shear pins, and yoke shall be considered in design.

a. Bushing. The bushing is designed as a boundary lubricated bearing. The bushing is proportioned such that the actual bearing stresses do not exceed allowable stresses. The bearing pressure between the trunnion pin and bushing is based on an effective area and the gate reaction forces. The effective bearing area is commonly based on the dimensions of the projected width of the trunnion pin onto the bushing (or bearing). The magnitude of bearing pressure is dependent on the length and diameter of the trunnion pin. The contact pressure is assumed to act uniformly across the diameter of the trunnion pin (even though it is theoretically nonuniform).

b. Spherical plain bearing. Design of the bearing is based on the strength of the bearing material. Serviceability requirements (limiting wear or deformations) are specified in paragraph 4-6. Material strength will usually control the design of tainter gate trunnion bearings because frequency of gate operation is relatively low. Bearing size is a function of direction and magnitude of trunnion reactions. Spherical bearings transmit radial forces (loads acting primarily in plane perpendicular to axis of trunnion pin) and moderate axial forces simultaneously (loads acting in a plane parallel to the axis of the trunnion pin). Where axial forces are large, thrust washers or an additional thrust bearing may be required to transmit axial loads.

c. Trunnion pin. The trunnion pin is designed as a simply supported beam with supports located at the centerlines of the yoke plates. Loading consists of the bearing stresses from the bearing or bushing. The length and diameter of the trunnion pin shall be proportioned such that the bearing pressure onto the bushing and flexural and shear stresses in the trunnion pin are less than the allowable stress limits as discussed in paragraph 4-3. The bearing, flexural and shear stresses are calculated using commonly accepted engineering practices. The retainer plate and shear pins are designed to carry frictional loads produced when the tainter gate is raised or lowered. The magnitude of torsion produced by friction is a function of the trunnion pin diameter, the coefficient of friction and the magnitude of the gate thrust. The weld connecting the retainer plate to the trunnion pin shall be sized to prevent rotation (see Figure 4-6). For spherical bearings, bearing movement occurs between the inner and outer rings only and not on the pin. Therefore, the pin is designed only to support the bearing inner ring.

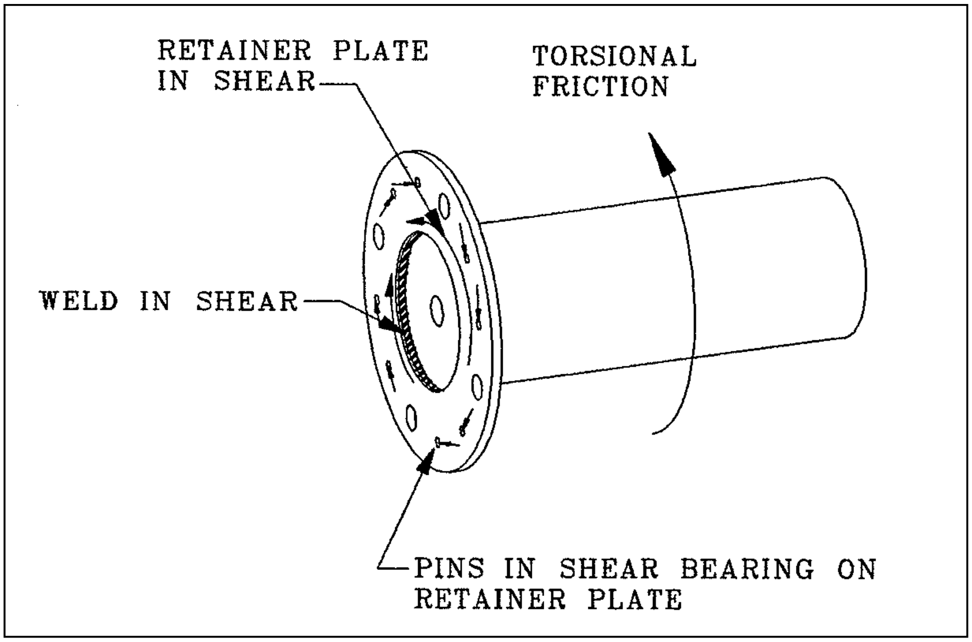


Figure 4-6. Generalized forces on trunnion pin and retainer plate

d. Yoke. The yoke side plate shall be sized to resist trunnion pin bearing load and lateral gate loads. The base plate and stiffeners shall be designed to resist contact pressure between the yoke bearing plate and trunnion girder based on gate reaction forces and stressing loads imposed by steel stud bolts, as shown by Figure 4-7. To determine the required strength, it is recommended that the base plate be analyzed as a simple beam supported by the parallel yoke plates with a distributed load equal to the bearing pressure.

e. Trunnion hub. The trunnion hub can be modeled as a cantilevered beam subjected to a distributed load from the trunnion pin as shown in Figure 4-8. The cantilevered portion of hub extends beyond the flange of the trunnion arm extension. Design checks for bending and horizontal shear are made along section A-A of Figure 4-8.

4-6. Serviceability Requirements

a. Access. The design shall consider features such as ladders, foot stands, railing, and passageways for access to the trunnion assembly for inspection and maintenance purposes.

b. Grease systems. Except where self-lubricating bearings are used, provisions to grease and lubricate the trunnion pin shall be provided. Manual or automatic grease systems may be used. Grease fittings are provided on the hub for both manual and automated systems. For automated delivery systems the grease fittings provided in the trunnion hub or through the trunnion pin to allow for lubrication in event of a pump failure. Automated grease systems dispense a measured amount of grease to the trunnion automatically at intervals established by the mechanical engineer. Provisions to flush automated systems should be considered. Pump units should be located near the trunnion to minimize grease line length. Grease lines should be stainless steel pipe of adequate wall thickness for the anticipated pressures.

c. Lubrication. Steel-on-steel bearings require lubrication at regular intervals. Lubricants that contain rust-inhibiting compounds are desirable. Lubrication should be considered for some

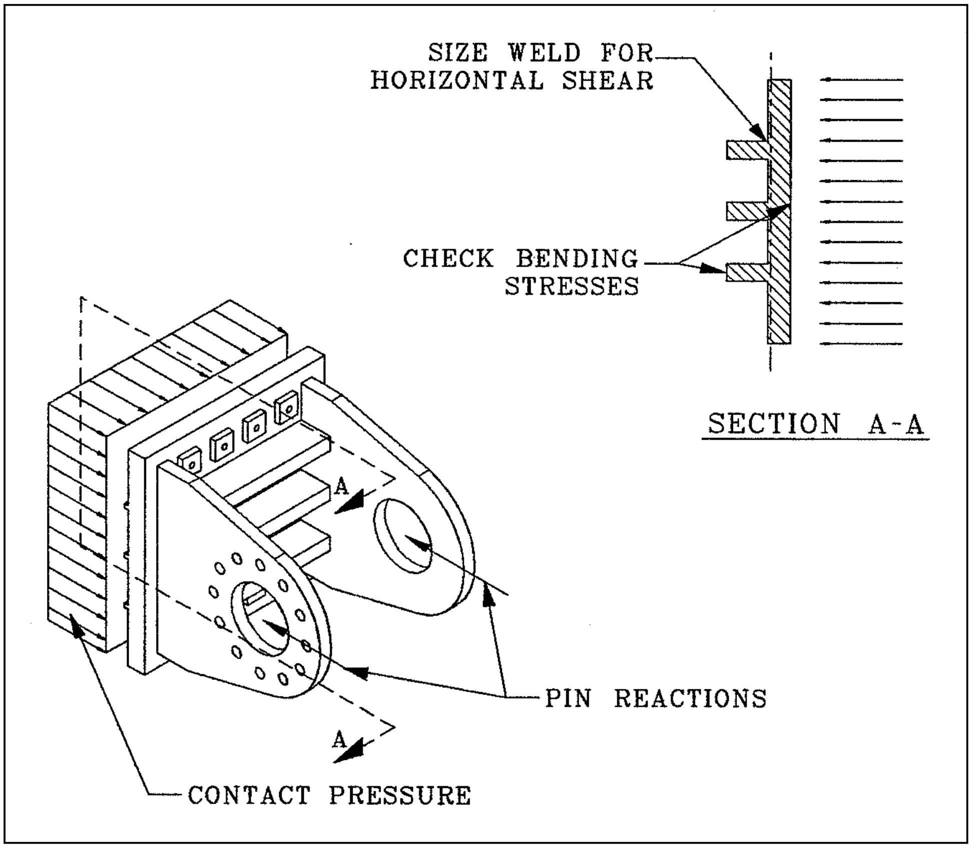


Figure 4-7. Design of base plate

maintenance-free bearings because friction is reduced, the lubrication acts as a seal to prevent dirt from entering the bearing, and corrosion protection is increased. Certain types of lubricants are detrimental to some maintenance-free bearings and other types of maintenance free-bearings should not be lubricated at all.

d. Bearing wear and deformation. Bearing size is a function of desired service life when designing for serviceability. Service life can be determined by use of empirical formulas and is based on magnitude and type of load, bearing movement, and expected contamination and corrosion. The manufacturer usually provides design procedures including empirical formulas and required safety factors. Angle of bearing tilt should be estimated for misalignments and gate movements based on judgement. Allowable angle of tilt of a spherical bearing is dependent upon bearing size, geometry, and design. The bearing manufacturer provides limits of tilt.

4-7. Design Details

a. Trunnion tolerance. Tolerances for the trunnion axis centerline with respect to the piers is based on clearance requirements between the side seal and seal plate embedded in the pier and between the gate bumper and the pier. If the trunnion centerline is not perfectly aligned, out-of-plane sweep (with respect to the pier) will occur when the gate is moved from the closed position to the fully raised position. Tolerance requirements may be relaxed if side-seal plates are terminated and a recess in the pier is provided above upper pool. The centerlines of trunnions at each gate arm must pass through a common axis to avoid unintentional friction loads due to binding as the gate rotates through its operational range.

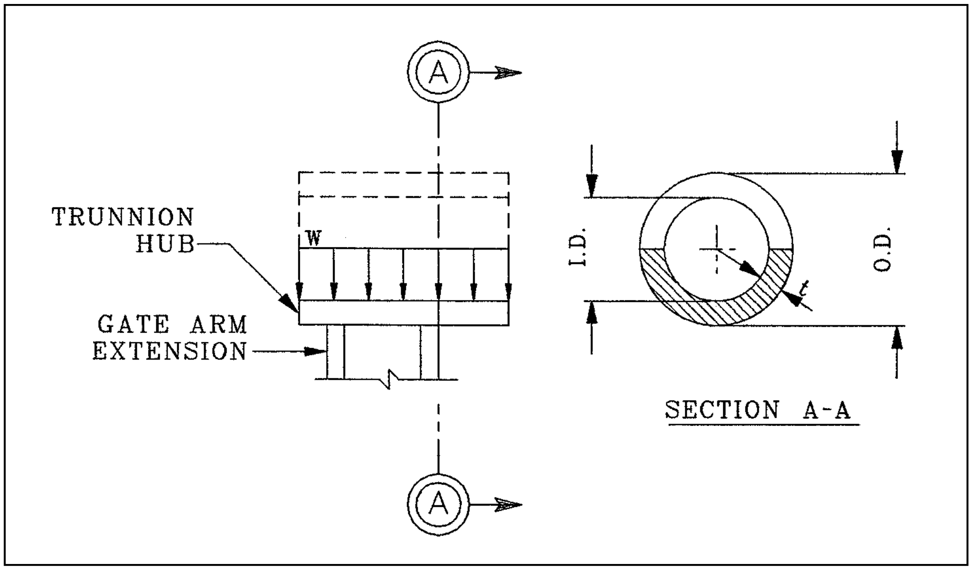


Figure 4-8. Trunnion hub design assumptions

Tolerance requirements should be determined based on gate size and should be included in project specifications.

b. Trunnion yoke adjustment. Horizontal and vertical jack screws are provided on the trunnion hub for setting and adjusting the trunnion yoke so that the trunnion hub axes are on a common horizontal line. Proper alignment is required to ensure that all cylindrical bearing surfaces of the yoke and hub rotate about a common horizontal axis without binding. Second-placement concrete or grout (zinc in the case of steel girders) can be used to fill the space between the trunnion girder and yoke and between the trunnion assembly and side bearing plate on the pier.

c. Bushings. Bushings are usually furnished with two side disk bushings and one cylindrical trunnion pin bushing. Shear pins between the side bushing and trunnion yoke plates are used to prevent bushing rotation. A light drive fit between the hub and cylindrical bushing is generally specified to prevent differential rotation between the hub and bushing. The bushing is usually given an overall finish of 1.6 μm (63 micro-in.) except for the bore finish of 0.8 μm (32 micro-in.).

d. Spherical bearings. Bearing rings that are mounted with an interference fit are positioned on the trunnion pin with spacers or shoulders placed against the inner ring. A housing shoulder is generally mounted to the trunnion pin on the side adjacent to the pier and a locking plate or spacer sleeve retains the ring on the opposite side. The housing end cover generally retains the outer ring. Retaining rings may be used to provide the bearings support along the axis of the pin. The bearing internal clearance (between inner and outer rings) must account for deformations induced by the interference fits. Figure 4-2 shows a typical spherical bearing arrangement.

e. Trunnion lubrication. For trunnion lubrication, grease grooves are commonly provided on the inside face of the bushing. The size, length, and location of the grease groove shall be sufficient to uniformly distribute lubricants to all bearing surfaces. This detailing shall be performed by the mechanical engineer. A hole may be drilled through the hub and bearing to inject the grease grooves with grease.

f. Trunnion pin. Trunnion pins may be designed with a hole drilled along the pin centerline for entry of a radioactive source to facilitate radiographic testing. The hole may also be tapped for handling, installation, and removal purposes.

Chapter 5
Gate Anchorage Systems

5-1. General Description

The trunnion girder is held in place by an anchorage system that extends into the concrete pier. Anchorage systems can be classified as prestressed or nonprestressed. The prestressed system includes high strength, preloaded components while the nonprestressed system incorporates structural steel components. A general arrangement of a gate anchorage system is shown in Figure 5-1.

a. Prestressed systems. Prestressed systems consist of groups of posttensioned members that anchor the trunnion girder to the pier. The posttensioning operation creates an initial compressive stress in the system that acts to maintain trunnion girder-to-pier contact and to reduce structural cracks in the pier concrete. Figure 5-2 describes a typical posttensioned anchorage system. The anchors are placed inside ducts embedded in the concrete and are tensioned after concrete has set and cured. Subsequent to tensioning, the annular space between the posttensioning steel and duct is grouted for corrosion protection. Generally, two groups of anchorage steel are installed, one near each pier face. A limited bearing area is provided directly under each anchorage group. A compressible material is placed between the anchorage bearing areas at the trunnion girder-to-pier interface to intentionally prevent the transfer of stress across this reach. This detail enhances structural performance by reducing negative bending moments in the trunnion girder, and a larger moment arm between anchorage groups is available to resist nonsymmetrical loads. Alternate anchorage systems are shown by Figure 5-3. These systems are comprised of additional plates or other structural members anchored into pier concrete and connected to the trunnion girder (e.g., embedded plates or separate structural members anchored to pier and connected to the trunnion girder). Loads are transmitted from the trunnion girder to the anchorage system through these additional members, creating a larger area of pier concrete to resist posttensioning forces and loads applied to the trunnion girder. Therefore, concrete stresses will be lower, and bursting and spalling reinforcement is reduced. Additional connections and components are required for these systems.

b. Nonprestressed systems. Nonprestressed systems may consist of embedded standard shapes, large-diameter rods or built-up sections. Nonprestressed systems are relatively easy to design and install. However, nonprestressed systems allow greater deflections of the trunnion girder, allow tension and possibly structural cracking in concrete (bonded anchors), and require a large cross-sectional area of steel. This type of anchorage system is not recommended except for projects with small tainter gates. This EM provides criteria only for prestressed systems.

5-2. Components

A complete posttensioned anchorage system includes tendons (bars or strands), anchorage devices or bearing plates, ducts, end caps, grout tubes, couplers, anchorage zones, and a corrosion protection system. Anchorage zones include a portion of pier in the vicinity of tendon anchorage at either end of the tendons. The anchorage zone is geometrically defined as the volume of concrete through which the concentrated tendon force applied at an anchorage device (or girder-to-pier bearing area) dissipates to an area of more linear stress distribution throughout a cross section some distance from the anchorage device.

a. Tendons and anchorage. Tendons can be high-strength, low-alloy steel bars or strands. The tendons are posttensioned at the trunnion girder (referred to as the live end) to hold the trunnion girder to the pier. They pass through the trunnion girder and terminate at embedded bearing plates or anchorage devices (referred to as the dead end) within the pier. The embedded length of the tendons is typically 9 to

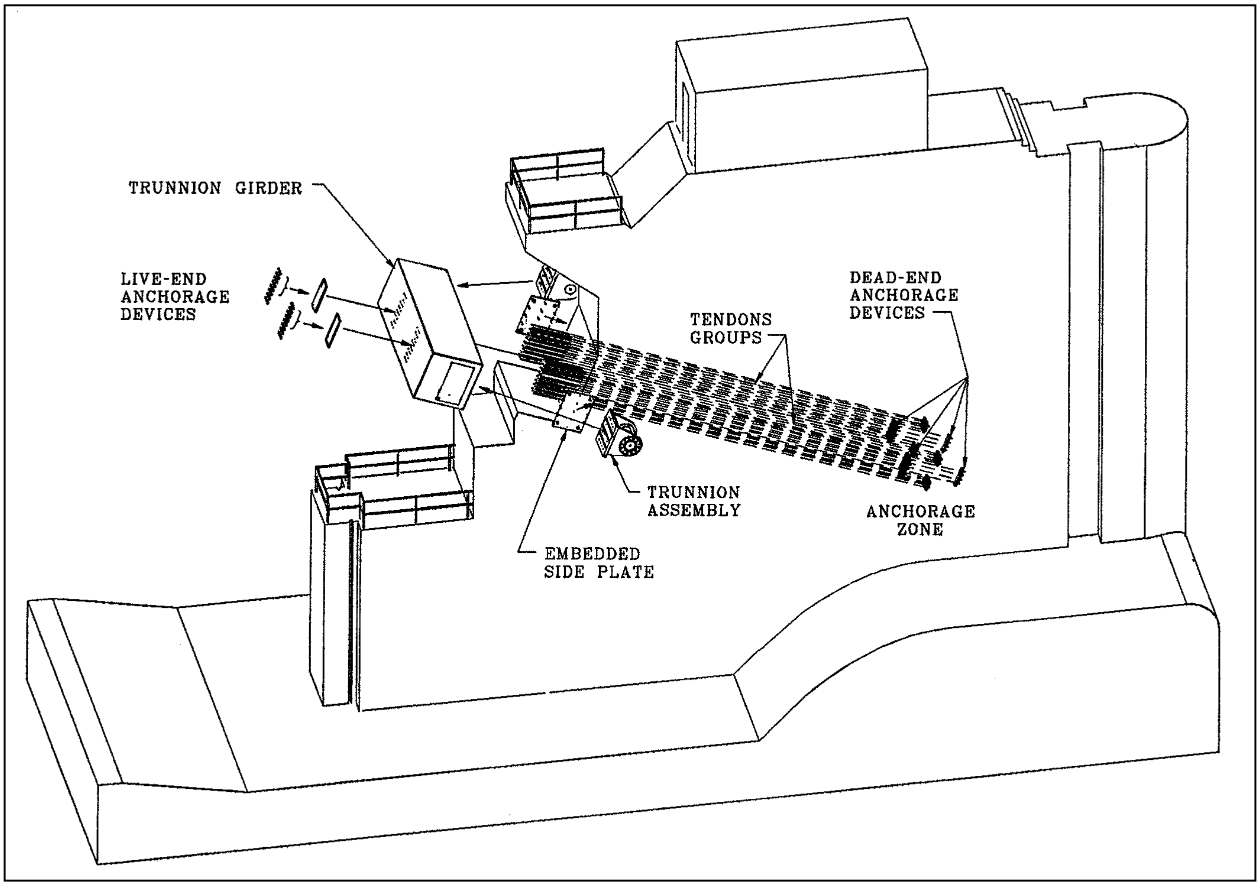


Figure 5-1. General arrangement of gate anchorage

15 m (30 to 50 ft). Longer lengths provide better control of posttensioned force and have higher pullout resistance since a larger area of concrete is effective in resisting the forces. Couplers are available to splice tendons; however, anchors are produced in sufficient lengths to make the use of couplers in typical trunnion girder installations unnecessary. The embedded ends of the prestressing steel are supported by a positive means rather than by gripping devices, which are vulnerable to slippage if grout penetrates into the anchorage device. Tendons are anchored at the dead end with embedded bearing plates that range in thickness from 25 to 63 mm (1 to 2-1/2 in.). The dead-end termination points of individual tendons are staggered from one another to distribute the transfer of load from the tendons to the concrete. Live-end anchorage devices may consist of a wedge, bell, or flat plate system and typically seat against the trunnion girder. Cable strands may also be continuous, extending from the live end to a fixed loop or 180-deg bend (that acts as the dead-end anchorage) back to the live end.

b. Tendon ducts or sheathing. Ducts encase the tendons to separate them from the surrounding pier and abutment concrete and allow tensioning after concrete has cured. The ducts also protect anchors during placement of concrete and act as part of the corrosion protection system. Ducts shall be rigid or semirigid, either galvanized ferrous metal or polyethylene, or shall be formed in the concrete with removable cores. Polyethylene ducts are usually corrugated to increase crushing resistance and to interlock with surrounding concrete.

c. Corrosion protection systems. The corrosion protection system for tendons consists of tendon ducts, duct fittings, connections between ducts and anchorages, grout tubes, end caps, and grout. A proper

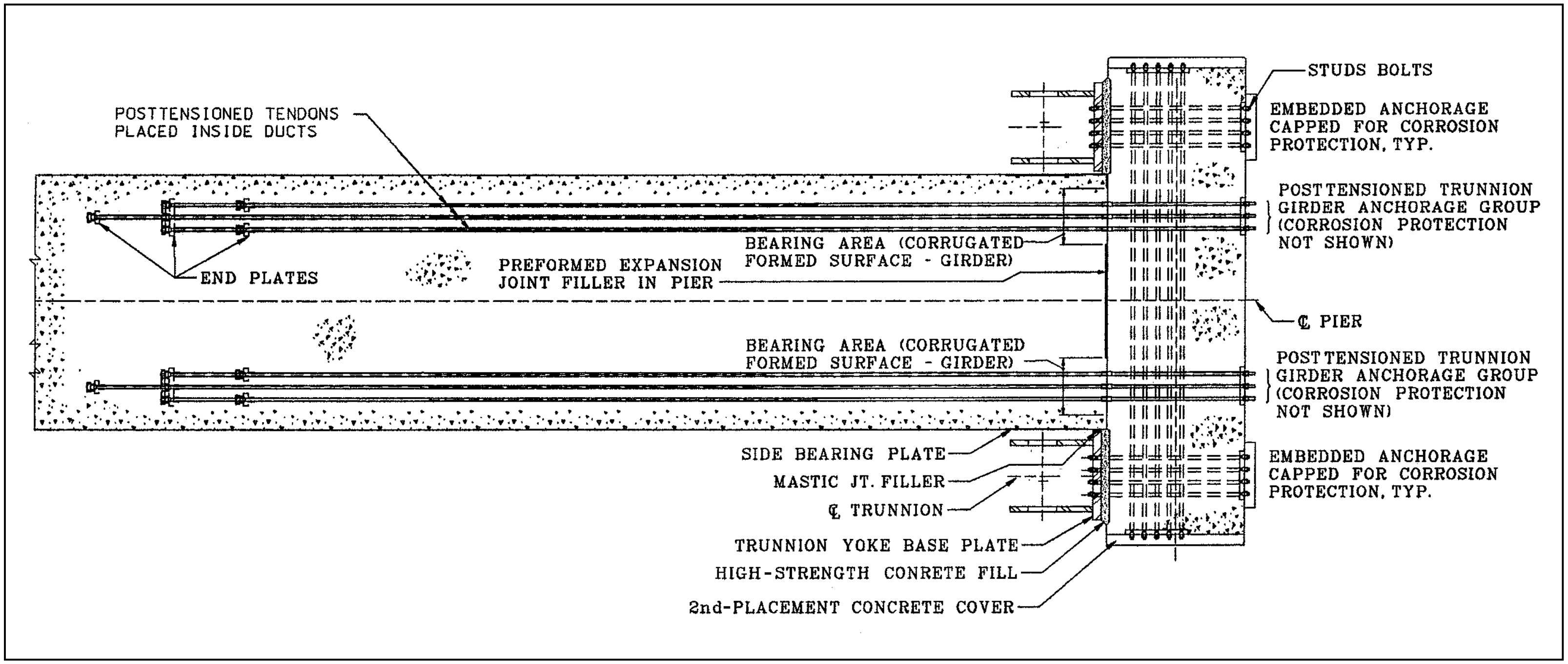


Figure 5-2. Typical posttensioned anchorage system

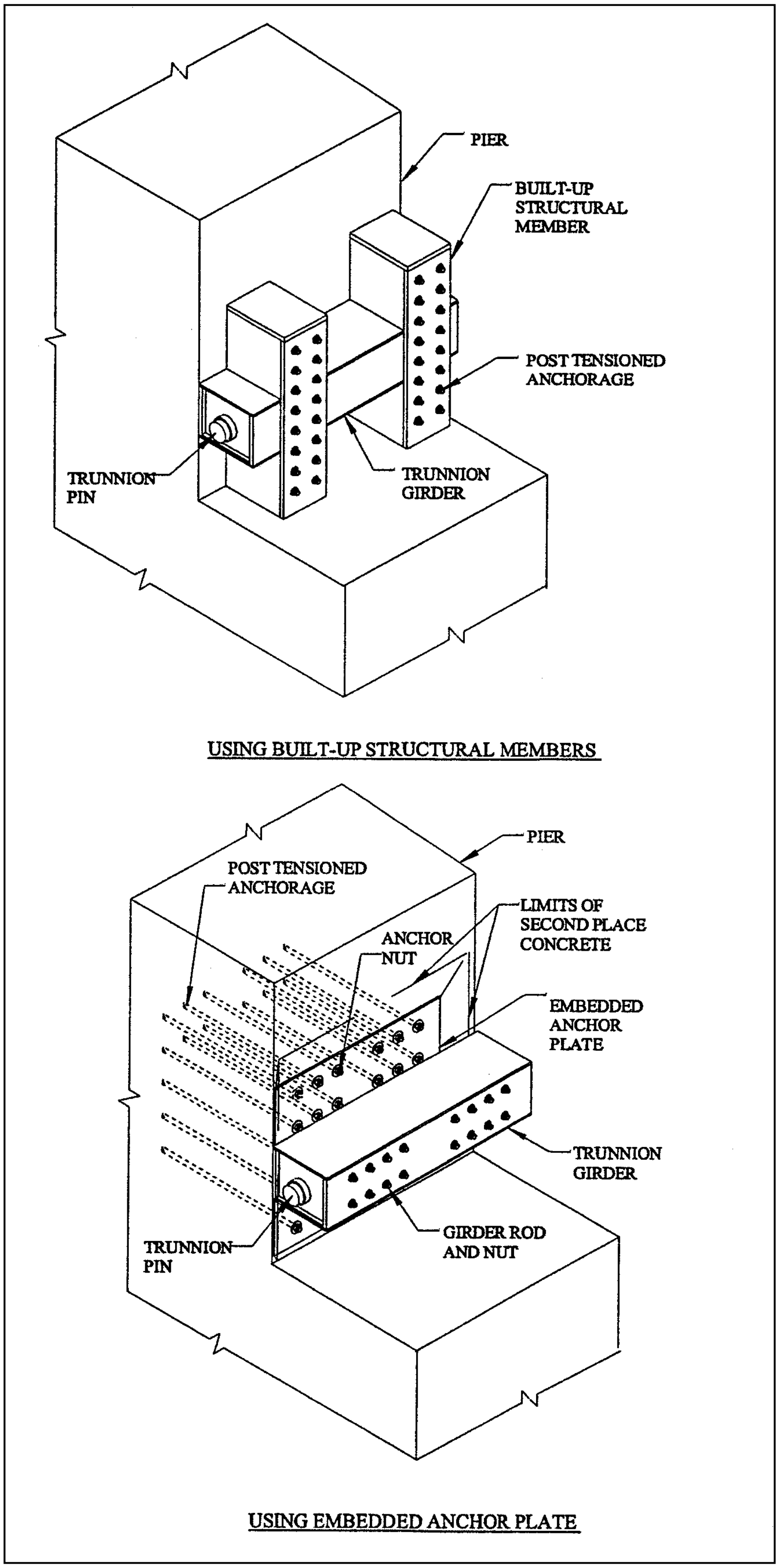


Figure 5-3. Alternate anchorage systems

duct system will prevent infiltration of moisture into the duct. Special fittings are provided for duct splices and connections between ducts and anchor plates using threaded or slip connectors and are provided with seals to prevent infiltration at these locations. Grout tubes extend from the sheathing to allow access for grouting. End caps are placed over the live end of the anchors and anchor nuts or wedges after stressing is complete and excess tendon removed. The grout encapsulates the tendon to prevent corrosion. Additional information on ducts can be found in Article 10.8.2, AASHTO LRFD Bridge Construction Specifications (AASHTO 1994).

5-3. Material Selection

a. Pier concrete. The minimum compressive strength of concrete in anchorage zones shall be 30 MPa (4500 psi). The minimum compressive strength of concrete between the anchorage zones shall be 28 MPa (4000 psi). Higher concrete strengths may be considered if required; however, specifications should include requirements on material controls and fabrication procedures to ensure the required strength. The maximum concrete aggregate size should be selected to pass between reinforcing bars.

b. Tendons. Posttensioning bars shall be of high-tensile alloy steel, conforming to the requirements of ASTM Designation A722. Cable strands shall conform to ASTM A416 with a minimum strength of 1860 MPa (270 ksi).

c. Steel for pier reinforcement. Reinforcing steel shall be deformed bars conforming to ASTM A615, ASTM A616 including Supplementary Requirement S1, ASTM A617, or ASTM A706. The specified yield strength shall be 400 Mpa (60 ksi). ASTM A706 shall be specified when bars are welded to form a cage.

d. Ducts. Galvanized steel pipe ducts shall conform to ASTM A53. Additional information on ducts can be found in Article 10.8.2, AASHTO LRFD Bridge Construction Specifications (AASHTO 1994).

5-4. Design Requirements

A properly designed anchor system will prevent structural cracking of concrete, limit deflections, account for all time-dependent stress losses, and safely accommodate specified loads for the life of the structure. Objectionable deflections and structural cracking may occur if the trunnion girder looses contact with the pier (lift-off). Lift-off can be controlled by an effective anchorage force that maintains residual compressive stresses between the trunnion girder and pier. The design of the gate anchorage system is based on strength and service limit states. The strength limit states ensure that the anchorage system will resist all factored loading combinations without failing. Service limit states are provided to restrict stresses, deformations, and cracks that adversely affect performance under typical or normal loading conditions at specified stages of use.

a. Design basis. Except as modified herein, the posttensioned anchorage system shall be designed in accordance with AASHTO (1994). The system shall be proportioned such that the limit states as specified in paragraph 5-4.c are not exceeded when the system is subject to loads as specified in paragraph 5-4.b. In general, the amount of anchorage steel required will be controlled by service limit states. Anchorage groups shall be sized such that the long-term effective anchorage force provides a minimum specified compressive stress between the girder and pier while the system is subject to the maximum trunnion reaction due to service loads. Additionally, the anchorage group shall have an ultimate strength greater than that required for factored loads. The anchorage zones, including spalling, bursting, and edge tension reinforcement shall be designed following procedures described by AASHTO (1994), Section 5.10.9, using factored jacking forces as specified in paragraph 5-4.b(1). The initial jacking force is based on the required long-term effective stress and includes all prestress losses (paragraph 5-5.h). A recommended analytical model to determine anchorage forces is described in paragraph 5-5.b.

b. Load requirements. The anchor system shall be designed to resist loads as follows. In general, the maximum load will occur when one gate is raised just off the sill and the adjacent gate (if applicable) is unloaded.

(1) Strength limit state. Load cases shall be as specified in paragraph 3-4, except that a uniform load factor of 1.9 shall be applied to each of the applied loads for the design of tendons. The anchorage zone shall be designed for the jacking load only using a load factor of 1.2.

(2) Service limit state. Load combinations shall be as specified in paragraph 3-4, except that a uniform load factor of 1.0 shall be applied to each of the applied loads.

c. Limit states. Limit states are identified for tendons, anchorage zone reinforcement, girder-to-pier bearing, and pier concrete.

(1) Strength limit state. The strength limit of the tendons shall be as specified AASHTO (1994), Section 5.9.3, and the resistance factor ϕ shall be 1.0. The strength limit of the pier concrete shall based on the criteria for an eccentrically loaded tension member as specified in AASHTO (1994), Section 5.7.6.2, and the resistance factor ϕ shall be 0.9. Resistance factors for the design of anchorage zones shall be as specified in AASHTO (1994), Section 5.5.4.

(2) Service limit state. The service limit states are intended to limit tendon and pier concrete stresses at three stages including jacking (tendons only), after transfer of jacking loads and before losses, and after losses with trunnion service loads. Tendon stresses shall be limited to those values provided in Section 5.9.3, AASHTO (1994). The tendons shall be sized to maintain a minimum compressive bearing stress between the girder and the pier of 1.4 MPa (200 psi). For pier concrete, stresses shall be limited in accordance with Section 5.9.4, AASHTO (1994), except that tension stresses are not allowed.

5-5. Analysis and Design Considerations

a. Primary axis orientation. The orientation of the tendons (or primary axis) of the gate anchorage system is usually set to coincide with line of action of the maximum trunnion reaction based on service loads. The maximum gate reaction usually occurs with unsymmetrical hoist support conditions.

b. Analytical model for anchorage forces. The trunnion girder is assumed to behave as a simply supported beam with cantilevered end spans as shown in Figure 5-4. The gate anchorage represents the simple supports. The position of support is assumed to lie at the center of gravity of the anchorage steel group. Reaction forces are determined by summing moments about the supports. The anchorage force is equal to the calculated reaction R_{max}.

c. Analytical model for anchorage zones. Analysis for the design of anchorage zones may include strut and tie models, elastic stress analysis or other approximate methods. These methods are described in Section 5.10.9 of AASHTO (1994).

d. Girder-to-pier bearing stresses. Compressive bearing stresses exist along the contact area between the girder and the pier due the posttensioning operation. Time-dependent losses and external

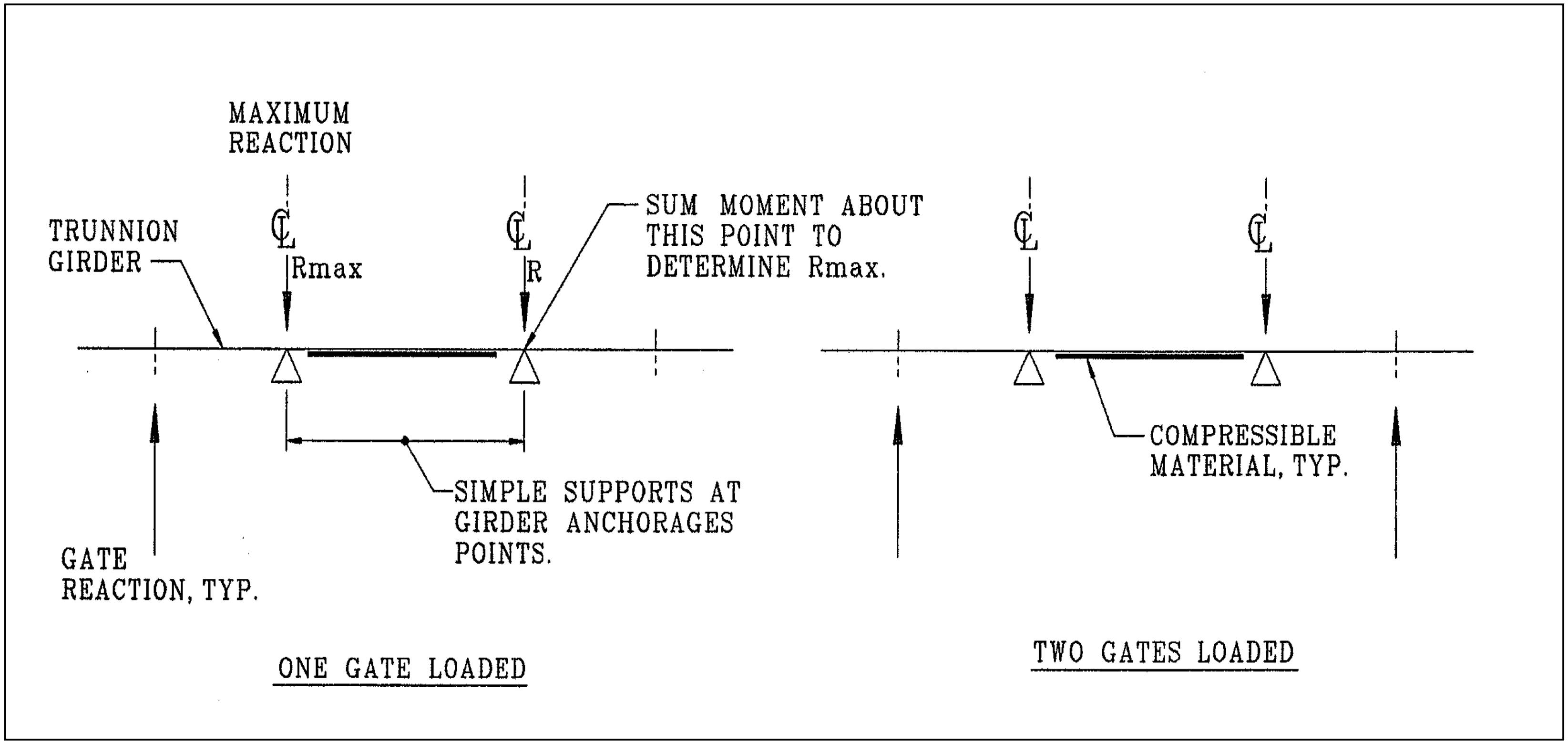


Figure 5-4. Analytical model to establish minimum anchorage force

gate loads act to reduce the bearing stresses. The net effect can be evaluated by superposition of stresses. Maximum stresses, located at the corners of the girder-pier bearing area are determined by superimposing the initial stress from the posttensioning operation, time-dependent losses, and the change in stresses due to trunnion reaction (gate) loads. The effect of gate loads can be represented as an equivalent set of normal force (P) and couples (M_x and M_y) due to eccentricity of the normal force about orthogonal axes of the contact area. The resulting bearing stresses are determined assuming that the girder acts as a rigid body. Figure 5-5 shows a typical design layout and assumed section properties of the bearing area. The bearing stress distribution for an unloaded gate and a loaded gate is shown in Figure 5-6. The change in bearing stresses due to gate loads $\Delta\sigma$ is determined as

$$\Delta\sigma = \frac{P}{A} \pm \frac{M_x c}{I_x} \pm \frac{M_y c}{I_y} \tag{5-1}$$

where

A = total bearing area

M_x = couple about the x axis

M_y = couple about the y axis

c = distance from the respective axis to the point in question

I_x and I_y = section moments of inertia as defined by Figure 5-5.

e. Finite element models. Elastic analysis of anchorage zone problems is discussed in Section 5.10.9.5, AASHTO (1994). Elastic analysis can be accomplished using 2-D or 3-D finite element models. Practical analyses are limited to linear behavior; however, models can be used to validate more simplified analyses and to determine required reinforcement in areas of complicated local conditions.

f. Anchorage depth. The depth of anchorage into the pier or abutment should be maximized to the extent possible to maximize the area of concrete effective in resisting the anchorage forces. Anchorage tendons of approximately 80 to 90 percent of the gate radius have been used with satisfactory performance. Interference with embedded metals (side-seal plates) usually limits the anchorage depth.

g. Girder movement. Girder movement across the girder-to-pier interface shall be investigated using the shear friction concept. As shown in Figure 5-7, a sufficient residual contact pressure must be available to resist shear and minimize movement. Unless other means such as dowels or mechanical confinement by adjacent concrete are provided to prevent relative movement between the trunnion girder and pier, the condition described by Equation 5-2 shall be satisfied. The right hand side of Equation 5-2 represents the available shear strength due to frictional resistance.

$$V_u \leq 0.85\mu R \tag{5-2}$$

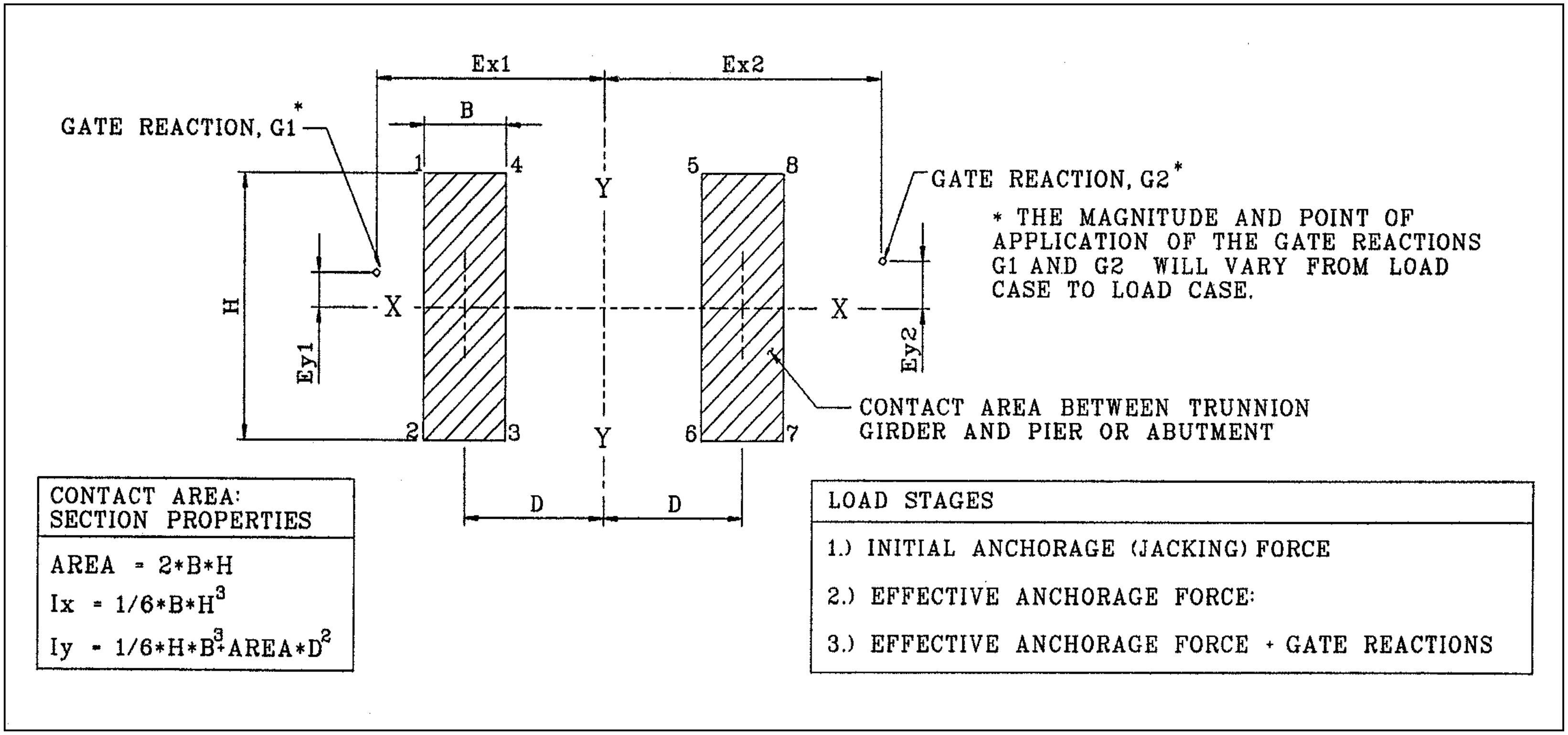


Figure 5-5. Analytical model to evaluate anchorage bearing stresses

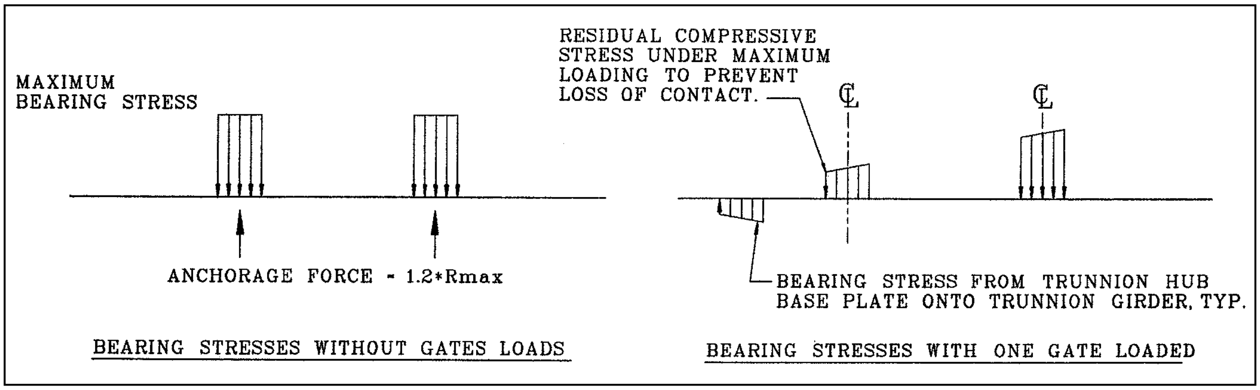


Figure 5-6. Stress distributions between pier/trunnion girder interface

where

V_u = factored shear force

μ = coefficient of friction for the interface

R = residual compressive force between the girder and pier

h. Effective stresses. Estimated lump-sum losses (assumed values not supported by computation) may be used for the initial design of prestressed concrete. However, the final design should account for individual losses due to elastic shortening, shrinkage, concrete creep, steel relaxation, anchorage set, and friction. For straight tendons, losses due to friction may be neglected. The value of anchor set loss can be assumed or obtained from the manufacturer and should be verified prior to construction. A more refined incremental time-step analysis of losses is typically not required. AASHTO (1994), Section 5.9.5, provides guidance for estimating prestress losses.

i. Anchorage zone reinforcement. A portion of pier concrete adjacent to the trunnion girder and anchor plates will be subject to tensile stresses. Tension exists in a portion of concrete located at the center of the anchorage tendons (bars) ahead of the anchorage device or girder. This area is termed the bursting zone. At the end of the pier adjacent to the girder, edge tension forces may exist along the sides and end surface. The edge tension forces along the end surface are referred to as spalling forces. Reinforcement shall be provided where required in the tendon anchorage zones to resist bursting and edge tension forces induced by tendon anchorages. Design guidance regarding bursting, spalling and edge tension is specified by AASHTO (1994), Section 5.10.9.3.

5-6. Serviceability

Corrosion of the tendons and anchorage components is the primary serviceability concern regarding design of the anchorage system. Corrosion of tendons must be avoided because it can lead to potential failure of the tendons and because the tendons are difficult to replace. The anchorage system shall be doubly protected against corrosion. With a properly constructed system, the tendons are doubly protected against corrosion since ducts and grout both prevent moisture penetration.

a. Duct and grout requirements. To eliminate the penetration of moisture or other corrosive substances, the duct system (including sheathing and all connections) shall be water or gas tight. Ducts shall be nonreactive with concrete, tendons, and grout. Ducts shall have an inside diameter at least 6 mm

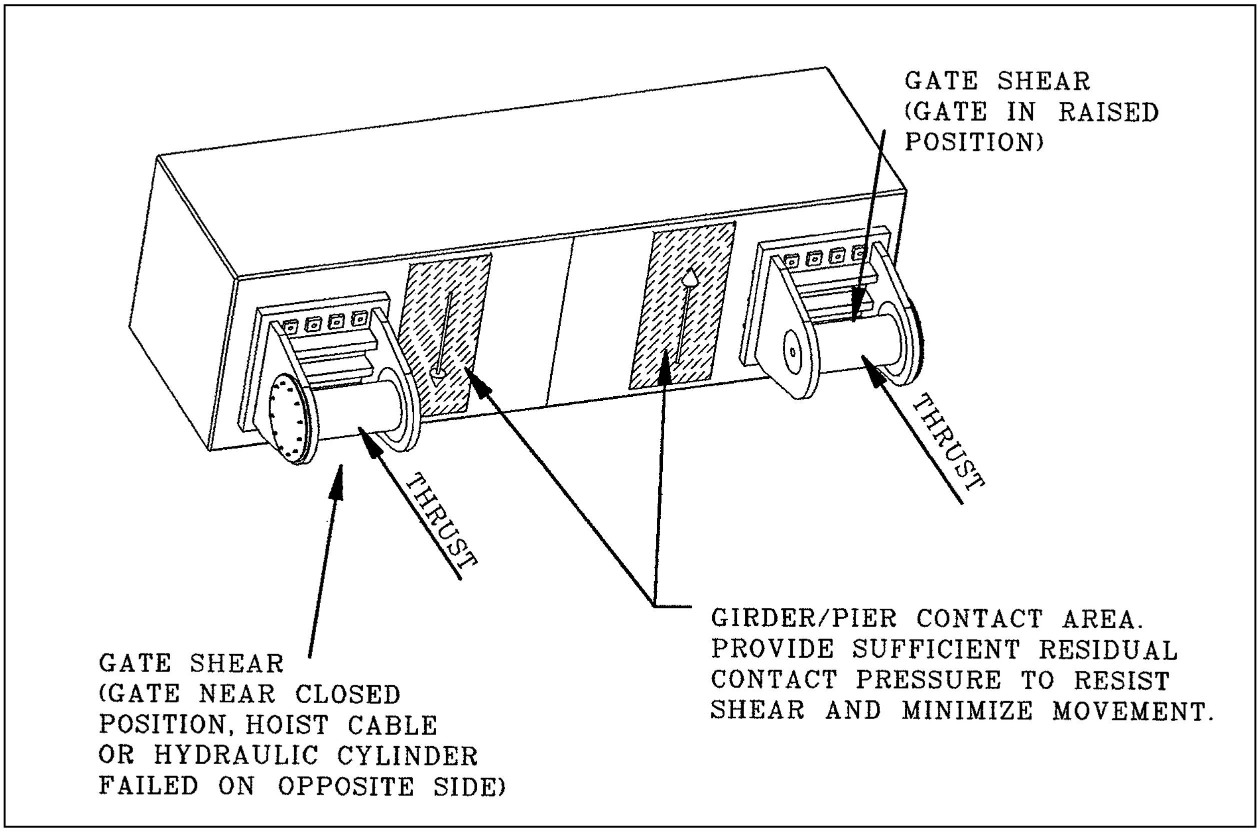


Figure 5-7. Trunnion girder movement

(1/4 in.) larger than the tendon diameter. Grout shall be placed such that tendons are completely encapsulated in grout with no air voids present. Where corrosion is of great concern, metalic sheathing should be avoided. A practice recommended in the previous version of this EM was to use a soft nonhardening, water-displacing, corrosion-inhibiting compound in lieu of grout. This was done to facilitate possible future removal and replacement of the trunnion girder; however, this practice is no longer recommended due to a poor performance record.

b. Anchorage devices and end caps. Anchorage devices shall be encapsulated by second placement concrete. Where aggressive chemical environments are encountered or where corrosion is of great concern, anchor plates and anchorage ends can be encapsulated in plastic. End caps shall be filled with grout or anticorrosion compound and should be fitted with a sealing device.

5-7. Design Details

a. Anchorage layout.

(1) The posttensioning anchorage steel should be installed in two groups with each group being located as close to the adjacent pier face as practicable. To place conventional pier steel reinforcement, clearance outside the prestressing bars of approximately 250 mm (10 in.) should be provided.

(2) Anchorage plates for alternate tendons should be installed in a staggered pattern so all anchorage loads are not transferred to one plane. This will significantly reduce the quantity of reinforcing steel that

would otherwise be required to control vertical tension cracks in the concrete. A staggered spacing of 0.6 to 1.2 m (2 to 4 ft) has been used successfully in the past to distribute anchorage forces.

b. Reinforcement considerations.

(1) Anchorage zones. Spalling zone reinforcement that consists of conventional grid reinforcement designed to resist approximately 4 percent of the total prestressing force will be adequate for most cases. The reinforcement should be placed as near to the surface as practicable. For typical arrangements, tensile stresses in the bursting zone have been shown to be a maximum of about 18 percent of the unit compression stress due to the longitudinal prestressing. Reinforcement for these stresses should be provided from the downstream pier face into the pier for a distance of approximately one-half the width of the pier.

(2) Pier corners. To prevent spalling of the concrete at the corners of the pier, it is suggested that the outer layer of reinforcement be welded to angles that are embedded along the vertical edges of the pier.

c. Construction considerations.

(1) To minimize integral action between the girder and the pier during posttensioning, all contact surfaces between the girder and the pier (except structural bearing zones) should be troweled smooth or separated by a premolded expansion joint material.

(2) Tendons can be placed horizontally if the gate trunnion is oriented so that the vertical component of thrust is negligible. With horizontal tendons, fewer lifts of anchorage zone concrete are required and construction is simplified.

(3) Pier prestressing tendons should be installed prior to installation of conventional reinforcement and forms. This will permit close inspection of the embedded ends to ensure proper construction.

Chapter 6
Trunnion Girder

6-1. General Description

The trunnion girder supports the tainter gate. The trunnion girder and associated anchorage are critical items since all loads acting on the gate are transferred from the trunnion through the girder and anchorage to the piers. The girders are located on the downstream pier face and are held in place by a prestressed or nonprestressed anchorage system that extends into the pier (Chapter 5.). Trunnion girders can be constructed of posttensioned concrete or steel. Selection of girder type (steel or posttensioned concrete) is dependent upon a variety of factors including availability of quality fabricators or precasters, site exposure conditions, economics, and designer preference. Posttensioned concrete trunnion girders are very stiff resulting in minimal deflections and offer significant resistance to torsional and impact loads. Steel girders are susceptible to corrosion and are more flexible than concrete girders but are more easily retrofitted and repaired. Due to their higher flexibility, steel girders are intended for use with smaller gates. Paragraph 6-6.c provides guidance on use of steel girders.

6-2. Components

a. Posttensioned concrete. A concrete trunnion girder may be of precast or cast-in-place construction. In either construction method, the girder is posttensioned longitudinally to control in-service deflections. (Posttensioning also increases girder resistance to flexure, shear, and torsion.) Longitudinal ducts are provided for the posttensioning tendons and recesses are commonly provided for second placement concrete pours between the trunnion assembly and pier. Conventional reinforcement is provided to resist shear, torsion, bursting, and reverse loading forces, and to control spalling. Figure 6-1 shows the upstream face of a typical concrete trunnion girder.

b. Structural steel. Steel trunnion girders are typically of welded construction and are I-shaped or box-shaped. I-shaped members can be made from plates or standard steel sections and are primarily used where torsion is not a significant concern. Box-shaped girders are fabricated from standard sections or plate steel and are much more efficient in resisting torsional loads. Box-shaped girders (especially those that include center-mounted pins) are more difficult to fabricate, coat, and inspect than I-shape girders. (Paragraph 4-1.b describes center-mounted pins.) Girder stiffeners are used to increase web stability in areas of high shear and where posttension anchorage forces are applied. Stiffeners also increase torsional stiffness and provide support for center-mounted pins. Figure 6-2 depicts a typical steel girder configuration.

6-3. Material Selection

a. Concrete girders. The minimum compressive strength of concrete shall be 33MPa (5000 psi). Higher concrete strengths may be considered; however, specifications should include requirements on material controls and fabrication procedures to ensure the required strength.

b. Prestressing reinforcement for concrete girders. The posttensioning bars shall be of high-tensile alloy steel, conforming to the requirements of ASTM Designation A722. Posttensioning strands shall be low-relaxation, high-tensile, seven-wire strands conforming to ASTM A416 with a minimum strength of 1860 MPa (270 ksi). Bars are generally preferred over strands because they are less susceptible to stress corrosion.

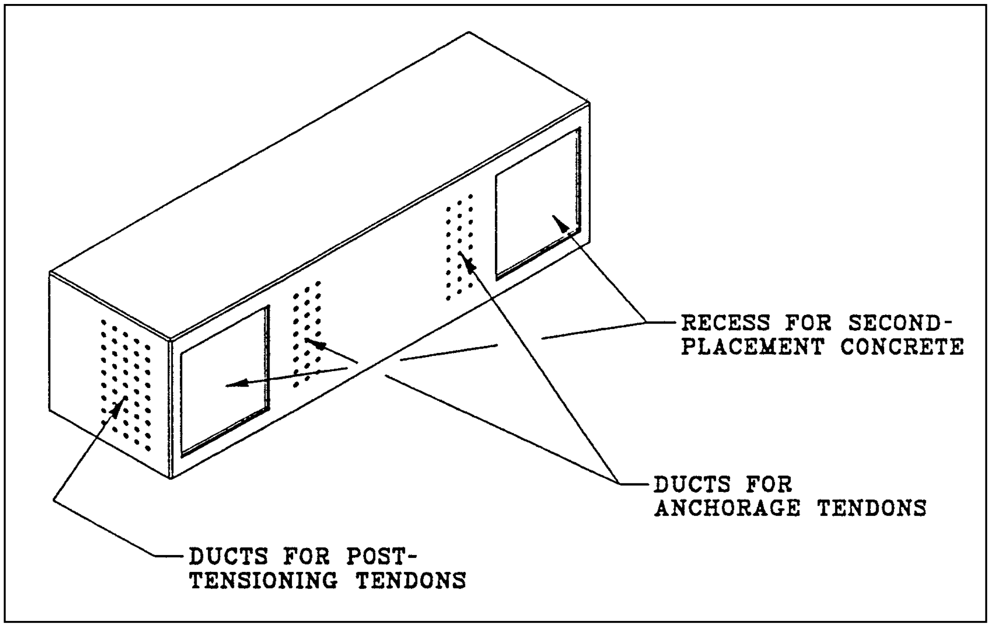


Figure 6-1. Posttensioned concrete trunnion girder

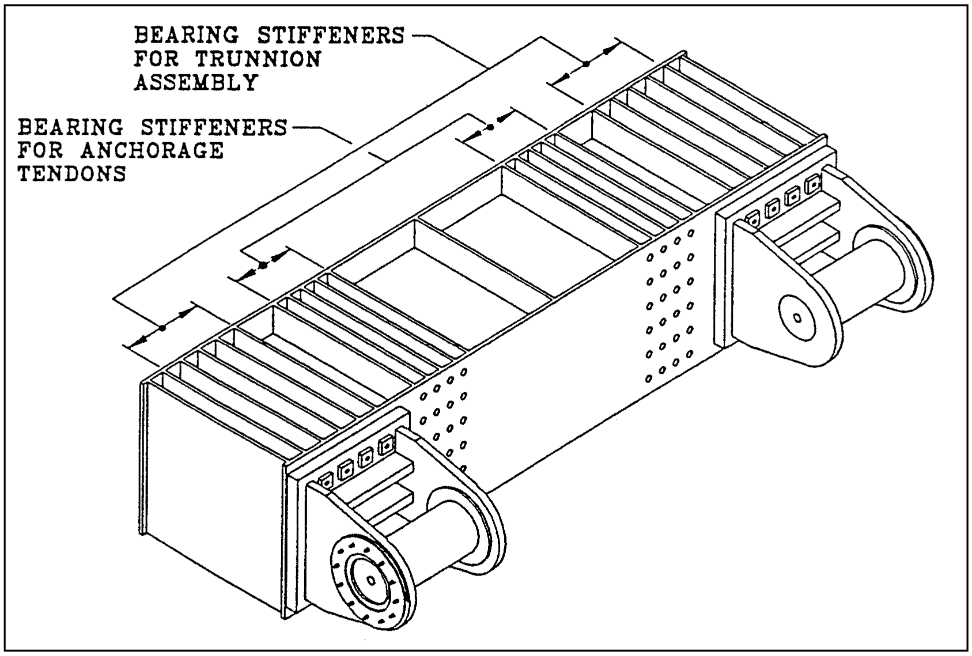


Figure 6-2. Steel trunnion girder

c. Mild steel reinforcement for concrete girders. Reinforcing steel shall be deformed bars conforming to ASTM A615, ASTM A616 including Supplementary Requirement S1, ASTM A617 or ASTM A706. ASTM A706 shall be specified when bars are welded to form a cage. The specified yield strength of deformed bars shall be 400 MPa (60 ksi).

d. Steel girder. Steel trunnion girders are considered fracture critical members (paragraph 3-8). Either ASTM A36 or ASTM A572 can be specified.

6-4. Design Requirements

Trunnion girders shall have sufficient strength to withstand combined forces of bending, torsion, shear, and axial compression due to trunnion reaction and anchorage forces. Girder torsion that occurs due to trunnion pin friction (as the gate is operated) and eccentric loads applied at the trunnion pin shall be considered. However, torsion due to trunnion pin friction shall not be considered if it counteracts torsion resulting from other loads. The design load cases shall be consistent with the operational range of the gate (i.e., consider loading from closed to open positions and intermediate positions). For multigate projects, the orientation of adjacent gates shall be considered (i.e., one gate closed, other gate open or closed or dewatered for maintenance) when evaluating the loading condition on the trunnion girder. Specific requirements for concrete and steel girders are provided in paragraphs 6-4a and b, respectively.

a. Concrete girders.

(1) Design basis. Concrete trunnion girders shall be designed based on the strength design method in accordance with ACI (1995), except as modified herein. Behavior under service loads shall be considered at all load stages that may be critical during the life of the project from the time posttensioning is first applied. The posttensioned anchorage zones shall be designed and detailed in accordance with AASHTO (1994), Section 5.10.9.

(2) Load requirements. The required strength of concrete trunnion girders shall be determined in accordance with the load combinations specified in paragraph 3-4.b, except the following load factors shall be used. A uniform load factor of 1.7 shall be applied to each of the specified loads for evaluating strength limit states and a load factor equal to 1.0 shall be applied to each of the loads for evaluating service limit states. A load factor of 1.2 shall be applied to the girder anchorage force.

(3) Limit states.

(a) Strength limit state. The design strength (nominal strength multiplied by a strength reduction factor) shall be greater than the required strength. The nominal strength and strength reduction factors shall be determined in accordance with ACI (1995).

(b) Service limit state. Service limit states are provided to limit tendon and concrete stresses at various stages including jacking (tendons only), after prestress transfer but before time dependent prestress losses, and after prestress losses have occurred (concrete only). These stages and prestress losses are discussed in Sections 18.4, 18.5 and 18.6, ACI (1995). Tendon stresses shall not exceed those specified in Section 18.5 of ACI (1995). Concrete stresses shall not exceed those provided in Table 6-1. The compressive strength of concrete at the time of initial prestress is denoted f'_{ci}, and f'_c is the specified compressive strength of concrete.

Table 6-1
Service Limit State Concrete Stresses

	Stress immediately after prestress transfer before prestress losses due to creep and shrinkage	Stress at service load after allowance for all prestress losses
Compression	$0.55\ f'_{ci}$	$0.4\ f'_c$
Tension	0.0	0.0

b. Structural steel.

(1) Design basis. Steel trunnion girders shall be proportioned and designed in accordance with paragraph 3-4 and EM 1110-2-2105.

(2) Load requirements. Steel trunnion girders shall be designed to resist trunnion loads due to load combinations specified in paragraph 3-4.b. A load factor equal to 1.2 shall be applied to the girder anchorage force.

(3) Limit states. Strength and serviceability limit states shall be considered in the design of steel trunnion girders.

(a) The strength limit states of yielding and buckling shall be evaluated in accordance with AISC (1994), Chapter H (for members under combined torsion, flexure, shear and axial forces), modified as required by paragraph 3-4.c. Resolved normal and shear stresses due to factored loads (i.e., required strength) shall not exceed the factored yield strength (i.e., design strength), and normal stresses due to factored loads (required strength) shall not exceed the factored critical buckling stress (design strength) of the member. When shear and normal stresses are of comparable magnitude, the affect of combined stresses can be evaluated by Von Mises stresses. The Von Mises stresses due to factored loads should not exceed the factored yield strength using $\phi = 1.0$ and α in accordance with paragraph 3-4.c. Careful consideration should be given for buckling in this analysis. Standard buckling equations apply to loads that are normal and transverse to the longitudinal axis of the member, whereas Von Mises stresses are usually not normal or transverse.

(b) The serviceability limit state shall be evaluated for maximum girder deformations. See paragraph 6-6.c for discussion of girder deformations.

(c) Fatigue limit states are typically not considered in the design of steel trunnion girders, since the number of load cycles is generally low and dynamic loading is not generally a concern. However, steel trunnion girders are considered fracture critical and proper material selection, welding procedures, joint details, and adequate quality control and testing is required. Paragraph 6-8 includes discussion of fracture considerations.

6-5. Analysis and Design Considerations

The trunnion girder shall be proportioned to satisfy the requirements of paragraph 6-4. The size of the trunnion girder is dependent on the magnitude of the flexural, shear, and torsion forces due to trunnion loads, and those forces due to anchorage jacking forces. The maximum shear and maximum bending forces act at the fixed end of the cantilever portion of the trunnion girder. These forces are combined with torsional and axial forces that result in a complex interaction of stresses. The design is accomplished by separating different stress contributions and designing for each individually. In general, shear and bending stresses due to transverse loading can be significant, while for most tainter gate configurations, axial stresses are minimal. Torsional shear stresses may be significant, especially for yoke mounted pins; however, these stresses can be limited by properly orienting the trunnion girder to minimize eccentricity. Maximum torsion will usually occur in the girder when the gate is partially raised and the pool is at maximum level. The trunnion girder should be designed with a compressible material over the center portion of the contact area between the girders and the back face of the pier as shown in Figure 5-2. This detail will provide a larger moment arm between the anchorage steel groups to resist gate reaction forces and will reduce negative bending moments in the trunnion girder when the main gate anchorage tendons are tensioned.

a. Analytical models. The trunnion girder can be modeled as a simply supported beam with cantilevered end spans. The supports are located at the centerlines of trunnion girder anchorages. The girder support is assumed to be torsionally fixed at the anchorage point. This model is shown in Figure 6-3.

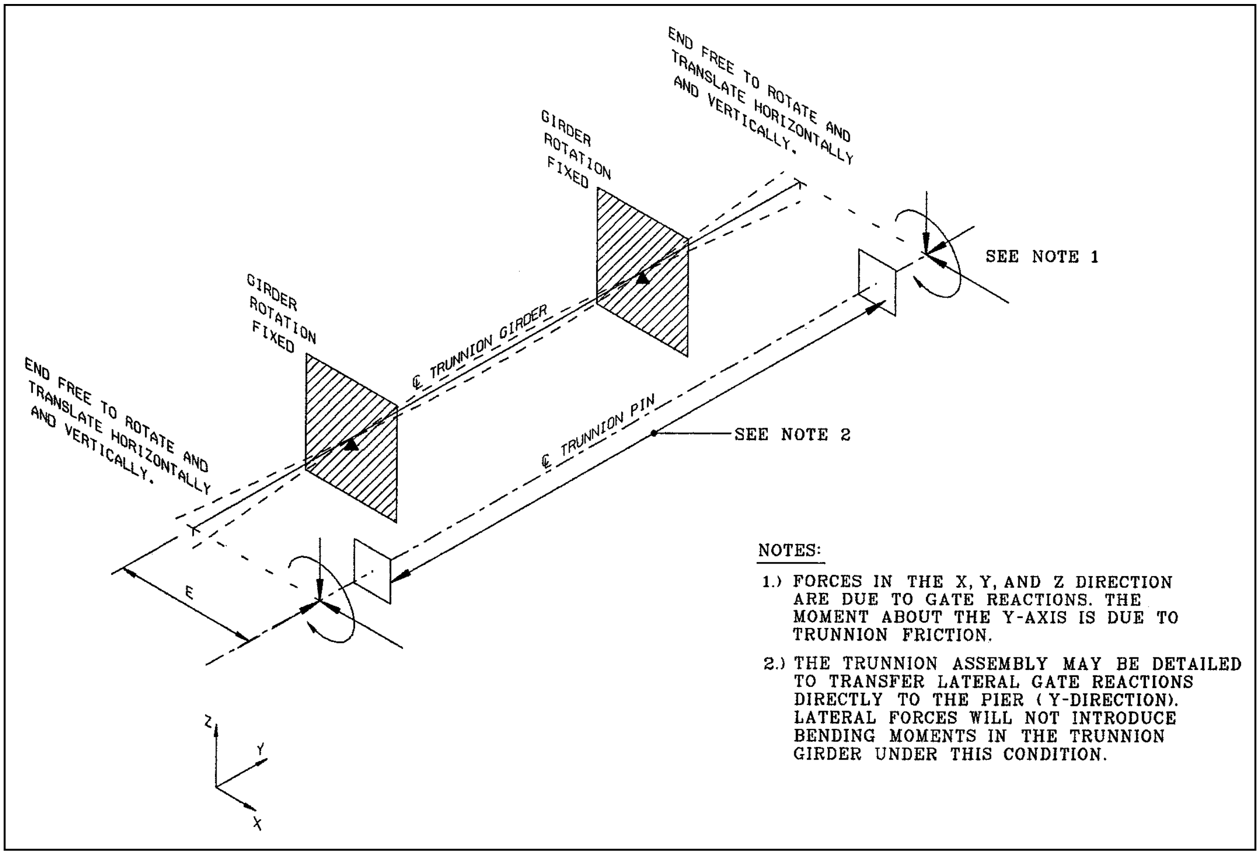


Figure 6-3. Trunnion girder analytical model

b. Concrete girders.

(1) Shear and torsion reinforcement. Shear and torsion reinforcement shall be designed in accordance with ACI (1995), Section 11.6. For design purposes, the critical section for shear and torsion shall be located at the first row of anchorage tendons closest to the trunnion. The minimum amount of web reinforcement should not be less than 0.003 *bs*, where *b* is the girder width and *s* the spacing of web reinforcement.

(2) Stressing sequence. Depending on the tendon stressing order, the controlling design stage may not be when all tendons are stressed. Consideration shall be given to bursting and spalling reinforcement needed as each tendon is tensioned in sequence. Any special requirements regarding stressing order shall be described in the contract specifications or noted on the contract drawings.

(3) Anchorage zones. Anchorage zone reinforcement to resist bursting, edge tension, and spalling shall be designed in accordance with AASHTO (1994), Section 5.10.9. This applies to anchorage zones corresponding to the girder longitudinal tendons and those due to girder anchorage forces (bearing areas between the trunnion girder and pier).

c. Steel girders.

(1) Stability. Beam flanges and webs should be proportioned to satisfy compact section requirements to avoid local buckling. Where compact sections are not practical, noncompact sections are allowed; however, slender elements shall not be used. Transverse stiffeners may be used to provide increased web stiffness. Where required, transverse stiffeners shall be designed in accordance with AISC (1994), Chapter F. The limit state of lateral torsional buckling is generally not a concern for box girders due to the relatively stiff cross sections. However, lateral torsional buckling may be a concern for I-shaped trunnion girders. In evaluating lateral torsional buckling, the unsupported length is generally determined assuming that the girder is braced at the centerlines of anchorage groups.

(2) Anchorage tendon supports. Specific members are generally required to provide support to resist the anchorage system posttensioning loads. These members may be comprised of steel plates, pipes, or tubes. Stiffener plates may be mounted on each side of an anchor and welded to the interior flanges of the girder. These plates may also serve as web or flange stiffeners. Pipes or tubes may also be mounted to the interior flanges and anchors passed through the inside of the pipes or tubes. Members shall be designed to resist jacking loads as described in paragraph 6-4.b(2). Steel plates should be designed as edge loaded flat plates, and pipes or tubes should be designed as axially loaded columns with boundary conditions that are consistent with the detailing.

6-6. Serviceability Requirements

a. Corrosion protection for concrete girder components. Ducts for tendons shall be morter tight and shall be grout-filled subsequent to posttensioning. Ducts shall be nonreactive with concrete, tendons, and grout and shall have an inside of diameter at least 6 mm (1/4 in.) larger than the tendon diameter. Anchorage devices shall be encapsulated by second-placement concrete.

b. Corrosion protection for steel girders. Corrosion protection for steel trunnion girders consists of protective coatings. I-shaped and open-end box-shaped girders may be painted, metalized, or hot-dip galvanized. Closed-end box-shaped girders may be galvanized. Box girders must include access holes for penetration of galvanizing material to the interior of the girder. The designer should locate potential galvanizers to assure that the size of girder can be accommodated. Bituminous fillers may be used to fill recesses and isolated pockets to promote proper drainage. Chapter 8 provides additional information on coatings.

c. Steel girder deflection. Deflections of steel girders due to service loads shall be evaluated for effects on tainter gate operation. Deflections shall be limited so that design stresses for bearings are not exceeded, maximum allowable bearing rotations are not exceeded (for spherical bearings), gate seal contact surfaces are maintained within acceptable tolerances, machinery loads are not exceeded, and gate and girder anchorage design assumptions are not compromised. For small gates (i.e., less than 15 m (50 ft) wide or 7 m (23 ft) high), acceptable deformations shall be determined by the owner and verified by standard engineering practice. For gates larger than this, steel girders shall be designed to achieve the same stiffness characteristics of a typical concrete girder proportioned to resist the same loading.

Alternatively, deflections may be calculated and the impact on operability determined. Such analyses shall be conducted using advanced 3-D analysis techniques such as the finite element method.

6-7. Design Details

a. Concrete girders.

(1) It has been common practice to require that the trunnion girder be completely posttensioned prior to placing adjacent pier concrete and tensioning the girder anchorage. This was done because shortening of the girder due to posttensioning would be restricted by bond to the adjacent concrete at points of bearing. This requirement can cause delays in the construction schedule. The use of second-placement concrete can be incorporated in the area between the pier and girder to eliminate this concern.

(2) Torsional reinforcement shall be provided in the form of closed stirrups. As an aid in construction, it is suggested that the conventional reinforcement be assembled as a cage with the web steel fabricated in a welded grid arrangement and welded to surrounding hoops and longitudinal steel. The longitudinal bars should not be too large since the posttensioning of the girders will have a tendency to cause buckling of these bars and may cause spalling of the concrete.

(3) The tendon spacing for the longitudinal posttensioning steel must be offset with respect to the trunnion girder anchorage tendons, allowing adequate clearance for concrete placement between ducts for longitudinal and anchorage steel. A 178-mm (7-in.) grid spacing for both the longitudinal girder and main gate anchorage tendons has been used satisfactorily in previous designs.

b. Steel girders.

(1) Consideration should be given for working room by selecting web depths and spacing of tendon supports and stiffeners so that welding can be performed without difficulty. It is recommended to provide a minimum of 200 mm (8 in.) of working room at a 45-deg angle to perpendicular members at a weld joint. If tighter working room is required by other constraints, potential fabricators should be consulted for requirements. Consideration should also be given for working room for weld test equipment.

(2) I-shaped girders are easier to fabricate than are box-shaped girders. Weld joints for flange-to-web welds and tendon support members are easily accessed. Box-shaped girders are more difficult to fabricate when center-mount trunnions and tendon supports are incorporated. The top plate may be installed in sections if welding to intermediate plates is required. Allowance for welding access may control member selection and sizing so that adequate working room is provided and quality welding can be assured.

6-8. Fracture Control

a. Material selection. Proper material selection and fabrication details and procedures will aid in control of brittle behavior of steel girders. To enhance ductile behavior, specified materials should either include mild carbon steels or maintain a carbon equivalent conducive to good welding. Weld metal should meet AWS matching filler metal requirements. Stringent toughness requirements should be considered for cold regions and where large welds (welds greater than 1 in.) are used. Toughness requirements shall be in accordance with EM 1110-2-2105.

b. Weld details. Girder end and interior stiffener or support plates should be coped at the corners to avoid intersecting welds and to provide access for coating materials. Where possible, large welds should be avoided by specifying double-bevel, full-penetration or partial-penetration groove welds, and by replacing single thick plates with multiple thinner plates. For all welding, AWS requirements should be specified for weld details, heat treatment, and matching filler metal. Stress relieving should be considered for girders with thick plates and high restraint.

c. Nondestructive testing. All structural joints and large welds should be 100 percent tested using ultrasonic or radiographic testing. Appendix B provides additional information on welding.

Chapter 7
Operating Equipment

7-1. Introduction

a. General. Traditionally, tainter gates have been operated by lifting with wire rope or chains attached to a hoist located above the gate. More recently, hydraulic hoist systems have also been utilized to operate tainter gates due to economy, reduced maintenance, and advantages concerning operating multiple gates. Chain hoists are not recommended for new designs due to past maintenance problems. The components of the lifting equipment associated with the gate shall be designed to withstand forces including the weight of the gate, silt and ice loads, and friction loads (side seals and trunnion) using the load cases defined in Chapter 3. Component design of the lifting machinery is the responsibility of the mechanical engineer.

b. Coordination. The design process is an iterative procedure where alternate hoist arrangements are evaluated jointly with the structural systems of the tainter gate and pier to arrive at a design that minimizes cost. Throughout this process, the structural engineer requires the technical assistance of a hydraulic engineer and a mechanical engineer. The hydraulic engineer specifies the opening requirements and flow nappes for various flood events. The location and layout of the machinery is dependent on gate opening requirements, and the flow nappe is needed so that the cylinder for hydraulic hoist systems can be located to avoid submergence. The mechanical engineer designs the operating machinery and determines the layout of operating machinery and operating machinery design loads. There are many acceptable solutions for the layout of the machinery and each alternate geometry will result in a different system design. For example, the requirements for a hydraulic cylinder (stroke length, bore, piston rod diameter, and operating pressure) and resulting machinery loads will be affected by the layout of the cylinders. The tainter gate design (and possibly the anchorage and pier design) is affected by the unique machinery loads of each layout. The optimum layout is determined by jointly evaluating overall costs of the hoist system, tainter gate, gate anchorage, and pier.

7-2. Machinery Description

a. Hydraulic hoist. A hydraulic hoist system consists of two synchronized hydraulic cylinders as described in paragraph 3-2.c (Figure 3-7). Each cylinder is mounted on the adjacent pier with a trunnion and is connected to the downstream side of the gate with a pin connection. The connection between the cylinder and gate is generally made near the end of a horizontal girder or at a connection in the end frame. The cylinder location should be selected to provide a practical connection to the gate and to minimize hoist loads while maintaining a relatively constant cylinder load through the required range of motion. The arrangement must accommodate required angular movement necessary to lift the gate through the range of motion. The cylinder position and point of connection of the cylinder to the gate affects the entire structural design and coordination between designers of the cylinder and gate is essential. Layout of the hydraulic cylinder is discussed in Chapter 3.

b. Wire rope hoist. A typical wire rope hoist consists of single or multiple wire ropes attached to each end of the gate, wound on overhead drums (Figures 3-6 and 3-8). The drums can be interconnected with a shaft and powered by a single motor or powered individually with synchronized motors. Although ropes can be attached to the upstream or downstream side of a gate, common practice is to attach ropes to the upstream face of the gate at lifting hitches located near the bottom of the gate. The hoist is typically located so that the wire ropes are in contact with the skin plate for the full height of the gate (in closed position) to prevent floating debris from becoming lodged between the gate face and the wire rope. A detailed discussion on the location of the wire rope and associated hoists is provided in Chapter 3.

7-3. Machinery and Gate Loads

For all of the load cases described in paragraph 3-4.b, the operating machinery may be loaded and include some force. Regarding gate design, the effect of the machinery is considered to be a gate reaction for cases in which the machinery supports the gate, and an applied load when the gate is supported otherwise. For all of the load cases with variable gate position, the machinery force varies in magnitude and direction throughout the range of motion. Therefore, for load cases 2, 3, and 4 of paragraph 3-4.b, all positions through the operating range must be accounted for in design of the gate and operating equipment.

a. Machinery loads. Operating machinery must have the capacity to support the gate during operation. Given the load requirements specified in paragraph 3-4.b, the required capacity is determined from the gate operating load conditions (load cases 2 and 3), in which the machinery loads are treated as gate reaction forces. The machinery load requirements are the reaction forces since these are the forces required to operate the gate. In determining the machinery design loads, coordination between the project structural and mechanical engineer is essential.

b. Gate loads. Due to application of appropriate factors of safety and machinery efficiency requirements, the actual maximum machinery capacity can be much larger than that required to operate the gate. Loads Q, described in paragraph 3-4.b(1) and applied in load cases 1, 4, and 5, are based on the actual machinery force. Determination of the magnitude of Q is discussed in paragraph 7-3.c.

(1) Hydraulic hoist. The hydraulic cylinders apply a concentrated load to the gate where the cylinder is attached to the gate. Hydraulic hoists may exert upward as well as downward forces when the gate is closed (load case 1) or jammed (load case 4). The cylinder and associated connections at the pier and gate must be designed to withstand the cylinder force. The force over the operational range must be considered, since the cylinder force and its affect on the tainter gate changes with changing gate position.

(2) Wire rope hoist. As opposed to hydraulic hoists, wire rope hoists can exert only upward forces. The wire rope includes a tension force, and where the rope bears on the skin plate, radial contact pressure is applied. The rope always exerts a concentrated force at the lifting hitch, and when the wire rope is not tangent to the top of the gate, a concentrated reaction force occurs at the change in rope profile (paragraph 3-2.c(5)). The lifting hitch and supporting members must be designed to withstand the concentrated rope force, and the skin plate and supporting members must be sized to withstand the contact pressure and any reaction due to change in rope profile. For design, the entire operational range must be considered, since the cable force and its affect on the tainter gate changes with changing gate position.

c. Gate load magnitude.

(1) Hydraulic hoist. Machinery loads applied to the gate (Q) are defined in paragraph 3-4.b(1), and the specific magnitude shall be determined in consultation with the mechanical engineer. The maximum downward load Q_1 is a function of the relief valve setting, the operational range of the relief valve, and dynamic effects of fluid pressure surges when gate movement stops upon impact with the sill. A back-pressure valve controls lowering of the gate. It is set to the minimum value required to hold the gate in any open position. A lowering control relief valve (with a backup) set at 330 Kpa (50 psi) above the back-pressure valve should be provided to relieve the pressure side of the cylinder and protect the system if the gate jams while closing and when the gate impacts the sill. This relief valve setting and the resultant downward load that the cylinder exerts establish the Q_1 load. If a relief valve fails, extreme loading may be possible. Use of relief valves in parallel limits the possibility of valve failure and is more economical than constructing a structural system capable of resisting this condition. As shown by Figure 7-1, the at-rest downward load Q_2 is a function of the dead weight of the piston and rod W_{ROD} and a hydraulic cylinder force P. The force P is that which

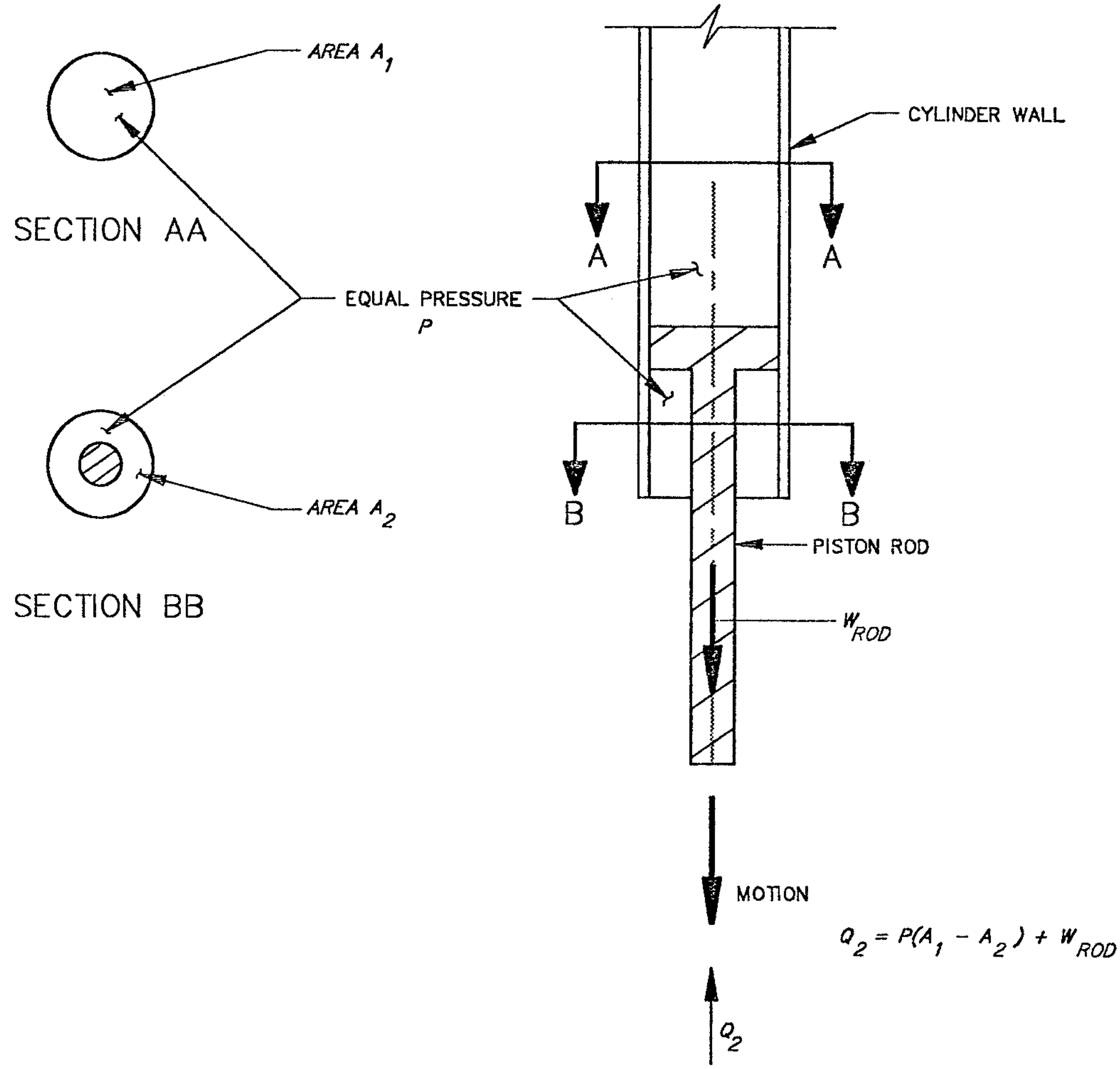


Figure 7-1. Definition of at rest downward operating machinery load Q_2

exists when the pressure across the piston equalizes when the gate is at rest on the sill. The maximum upward load Q_3 is the hoist lifting capacity based on the system pressure relief valve setting and is generally 10 to 30 percent more than that required for one hoist to raise the gate. A second (backup) pressure relief valve should be provided to protect the system if the first valve fails.

(2) Wire rope hoist. The maximum applied machinery load Q_3 is generally two to five times the required lifting load; however, the specific magnitude is generally dependent on some load limit device and shall be determined in consultation with the mechanical engineer. For efficiency, operating machinery is typically oversized. To avoid extreme loading and possible damage to the gate, load limit devices are often utilized. The

overall reliability of the structure is influenced and possibly controlled by the reliability of these load limiting devices. Certain devices have had low reliability (i.e., mechanical switches) and should be avoided. A load limit system with a high degree of reliability should be used and the load limit should be well defined.

7-4. Machinery Selection

The type of operating machinery can include either the wire rope hoist or hydraulic hoist and the selection of type should be based on project specific conditions. For a new design, the primary advantages of the wire rope hoist are that the connection is easily made on the upstream face of the gate (increases moment arm for lifting force) and that there are no environmental concerns of oil spillage. For repair or rehabilitation of existing projects, wire rope hoists may be the only practical alternative. Advantages of hydraulic hoists include the following: a) they are generally more economical especially for relatively large lifting capacities; b) they can apply force in opening and closing; c) several gates can be operated with the same power unit; d) they require low maintenance compared to wire rope systems; and e) they generally require shorter piers for support compared to wire rope systems.

Chapter 8
Corrosion Control

8-1. General Considerations

Corrosion damage will occur over time and can seriously impair structural and operational capacity of tainter gates. To minimize future structural problems and high maintenance and rehabilitation costs, resistance to corrosion must be considered in the design process. Tainter gates are vulnerable primarily to localized corrosion (i.e., crevice corrosion or pitting corrosion), general atmospheric corrosion, or mechanically assisted corrosion. Brief theoretical discussions on corrosion are presented in EM 1110-2-3400 and CASE (1993). Prudent design and maintenance practices can minimize the occurrence of these types of corrosion. Corrosion in tainter gates is best controlled by application of protective coatings but can be minimized with the proper selection of materials and proper design of details. Cathodic protection systems can be applied but are not very common on tainter gates. The selection of corrosion protection alternatives is highly dependent on the particular environment in which the gate will function.

8-2. Material Selection and Coating Systems

a. Material selection. Except for unusual cases, there are few options in selecting materials for the construction of tainter gates. Structural members on tainter gates should be constructed of all-purpose carbon grade steel (such as ASTM A36) or high-strength, low-alloy steel (such as ASTM A572, grade 50). Steel that is available today often has a dual certification. Weathering steel (atmospheric corrosion resistant, high-strength low-alloy steel, ASTM A242 or ASTM A588) that is uncoated is not recommended for use in construction of tainter gates or trunnion girders. Coated weathering steel might be warranted in certain conditions. Protective coatings applied to weathering steel typically provide longer corrosion life than those applied to other steels. The initial cost of weathering steel is generally higher than that of other high-strength, low-alloy steels, but the additional cost of coated weathering steel may be offset by the reduced maintenance costs. Embedded items that are difficult to maintain (including the sill plate and side-seal rubbing plate) should be constructed of stainless steel, and trunnion bushings are generally bronze. All carbon grade or high-strength, low-alloy steel should be coated. Where dissimilar metals are in contact, rubber gaskets or equivalent insulators should separate them. Generally, this is not necessary for stainless steel bolts, because the area of contact between the bolt and structural steel is very small.

b. Coating systems. Application of coating systems is the primary method of corrosion protection for tainter gates. For normal atmospheric exposure, alkyd enamel and aluminum-based systems provide adequate protection. For gates subject to frequent wetting and fresh water immersion, vinyl systems generally perform the best. For salt water and brackish water environments, coal tar epoxy systems are most effective. In general, vinyl systems are the most appropriate for tainter gates. Metalizing should be considered for conditions that are predicted to include extreme abrasion conditions due to ice and debris or where there are stringent regulations governing the use of volatile organic compounds (VOC). EM 1110-2-3400, CWGS 09940, and CWGS 05036 provide detailed information on selection, application, and specifications of coating systems.

8-3. Cathodic Protection

For gates or portions of gates that are usually submerged, cathodic protection should be considered to supplement the paint coatings. Since corrosion is a continuing process of removing electrons from the steel, cathodic protection introduces a slow current to counteract this effect. This essentially causes all parts of the structure to be cathodic. Cathodic protection is achieved by applying a direct current to the structure from some outside source. The direct current can be invoked either by impressed current or sacrificial anodes attached to the gate. CWGS-16643, provides guidance on impressed current cathodic protection systems for miter gates. Many of the same basic principles can be applied to tainter gates.

8-4. Design Details

Crevices, areas where ponding water may accumulate, locations where dissimilar metals are in contact, and areas subject to erosion are all susceptible to corrosion. Structural detailing has a significant impact on the structure's susceptibility to corrosion. Structures should be detailed to avoid conditions that contribute to corrosion. The following items should be considered in the design process.

a. Structural members should be detailed such that all exposed portions of the structure can be painted properly. Break sharp corners or edges to allow paint to adhere properly.

b. Drain holes should be provided to prevent entrapment of water. Extra large drain holes located in areas where the silt may be trapped (i.e., where member connections form open-ended chambers) are appropriate.

c. Lap joints should be avoided, but where used, the joint should be welded so that water can not be trapped between the connected plates.

d. Weld ends, slag, weld splatter, or any other deposits should be ground from the steel. These are areas that form crevices and can trap water. Use continuous welds.

e. Where dissimilar metals are in contact (generally carbon steel and either stainless steel or bronze), an electric insulator should be provided between the two metals. Large cathode (stainless steel)-to-anode (carbon steel) area ratios should be avoided. Surfaces of both metals should be painted. If only the anode metal is painted and there is a small defect in the coating, the cathode-to-anode area ratio will be very large and rapid corrosion can occur.

f. Where possible, welds should be used in lieu of bolts, considering the effect on fracture resistance. In general, welded connections are more resistant to corrosion than bolted connections. In bolted connections, small volumes of water can be trapped under fasteners and between plies that are not sealed.

g. In some cases, specifying a uniform increase in member component thickness provides a structure with increased resistance to corrosion damage. However, this is not recommended as a general practice. This is not effective for localized corrosion, the total structural cost is increased, and the increase in member resistance to tension, compression, and bending effects is not uniform.

h. If a protective coating must be applied, steel must generally be sandblasted prior to painting, and accessibility for sandblasting should be considered. A sandblasting hose generally cannot be bent.

Appendix A
References

A-1. Required Publications

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Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures

ER 1110-2-1150
Engineering and Design for Civil Works Projects

ER 1110-2-8157
Responsibility for Hydraulic Steel Structures

EM 1110-2-1603
Hydraulic Design of Spillways

EM 1110-2-1605
Hydraulic Design of Navigation Dams

EM 1110-2-2105
Design of Hydraulic Steel Structures

EM 1110-2-2607
Planning and Design of Navigation Dams

EM 1110-2-3400
Painting: New Construction and Maintenance

EM 1110-8-1 (FR)
Winter Navigation on Inland Waterways

CWGS 05036
Metallizing: Hydraulic Structures

CEGS 05090
Welding, Structural

CEGS 05091
Ultrasonic Inspection of Weldments

CWGS 05101
Metalwork Fabrication, Machine Work, Miscellaneous Provisions

CEGS 05120
Structural Steel

CWGS 05502
Metals: Miscellaneous, Standard Articles, Shop Fabricated Items

CWGS 05913
Tainter Gates and Anchorages

CWGS 09940
Painting: Hydraulic Structures

CWGS 16643
Cathodic Protection System (Impressed Current) for Lock Miter Gates

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American Society for Testing and Materials (ASTM) 1997
American Society for Testing and Materials, A276, *Standard Specification for Stainless Steel Bars and Shapes,* West Conshohocken, PA.

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Appendix B
Design and Specification Considerations for Fabrication and Erection

B-1. General Considerations

This appendix presents general design considerations and provides guidance on preparation of technical project specifications regarding fabrication and erection of tainter gates. Civil works guide specifications (CWGS) and military construction guide specifications (CEGS) may be used as a guide for preparing project specifications; however, the engineer must ensure that these specifications are tailored to reflect project specific requirements as required by Engineer Regulation (ER) 1110-2-8157.[1] Applicable CWGS and CEGS include CWGS 05913, CWGS 05101, CWGS 05502, CEGS 05120, CEGS 05090, and CEGS 05091. Engineer Manual (EM) 1110-2-2105 provides guidance for preparation of project technical specifications for steel used in hydraulic steel structures.

a. General specification requirements. Generally, the requirements for gate fabrication, installation, erection, and operational testing given in CWGS 05913 will apply. CWGS 05913 shall be edited to include project specific requirements consistent with guidance specified herein. Special considerations for critical or fracture critical members must be developed and specified. Although not included in CWGS 05913, bolted connections and framing members of American Society for Testing and Materials (ASTM) A 572 structural steel may be used in the construction of tainter gates. Where bolted construction and/or ASTM A 572 structural steel is specified, drawings and specifications shall be edited accordingly. In conformance with EM 1110-2-2105, project specifications should require full pretensioning of high-strength bolts for all bolted connections. Construction erection shall conform to requirements of American Institute of Steel Construction (AISC) S303, Sections 7 and 8.

b. Access and dimensional tolerance.

(1) Access design considerations. Members should be located and proportioned to provide sufficient access for workers and equipment required for fabrication, painting, and inspection. For example, there should be sufficient access to ensure that a welder has an unobstructed view of the weld root and sufficient space to apply quality welds with correct electrode angle. There should be sufficient access for sandblasting, painting, and inspection equipment and should be provisions (such as access hatches and safety railing) to provide an inspector access to frequently inspected areas.

(2) Specification tolerance requirements. The project specifications shall include provisions to ensure proper fabrication by conformance to American National Standard Institute (ANSI)/American Welding Society (AWS) D1.1 (1996) and AISC S303 (1992) requirements for fabrication and erection tolerances. (CWGS 05913 provision for alignment of elements that support skin plates is appropriate for welding of skin plate to supporting elements.) Provisions shall also be included to ensure proper gate operation and function. CWGS 05913 specifies tolerances for trunnion location and alignment and flatness of side-seal and sill plates. The specified tolerances for trunnion location are generally appropriate; however, these tolerances shall be determined by the engineer and should based on gate size and geometry.

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[1] References are listed in Appendix A.

B-2. Shop Fabrication

In conformance with CWGS 05913, gates shall be shop fabricated. Where fabrication of the gate in separate segments is necessary for handling and shipping, segments shall be proportioned to facilitate assembly and to minimize the number of joints to be field-welded. Specifications shall include specific project dependent requirements on shop fabrication, preassembly, and field fabrication. Splices shall be at areas of low stress, and segments shall be of strength and stiffness to withstand forces imparted during shipping and erection (considering temporary supports and bracing). When gates must be fabricated in sections, the gate shall be preassembled in the shop and appropriate measure shall be taken to reproduce the assembly in the field. Splice locations and associated connection details shall be determined by the design engineer and shown on the contract drawings or determined by the contractor and approved by the contracting officer.

a. Forging and casting. Items to be forged or cast shall be indicated on the drawings. Specifications shall be edited to require that forging and casting for the prescribed material be done in accordance with the applicable ASTM standard. Recommended ASTM standards are provided in CWGS 05913 and Chapter 3.

b. General welding considerations. General welding requirements shall conform to ANSI/AWS D1.1 (1996) as provided in CWGS 05101 and CEGS 05090. For special circumstances not covered (i.e., welding stainless steel), the engineer shall develop the specific welding requirements and edit the specifications accordingly, or require approval of contractor developed procedures. Guidance on welding is provided in EM 1110-2-2105.

c. Complex details. Special considerations may be warranted for fabrication of complex welded details that join main structural members or that include thick plates subject to high constraint. The engineer shall develop necessary provisions for complex details considering the following guidance and edit the project plans and specifications (CWGS 05101 and CWGS 05913) accordingly.

(1) Vertical rib-to-horizontal girder connection design considerations. The welded vertical rib-to-horizontal girder connection shown in Figure B-1 is difficult to fabricate to provide optimum fracture resistance. Unfavorable conditions that are unavoidable include separation of rib and girder flange surfaces due to rib curvature, and intersection of vertical (transverse to girder flange) and horizontal (transverse to rib flange) welds. Additionally, relatively poor access for field welding exists since the clearance between girder flange and skin plate is limited by rib depth. Although this connection has been suitable in most projects, there have been several cases involving cracked and failed connections and special considerations are warranted. Historically, welded connections have been used and will continue to be used; however, with appropriate design, bolted connections are appropriate and have some advantages over welded connections.

(a) Welded connection considerations. The connection between the rib and girder is generally field-welded subsequent to fabrication of the skin plate assembly and girders. The designer shall ensure that rib depth is enough to provide adequate welding access for vertical welds. Due to the curvature of the vertical ribs, a flat joining of faying surfaces between the flanges of the girder and rib is not possible. Filler plates of thickness necessary to provide bearing between the curved rib flange and girder flange shall be included. A vertical seal weld of size necessary to fill the gap and cover the fill plate is generally provided. If the vertical weld is designed as a structural weld, then ANSI/AWS D1.1 (1996), Section 2.13, applies, and filler plates shall be selected such that the maximum gap between the girder flange and rib flange does not exceed 5 mm (3/16 in.). Generally, the vertical weld joining the edge of

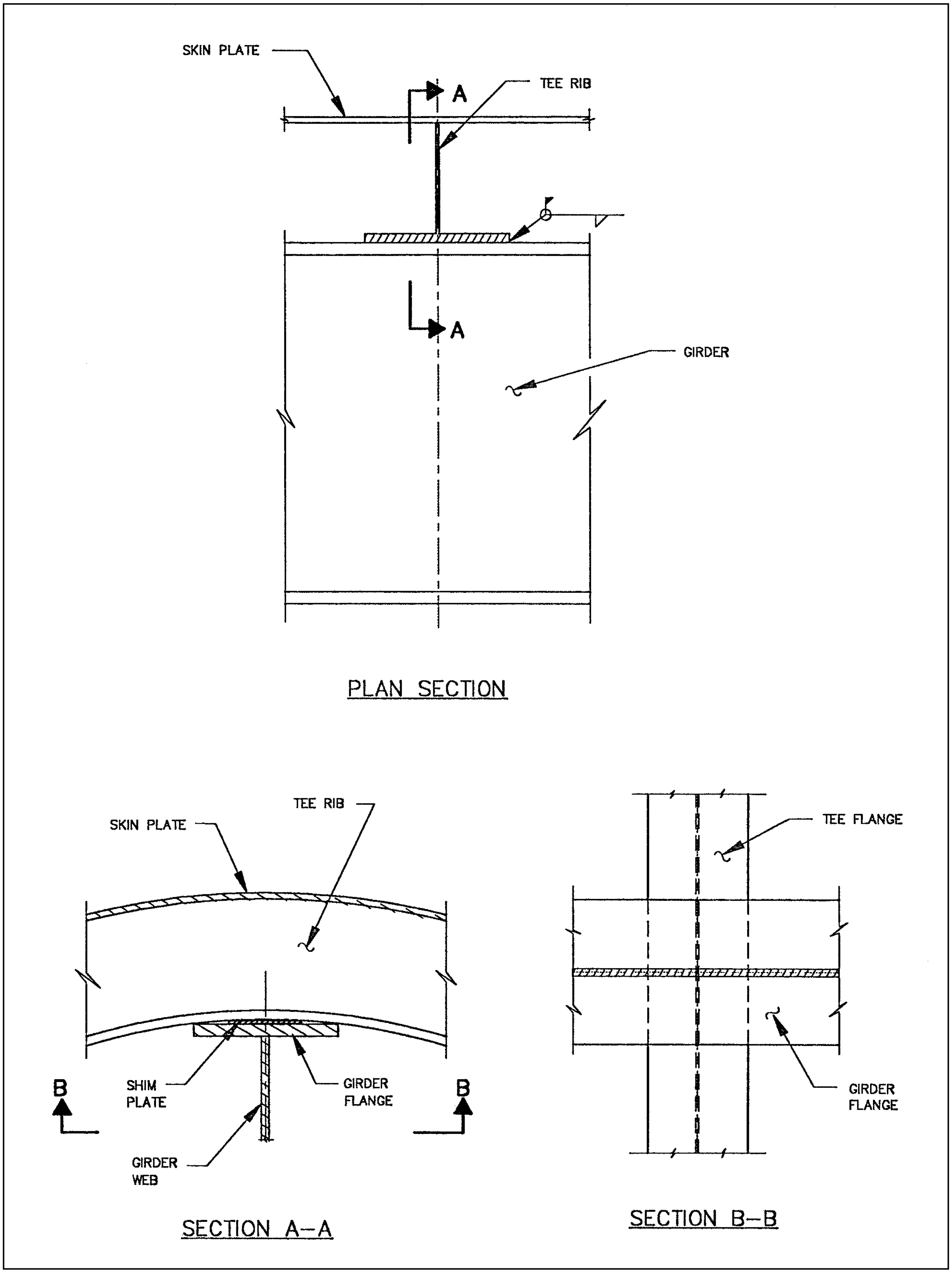


Figure B-1. Rib-to-girder connection (welded)

the rib flange to the girder flange is not a structural weld, since the horizontal weld is of adequate strength. Additionally, the horizontal and vertical welds are deposited on opposite sides of a common plane of contact to join the rib flange and girder flange. AISC (1994), Section J2.2, and ANSI/AWS D1.1 (1996), Section 2.4.7.2, require that welds deposited on opposite sides of a common plane of contact between two parts be interrupted at a corner common to both welds. This is not possible if the overlapping area is sealed. However, to minimize the adverse effects, the horizontal and vertical weld size shall be no larger than that necessary to provide a seal within 25.4 mm (1 in.) of their intersection.

(b) Bolted connection. The bolted connection alternative consists of an arrangement of four bolts joining the girder flange and rib flange (Figure B-2). The primary considerations in design are to select the appropriate bolt size to withstand the applied loads and to provide a solid bearing between the nut of the bolt and the bolt head. This can be accomplished with specially fabricated shims or by using a combination of shims and bevel washers. There must be adequate bearing to withstand full pre-tensioning of the bolts. The bolted connection alternative alleviates the problems associated with welding including intersecting welds and clearance between the girder flange and skin plate. The primary disadvantage is that the overlapping area between flanges of the rib and girder is not sealed from water intrusion.

(2) Full-penetration flange welds and weld access holes. At member intersections or splices where the flange of a rolled or built-up shape is welded with a full-penetration weld, weld access holes should be provided and detailed in compliance with ANSI/AWS D1.1 (1996), Section 5.17. Groove welds that do not have steel backing or back gouging are considered partial-penetration welds regardless of size, and backing is not possible without a weld access hole through the web. Additionally, without an access hole, an unfavorable residual stress condition exists where the flange and web welds intersect. This condition exists in the strut-to-girder connection, splices that may exist between the trunnion hub assembly flange plates and end frame struts, and at intersections of end frame members. At critical connections for intersecting members, the engineer shall show access hole requirements on the drawings or require approval of contractor submittals.

(3) Thick plate weldments. Weldments involving thick plates are particularly susceptible to cracking compared to those of thin plates. (A thick plate is generally considered to be 38 mm (1-1/2 in.) or greater in thickness.) The structural engineer shall develop a welding procedure (or require approval of a contractor developed procedure) for critical thick plate weldments and edit project specifications accordingly. General requirements for welding of thick plates are provided in EM 1110-2-2105. Trunnion yoke plates, trunnion bushing assembly, cable attachment brackets, steel trunnion girders, and built-up members generally include weldments with thick plates and/or high constraint. For the trunnion hub and yoke assemblies, plates are subject to large through-thickness stresses. The designer may require improved through-thickness properties by specifying steel with a maximum sulfur content of 0.01 percent and/or verification of resistance to lamellar tearing by testing in accordance with ASTM A 770/A 770M.

d. Trunnion. The engineer shall determine specific requirements for yokes, pins, and associated cladding, hubs, and bushings in accordance with guidance provided in Chapter 4 and edit project plans and specifications (CWGS 05913), accordingly.

e. Trunnion girder and anchorage. The engineer shall determine specific requirements for the trunnion girder and anchorage in accordance with guidance provided in Chapter 5 and Chapter 6 and edit project plans and specifications (CWGS 05913), accordingly.

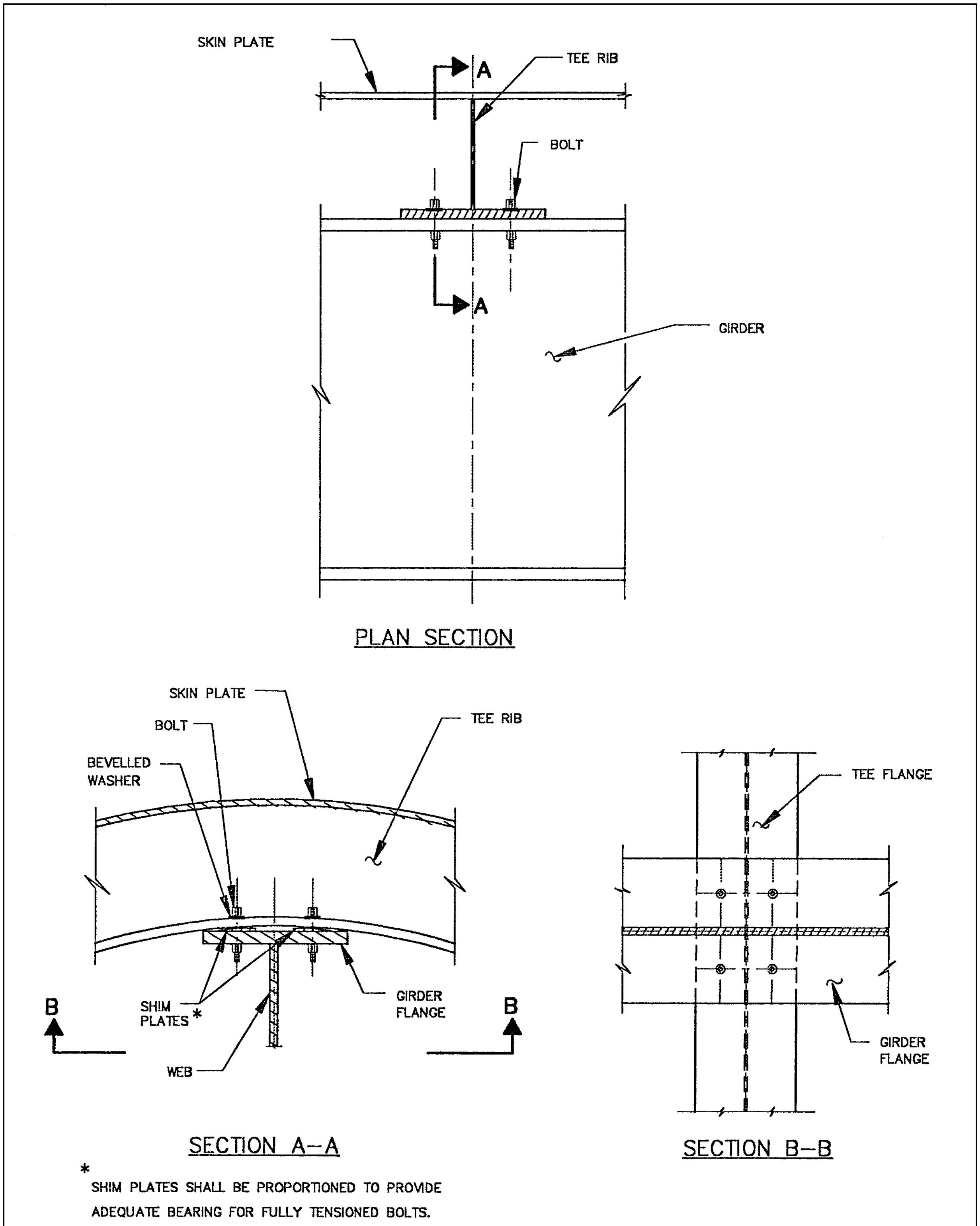


Figure B-2. Rib-to-girder connection (bolted)

f. Welding inspection requirements. CWGS 05101 shall be edited to specify contractor requirements for weld inspection considering general weld inspection requirements specified in EM 1110-2-2105 and the following:

(1) Prior to welding, thick (greater than 38 mm (1-1/2 in.)) plates of the trunnion hub and yoke assemblies that are subject to through-thickness weld residual stresses shall be examined in accordance with ASTM A435. In the vicinity of the weld, 100 percent of the plate surface shall be inspected.

(2) Critical groove welds including those in steel trunnion girders, at girder splice locations, at the end frame to girder connection, and trunnion weldments shall be inspected by ultrasonic inspection (UT) and/or radiographic inspection (R). Critical fillet welds are generally best inspected using penetrant inspection (PT) and/or magnetic particle inspection (MT). Critical welds and associated testing methods shall be determined by the engineer and shall be designated on the contract drawings.

(3) In accordance with ANSI/AASHTO/AWS D1.5 (1995), all tension butt welds of fracture critical members (FCM) shall be inspected using R and UT. All other tension groove welds in FCM shall be inspected by UT. The engineer shall determine all FCM and shall clearly designate all FCM and required nondestructive examination on the contract drawings.

B-3. Field Fabrication and Erection

Project specifications (CWGS 05101 and CWGS 05913) shall be edited to ensure conformance to fabrication, erection, and installation requirements specified herein. Specifications shall ensure that construction erection conforms to requirements of AISC S303 (1992), Sections 7 and 8, and the specifications shall require approval of contractor submitted erection plan prior to construction.

a. Gate assembly. Field assembly of the gate shall be conducted such that the assembled components are not over-stressed or unstable during assembly. The gate shall be assembled in conformance with the shop match markings using temporary supports that shall be removed after fabrication is completed. Prior to field welding and/or bolting of permanent connections, controlling dimensions and alignments shall be within specified tolerances. All field welding and associated nondestructive examination shall conform to requirements specified herein for shop fabrication. Skin plate welds shall be tested for water tightness after the gates are installed but prior to painting and mounting of seals, and disclosed leaks shall be sealed by appropriate means to be approved by the contracting officer.

b. Trunnion installation. Trunnion yokes and pins shall be installed and aligned in accordance with requirements provided in Chapter 4. For greased trunnion systems, the trunnion lubrication system lines shall be purged and filled with grease before the final connection to the gate. Bearing surfaces of trunnion pins and bushings shall be cleaned and coated with grease prior to installing the trunnion pins. For self-lubricating trunnion systems, bearing surfaces of trunnion pins and bushings shall be cleaned prior to installing gates.

c. Anchorage system and trunnion girder installation. Procedure for installation and anchorage of the trunnion girder shall conform to requirements of Chapters 5 and 6.

d. Protective coating and cathodic protection. The engineer shall determine the corrosion protection considering the project specific environment as discussed in Chapter 2, and guidance provided in Chapter 8 and CWGS 05913 shall be edited accordingly.

Appendix C
Operation and Maintenance Considerations

C-1. General

To maintain operational capacity is a key consideration in the design of tainter gates. Operational capacity may be inhibited by various circumstances including normal deterioration due to corrosion and unintended loading (vibration, accumulation of ice, obstruction, etc.). Optimum operation can be assured by selection of appropriate design details and by providing continued maintenance and inspection.

C-2. Design Considerations

Tainter gate designs shall include provisions to minimize effects of corrosion and unintended loading and provisions to provide access required for inspection and maintenance. In addition to a protective coating system, corrosion can be controlled by appropriate detailing. Unintended loading is minimized by proper design of seal details, ice control, and trunnion lubrication. Bulkheads and safety devices provide access for inspection and maintenance.

a. Design details for corrosion control. Using continuous welds instead of intermittent welds, rounding sharp corners, and seal welding lap joints can minimize localized corrosion. Structural members should be located such that access for future inspection and necessary maintenance is provided. All areas should be visually accessible and provide access for inspection and painting equipment. For very large members, access manholes may be necessary. Provision for drainage should be included where water can accumulate such as on the webs of the girders, end frames, and bracing members. Drain holes should be located in areas that are lowest while the gate is in normal position. The size of hole is dependent on the size of drainage area and generally ranges from 25 mm (1 in.) to 75 mm (3 in.) in diameter. The cut edges of holes should be smooth and free of notches, especially in areas where the cut member is subject to tensile forces.

b. Control of unintended loading.

(1) Design details to control flow-induced vibration. Flow-induced vibration may occur due to flow under the gate or over the gate. Regarding flow under the gate, bottom seals of various configurations have been a source of vibration problems in tainter gates. This vibration can be minimized or eliminated with use of proper lip and bottom seal configurations as discussed in paragraph 3-7.a(2). (Other configurations are discussed in EM 1110-2-1605.)[1] The girder and bottom lip should be sized to have adequate stiffness to limit flexing that would permit flow between the lip and sill. Bracing of the cantilevered portion of the skin plate provides rigidity. Vibration (or debris) loading may also occur due to overtopping water and debris that may impact the upper girder. For conditions where this is a concern, deflector plates positioned to protect the upper horizontal girder will minimize the effects of impacting water and debris.

(2) Deicing systems. Devices for preventing the formation of ice, or to thaw ice adhering to the gates and seals, are necessary for gates that must be operated in subfreezing weather. Ice formation may freeze a gate in place or obstruct motion, and buildup of ice may add sufficient weight to overload gate hoists. Various solutions exist to control ice formation and buildup (Haynes et al. 1997). The most

[1] References are listed in Appendix A.

common methods of ice control for tainter gates are use of direct heating systems or air bubbler systems. Direct heating is the most effective means of controlling ice. Air bubbler systems for tainter gates are generally slow and inefficient; however, they may be beneficial when used to supplement direct heating systems where unusually severe climatic conditions exist.

(a) Direct heating systems. Direct heating units can be installed at appropriate locations on or around a gate. Generally, heater units consist of steel cells with embedded electric heating elements or heat tape with heat transfer fluid. Heating elements are generally helical formed coil of chrome-nickel, encased in a seamless sheath of corrosion-resisting, nonoxidizing metal. All heater units should be designed as removable units so replacement is easily accomplished. Side-seal heaters generally consist of a tubular steel cell with embedded heater elements that is fabricated to fit in a recess behind the side-seal rubbing plate along its length. Another method of heating the side seal is to heat the J-seal directly by inserting heat tape into the hollow portion of a hollow J-seal. The heated J-seal should be used to supplement embedded heaters, since the J-seal would prevent ice formation only along the J-seal, while the remainder of the side-seal rubbing plate could accumulate ice. Heat tape is discussed in EM 1110-8-1 (FR). To control ice formation on gate surfaces, box-type heater units can be attached directly to the downstream side of the skin plate and/or along the end frame members. These units are essentially flat rectangular steel cells that encase heating coils. The cells are sealed and insulated to direct heat to the gate surface. Radiant (or infrared) heaters provide another means to control ice on gate surfaces (Haynes et al. 1997). These heaters can be suspended in close proximity to the structure and may be practical in areas that are difficult to access.

(b) Air bubbler systems. Air bubbler systems are easy to use and can be custom designed for a particular project purpose to move ice and reduce ice growth. Bubbler systems consist essentially of compressed air circulated through a pipe system. Air can be released through outlets located in front of the gate sills to create a circulation of warmer water from the bottom of the reservoir toward the face of the gate. This will prevent the formation of ice on the face of the gate and tends to keep an area of open water upstream from the gates. Air bubbler systems are discussed in EM 1110-8-1 (FR) and Haynes et al. (1997).

(3) Lubrication. Lubrication of the trunnion is a primary design consideration. Lubrication reduces trunnion friction to minimize lifting forces and flexural forces in end frame members. The surface of the trunnion pin and bushing must be sufficiently lubricated for efficient operation of a gate. Lubrication can be accomplished by direct application of grease through the hub and bushing to the bushing-pin interface. Grooves are fabricated on the inside face of bushings to contain grease that is injected through holes drilled in the hub and bushing. Another option is to use self-lubricating bronze bushings.

(4) Debris protection. Unintended forces may occur due to accumulation of debris. Debris may become lodged between the gate and adjacent piers resulting in gate binding during lifting or may accumulate on gate members adding to the gate weight. Deflector plates can be attached to the end frame struts and on the downstream flange of girders at appropriate locations to provide debris screens. It is recommended that deflector plates be fabricated of abrasion resistant and lightweight material such as ultra-high-molecular-weight polyethylene.

c. Access.

(1) Temporary closure. Means to provide temporary closure between piers for emergency situations or gate maintenance or repair operations should be considered in design. In some cases, project operation may permit gates to be out of service for a known period, and temporary closure may not be needed.

Vertical lift gates and sectional bulkheads are the most common types of temporary closure structures. A discussion on maintenance and emergency closure facilities is included in EM 1110-2-2607. Maintenance bulkheads are generally provided upstream of tainter gates, and downstream if necessary, to accommodate unwatering between piers to provide a dry area for maintenance or repair activities. Since closure is for planned inspection and maintenance, maintenance bulkheads are designed for static heads and cannot be installed in flowing water. Bulkhead guide slots and sills should be located such that there is adequate space to permit the installation of maintenance scaffolding for use in maintenance and repair operations. Other factors that should be considered in determining bulkhead location are pier size and location and capability of bulkhead lifting equipment. The emergency bulkheads provide closure between piers under flowing conditions. Emergency closures can be provided upstream or downstream of the tainter gates; on many navigation projects, it may be advantageous to provide closure downstream of the service gate since many accidents involve barges on the upstream side. The decision to include emergency closure structures should be based on economic analysis of costs and associated benefits of emergency closure given a gate failure.

(2) Safety and critical items. Gate stops are attachments that provide a block to prevent a gate from being raised past the maximum elevation for safety and operational concerns. Gate stops are generally welded attachments that extend from the ends of the lower girder between the outside of the gate and the surface of the pier. When the gate is in the fully raised position, the stops contact stop beams that are mounted on the pier. To provide safe and easy access for inspection and maintenance, handrails and ladders may be attached to gate members using welded or bolted connections. The connections should be designed to minimize adverse effects (stress concentration, residual stress) on main structural members. Attachments should be located away from tension zones if possible and good detailing practice is essential.

C-3. Inspection

ER 1110-2-100 prescribes general requirements for periodic inspection of completed civil works structures that are applicable to all project features. Supplemental more specific requirements for inspection of hydraulic steel structures (HSS) are specified in ER 1110-2-8157. The structural engineer shall develop an inspection plan for each tainter gate in accordance with requirements of ER 1110-2-8157 and those specified herein. To conduct a detailed inspection for each tainter gate on a project is not economical, and detailed inspection must be limited to critical areas. The inspection plan should identify which elements require inspection and what nondestructive testing is required for each. The focus of an inspection should be on fracture critical members, and then critical members and connections most susceptible to various forms of degradation. Design drawings and computations, previous inspection reports and all operations/maintenance records since the most recent inspection should be reviewed to develop the inspection plan.

a. Identification of critical areas. The most critical elements are those that are determined to be fracture critical. Fracture critical elements are those that are subject to tensile stresses whose failure would cause the structure to collapse. Other critical structural elements include those that are most susceptible to degradation including fracture and corrosion.

(1) Fracture. Stress level, stress concentration, material thickness (affects residual stress, toughness, and constraint), quality of fabrication (i.e., weld quality, tack welds, intersecting welds, or poor accessibility), operational vibration or overload, displacement-induced secondary stress, and load distribution are each factors that may contribute to fracture. Fracture is most likely to occur at locations where high-tension stress and severe stress concentration exist. To identify critical areas for fracture,

locations of moderate-to-high nominal tensile stress level and details that include significant stress concentrations must be identified. The effects of stress level and sensitive details are combined to determine the critical areas for inspection. Stress levels are determined by appropriate structural analysis and the severity of stress concentration imposed by a particular detail is reflected by its particular fatigue category. Fatigue categories are specified in AISC (1994). Tainter gates generally have significant tensile stresses in the downstream flanges of the girders at the midlength (lower girders are more critical), in the upstream girder flange and the outside flange of end frame struts near the girder-strut connections, and where the end frames join the trunnion assemblies. Significant tensile stresses may also occur in end frame bracing members (due to trunnion pin friction), and in the upstream flange of skin plate ribs at the horizontal girders. For in-depth engineering inspections, critical areas that should be inspected for cracking are:

(a) Fracture critical components. Fracture critical components may include lifting brackets, lifting cable or hydraulic machinery components, various components of the trunnion assembly and trunnion beam, girders, and various end frame members. Fracture critical components may vary on different projects and shall be determined by the design engineer.

(b) Locations identified as susceptible to fracture or weld-related cracking. Trunnion weldments, trunnion girders (steel), lifting bracket weldments, the girder-rib-skin plate connection on the upstream girder flange near the end frames, the bracing-to-downstream girder flange connection near midspan, and the girder-to-strut connection are areas most susceptible to fracture. At intersecting welds and where previous cracks have been repaired by welding are also areas of concern.

(2) Corrosion. Corrosion can occur at any location on a gate; however, certain areas are more susceptible than others. Sensitivity to corrosion is enhanced at crevices, locations of dissimilar metals, areas subject to erosion, and areas where ponding water or debris may accumulate. Other areas that are susceptible to corrosion include where it is difficult to adequately apply a protective coating, such as at sharp corners, edges, intermittent welds, and locations of rivets and bolts. Corrosion-susceptible locations on tainter gates include trunnions with dissimilar metals, seal connection plates and rib-girder connections (crevice corrosion and dissimilar metals), locations of bolts (crevice corrosion), drain holes, and general areas where sharp corners, edges, and intermittent welds are located. Other areas where corrosion is likely to occur is where heaters are located and at the normal water line.

b. Inspection procedure.

(1) Inspection for cracks. Common nondestructive field methods to inspect for cracking include visual, penetrant, ultrasonic, and/or magnetic particle inspections. These and other methods are described in detail by ANSI/AWS B1.10 (AWS 1986). Visual examination is the primary inspection method and shall be used to inspect all critical elements. A visual inspection is "hands-on" and requires careful and close examination (particularly with the aid of a magnifying glass). Critical areas should be cleaned prior to the inspection and when necessary, additional lighting should be used. If cracks are suspected, penetrant, ultrasonic, and/or magnetic particle inspections should be used to confirm the extent of cracking.

(2) Inspection for corrosion. Appropriate tools that may be used to measure and define corrosion damage include a depth micrometer (to measure pitting), feeler gages (to quantify the width of a crevice exhibiting corrosion), an ultrasonic thickness gage (for measuring thickness), a tape measure, and a camera. Guidelines to quantify the severity of corrosion damage are given by Greimann, Stecker, and

Rens (1990). Nondestructive inspection techniques to inspect for corrosion damage include visual and ultrasonic inspections.

(a) Visual inspection. A visual inspection of all corrosion susceptible areas should be made to locate, identify, and determine the extent of corrosion. When corrosion is observed, the type, extent, and severity should be reported. Any failure of the paint system should also be identified. The extent of paint system failure and regions of localized discoloration of gate components should be recorded. In areas where paint failure has occurred, the gate surface should be visually examined for pitting. When pitting is present, it should be quantified by using a probe type depth gage following ASTM G46 (1994).

(b) Ultrasonic inspection. Ultrasonic inspection is useful to measure thickness loss and can be used to obtain a baseline reference for thickness when it is unknown. The thickness of a steel plate or part can be determined to an accuracy of approximately 0.13 mm (0.005 in.). The technique can be conducted through a paint film or through surface corrosion with only a slight loss in accuracy. Where only one side of a component is accessible, the open surface can be scanned to identify thickness variation over the surface to determine where corrosion has occurred. Ultrasonic inspection equipment may not be reliable when pitting corrosion is prevalent, because the size and depth of the pitting impair the output signal of the transducer.

(3) Operational components. Mechanical and electrical components including seals, lifting mechanisms, bearings, limit switches, cathodic protection systems, and heaters are critical to the operation of spillway gates and should be inspected appropriately. These components should be checked for general working condition, corrosion, trapped debris, necessary tolerances, and proper lubrication.

c. Inspection reports. Inspection reporting shall be in accordance with requirements of ER 1110-2-8157.

Appendix D
Data for Existing Tainter Gates

Spillway tainter gates of various types and sizes have been used on more than 150 USACE projects. New gates that are designed likely have similar characteristics to some that have been designed in the past. Past designs should be utilized as much as possible to avoid duplication of effort. The following table includes a listing of selected existing projects that include tainter gates. Table D-1 provides basic information including gate height (H), gate width (W), gate radius (R), gate design head (DH), gate weight (WT), and miscellaneous information. A more complete listing is provided by Cox (1972). Figures D-l and D-2 show details of flood control and navigation projects, respectively.

Table D-1
Data for Existing Tainter Gates

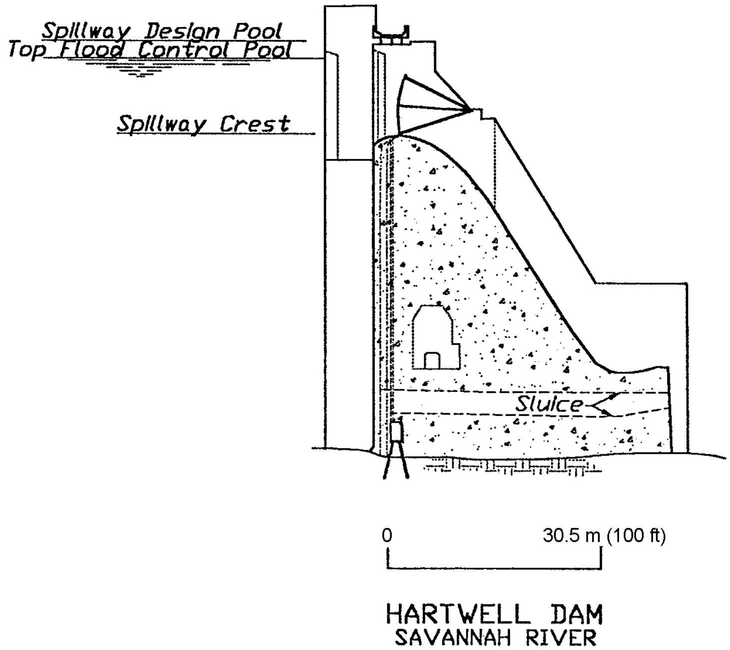
Project	H m (ft)	W m (ft)	R m (ft)	Year	DH m (ft)	WT KN (kips)	Miscellaneous
Holt Lock and Dam Black Warrior River Mobile District	10.7 (35)	12.2 (40)	13.7 (45)	1966	10.7(35)	663 (148)	Navigation Project Inclined end frames Steel trunnion girders
Dear Creak Lake Huntington District	10.7 (35)	12.8 (42)	9.4 (31)	1968	11.9 (39)	394 (88)	Flood Control Project Inclined end frames
Carters Project Coosawattee River Mobile District	11 (36)	12.8 (42)	11.6 (38)	1973	8.5 (28)	659 (147)	Parallel end frames
Pine Flat Dam Kings River Sacramento District	11.6 (38)	12.8 (42)	11.9 (39)	1953	11.6 (38)	520 (116)	Flood Control Project Inclined end frames Chain hoist system
John W. Flannagan Lake Huntington District	11.6 (38)	12.8 (42)	11.6 (38)	1967	11 (36)	565 (126)	Flood Control Project Inclined end frames
Lock and Dam 5 Mississippi River St. Paul District	4.6 (15)	10.7 (35)	7.6 (25)	1935	4.6 (15)	233 (52)	Navigation Project Parallel end frames
Lock and Dam 26 Mississippi River St. Louis District	12.8 (42)	33.5 (110)	16.5 (54)	1992	12.8 (42)	4,384 (978)	Navigation Project Parallel end frames
Lake Shelbyville Kaskaskia River St. Louis District	11.3 (37)	13.7 (45)	10.7 (35)	1970	11.3 (37)	515 (115)	Flood Control Project Inclined end frames
Kaskaskia River Navigation St. Louis District	9.2 (30)	18.3 (60)	13.7 (45)	1973	9.2 (30)	1,031 (230)	Navigation Project Parallel end frames
Hartwell Dam Savannah River Savannah District	11 (36)	12.2 (40)	12.2 (40)	1963	10.7 (35)	489 (109)	Flood Control Project
Dwarshak Dam Northfork Clearwater River Walla Walla District	17 (56)	15.2 (50)	16.8 (55)	1971	17 (56)	1,255 (280)	Flood Control Project Inclined end frames

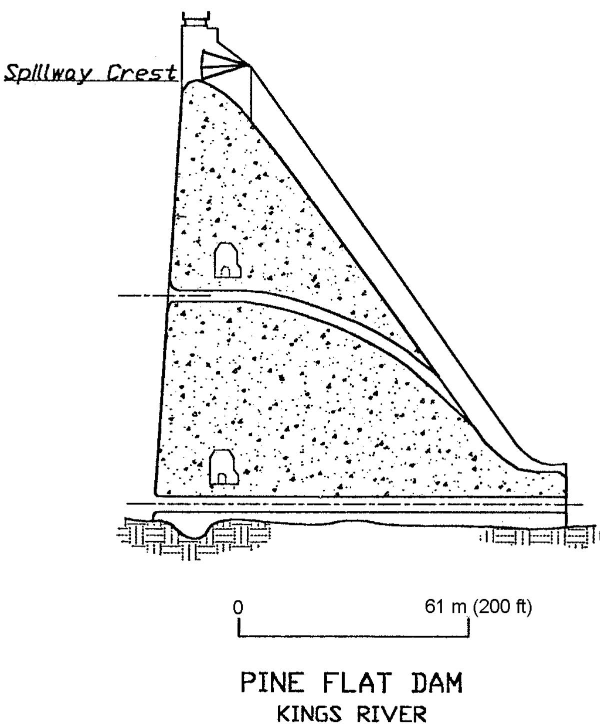
Note: All gates have wire rope hoist systems except as noted.

(Continued)

Table D-1 (Concluded)

Project	H m (ft)	W m (ft)	R m (ft)	Year	DH m (ft)	WT KN (kips)	Miscellaneous
Lower Granite Lock and Dam Snake River Walla Walla District	18.3 (60)	15.2 (50)	18.3 (60)	1975	18.3 (60)	1,345 (300)	Flood Control Project Inclined end frames
Willow Island Lock and Dam Ohio River Huntington District	8.5 (28)	33.5 (110)	14.9 (49)	1976	7.9 (26)	2,600 (580)	Navigation Project Parallel end frames
Racine Lock and Dam Ohio River Huntington District	10.3 (34)	33.5 (110)	17.4 (57)	1971	10.3 (34)	3,138 (700)	Navigation Project Stressed skin type fabrication
Old River Auxiliary Control Structure Mississippi River New Orleans District	22.9 (75)	18.9 (62)	21.6 (71)	1981	55	3,810 (850)	Framed with vertical end girders and seven horizontal girders
Winfield Lock and Dam Huntington District	8.5 (28)	33.5 (110)	14.6 (48)	1997	7.9 (26)	2,564 (572)	Navigation Project Parallel end frames Hydraulic hoist system



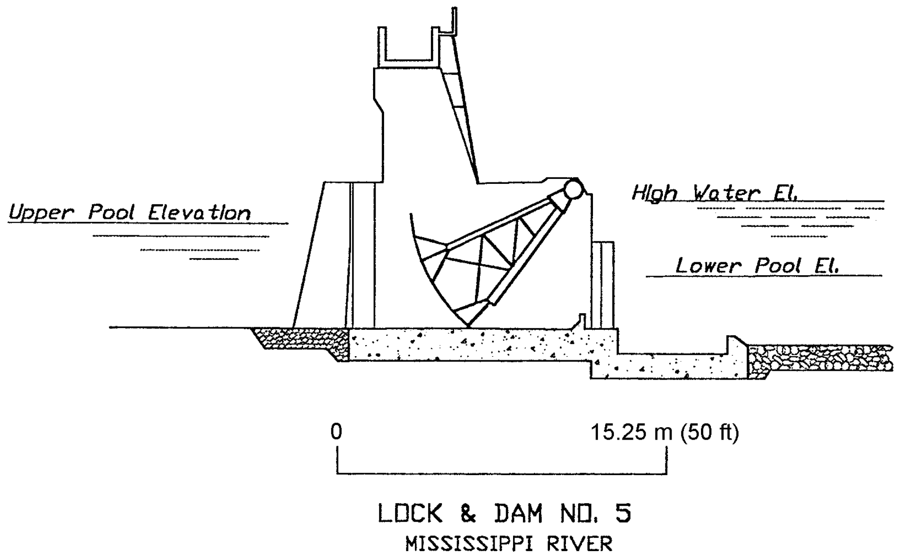


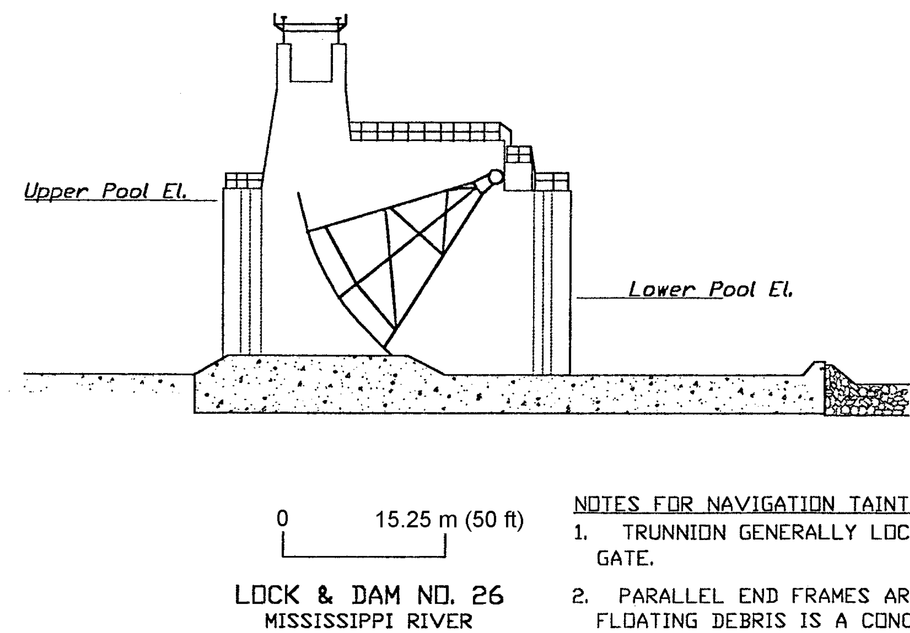
NOTES FOR FLOOD CONTROL TAINTER GATES

1. TRUNNION GENERALLY LOCATED AT APPROXIMATELY ONE THIRD OF THE GATE HEIGHT ABOVE THE SILL.

2. END FRAMES ARE GENERALLY INCLINED.

Figure D-1. Typical flood control projects





NOTES FOR NAVIGATION TAINTER GATES:

1. TRUNNION GENERALLY LOCATED NEAR TOP OF GATE.
2. PARALLEL END FRAMES ARE PREFERRED WHERE FLOATING DEBRIS IS A CONCERN.

Figure D-2. Typical navigation projects