

Preferred Practices
for
Steel Bridge
Design,
Fabrication, and Erection

2005

Texas Steel
Quality Council

Texas Department
of Transportation
(TxDOT)

Preferred Practices for Steel Bridge
Design, Fabrication, and Erection

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1. Introduction

1.1. Overview

This document provides guidance to help steel bridge designers working on Texas Department of Transportation (TxDOT) projects to achieve optimal quality and value in steel bridges.

It is maintained by the Texas Steel Quality Council, which is a joint owner-industry forum comprised of the following:

- TxDOT design, fabrication, and erection engineers; TxDOT inspectors
- FHWA bridge engineers
- Academics
- Steel bridge fabricators, detailers, and trade association representatives
- Steel mill representatives; and design consultants.

The Council meets regularly in an open forum to discuss best practices for achieving the best steel bridges, and this document reflects the Council's agreements.

Open and informed participation by representatives from all aspects of steel bridge construction is instrumental to the Council's success, and the Council welcomes and encourages all comments. Submit comments and suggestions to the TxDOT Steel Quality Council, c/o Steel Bridge Fabrication, Materials Section, Construction Division, TxDOT, 125 E. 11th St., Austin, Texas, 78701-2483, or send comments by email to hgilmer@dot.state.tx.us, jholt@dot.state.tx.us, or rmedloc@dot.state.tx.us.

1.2. Revision History

Version	Publication Date	Summary of Changes
1	November 2000	New Document
2	October 2005	Updated to accommodate AASHTO Load and Resistance Factor Design (LRFD) specifications.

2. Design

2.1. Preliminary Design Considerations

The success of a steel bridge design depends on the preparation of the designer:

- Have a well founded knowledge of design specifications.
- Be familiar with the construction and fabrication specifications and standard drawings that apply to steel structures.
- Be aware of construction and fabrication specifications' influence on design and any modifications or special provisions they may require.
- Take time during initial decision making to consult with fabricators, steel erectors, and contractors for ideas on achieving economical, easily built designs.
- While designing, think about how everything fits together—for example, how rotation, deflection (especially differential deflection), twist, stiffness (vertical bending, lateral bending, and torsion), and skew affect interaction between different elements.
- Always provide clear and distinct load paths that mitigate or, preferably, eliminate out-of-plane bending.
- Always provide enough access for bolting, welding, and painting. Keep designs simple by maximizing the use of common details and minimizing the number of plate sizes and rolled shapes fabricators are required to purchase. Complicated details are always hard to fabricate and build.
- Never use details that permit water and debris to collect on girders.

Designs that merely satisfy design specifications are rarely good. Good designs reflect consideration of the requirements of fabrication, construction, and maintenance.

2.1.1. Steel Grade Selection

If it is acceptable for the location, use weathering steel (A 709 Grades 50W and HPS 70W) left unpainted. FHWA Technical Advisory T 5140.22 and Section 2.1.2.3 of this document provide guidelines on acceptable locations. Some TxDOT districts object to the use of weathering steel, so obtain district approval before using weathering steel in a design.

Although weathering steel is slightly more expensive initially than non-weathering steel, it is ultimately more economical because it does not require initial or maintenance painting.

Weathering steel performs as well but not better than non-weathering steel in painted applications, so avoid requiring weathering steel in painted applications. Fabricators

may prefer to use weathering steel for painted bridges, and TxDOT allows substitution of weathering steel for non-weathering steel if all other material requirements are satisfied.

Avoid using hybrid girders consisting of A 709 Grade 36 webs with A 709 Grade 50 flanges for painted bridges. Although these hybrids may seem economical, the cost difference of the two grades of steel is actually minimal.

In contrast, A 709 Grade HPS 70W may be economical only in hybrid girders. With Grade 50W webs, use a hybrid configuration with HPS 70W tension flanges and HPS 70W compression flanges in negative moment regions. The use of HPS 70W steel may be restricted by:

- The fact that its stiffness is not increased over lesser grades.
- Its availability. Do not specify HPS 70W steel in a design unless availability is verified with local fabricators—for example, rolled sections are currently not available in this steel.

A 709 Grade HPS 50W is also a high performance steel, but TxDOT does not recommend designs requiring its use.

A 709 Grade 50S (bridge steel equivalent to A 992) is acceptable for painted rolled beam bridges and anywhere structural shapes are used in painted applications.

Avoid using A 709 Grade 100 or 100W steel; Grade 100 steel has no real application for Texas because typical bridges are short- and medium-span bridges. Use A 709 Grade HPS 100W if 100-ksi steel is needed.

For cost analyses in the design phase, use the actual price of steel per pound instead of a typical bid price per pound. Fabricators, steel suppliers, or AISC can be consulted during the design phase of a job to provide the latest information about steel costs.

2.1.2. Proper Use of Weathering Steel (A 709 Grades 50W and HPS 70W)

You can easily ensure good performance from weathering steel and reduce or eliminate unsightly concrete staining. For detailed recommendations, see TxDOT's report on Project 0-1818, "Performance of Weathering Steel in TxDOT Bridges," Chapter 4.

2.1.2.1. Design Strategies

Provide drip plates (also called drip tabs) to divert runoff water and protect abutments and columns from staining. Provide them on every girder because staining may occur before slab placement. Consider what the diverted water will stain—for example, do not place drip plates so close to substructure elements that wind blows diverted water onto the substructure. Drip plates over concrete riprap at bridge embankments cause stain spots on the riprap, but the alternative is widespread staining of both the abutment and riprap.

If necessary, provide stainless steel drip pans to protect abutments and columns. Doing so is impractical with large tub girders and may not be necessary for plate girders with good drip plate details.

Provide details that take advantage of natural drainage.

Provide adequate drainage beneath overpass structures to prevent ponding and continual traffic spray from below. Communicate the importance of adequate drainage to roadway designers.

Provide stiffener clips for proper ventilation and drainage. The stiffener clips shown on TxDOT standard drawings SGMD, “Steel Girder Miscellaneous Details,” and SBMD, “Steel Beam Miscellaneous Details,” are adequate for this purpose.

Eliminate details that retain water, dirt, and other debris.

Do not detail deck drains that can discharge water onto the steel, especially in regions that use de-icing chemicals.

Either completely seal box or tube members or provide adequate drainage and venting to allow condensation in unsealed tubular and box sections to dissipate. See Section 2.5 for more on this subject.

Use sealed expansion joints. See Section 2.1.5 for more guidance on expansion joints. Avoid any type of open joint that allows runoff to reach the steel.

Specify application of an adequate protective coating to surfaces that may be subject to standing water. However, avoid details that create this situation.

Specify application of an adequate protective coating to weathering steel that will be embedded in soil or gravel pockets. The coating should be one of the types used on carbon steel in the same environments, and it should extend above the interface of the embedment for several inches.

2.1.2.2. *Fabrication and Construction*

The 2004 TxDOT Standard Specifications require the following:

- Fabricators must blast clean (SSPC-SP6) fascia surfaces of weathering steel elements before shipping them.
- The Contractor must spot clean fascia steel surfaces in the field after the slab concrete has been placed.

Eliminate identification and other markings on the outside face of any fascia beam, including any markings placed during erection.

2.1.2.3. *When Not to Use Weathering Steel*

Do not use weathering steel in the following conditions:

- If the atmosphere contains concentrated corrosive industrial or chemical fumes.
- If the steel is subject to heavy salt-water spray or salt-laden fog.
- If the steel is in direct contact with timber decking; timber retains moisture and may have been treated with corrosive preservatives.
- If the steel is used for a low urban-area bridge or overpass that creates a tunnel-like configuration over a road on which deicing salt is used. In this situation, road spray from traffic under the bridge causes salt to accumulate on the steel.
- If the location has high rainfall and humidity or is constantly wet. This situation is rare in Texas.
- If the structure provides low clearance (less than 8 to 10 feet) over stagnant or slow-moving water.

2.1.3. **Paint**

TxDOT district offices usually select paint systems. If the plans do not specify a paint system, System II is the default paint system specified in the 2004 TxDOT Standard Specifications.

The 2004 TxDOT Standard Specifications have three paint systems for new steel bridge construction:

- System II—two-coat system with epoxy zinc primer.
- System III—three-coat system with inorganic zinc (IOZ) primer.
- System IV—two-coat system with inorganic zinc primer

Item 446, “Cleaning and Painting Steel,” provides a more detailed description of these systems.

TxDOT Standard Specifications require the inside of all tub and box girders to be painted with a light-colored paint (white polyamide cured epoxy) to facilitate future inspection. This paint is not intended to provide corrosion protection.

2.1.4. **Span Configuration**

Span configuration plays an important role in the efficient use of steel. Two-span continuous girders are not efficient because of high negative moments. However, they can be economical if maximizing prestressed concrete beam approaches leaves only enough room for a two-span continuous unit to fill in the remainder of the structure. Three- and four-span units are preferable but not always possible. Units more than four spans are not recommended. For three- and four-span units, make interior spans about 20 to 30 percent longer than end spans. If end spans are short in relation to interior spans,

uplift can be a problem at the girder ends. If end spans are too long in relation to interior spans, a disproportionate amount of steel will be required for the end spans.

Always consider the presence of uplift at ends of continuous girders, particularly with light, rolled beam units or short end spans. Commentary to *AASHTO LRFD Bridge Design Specifications*, Article C3.4.1, indicates uplift to be checked as a strength load combination and provides guidance in the appropriate use of minimum and maximum load factors. Uplift restraint, when needed, should satisfy the Strength limit state and the Fatigue and Fracture limit state.

Using a continuous slab with simple span girders, typically done in Texas with prestressed concrete beams, can be an economical framing method. Advantages over continuous girder designs include elimination of costly air splices and heavy lifts during girder erection. Construction is also faster with simple span girders than with continuous girders; however, a potential drawback is loss of internal redundancy. In addition, more section depth is required. Investigate this framing method on a case-by-case basis to determine if it is economical.

Another economical framing method may be use of simple spans for dead load (beam and slab) and continuous for live load (and dead loads applied to the composite sections). TxDOT has done this with prestressed concrete I beams, but it did not prove economical. However, steel beams may be different if continuity details are simple enough. No detailed recommendations or suggestions are offered with this framing system.

2.1.5. Expansion Joints

Try to limit expanding lengths to allow use of standard strip seal expansion joints. Modular joints and finger joints are expensive and difficult to construct and maintain, and they have shown poor long-term performance; use them only as a last resort.

See TxDOT standard drawings SEJ-A and SEJ-P, “Sealed Expansion Joint Details (Without Overlay),” for strip seal expansion joint details. TxDOT prefers the joint on the SEJ-A standard for most bridges. The SEJ-P standard drawing has a larger rail section, and TxDOT recommends it only for structures carrying a large amount of heavy truck traffic such as those on NAFTA routes and interstate highways. The SEJ-P joint also requires more slab depth at the joint location than the SEJ-A joint because of the larger rail. Ensure slab depth is adequate for whichever joint is used.

Consider using an inverted-T bent rather than a modular or finger joint if you need to accommodate more thermal expansion than a 5-inch strip seal expansion joint’s capacity. You can extend the stem of the inverted-T bent through the slab to become the finished riding surface. You then place an expansion joint at each face. This type of bent is often designed with an assumed 6- or 8-inch riding surface placed on top of the stress-carrying cap after the bridge slabs in the adjacent units are placed. Use adequate reinforcing steel in the riding surface to accommodate live load strains. You can use this technique to eliminate finger or modular expansion joints. Aesthetics is a consideration, but you can usually design a corbel with a shape that complements the rest of the bridge. If the SEJ-P

standard is used with this system, the riding surface should be 10 inches above the top of the cap so that the large joint rail will fit.

With a finger joint, use a trough consisting of a steel channel (C shape) and a deflector plate underneath the joint to direct water away from the girders. Neoprene troughs have been used in the past under finger joints but have performed poorly. The channel/deflector system should stop at slab edges and allow water to be discharged directly out low ends. Drain pipes are typically not necessary with this system, and if provided, they may quickly become clogged. Use a channel/deflector system on at least a 4-percent grade to ensure good drainage.

2.1.6. Girder Spacing (Plate Girders, Tub Girders, and Rolled Beams)

Many studies show that the weight of structural steel per square foot of deck area decreases as girder spacing increases. However, you should limit girder spacing (or web spacing with tub girders) to 10 feet for the following reasons:

- TxDOT standard drawings do not support girder spacings of more than 10 feet.
- Slabs (or floor systems) cannot adequately support certain overloads.
- Re-decking while maintaining traffic is more challenging.

When setting the girder spacing, also consider using prestressed concrete panels to form the deck. (See TxDOT standard drawing PCP, “Prestressed Concrete Panels.”) This deck-forming option is the most economical slab-forming method. TxDOT allows panels only on straight girders because TxDOT wants the stiffness full-depth cast-in-place decks provide for curved girders. However, TxDOT accepts use of prestressed panels for most straight girder applications with girder spacing of 10 feet or less.

TxDOT prefers a minimum of four I-shaped beams/girders for a vehicular bridge span. TxDOT prefers a minimum of three tub girders in order to eliminate fracture-critical designation. Three tub girders are not possible on one-lane connectors, forcing the use of a two-tub-girder cross section.

2.1.7. Available Length of Material

Design consideration for splice location depends in part on the length of plate that is available. No single maximum length is supplied for all plate sizes. The maximum plate length depends on a couple of factors:

- Weight of the material—The thicker the material, the shorter the maximum available plate lengths.
- Material type—Fracture-critical materials may have special length limitations depending on the procedures and equipment used by the mill. For example, steel that must be normalized must be short enough to fit inside a furnace.

Consult a fabricator or steel mill on typical length limits. These limits vary from mill to mill and with material type and thickness.

2.2. Plate Girders

TxDOT standard drawing SGMD, “Steel Girder Miscellaneous Details,” provides common details for use with typical plate girder spans.

When you consider material transitions, weigh labor and welding costs against potential material savings. When these costs are high, minimize the number of splices. Allow fabricators the flexibility to adjust the number and location of splices with designer approval. Designers should include a note in design details stating that adjustment to the number and location of transitions may be allowed with designer approval.

2.2.1. Flange Width

For curved girders, flange width should be approximately one-third the web depth and no less than 30 percent of the web depth. For straight girders, a flange width of about one-fourth of the web depth should be sufficient. Do not use flanges less than 15 inches wide. The extra width for curved girders enhances handling stability and helps keep lateral bending stresses within reason.

Maintain a constant flange width for each girder field section. Girders adjacent to each other should have the same flange width dimension to simplify slab formwork and to prevent variation in diaphragm or cross-frame geometry at interior bearings.

Flange width transitions are permissible only at field splices. All girders should have the same width transition at the same field splice location.

Width increments should be in whole inches.

If you allow panel forming, which is recommended for straight girders, the designer is responsible for ensuring that the required studs can be placed on the girder without interfering with the panels.

Flange width affects girder stability during handling, erection, and deck placement. Keep the girder length (field section length) to flange width ratio below 85.

In most cases, top and bottom flanges should be the same width. Girders in positive bending that are composite with a slab can have a top flange narrower than the bottom flange, but assess weight savings against reduced lateral stability before hardening of the deck.

2.2.2. Flange Thickness

TxDOT prefers a minimum flange thickness of 1 inch. For straight girders, you may reduce flange thickness to $\frac{3}{4}$ inch. Thinner plate will “cup” excessively when welded to the web.

TxDOT prefers a maximum flange thickness of 3 inches. Grade 50 and HPS 70W steels are not available in thicknesses greater than 4 inches. Weld time increases disproportionately when splicing plates thicker than 3 inches.

Flange thickness must satisfy AASHTO b/t requirements. AASHTO requirements for compression flange b/t ratios are more stringent than for tension flanges. Remember that flanges in tension under dead load can experience compression with certain live load positions or with construction sequencing.

Use a 10-foot minimum length for any given flange thickness on a girder. This is more a practical than a design consideration.

Use only a few flange sizes, and do not use a small quantity of one flange size. An economical girder will have as many as four or six flange sizes on a continuous girder and as many as two or three sizes for a simple span. On jobs with multiple structures, designers should communicate with each other and establish a preliminary list of no more than eight flange plate thicknesses to use. Refine the list as the designs progress if beneficial.

You can probably economically introduce a flange splice if you can save about 800 to 1000 pounds. These numbers are approximate and are a function of the current cost of steel plate.

TxDOT prefers flange thickness increments of 1/4 inch from 1 to 3 inches, and 1/2 inch from 3 to 4 inches. Fabricators prefer no plate thickness changes that are relatively small—for example, from 1 to 1.125 inches. Generally, the thicker plate should provide at least 25 percent more area than the thinner plate. In addition, the thinner flange should be no less than half the thickness of the thicker flange.

If nominal fatigue resistance is not satisfied at Category C and C' details, increase flange thickness to eliminate the problem. The extra weight of a slightly thicker flange is less expensive than bolted stiffener tabs. Category C (stud connectors) and C' (toe of stiffeners) are typically the controlling fatigue details for plate girders. Avoid details more critical than these Categories.

Do not use lateral bracing. Increase flange thickness if necessary. Lateral bracing creates fatigue-sensitive details and is costly to fabricate and difficult to install.

For curved girders, it is more economical for some fabricators to heat-curve the girders than to cut the flanges from a wide plate, producing waste. For other fabricators, the cost of the waste can be less than the time and labor to heat-curve. Designs should allow heat-curving (see Steel Bridge Fabrication Guide Specification, AASHTO/NSBA Steel Bridge Collaboration S2.1-2002). Let fabricators decide the most economical way to produce curved flanges.

“Slabbing and stripping” refers to a process in which wide plates of different thickness are welded to each other and individual flanges are then cut from this assembly. Because plates typically come in 42-inch or 48-inch minimum widths, this process reduces the number of individual flange splices required. It also reduces flange plate handling costs in the shop. The fabricator may not elect to slab the flanges, and it is not always feasible with curved girders. However, straight girder designs should allow the possibility of slabbing and stripping.

2.2.3. Flange Splice Locations

Locate splices at least 6 inches away from a web splice or transverse stiffener in order to facilitate non-destructive testing of welds.

Splices should be at least 10 feet apart.

Change flange width only at field splices, which are also good locations to change flange thickness.

2.2.4. Web Depth

The recommendations in *AASHTO LRFD Bridge Design Specifications*, Article 2.5.2.6.3, provide a good estimate of a minimum web depth for straight girders. Consider this depth a starting point. If vertical clearance is not a problem, adding depth can result in lighter girders. For curved girder web depth, use either the AASHTO recommended minimum depth for straight girders, increased by 10 to 20 percent, or use LRFD Equation 2.5.2.6.3-1 as a starting point. Aesthetics also has a role in girder depth. A rule of thumb for a well-proportioned superstructure is to have total section depth (slab plus girder) in the range of 0.033L to 0.04L ($L = c-c$ brg length).

Use web depths in whole-inch increments.

At girder ends, dap webs such that total superstructure depth, including bearings, closely matches that of an adjacent span if the difference in depth is much more than 6 inches. This is done primarily for aesthetic reasons and to minimize tall reinforced concrete pedestals for bearings on substructure elements. Dapping girder ends can also be done at abutments to keep backwall heights within reason. Show the web dap to the nearest whole inch. No more than 40 percent of the web depth should be dapped.

Dap girder ends by cutting the web at a slope (1:1 minimum, 4:1 maximum) and allowing for cold bending of the flange to fit the web. Although AWS D1.5, Article 12.12, prohibits cold bending of fracture critical members, neglect this AWS provision because the stress range is very low near the support (dap location). Make a provision for an allowable shop splice in the flange immediately beyond the bends so that the fabricator does not have to work with a long piece of flange during bending operations. The slope should begin a sufficient distance away from the face of the substructure (6 inches minimum, taking substructure skew into account) so that the girder will not hit the

substructure when it expands and so that erectors will have some latitude in moving the girder longitudinally.

Do not use haunched webs where the web is deeper over interior supports for spans less than 400 feet. Haunched webs have more potential for fabrication error and material waste can be excessive, and with the advent of A 709 Grade HPS70W, there is little reason to use them with spans less than 400 feet. The only arguments for their use are a possible aesthetic advantage over normal girders and maximizing freeboard for river crossings.

2.2.5. Web Thickness

Minimum web thickness is 1/2 inch. Thinner plate is subject to excessive distortion from welding.

For web depths up to 96 inches, provide sufficient thickness to preclude the need for longitudinal stiffeners. Longitudinal stiffeners do not provide economical designs and present fabrication and fatigue problems that make their use in a design unwise. They may be justifiable in deep girders, but you can easily design most structures built for TxDOT without them.

Web thickness should also be sufficient to eliminate the need for transverse stiffeners either entirely or partially. In high shear regions if transverse stiffeners spaced at about 8 to 10 feet prevent the need for a thicker web, you can justify using the stiffened web. Consider diaphragm or cross-frame connection plates as transverse stiffeners if they are needed in order to obtain a higher shear capacity, provided their spacing does not exceed AASHTO requirements. TxDOT discourages the use of fully stiffened web designs.

Optimum designs have few sizes, similar to flanges. A reasonable target would be three or fewer sizes for a continuous girder and one or two for a simple span.

Web thickness increments should be 1/16 inch up to a plate thickness of 1 inch. Use 1/8-inch increments beyond this.

2.2.6. Web Splice Locations

Splices should be at least 10 feet apart.

Locate web shop splices at least 6 inches away from a flange splice or transverse stiffener in order to facilitate non-destructive testing of welds.

2.2.7. Web-to-Flange Welding

Design web-to-flange welds and show them in the span detail drawings on the girder elevation. In most cases, the American Welding Society (AWS) minimum size weld (5/16 inch; see AASHTO/AWS D1.5) is sufficient. Welds more than 3/8 inch require multiple passes and drive fabrication costs up significantly.

2.2.8. Stiffeners

Plates provided only as a means to connect diaphragms or cross frames to girders are included as stiffeners even though they are not typically considered stiffeners.

TxDOT standard drawing SGMD shows stiffener lengths and how the stiffeners are welded to the girders. If a design requires welds larger than AWS minimums, indicate this on the span detail drawings. The SGMD standard drawing also provides direction to the fabricator on the orientation of the stiffeners—for example, plumb or perpendicular to the girder. Corner clips are also detailed on the SGMD standard drawing.

Fabricators strongly discourage full-penetration welding of bearing stiffeners to flanges. Full-penetration welds distort the bearing area of the bottom flange. Use finish to bear at the bottom flange and tight fit at the top. If a diaphragm or cross frame is attached to the bearing stiffener, use fillet welds to connect the stiffener to both flanges as shown on the SGMD standard drawing.

2.2.8.1. *Width*

Bars are more economical than plates for stiffeners. Bar widths come in 1/4-inch increments for widths under 5 inches and in 1/2-inch increments for widths of 5 to 8 inches. To take advantage of bar use, specify stiffener widths in 1/2-inch increments. Specify thickness in 1/8-inch increments using 3/8-inch as an absolute minimum.

The stiffener's width should be sufficient to provide clearance for field welding of diaphragm and cross-frame members to the stiffener, particularly when field welding is required near the face of the web. Welders recommend 4 inches or more of clearance between the web face and vertical welds on a gusset plate/diaphragm member. Welders recommend three inches or more of clearance between a gusset plate/diaphragm member and a flange. The SGMD standard drawing specifies minimum stiffener widths, which are based on these recommendations. (See Figure 1.)

Bearing stiffeners should extend to about 1/2- to 3/4- inch from the flange edge. They should be wide enough to facilitate field welding of diaphragm members as shown in Figure 1.

Stiffeners can extend beyond the flanges if they do not interfere with slab forms and bearing anchor bolts and lateral guides.

If a diaphragm or cross frame is not connected to the stiffener, use the minimum size stiffener allowed by AASHTO.

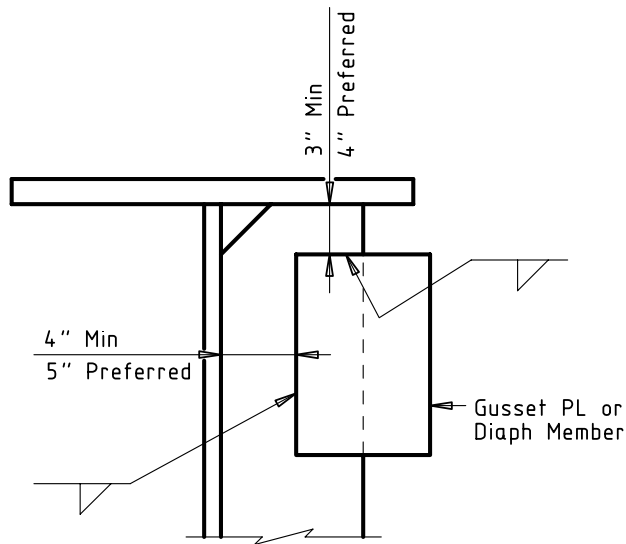


Figure 2-1. Recommended Clearances for Field Welding

2.2.8.2. Thickness

Use few stiffener sizes for a girder. For example, if the design requires minimum bearing stiffener thicknesses of 1 inch, 1.25 inches, and 1.5 inches, use 1.5-inch stiffeners at all locations. Use a bearing stiffener thickness that matches a flange thickness.

Size intermediate bearing stiffener thickness according to AASHTO requirements, and round up to the nearest 1/4 in.

Bearing stiffeners should be thick enough to preclude the need for multiple bearing stiffeners at any given bearing. Multiple stiffeners present fabrication difficulties and usually are not needed.

2.2.9. Bearings

For most plate girder spans, select bearings from TxDOT standard drawing SGEB, or use a modified version of a bearing on this drawing. Avoid costly proprietary High Load Multi-Rotational (HLMR) bearings (disc, pot, and spherical bearings). Always provide enough cap width, length, and depth to accommodate bearings and their anchor bolts.

On skewed structures, ensure that sole plates do not conflict with abutment backwalls, inverted-T stems, or beams in an adjacent span.

2.2.10. Field Splices

Show field splices in the design detail drawings as welded splices. Offer bolted splices as an option, and show them in the design details unless they are not desired for aesthetic reasons or if splice plates interfere with diaphragm or cross-frame locations. In the design details, note that structural steel pay weight is based on welded field splices regardless of splice type chosen by the contractor. No design for a welded field splice is typically

required other than locating the splice and ensuring that nominal fatigue resistance is satisfactory. Item 448, “Structural Field Welding,” in the 2004 TxDOT Standard Specifications provides fabricators and erectors the necessary details of welded field splices.

See Section 2.7, “Bolted Connections,” for more information related to bolted field splices.

Locate field splices at points of dead load contraflexure. They do not have to be at the exact contraflexure point but should be reasonably close. Field splices do not have to be present at every contraflexure point. If the spans are short enough, you may be able to skip a contraflexure point without violating length limitations.

Make girder field lengths about 130 feet maximum, keeping in mind site access and the stability criteria in Section 2.2.1. Provide optional field splices in the design if girder field length is longer than 130 feet. Keep the stresses in the girder at an optional splice as low as possible. A flange splice is not a good location for an optional splice because the stresses in the thinner flange, by design, are usually close to the maximum permitted.

Splices must be far enough away from diaphragms or transverse stiffeners to allow room for splice plates.

For curved girders, do not let the girder sweep plus the flange width exceed 6 feet for ease of shipping. The current legal vehicle width is 8 feet 6 inches without a permit. Limiting the overall shipping width of curved girders to 6 feet permits fabricators to offset the girder on the trailer, as is frequently done, while not exceeding an overall width of 8 feet 6 inches. Add optional field splices if required, as noted above.

For straight girders, pay close attention to the interaction between the panel bedding strips and top flange splice plates and bolts where prestressed concrete panels may be allowed as a slab-forming option. Additional slab haunch may be required to accommodate bolt head height.

The legal vehicle height limit is 14 feet. Most trailers are approximately 4 feet high. Assume approximately 6 inches for dunnage. Overall girder depth, including stud connectors, should not exceed 9 feet for ease of shipping.

Optional bolted field splices often require fill plates in flange splices. The steel grade specified for the girders is frequently not available in thicknesses of 3/8 inch or less. If you need flange fill plates less than 3/8 inch thick, allow optional fill plate material (A 606, A 570, etc.) that meets design requirements.

Do not bring HPS 70W steel into a field splice unless stress demands require it. At low stress regions, where splices should be located, AASHTO’s minimum splice strength requirement forces a greater number of bolts and larger splice plates than would otherwise be required.

2.3. Rolled Beam Sections

TxDOT standard drawing SGMD, “Steel Girder Miscellaneous Details,” provides common details for use with rolled (wide-flange) beam spans.

Rolled beams can be more economical than plate girders for their applicable span lengths because of decreased fabrication costs.

2.3.1. Sections

Select beams that have a top flange that is sufficiently wide to provide adequate spacing for three stud connectors per row. If prestressed concrete panels are allowed as a forming option, the flanges should be at least 12 inches wide. The designer is responsible for ensuring that the required stud connector spacing does not create a conflict with prestressed concrete panels if panels are allowed as an option.

Satisfy flange proportion limits in the *AASHTO LRFD Bridge Design Specifications*, and ensure that flange width is sufficient for handling and erection stability (see Section 2.2.1).

For continuous spans, if changing weights at splices, beams must be from the same rolling family as given in American Institute of Steel Construction (AISC).

The beams should be large enough that the elastic neutral axis of the composite section is within the steel beam, not within the slab or haunch.

Do not use cover plates. Their fatigue category is too low.

Do not use sections smaller than W21, which would require modifications to the SGMD standard drawing.

2.3.2. Stiffeners

Rolled beams usually do not need bearing stiffeners. Verify this using the provisions in AASHTO LRFD Article D6.5.

2.3.3. Bearings

Select bearings from TxDOT standard drawing SGEB, or use a modified version of these bearings. For simple spans with rolled beams, bearing designs depicted on TxDOT standard drawing SBEB are more economical designs. These bearings were designed for TxDOT’s standard steel beam spans only, but they may work for custom-designed bridges if analyzed for adequacy.

If calculated uplift is present at ends of continuous units, an uplift restraint satisfying the strength and fatigue limit states is required. The possibility of uplift during slab placement must be investigated and accommodated if present.

Do not use a continuous beam that has calculated uplift at end bearings with the Service I load combination under any circumstances.

2.3.4. Field Splices

The information in Section 2.2.10 applies.

2.3.5. Camber

Camber rolled beams for all continuous beams and for simple spans over about 50 feet. Camber continuous beams for total DL deflection and roadway vertical curves. Camber simple spans for total DL deflection only. For rolled beams, specify the welded plate girder camber tolerances in AWS D1.5 in the design detail drawings. Consult fabricators to determine whether the required camber can be achieved with the proposed beam section.

For detailing required camber, show only the mid-ordinate of simple-span beams in the design details. For simple span beams less than about 50 feet, note in the design details to erect beams with natural camber up.

2.4. Tub Girder Sections

There are no standard TxDOT details for tub girders at present. Most of the guidelines outlined are for curved, continuous tub girder units. Application of these guidelines to tangent and simple span construction is at the designer's discretion.

Tub girders should have a constant trapezoidal or rectangular shape and should be rotated with the cross slope. Keep the top-of-slab to top-of-web dimension constant. See Figure 1 for preferred horizontal geometry. Departure from the shown geometry can result in extreme difficulties generating shop drawings. The profile grade line and horizontal control line locations in Figure 2-2 are for example purposes only.

Take the centerline-of-bearing offset into account when substructure elements are detailed.

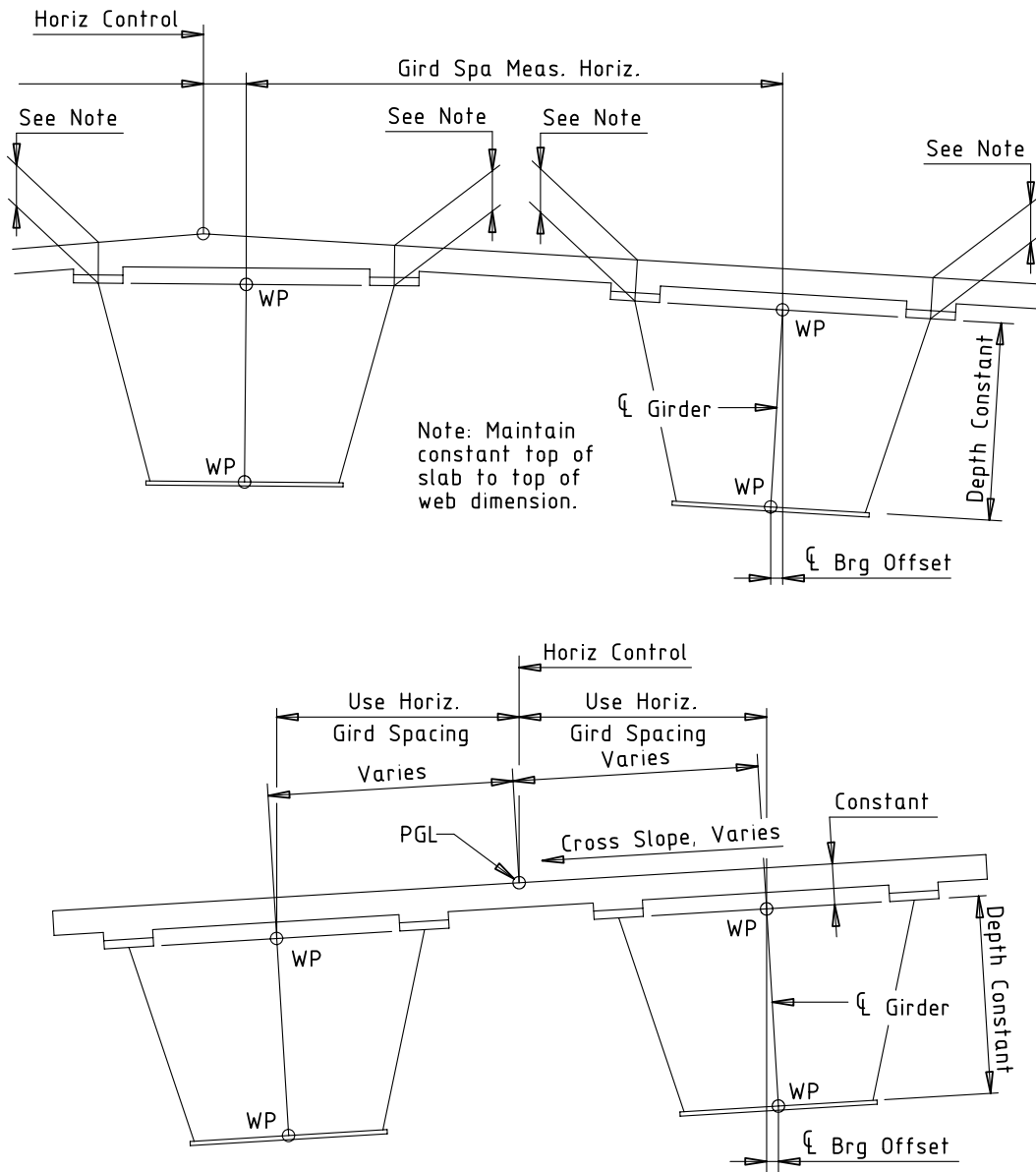


Figure 2-2. Recommended Tub Girder Horizontal Control

2.4.1. Flanges

In addition to AASHTO requirements, top flanges for tub girders should follow the suggestions for plate girder flanges in Section 2.2.

For bottom flanges, plate distortion during fabrication and erection can be a problem. Check with fabricators when using bottom tension flange plates less than 1 inch thick to determine whether practical stiffness needs are met. Bottom tension flanges should never be less than 3/4 inch thick. In addition, the bottom tension flanges should have a w/t ratio of 80 or less.

At present, no information is available regarding the possible economic benefits of using a thinner, longitudinally stiffened compression flange over a thicker, unstiffened flange. Until such information is available, discuss options with fabricators who are experienced with tub girders.

If using longitudinal stiffeners, try to maintain a clear distance between longitudinal stiffeners of no less than 24 inches (more is better) to accommodate automated welding equipment. Therefore, the minimum flange width, between webs, is 48 inches when using one stiffener and 72 inches for two stiffeners. Do not use more than two stiffeners per flange.

For straight girders, plates or bars are recommended over WT shapes for longitudinal stiffeners as long as they meet AASHTO criteria. Plate and bar sections are less expensive and easier to splice than WT sections. For curved girders, WT sections are recommended. If WT sections are used, the suggested ratio of the depth to one-half the WT flange width should be greater than 1.5 to provide good welding access. Try to allow termination of longitudinal stiffeners at a bolted field splice such that fatigue is not a concern at the stiffener's end.

Bottom tension flanges of two-girder spans are classified as fracture-critical. Top tension flanges are not fracture-critical unless only one girder per span is used. The tension portion of each web plate attached to a fracture-critical flange plate is fracture-critical itself. Avoid details more critical than Category C with fracture critical members (and for non-fracture-critical members as well).

Bottom flange edges should extend at least 2 inches beyond the web centerline to facilitate automated welding.

2.4.2. Webs

The suggestions in Sections 2.2.4 and 2.2.5 apply.

2.4.3. Stiffeners

For stiffeners and connection plates for internal cross frames, a good option to provide fabricators is shown in Figure 2-3. Cutting the stiffeners short of the bottom flange facilitates automated welding of the web to the bottom flange. After this welding is complete, the stiffener can then be attached to the bottom flange with an additional plate.

Refer to Section 2.11, Bearing Replacement, for information on stiffeners to be used for future bearing replacement.

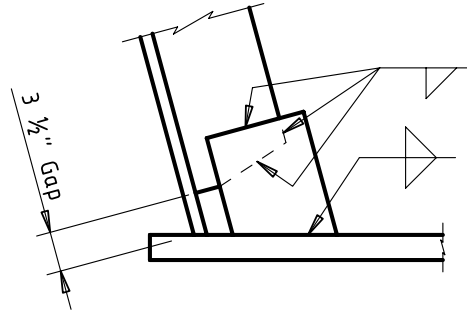


Figure 2-3. Recommended Tub Girder Stiffener Option

Do not specify complete penetration groove welds to connect bearing stiffeners to bottom flanges. Weld-induced flange distortion is even more of a problem with tub girder flanges than with plate girders.

2.4.4. Top Flange Lateral Bracing

Use lateral bracing in straight tub girders and in curved tub girders.

Bolt lateral bracing directly to the top flange. Provide enough slab haunch that formwork does not interfere with the bracing. Do not use shims or fills between the lateral bracing and top girder flange that increase eccentricity of the connection.

TxDOT prefers single-laced lateral bracing over double-laced bracing. The angle between the girder flange and bracing should be at least 35 degrees. An angle closer to 45 degrees is ideal.

Consider erection loads and sequential concrete placement when determining the worst-case loading for lateral bracing.

2.4.5. External Diaphragms and Cross Frames (between Piers)

External diaphragms or cross frames are normally used to control relative displacement and twist of girders during slab placement. Once the slab has matured sufficiently, they may be removed, which is done primarily for aesthetic reasons. If they are to remain in place, they should complement the overall structural aesthetics and should contain fatigue-resistant details.

For curved tub girders, external diaphragms or cross frames at span quarter or third points are usually sufficient; adding more is unnecessary. With straight tub girders, one external cross frame or diaphragm at mid-span should be sufficient. External diaphragms or cross frames must be backed up with an internal diaphragm or cross frame.

2.4.6. Internal Diaphragms and Cross Frames (between Piers)

Internal diaphragms and cross frames are used to control cross-section distortion. For curved tub girders, locate an internal cross frame or diaphragm at every other lateral

bracing point, which should result in a spacing of 14 to 18 feet. Place horizontal struts, usually angle sections, at the lateral brace point between internal cross frames to control horizontal bending of the flange during concrete placement. Like lateral bracing, they should be attached directly to the flanges.

For straight tub girders, internal cross-frame or diaphragms can be spaced every third or fourth lateral bracing point.

2.4.7. Pier Diaphragms and Cross Frames

Assuming one bearing per girder, diaphragms at bents should be plate girder sections that are approximately the same depth as the girders themselves, and they should connect to the tub girder's flanges and webs if their span-to-depth ratio is 3 or more. With two bearings per girder, a cross frame may be a better choice at piers. Verify that bearing assemblies do not interfere at the bottom flange connection. Carefully consider constructibility of diaphragms at the girder ends as the presence of abutment backwalls, stems of inverted-T bents, or other girders can complicate bolting these diaphragms in place. Provide an inspector access hole through the diaphragm web plate at intermediate supports. See Section 2.5.3 for recommended opening sizes. See Figure 2-4 for a sketch of a typical pier diaphragm between girders (with one bearing per girder).

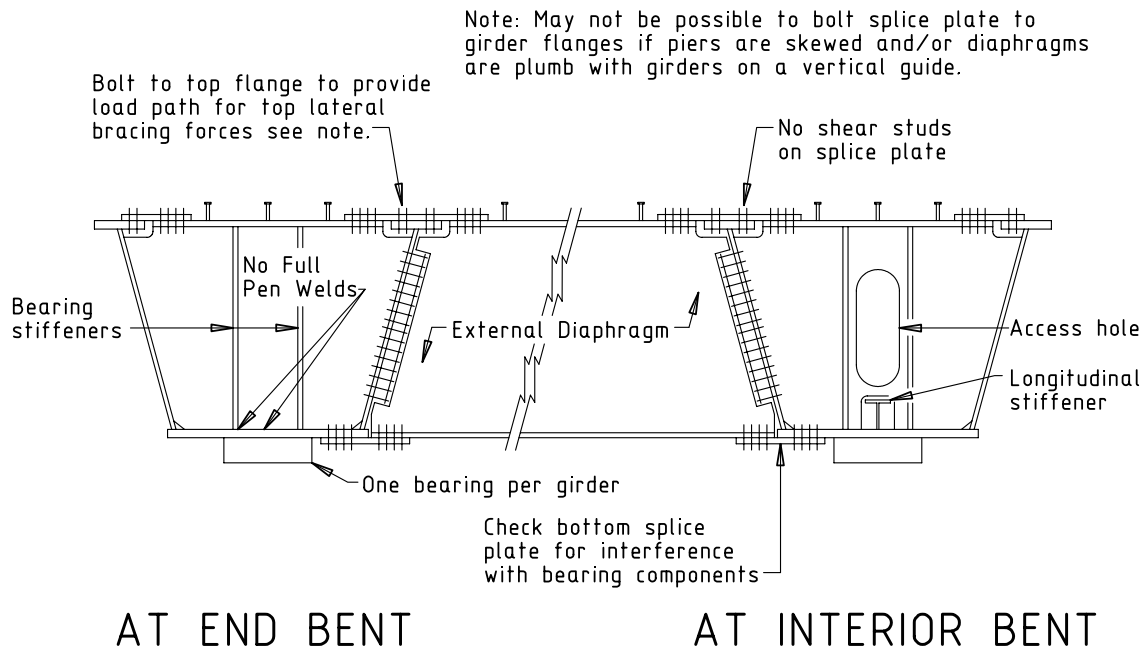


Figure 2-4. Moment-Connected Pier Diaphragm

2.4.8. Field Splices

Bolt field splices. The suggestions in Section 2.2.10 apply. In addition, overall girder width, including sweep, should be no more than 14 feet for ease of shipping.

2.4.9. Bearings

TxDOT prefers one bearing per girder at each support. A girder may not bear evenly on both bearings or, at worst, on only one of the bearings with two bearings per support. This is especially true with skewed piers.

High Load Multi-Rotational (HLMR) bearings may be necessary. TxDOT prefers neoprene bearings over HLMR bearings. HLMR bearings are good only with very large reactions, more than 1,200 kips.

TxDOT prefers bearing designs without anchor bolts (except anchor bolts through masonry plates only). Do not require anchor bolts to pass through the girder flange. A better method of detailing for restraint is to design an external alignment device that is flexible in terms of placement after girder erection.

Bearing designs should accommodate bearing replacement with a minimal amount of lifting.

2.4.10. Electrical Service and Inspection Access

Design details should provide for electrical service on the inside of the girders, with outlets spaced at no more than 100 feet to facilitate maintenance and inspection during the life of the bridge. The long girder length between access holes or doors necessitates this provision.

Provide an access hole with lockable door or cover in the bottom flange near each end support for inspector access. The door or cover must be light enough to be easily managed by an inspector (suggested weight is 25 pounds or less).

2.5. Box Girder Sections (Closed Boxes for Straddle Bents)

Straddle bents are sometimes employed when there is limited vertical clearance between a bent and lower roadway. Steel box girders are a good solution in these situations.

Longitudinal beams/girders should be supported on top of the straddle bent, but they may be supported on the sides of the bent if necessary to satisfy vertical clearance requirements. TxDOT standard drawing MEBR(S), “Minimum Erection and Bracing Requirements (Steel I-Beams and Plate Girders),” does not apply to straddle bents, and the designer should address beam/girder bracing requirements through special details or notes.

Provide two inspector access doors or hatches in all box girders. Typically, a hatch-type, lockable door at each end of the box is sufficient. It is important to provide access holes at each end of the box.

Boxes can be designed to be completely sealed (preferred) or to be well drained. Coordinate all aspects of box design toward one or the other of these designs. For example, a design that calls for a sealed door should not have drain holes or corner clips

on outside plates. Remember that small openings into the box occur at bolted field splices if a sealed box design is being considered. Keep any openings as small as possible or install screens to keep birds and bats out of the boxes as they can plug the drains.

Recommendations for plate girders generally apply to straddle bent box girders.

2.5.1. Flanges

Tension flanges for straddle bents are fracture-critical.

Extend flanges past the outside edge of each web a minimum of 2 inches to allow for automated welding equipment.

Flange width depends somewhat on the need for enough room inside the box girder to allow passage of inspection personnel. If fabrication is required within the box, provide at least 48 inches between webs.

See Section 2.2.1 for more details.

2.5.2. Webs

When estimating web depth for a straddle bent, use the span length divided by 12 ($L/12$) to obtain a reasonable starting point for design.

The tension half of each web plate is fracture-critical, so take care to avoid weld details on the web that are more critical than Category C'. Also, if a detail welded to a fracture-critical member is long enough, it becomes fracture-critical itself (see AWS D1.5, Section 12.2).

2.5.3. Stiffeners

Stiffeners, both intermediate and bearing, should consist of plates sized to match the box's interior dimension with an opening that is sufficiently large to permit fabrication and inspection functions within the box. The absolute minimum opening size is 18 inches wide by 30 inches deep. If possible, provide openings that are at least 32 inches wide by 36 inches deep. Place the holes at mid-depth and concentric with the box. The opening corners should have a radius equal to one half of the opening width.

If multiple bearing stiffeners are required at a bearing, space them far enough apart to provide adequate welding access. Although not always possible, stiffeners should be spaced 36 inches apart to facilitate welder access.

Stiffeners should be welded to all four inside box surfaces. Avoid a tight fit condition without a weld because it can create an unstiffened web gap from which fatigue cracks can propagate. Avoid complete penetration groove welding because of weld-induced distortion that invariably occurs in the flanges.

See Figure 2-5 for a sketch of a typical box girder stiffener

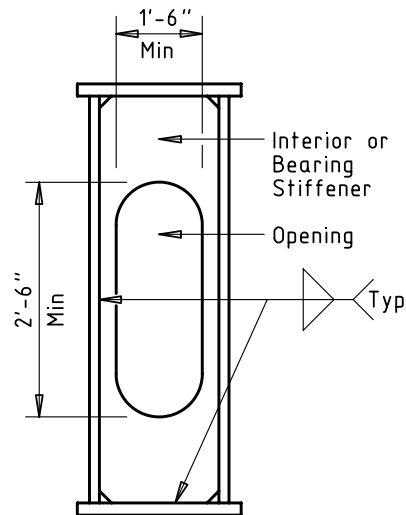


Figure 2-5. Recommended Box Girder Stiffener Detail

2.5.4. Bearings

A preformed fabric pad is the type of bearing most often employed with straddle bents. Base pad thickness primarily on rotational capacity requirements set forth in the *AASHTO LRFD Bridge Design Specifications*. Reinforced neoprene bearings are a good alternative if rotation is difficult to satisfy with preformed fabric pads.

If the straddle bent is not level, use a sole plate, beveled to match the slope of the cap.

If feasible, use a bearing system that is forgiving of anchor bolt misplacement.

2.5.5. Field Splices

Field splices, if needed, should be bolted. See Sections 2.2.10 and 2.7 for more information related to bolted field splices.

2.5.6. Flange-to-Web Welding

For flange-to-web welds on box girders, designers should provide options. These may include the following:

- Double fillet welds
- Double unbalanced fillet welds
- A full penetration weld
- A partial penetration weld with fillet backing

These options are shown in detail in Figure 2-. Allowing these options provides flexibility to the fabricator, which helps to ensure the most economical product. The choice of joint detail for a box girder corner has a great effect on quality and economy. Access inside the boxes is generally limited, especially in smaller boxes, and it is usually

best to minimize the amount of welding that must be accomplished from inside the box. Fillet welds are generally more economical than groove welds, so fillet welds are encouraged. Additionally, large welds can cause distortion problems during fabrication.

One flange can be welded to the webs with relative ease. The need for options becomes more pronounced for the remaining flange, sometimes referred to as the “lid.” Detail the tension flange to web welds as double fillet welds, and provide options for the compression flange. This assumes that the tension flange changes at a field splice or that the box is simply supported. If the tension flange changes sides without a field splice, the weld detail should remain constant per side.

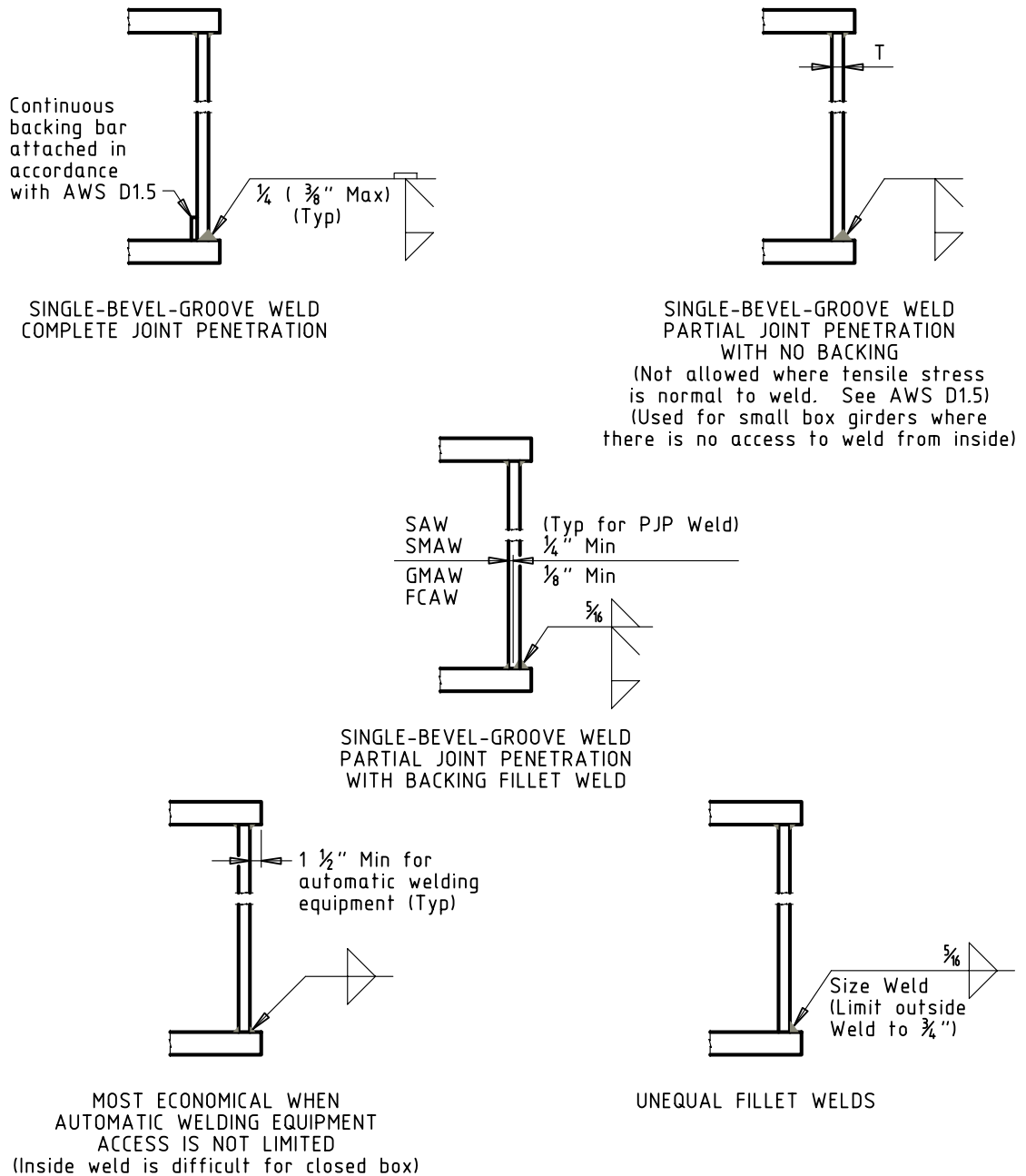


Figure 2-6. Recommended Box Girder T-Joint Weld Details

2.6. Diaphragms and Cross Frames

TxDOT has traditionally used field welding as the preferred method of connecting diaphragms and cross frames to girders because it is more forgiving with respect to

erection tolerances than bolted connections. For bolted connections, use standard size holes to control girder geometry.

The diaphragms and cross frames shown on TxDOT standard drawing SGMD should be acceptable for the beam spacing and depth limits noted on those standards. However, because of the variability in steel bridges, always confirm the adequacy of these standard diaphragms and cross frames, including their connections, before using them with your design details.

For straight beams and girders, observe the 25-foot spacing limit from the AASHTO *Standard Specifications* as a starting point. A larger spacing is acceptable if a larger spacing can be achieved without temporary bracing and if all other limit states are satisfied. A tighter-than-normal diaphragm/cross-frame spacing near interior supports may prove beneficial in terms of increasing negative moment bending capacity. This should be investigated for each straight continuous bridge design. For curved girders, TxDOT prefers that diaphragms or cross frames be placed at 15 to 20 feet maximum to help limit lateral flange bending stresses and cross-frame/diaphragm member forces.

You may need to consider bracing beyond what permanent diaphragms and cross frames provide for erection and slab placement. In some cases fascia beams have twisted during slab placement, a problem that permanent diaphragms have not prevented.

2.6.1. Member Selection

Equal leg angles are often more cost-effective than unequal leg angles. Fabricators discourage back-to-back angles used as cross-frame members. Some common angle sizes for diaphragms are L3.5 x 3.5 x 3/8, L4 x 4 x 3/8, and L5 x 5 x 1/2.

Fabricators discourage the use of WT shapes, especially in small quantities. If channel sections are used, C shapes are preferable to MC shapes.

If you use a channel, provide an option for the fabricator to bend a plate into an equivalent channel shape instead. A bent-plate diaphragm, in the shape of a channel, is a possible option to provide to the fabricator for diaphragms on shallow plate girders (4-foot-deep web or less) or rolled beams.

It may be beneficial to use a larger shape than is required if the larger shape is being used in significant quantities elsewhere in the project.

Design and detail cross frames such that they can be erected as a single unit. Fabricators and erectors discourage diaphragms that require erection in separate pieces.

Detail cross frames such that all welding during fabrication can be done from one side to minimize handling costs.

2.6.2. Stage Construction and Skews

If the bridge is to be built in stages or if the skew is large, differential deflection between girders due to slab placement can be significant. Special diaphragm/cross-frame details may be required for these cases. An example is slotted holes for erection bolts and the requirement of field welding after slab placement if the designer has ensured that erection bolts alone can accommodate the loads.

TxDOT standard drawing SGMD covers skew angles up to 45 degrees. Anything beyond this requires special design details showing diaphragm/cross-frame attachment to the girders.

2.6.3. Diaphragm and Cross-Frame Orientation

Standard drawing SGMD indicates that diaphragm/cross-frame lines at end bearings are parallel to the skew up to a 20-degree skew. Between 20- and 45-degree skews, diaphragm/cross-frame lines at end bearings are not quite parallel to the centerline of bearing.

AASHTO permits interior diaphragm/cross-frame lines parallel to the skew up to 20-degree skews. When diaphragms/cross frames are placed along the skew, the designer should be aware that Dart welders are commonly used industry-wide to attach the stiffeners to the girder webs. Dart welders can weld a stiffener plate that is skewed to the web up to 20 degrees, which works well with the AASHTO limitation. Beyond 20 degrees, fabricators will have to use a more costly welding method.

Placing all diaphragms/cross frames along the skew is acceptable for skews up to 20 degrees. All other diaphragms/cross frames should be normal to the girders. Curved girders are an exception and should always have radial diaphragm/cross-frame lines at intermediate locations.

A good, economical design minimizes the number of diaphragms/cross frames with different geometry. Superelevation changes, vertical curves, different connection plate widths, and flaring girders all work against this goal.

2.7. Bolted Connections

One-inch, 7/8-inch, and 3/4-inch diameter bolts should be the only sizes considered for bridges. One-inch bolts often provide the most economical design. However, for small rolled beam flanges, smaller bolts may be better due to net area requirements. Do not mix sizes within a splice or within a unit unless A490 bolts are used with A325 bolts. (See Section 2.7.2.)

Web and flange splice plates should be at least 1/2 inch thick.

Provide more edge distance for bolt holes than the AASHTO minimums. If the drill drifts during the drilling operation, the hole could violate minimum edge distances. Add 1/4 inch to the AASHTO minimums.

2.7.1. Slip Coefficient

The 2004 TxDOT Standard Specifications allow painted faying surfaces if the paint is documented to meet slip and creep requirements, so show the slip coefficient assumed in the design on the design detail drawings.

TxDOT recommends using Class A surface conditions (slip coefficient = 0.33) for design for the following reasons:

- It allows for surface deterioration before the splice is made.
- Slip might not control the design, and this information is not normally conveyed in the plans. If a 0.33 design slip coefficient is adequate, it permits more flexibility to fabricators in the coating types to be allowed on faying surfaces.

For unpainted weathering steel structures, faying surfaces must be blast cleaned and be free of any mill scale. Research attests that mill scale on weathering steel plate is more slippery than mill scale on non-weathering steel plate, and it is detrimental to the slip resistance of connections. The Standard Specifications call for an SSPC-SP 10 (“near-white”) blast-cleaning to ensure that all mill scale is removed, but the near-white finish is not required to remain at the time of erection. See further discussion under Section 4.2.

2.7.2. A325 vs. A490 Bolts

For the following reasons, do not use A490 bolts unless absolutely necessary:

- TxDOT’s bolt installation procedure (in TxDOT Standard Specification Item 447) specifies the use of fit-up bolts, which are used to bring all the plies into full contact. The erector is often able to release the crane from the member using these bolts to support the joint as part of the erection procedure. Before the joint is complete, these fit-up bolts must be loosened. They can be retightened if A325 bolts are used but must be replaced if A490 bolts are used. Contractors strongly prefer loosening and retightening to replacement, and TxDOT inspection procedures cannot ensure that the A490 bolts will be replaced.
- A490 bolts are much more sensitive to tightening procedures. If over-tightened, these bolts can unload significantly below their proof load. A325 bolts have much more ductile behavior, so they can be tightened well beyond their proof load and still maintain the required tension.
- A490 bolts require impact wrenches of ample strength and quality that are sometimes not available at construction sites.

A325 bolts and A490 bolts of the same diameter are easily confused. If A490 bolts are necessary, use them in all similar connections or make them a different diameter than A325 bolts used for the bridge. But use common sense—for example, if A490 bolts are required for some lateral bracing connections, use the same size A490 bolts for all lateral bracing connections. Switching bolt grade/size at a common field splice location for

adjacent girders would require re-calibration of the tightening equipment in the middle of the erection process, or require separate wrenches calibrated for each bolt grade/size. A good design is not a source of confusion and delay in the field.

2.8. Anchor Bolts and Rods

The most economical anchor bolt for bridges is a mild steel anchor bolt. Mild steel anchor bolts are usually sufficient. Alloy steel anchor bolts may not provide the best value in the long term because their superior engineering properties are not realized on typical bridges.

Anchor bolts should be hot-dip galvanized as specified in the TxDOT Standard Specifications. Do not be concerned about contact between galvanized bolts and weathering steel. The zinc coating resulting from the hot-dip process is thick and sacrifices itself at such a low rate that the service life of the bolt is not compromised.

Use fatigue detail Category E when evaluating anchor bolts of any material for fatigue. Use the bolt tensile area, not the nominal area, when evaluating anchor bolt fatigue.

Despite the best efforts of contractors, anchor bolts are occasionally placed in the wrong location. Any bearing detail incorporating anchor bolts or rods should be able to accommodate their misplacement.

2.9. Stud Connectors

TxDOT has traditionally used stud connectors in only the dead load positive moment regions of plate girders and rolled beams. Studs applied full length of the girders may provide better distribution of deck cracks resulting from shrinkage. If you are placing studs along the full length of girder, do not consider slab reinforcement as part of the negative bending section. Tub girders require stud connectors the full length of the girder to ensure that the box section is “closed” along its entire length.

TxDOT standard drawing SGMD provides details for stud connectors. The designer must show their spacing on girder/beam elevations in the design details. AASHTO requires a minimum center-to-center stud spacing of four stud diameters transversely and six stud diameters longitudinally. Recent research sponsored by TxDOT indicates the longitudinal spacing could be lower as long as concrete consolidation can be achieved. Therefore, a longitudinal stud spacing of 4 inches is acceptable.

You must modify stud connector spacing when allowing the use of prestressed concrete panels as a slab-forming option. To verify that panels will fit with the studs, assume a minimum clearance between panel edges and stud connectors of 5/8-inch, the minimum clearance used with prestressed concrete beam horizontal shear reinforcement.

Designers may present a stud connector spacing modification in the design details other than what is given on the SGMD standard drawing.

Shear connector studs should not be required on top of flange splice plates.

2.10. Design Details

Both web camber diagrams and total dead load deflections should present camber requirements for plate girders. Most fabricators use total dead load deflections in determining web camber. However, some fabricators may opt to use a web camber diagram.

Do not provide web camber diagrams for tub girders because they supply no useful information to the shop drawing detailer. Steel detailers require only total dead load deflections to determine tub girder camber.

Avoid the term “web-cutting diagram” in design details because webs are not actually cut to these diagrams. Rather, fabricators make the adjustments they predict are needed such that the final product will meet specification requirements. Instead use phrases such as “camber diagrams” or “camber in the unstressed condition.”

See the current TxDOT *Bridge Detailing Manual* for more information on requirements for detailing of steel structures.

2.11. Bearing Replacement

The potential need to raise a bridge at some point in its life always exists because of unforeseen circumstances, such as bearing deterioration or failure.

Bearing designs should accommodate bearing removal with minimal lifting (1/4-inch lift requirement is suggested).

Raising a bridge with jacks is typical during bearing replacement. Jacking points on girders should be underneath a stiffened web. On tub girders, where it would be extremely difficult to add stiffeners to the girder webs once the slab is in place, include stiffeners for jacking in the initial girder fabrication. If there is enough room on the bent cap and under the girder to accommodate the predicted jack size, place the stiffeners accordingly. If the bent cap is too narrow to accommodate the jacks or the clearance under the girder is too small, place the stiffeners approximately 1 foot beyond each face of the cap. In this situation, shoring towers will be required in conjunction with jacks to raise the bridge. Place stiffeners symmetrically about the girder and bearing centerline so that no torsion will be induced during lifting operations.

On conventional I girders where it would be relatively easy to add stiffeners at jack locations, do not include these stiffeners in the initial girder fabrication. If the bearings need to be replaced, the absence of stiffeners gives the contractor more latitude in determining the best jack locations.

2.12. Bent Locations for Replacement or Widening

If existing bents are used for replacement or widening, use field-verified bent locations to prepare the plans. Do not rely on existing plans because the actual bent location/skew may differ enough from those plans to create field fit-up problems.

3. Fabrication

3.1. Shop Drawings

3.1.1. Shop Drawing Review

You need not check the welding procedures when reviewing shop drawings; however, you must check the welding symbol.

When stamping drawings, a reviewer need not initial the drawings. The approval engineer is responsible for reviewing shop detail drawings for conformation with the design details and specifications only, and the contractor and fabricator are responsible for all dimensions and fit of the structure.

See AASHTO/NSBA Steel Bridge Collaboration G1.1, “Shop Detail Drawing Review/Approval Guidelines,” for dimensions and material requirements that need to be checked and for other guidelines concerning shop drawing review.

3.1.2. Distribution of Approved Shop Drawings

The design office—which is the district design office, the Bridge Division, or a contracted design firm—handles distribution of approved shop drawings. One set of drawings is retained by the design office, one is sent to the shop drawing detailer (usually the fabricator), one is sent to the contractor, two are sent to the district (one for the area office and one for the field office), and two are sent to the Materials Section of the Construction Division (one for the Austin office and one to be forwarded to the in-shop TxDOT inspector). An additional set of drawings is needed if the structure is a railroad underpass, and another set is provided to the Bridge Division if the designer is an outside consultant. Some Districts choose to handle the distribution themselves. In 2006, TxDOT will convert to electronic submittal, review, and distribution of shop plans. See the Guide to Bridge Division Electronic Shop Drawing Submittal and other information posted on the internet at <http://www.dot.state.tx.us/brg/FO/Fabrication.htm>.

3.1.3. Shop Camber Checking

TxDOT inspectors check the actual camber condition during laydown. In accordance with the AWS D1.5 Bridge Welding Code, they check camber over the entire span, not just on the individual member.

3.2. Non-Destructive Testing

3.2.1. Use of Edge Blocks for Radiographs

When radiography testing is conducted at width transition splices, the location of the transition is moved 3 inches back from the splice to allow proper fit of the edge blocks. See Figure 3-1. This detail is shown on TxDOT standard drawing SGMD.

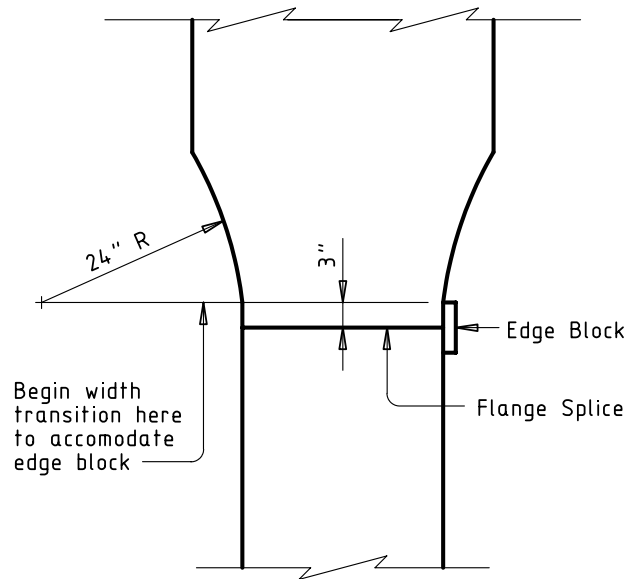


Figure 3-1. Flange Width Transition Detail

3.3. Cleaning and Painting

3.3.1. Painting Box and Tub Girder Interiors

The 2004 TxDOT Standard Specifications Sections 446.2.B and 446.4.F.3.d specify the type of paint (a white polyamide-cured epoxy) to be used on the interior of box girders.

Although paint on the inside of boxes provides some protection from corrosion, its primary purpose is to facilitate in-service inspections. The paint is intended to be “surface tolerant” so that it will adhere well to surfaces that have been cleaned free of grease, oil, and dust but that have not been blast-cleaned to near-white metal.

For the interior paint, the 2004 TxDOT Standard Specifications require a dry film thickness (DFT) of 2–3 mils, the minimum to ensure adequate coverage and the maximum to prevent masking of cracks.

3.3.2. Painting of Faying Surfaces

Faying surfaces may either be blast-cleaned and left bare, or painted with a prime coat after blast cleaning. Painting them might provide some corrosion protection. However, if the splice components are thin enough, bolt tightening will pull the plates together so that there will be no moisture access. Painting the faying surfaces also helps protect them during the time between when they are prepared in the shop and when they are erected in the field. The 2004 Standard Specifications leave the choice of painted or unpainted faying surfaces up to the fabricator, although designers may specify painted surfaces if they find this necessary in a particularly corrosive environment. If you choose to have the faying surfaces painted, ensure the following:

- The coating has been evaluated for actual coefficient of friction values.
- The coating has been evaluated for creep.
- The conditions under which the evaluation took place were representative of conditions under which the coating will actually be applied. If not, the manufacturer of the paint must be consulted regarding appropriate cure time.

The friction test and the creep test are described in the Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, which can be found in the AISC Manual of Steel Construction. Paint suppliers provide specific coefficient of friction values for each coating, and they certify that the coating meets AISC requirements.

If a steel structure with bolted connections will be painted but the faying surfaces will not, a small amount of paint in the faying surfaces around the perimeter of the connection is not detrimental. Although it is very important to avoid even small amounts of overspray on the faying surfaces of the connection, paint may be applied on the faying surfaces to within one inch or one bolt diameter, whichever is greater, of the perimeter holes (see Figure 3-2). This allows flexibility in fabrication because the painter may mask inside the edge of the connection instead of trying to mask precisely where the edge of the connection is expected to be. This also provides better protection because carrying the prime coat slightly under the splice plate minimizes the discontinuity in the primer that naturally occurs between splice plates and main members.

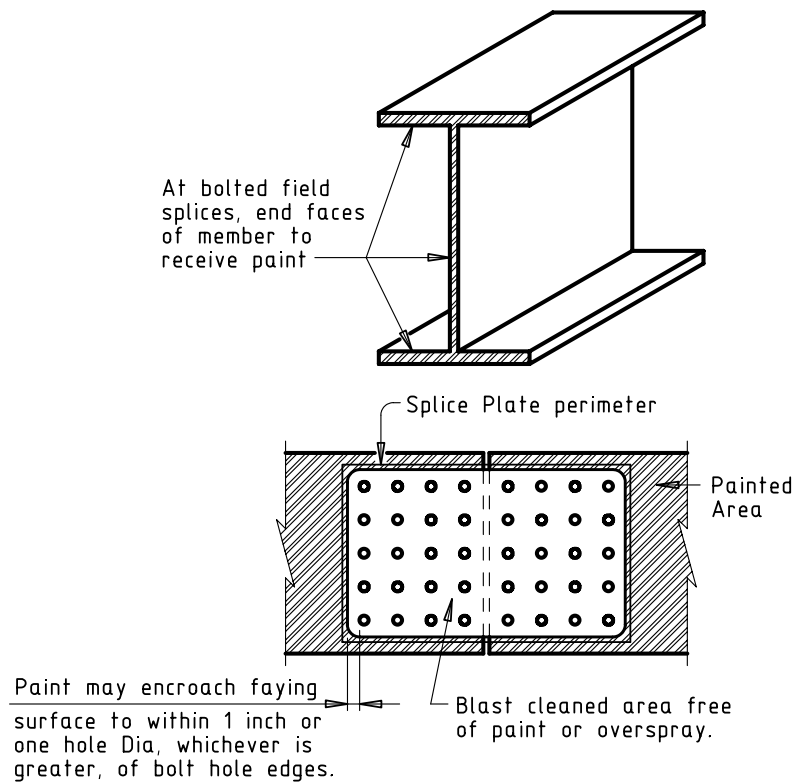


Figure 3-2. Paint Masking Requirements for Bolted Splices

4. Erection/Construction

4.1. Shipment of Bolts

Bolts for steel bridge structures are usually installed in the field although occasionally some are installed in the shop. When bolts will be installed in the field, they should be shipped directly to the field in containers that prevent exposure to the elements and maintain the integrity of the fasteners. All fasteners must be tightened in accordance with specification requirements for tightening. Diaphragm or cross-frame erection bolts, shown on TxDOT standard drawings SGMD and SBMD, are an exception to this rule and need only to be snug tight because they are used only as pins to align the diaphragms/cross frames prior to field welding the final connections.

The actual fasteners to be used on the job should never be shipped installed in the members unless they are completely tightened. This is important for two reasons:

- When fasteners are shipped to the field partially tightened, they are exposed for indefinite lengths of time, compromising the lubricant condition of the threads and washers.
- When fasteners are shipped to the field partially tightened (for example, snug-tight), it is not possible to know just how tight they are. This makes it impossible to tighten the fasteners properly.

4.2. Condition of Weathering Steel Bolted Splice Faying Surfaces

The Standard Specifications call for an SSPC-SP 10 (“near-white”) blast-cleaning to ensure that all mill scale is removed, but the near-white finish is not required to remain at the time of erection. In fact, research has shown that some amount of tightly adhering rust can enhance the slip resistance of a connection, and an SSPC-SP 6 finish is sufficient. Unless the girder has been exposed to particularly corrosive conditions during storage, pressure-washing the faying surfaces at the jobsite shortly before erection will remove any loose material detrimental to the connection.

