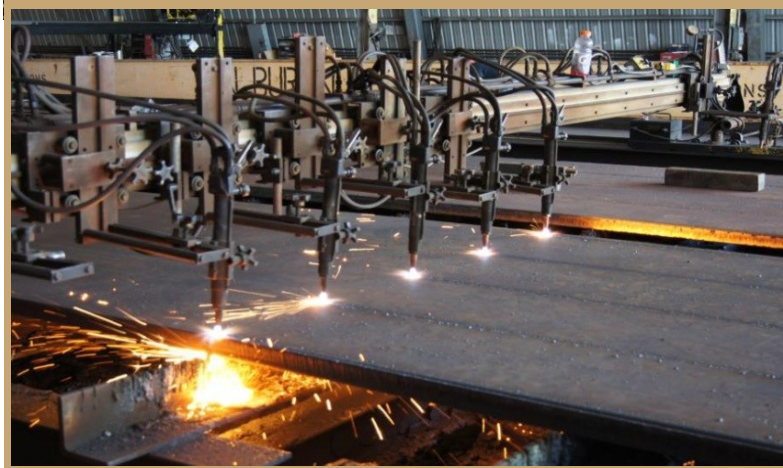


Guidelines to Design for Constructability and Fabrication

G12.1-2020



AMERICAN ASSOCIATION
OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS

AASHTO



American Association of State Highway and
Transportation Officials

National Steel Bridge Alliance

AASHTO/NSBA Steel Bridge Collaboration

Preface

This document presents guidelines developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each document represents the consensus of a diverse group of professionals.

It is desired that Owners adopt and support Collaboration guidelines in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the guidelines recommended herein. In such cases, Owners may adopt these guidelines with the exceptions they feel are necessary.

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FOREWORD

The original G12.1 document was published in 2003. Review comments on the original document were requested from Owners, steel suppliers, Fabricators, Erectors, university professors, and Designers. Based on the review comments and subsequent task group discussions, the document was updated to reflect then current practice and published in 2016. Subsequently, the document was reviewed by another group of independent reviewers, along with task group discussions, comments, and revisions. This current updated document reflects the latest practice for steel mills, fabrication, detailing, erection, and design.

This document has been prepared as a guide and thus much of the information is general in nature, representing the latest steel industry positions. Recommendations should not be considered as strict rules to be followed by any Contracting Engineer, Authority, Fabricator, or Contractor. Also, this document should be used in conjunction with the other Collaboration documents for further clarification on specific issues.

FURTHER READING

AASHTO LRFD Bridge Design Specifications (2020)

AASHTO LRFD Bridge Construction Specifications (2017)

G1.2, Design Drawings Presentation Guidelines (2003)

G9.1, Steel Bridge Bearing Design and Detailing Guidelines (2004)

G13.1, Guidelines for Steel Girder Bridge Analysis (2019)

S8.1, Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges (2014)

S10.1, Steel Bridge Erection Guide Specification (2019)

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SECTION 1 DESIGN

1.1—ROLLED BEAMS VERSUS WELDED PLATE GIRDERS

Where choice is possible between rolled beams and welded plate girders for short span applications, typically rolled beams are considered first (for spans up to 100 ft in length). However, allow the Fabricator to substitute welded plate girders that satisfy design requirements.

1.2—GIRDER SPACING

When choosing the number of girders in the bridge cross-section, consider the following:

- Owner preferences and limitations
- Cost of steel fabrication, transportation, erection, and coating
- Deck thickness and forming methods
- Provisions for future widening
- Stability and redundancy of the structure during future staged/partial re-decking
- Vertical clearances

C1.1

Generally, rolled beams are more economical than welded plate girders for short spans. Market conditions, girder depth restrictions, and product availability may also affect the decision. Standard designs, including details for rolled beam and welded plate girder solutions (developed by the Short Span Steel Bridge Alliance) are available at <http://www.ShortSpanSteelBridges.org>, or search engine key words: “Short Span Steel Bridge Alliance Standard Designs.”

For increased flexibility, Designers may want to include a plate girder option in the design plans. A welded plate girder option should especially be developed if one of the following criteria is met:

- For horizontally curved members with a radius less than 1,200 ft
- For members requiring camber greater than $\frac{1}{4}$ of the depth of the member (e.g., 6-in. camber for a 24-in.-deep member)
- For members requiring camber greater than 2½ in. on a 50-ft length
- If cover plates are required for the rolled beam option
- The span length is over 60 ft
- Rolled section is not available on short notice

C1.2

Many Owners have their own preferences and guidelines for girder spacing. The Designer should refer to these Owner preferences and guidelines, as applicable, when choosing a girder spacing.

For simple spans up to 140 ft, the standards published by the Short Span Steel Bridge Alliance (SSSBA) should be considered.

In general, the use of fewer girders in the bridge cross-section will result in a more economical design because less steel is used.

- Overhang width

The overhang width is generally determined such that the moments and shears in the exterior girder are similar to those in the interior girder. In addition, the overhang is set such that the positive and negative moments in the deck slab are balanced. A common historical practice is to make the overhang approximately 0.25 to 0.35 times the girder spacing.

The NSBA website has Steel Span to Weight Curves on the Design Resources page, which show trendlines of data for various ranges of girder spacing as a function of average span length. Wider girder spacings often result in a lower structural steel unit weight (unit weight taken as pounds of structural steel per square foot of deck area).

Generally, for a bridge with an average span length less than 175 ft, there is not an appreciable difference in the structural steel unit weight for the various girder spacings. For a bridge with an average span length more than 175 ft, the Designer may want to consider a wider girder spacing, perhaps between 11 ft and 13 ft, as this wider girder spacing trends to a lighter steel superstructure. However, as noted later in this commentary, there are tradeoffs associated with a wider girder spacing—such as a thicker concrete deck or deeper girders—that may infringe on vertical clearances. The decision on whether or not to choose a wider girder spacing must be made with consideration of other factors and not structural steel unit weight alone.

Typically, the following benefits are derived from the use of wider girder spacing:

- Lower total structural steel weight
- Fewer girders to fabricate, inspect, handle, coat, transport, and erect
- Fewer cross frames to fabricate, inspect, handle, coat, transport, and erect
- Fewer bolts and connections
- Reduced time of fabrication and erection
- Fewer bearings to purchase, install, and maintain

The following issues need to be evaluated during the decision-making process when wider girder spacing is being considered:

- Potential for a thicker concrete deck resulting in more weight, concrete, and reinforcing steel
- Methods for forming the deck
- Optimal (larger) fascia overhang versus an Owner-specified limit

1.3—MINIMUM THICKNESS FOR STIFFENERS, WEBS, AND FLANGES

For welded girder construction, certain minimum requirements for material thicknesses are normally recommended to reduce deformation and the potential for weld defects, as well as increased corrosion resistance/durability.

Use a $\frac{1}{2}$ in. minimum thickness for stiffeners, connection plates, and webs. Use a $\frac{3}{4}$ in. minimum thickness for flanges.

It is preferable to detail stiffeners and connector plates, and smaller gusset plates, so that they can be fabricated from bar stock or cut from larger plate at the Fabricator's discretion.

At girder field splices with plate thickness transitions across the splice, fill plate thicknesses of $\frac{1}{16}$ in. or less should be avoided.

For a web thickness change of $\frac{1}{8}$ in., use a $\frac{1}{8}$ -in. fill on one side of the web rather than $\frac{1}{16}$ -in. fills on each side.

- Girder depth and infringement on vertical clearance

C1.3

Preferred minimum thicknesses depend on the welding equipment used.

To allow the use of flat bar for stiffeners and connection plates, dimensions of these members should follow the parameters below:

- Widths less than 8 in.: increase width in increments of $\frac{1}{2}$ in.
- Widths between 8 in. and 12 in.: increase width in increments of 1 in.
- Thickness: $\frac{1}{2}$ in., $\frac{5}{8}$ in., $\frac{3}{4}$ in., $\frac{7}{8}$ in., 1 in.

Fill plates with thicknesses of $\frac{1}{16}$ in. or less pose difficulties in fabrication and handling. If the fill plates are blast-cleaned, distortion from the blasting operations can become problematic. Handling of large $\frac{1}{16}$ in.-thick fill plates is also difficult due to the plate flexibility. Note that no filler plates are required for web splices with thickness differences of $\frac{1}{16}$ in. or less, per the *AASHTO LRFD Bridge Design Specifications*, Article 6.13.6.1.3c.

If using CNC to drill flanges without assembly, the $\frac{1}{16}$ -in. offset in the web centerline can be programmed, or the $\frac{1}{16}$ -in. hole oversize may be sufficient to accommodate flange assembly.

1.4—MATERIAL SIZE AVAILABILITY

1.4.1 Plate Material Size Availability

When sizing girder flanges, maximum lengths available for the various plate widths and thicknesses should be considered.

For the design, select material that is readily available. Tables 1.4.1-1 and 1.4.1-2 show dimensions of typically-available plates. Material in the shaded area is available from three U.S. rolling mills as of press time. Contact a mill or Fabricator for the latest plate availability information.

C1.4.1

The availability of material sizes varies from mill to mill. The minimum width available from one mill is 48 in. and from two others is 60 in. Plate is also available in widths up to 190 in. from one mill.

Designers are encouraged to be aware of this example maximum plate length availability data, but generally should not engage in over-analysis of their designs in an effort to maximize plate usage. The layout of flanges and webs within a given plate to maximize plate usage is best left to steel detailers, who develop the exact shape and dimensions of each plate, understand the intricacies of flange and web nesting within a plate (including issues related to runout, camber, heat curving versus cut curving, cutting loss, etc.), and can best address the specific capacities and preferences of a given mill and Fabricator.

Instead, this data is useful to Designers for identifying the likely need for welded shop splices in flanges and webs. In general, it is prudent to allow the use of optional flange and web shop splices at the discretion of the Fabricator, and to include the preferred details for such splices in the contract plans. Also, when evaluating the economics of introducing flange thickness changes (and their associated mandatory flange shop splices) as discussed in Article 1.5.4, it is useful to have some idea of maximum available plate lengths. If the length of a particular flange exceeds the limits in Tables 1.4.1-1 or 1.4.1-2, that may provide more reason to introduce a thickness change.

In all circumstances, though, keep in mind the guidance provided in Articles 1.5.1 and C1.5.1 regarding limiting the number of different plate thicknesses used in a design.

**Table 1.4.1-1—Example Maximum Plate Length Availability
ASTM A709 Grades 36, 50, 50W (all dimensions in inches)**

Plate Thickness	Plate Width				
	72	84	96	108	120
½	972	972	972	972	972
¾	1,035	1,035	1,035	1,035	1,035
1	1,035	1,035	1,035	980	808
1½	1,035	1,035	1,035	720	680
2	1,035	1,035	1,035	720	680
2½	1,035	1,006	880	720	680
3	970	838	734	652	587
3½	830	920	800	635	600
4	720	800	685	600	600

Notes: Widths and thicknesses are grouped for convenience. Other widths and thicknesses are available in similar lengths. Interpolate between adjacent values for other size plates. Material in the shaded area is available from three U.S. rolling mills as of press time.

**Table 1.4.1-2—Plate Length Availability
ASTM A709 Grade HPS 70W (all dimensions in inches)**

Plate Thickness	Plate Width				
	72	84	96	108	120
½	580	580	580	580	580
¾	580	580	580	580	580
1	580	580	580	580	580
1½	580	580	580	580	580
2	580	580	580	580	580
2½	600	600	600	600	600
3	600	600	600	600	600
3½	600	600	600	600	600
4	600	600	600	600	600

Notes: Widths and thicknesses are grouped for convenience. Other widths and thicknesses are available in similar lengths. Interpolate between adjacent values for other size plates. Material in the shaded area is available from three U.S. rolling mills as of press time.

1.4.2 Wide Flange Beam Length Availability

Structural shape sections of various sizes are produced domestically.

Refer to the American Institute of Steel Construction (AISC) website for specific section availability at <https://www.aisc.org/steelaavailability/>.

C1.4.2

Examples of common rolled beam bridge sections are shown in Table 1.4.2-1. Longer lengths may be available, depending on the Producer.

**Table 1.4.2-1: Example Maximum Wide Flange Beam Length Availability
ASTM A709 Grades 50 and 50W**

Section Size & Foot-Weight Range	Maximum Length (ft)
W44 × 230–335	100
W40 × 431	80
W40 × 149–397	100
W36 (all)	100
W33 (all)	100
W30 (all)	100
W27 × 368	80
W27 × 336	90
W27 × 301–307	100
W27 × 94–281	100
W24 (all)	65
W21 (all)	50

Note: Information provided by Nucor-Yamato Steel (April 2014)

1.5—PLATE SIZING

1.5.1 Flange Plate Thickness

Limit the number of different plate thicknesses for a project. Select flange thicknesses in at least $\frac{1}{8}$ -in. increments up to $2\frac{1}{2}$ -in. and $\frac{1}{4}$ -in. increments over $2\frac{1}{2}$ in. See Article 1.3 for minimum flange plate thickness criteria.

When locating flange thickness transitions (shop flange splices), include no more than two butt splices or three different flange thicknesses for an individual flange between field splices, except for unusual cases such as very long or heavy girders or mill length availability limits.

At welded flange splices, the thinner plate should not be less than one-half the thickness of the thicker plate as a rule of thumb.

C1.5.1

An economical individual girder shipping piece has from one to three thicknesses per flange, with each flange having zero to two shop-welded splices. More flange thickness changes are usually not economical and should be avoided unless the girders are unusually heavy or limits on available plate lengths necessitate additional shop flange splicing with or without a thickness change. Availability of material sizes varies from mill to mill; see Article 1.4 for more information. Minimizing the number of flange plate thicknesses for a project reduces mill quantity extras and simplifies fabrication and inspection operations. See Table 1.5.4-1 for information on when thickness transitions are economically justified.

1.5.2 Flange Plate Width

Size flange material so that flanges can be economically cut from plate between 60 in. (preferably 72 in.) and 96 in. wide, even where girder flanges vary from girder to girder.

Keep individual flange widths constant within an individual shipping piece. When changing flange widths is unavoidable, avoid changing flange width at welded shop splices.

For straight plate girders, group flanges to make efficient use of material. For straight plate girders comprised of three flange plates, keep the center plate width, thickness, and length constant between girder lines so that shops can order material as wide plate. Keep the end plates the same width as the center plate, and use a common thickness so that shops can order material as wide plate, and then splice it as shown in Figure 1.5.2-1.

Larger order quantities of plate cost less. Similar sizes of flanges obtained during preliminary design should be grouped to minimize the number of thicknesses of plate that must be ordered. For example, if preliminary design results in eight thicknesses of $1\frac{1}{4}$, $1\frac{3}{8}$, $1\frac{1}{2}$, $1\frac{3}{4}$, $1\frac{7}{8}$, 2, $2\frac{1}{8}$, and $2\frac{1}{2}$ in., consider optimizing the design to four plate thicknesses of $1\frac{1}{4}$, $1\frac{1}{2}$, 2, and $2\frac{1}{2}$ in., or a similar grouping.

C1.5.2

The most economical size plate to buy from a mill is between 72 in. and 96 in. wide. For size availability, see Article 1.4. Fabricators order plate with additional width and length to account for cutting ($\frac{1}{8}$ in. per cut between plates and along sides), plate sweep tolerance, and waste (about $\frac{1}{2}$ in. on each outside edge). For example, a Fabricator might order a plate 74 in. wide to cut five 14 in. wide plates (4 in. of wasted material).

For straight girder bridges, Fabricators order girder flange material from wide plate and splice it either as wide plate or as individual flanges after cutting to width. For constant-width flanges, advantages to welding wide plate rather than stripping and then splicing include having one set of run-on tabs and run-off tabs, fewer crane moves, and considerably fewer weld starts and stops. Changes in thickness rather than width in a field section save as much as 35 percent of the labor required to join the flanges. However, shops frequently decide whether to weld first or strip first depending on crane capacity, hook height, and other individual preferences.

Because flange material with butt splices must be ordered as wide plate and then spliced and stripped or stripped and spliced, a Designer should size the flanges so that plates can be ordered with minimal waste. For bridges with non-parallel supports where the geometry of the flanges could vary from girder to girder, a Designer should consider how material might be ordered and spliced. See Figure 1.5.2-1 as an example.

For curved plate girders:

- Size flanges to get as many pieces as possible from a wide plate.
- Keep flanges in each area the same thickness and approximate length to allow splicing as shown in Figure 1.5.2-2.
- Maintain constant flange widths full length within a field section and consider nesting during sizing of plates for curved girders. (In fabrication, nesting is the technique of laying out component cutting patterns on a plate to optimize material use.) See Figure 1.5.2-2.

When in doubt, consult a Fabricator.

For curved-girder bridges, if the Fabricator chooses to heat-curve the members, the approach will be the same as for bridges with non-parallel supports, and the shop will curve the members after completing most of the fabrication. If the Fabricator chooses to cut-curve the members, the amount of material that will be wasted in cutting the curve is an additional consideration.

As an example of the material wasted, if the radius for the flanges in Figure 1.5.2-2 was 700 ft and the center plate was 2 in. thick by 60 ft long, the amount of waste for the center plate (the shaded area) would be about 3,100 pounds whether the plate cuts four flanges or one flange. Depending on whether adjacent girders use common flange thicknesses and transition points, some Fabricators may choose to splice the flanges as wide plate similar to straight girders and some will cut curve the plates to width prior to splicing. In either case, the amount of waste material may be significant. In the interest of economy, the Designer should consider how material might be ordered and spliced. See Figure 1.5.2-2 as an example.

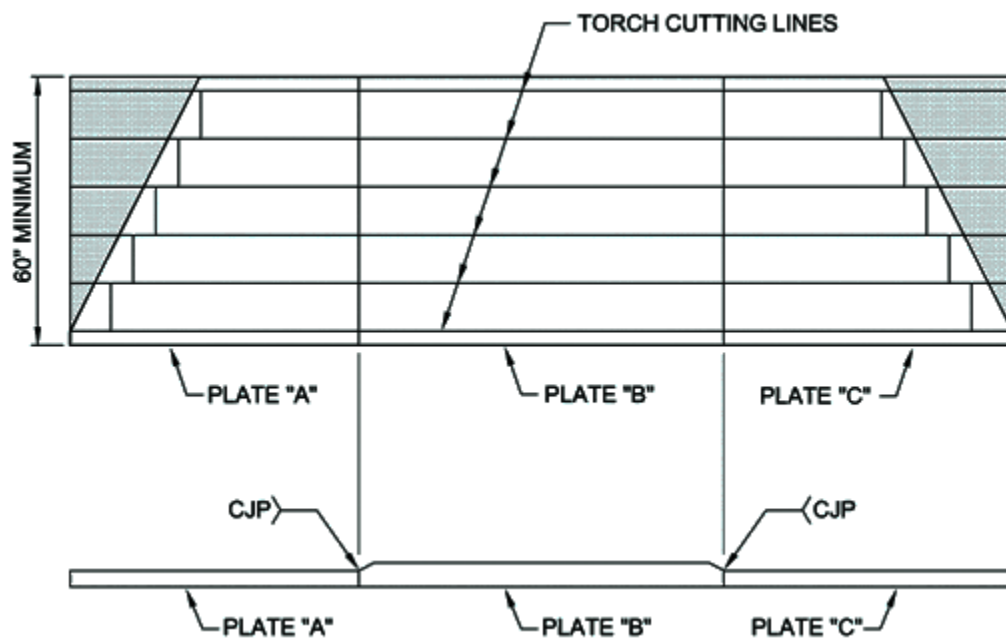


Figure 1.5.2-1

Straight Girder Example of Welding Wide Plates, then Stripping out the Flanges

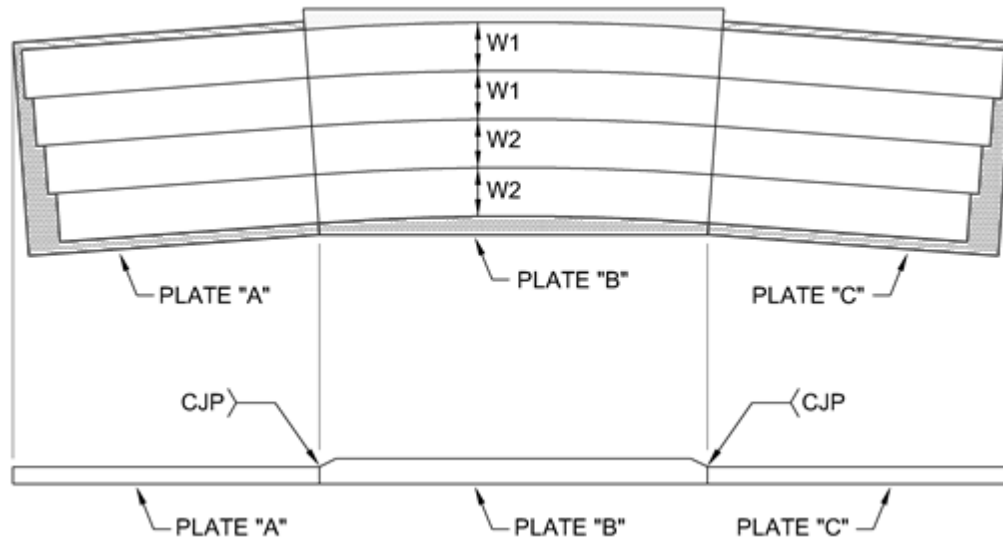


Figure 1.5.2-2

Curved Girder Example of Welding Wide Plates, then Stripping out the Flanges

1.5.3 Web Sizing

See Article 1.3 for minimum web plate thickness.

Minimize the use of transverse intermediate stiffeners.

The selection of web depth should consider geometric issues (such as minimum vertical clearances), fabrication and transportation issues, and overall economy of the design.

C1.5.3

A web thickness 1/16" less than that required to achieve an unstiffened web typically leads to a "partially stiffened" web design in which a few stiffeners may be needed near the supports. Partially stiffened web designs are generally considered the most economical choice for routine steel girder highway bridges. Excessive use of intermediate stiffeners is usually uneconomical due to the increased cost of fabrication.

When establishing girder web depths, first verify that minimum vertical clearance requirements under the bridge can be met. Other considerations include fabrication issues (maximum plate widths versus the need to introduce longitudinal shop splices in the web), transportation issues (maximum girder dimensions for transportation), and overall economy.

Economy is achieved using girders that can be shipped web vertical by truck, which is limited by overhead clearances on the shipping route. Girders that are under 9' deep can generally be shipped vertically on a truck.

Above this height it will depend on the route from the Fabricator to the site. Girders that are too deep to ship vertically can be shipped with the web horizontal, but supporting the full length with the girder's weak axis in bending tends to be much more challenging and costly. Horizontal limits also depend upon constraints along the route, such as toll booths, bringing the girder vertical at the jobsite, and other issues of practicality. It may be possible to ship girders 18' deep horizontally, but for bridge members at this depth it is prudent to check with potentially affected state Departments of Transportation (DOTs) regarding permits, a major Fabricator, and a hauling contractor about actual constraints.

Deeper girders are generally more economical, but only up to a point. To assess overall economy, it may be valuable to perform a web depth study where the web depth is incrementally increased, the girder is redesigned (targeting a partially stiffened web design), and the resulting girder weight versus web depth is recorded. These data points (girder weight versus web depth) can then be plotted to determine the optimum (minimum girder weight) web depth. Some steel girder design software packages (e.g., LRFD SIMON by NSBA) offer automated web depth study features; otherwise the study can be performed by simply iterating the design with different web depths.

Avoid the use of longitudinal web stiffeners in routine steel girder bridges.

Typically, the use of longitudinal web stiffeners is only economical in long-span steel girders where using very slender webs can lead to significant reductions in girder weight.

See Articles 2.1.2.3 and 2.1.2.4 for further discussion.

1.5.4 Shop-Welded Splices

Introduce a shop flange splice and a flange thickness transition when the weight savings will justify the work associated with the welded splice. Table 1.5.4-1 provides a method to make the evaluation.

Specify a shop-welded splice when the savings in flange material and when plate length limitation or special circumstances dictate. Table 1.5.4-1 provides a method to make the evaluation.

C1.5.4

Efficiently locating thickness transitions in plate girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. The parameters affecting the cost of shop-welded flange splices vary from shop to shop. For both straight and curved girder bridges, Fabricators often request to eliminate a shop splice by

In the design or specifications, provide criteria the Fabricator may follow to eliminate shop-welded flange splices by extending thicker plate.

extending a thicker flange plate. Design and specifications should consider allowing this practice, subject to the approval of the Engineer. When evaluating the request, Designers should review the percent change in deflections and stresses.

Many Owners have guides for economical flange thickness transitions. Some have graphs based on thickness change, length of change, and the thicker plate, but others use experience-based guidelines (e.g., the Texas Department of Transportation, a.k.a. TxDOT, estimates saving 800 to 1,000 pounds may justify a butt splice). Table 1.5.4-1 shows weight savings per inch of flange width that may be used to evaluate placement of shop splices. The criteria vary, especially for large curved girders, so Fabricators should be consulted whenever possible.

The following example demonstrates the use of Table 1.5.4-1:

Evaluate splicing a plate 16 in. \times 1 in. \times 35 ft to a plate 16 in. \times 1½ in. \times 35 ft versus using a plate 16 in. \times 1½ in. \times 70 ft. The weight saved by adding the splice is equivalent to the weight of a plate 16 in. \times ½ in. \times 35 ft (16 in. \times 0.5 in. \times 3.4 lbs/in.²/ft \times 35 ft = 952 lbs), about 950 pounds. The weight savings needed to justify adding the splice is determined by using a factor of 70 pounds per inch from Table 1.5.4-1, times the plate width of 16 inches, resulting in a value of 1,120 pounds. Because the actual saving is 950 pounds, Table 1.5.4-1 indicates that it is more economical to extend the 1½ in. plate for the full 70 ft than to add the shop splice.

**Table 1.5.4-1: Estimated Weight Saving Factor per Inch of Plate Width
For ASTM A709 Grade 50 Non-Fracture-Critical Flanges Requiring Zone 1 CVN Testing**

Multiply weight savings/inch × flange width (length of butt weld)							
Thinner Plate at Splice (inches)	Thicker Plate at Splice (inches)						
	1.0	1.5	2.0	2.5	3.0	3.5	4.0
1.0	70	70	70	—	—	—	—
1.5	—	80	80	80	80	—	—
2.0	—	—	90	90	90	70	70
2.5	—	—	—	100	100	80	80
3.0	—	—	—	—	110	90	90
3.5	—	—	—	—	—	110	110
4.0	—	—	—	—	—	—	130

Notes:

- Source: compiled from various Fabricators, November 2001.
- Weight factors for non-fracture-critical Zone 2 material are the same as for Zone 1, as shown, except that in the shaded areas the factors should be reduced by 20 percent.
- For compression flanges where Charpy V-Notch (CVN) testing is not required, the factors should be increased by about by about 10 percent, except the bottom two rows should increase by about 30 percent.
- For fracture-critical material, the factors should be reduced by values between 10 percent and 25 percent, depending upon the thickness.
- Materials other than A709 Gr. 50 will have values that will vary from those shown in the table.
- For intermediate thicknesses, interpolate between closest values.
- Where equal plate thicknesses are joined, table values indicate welded splice cost in terms of steel weight. Steel cost per pound is based on unfabricated steel plate, not the bid price of fabricated, delivered steel.

1.6—DIFFERENTIAL DEFLECTIONS

1.6.1 Fit and Differential Deflections

When choosing a fit condition for a steel I-girder bridge, consider:

- Differential deflections at each cross frame
- Span length
- Radius (for curved bridges)
- The need for lateral rotational capacity of the bearings

For recommendations on what fit condition is appropriate for a given bridge, see the document “Skewed and Curved Steel I-Girder Bridge Fit,” published by NSBA in 2016.

In practice historically, contractors or Fabricators have made the fit choice when not specified by the Designer in the contract plans, but considering the influence the fit condition can have on member loads (as well as constructability), it is prudent for the Designer to make this decision, as is currently

C1.6.1

In an I-girder bridge, “fit” refers to how the cross frames are detailed and fabricated to fit to the girders. They may be:

- Detailed to fit when all dead loads are applied (“total dead load fit,” TDLF, or “final fit”);
- Detailed to fit at erection (“steel dead load fit,” SDLF, or “erected fit”);
- Detailed to fit in the no-load condition (“no-load fit,” NLF, or “fully-cambered fit”); or
- Detailed to fit at some other condition in between.

The girders and cross frames may actually fit at more than one or all of these conditions. The distinction here is not whether or not the bridge components actually fit in these

required by the *AASHTO LRFD Bridge Design Specifications*.

conditions, rather it is how the bridge is detailed to fit.

The detailed fit condition can influence:

- The ability to construct the bridge. For example, choosing TDLF for a sharply curved bridge can make the bridge unconstructable; and
- Internal loads associated with the fit condition.

“Differential deflections” refer to the difference in girder deflection at either end of each cross frame. When differential deflections exist (as they must on skewed and curved bridges), cross frames tend to deflect a different amount on either end. Since the cross frames are very stiff, they cannot easily distort to accommodate these differential deflections, so the result for most bridges is that the girders twist.

For bridges detailed to TDLF, there will be layover (twist) at erection, but generally the girders will come back to plumb under total dead load. Conversely, on a skewed bridge detailed to steel dead load fit, girders will be plumb at erection but will experience some final layover under total dead load. For this reason, the fit condition is sometimes referred to as the “plumb” condition. However, it is not recommended to refer to fit in this way because it confuses the issue—particularly when the “plumb” discussion is extended to curved girders in which layover and plumb do not work the same way.

The NSBA document referenced in the recommendation provides more explanation about these choices and phenomena.

1.6.2 Deflection due to Phased Construction

Consider using single member top and bottom struts; omission of the cross frames or diaphragms between construction phases; or use field-drilled holes, slotted holes, or field-welded connections where phased construction would cause significant differential deflection in the bay between previous and new construction (phase 1 versus phase 2, existing versus widening, etc.). Note that for curved girders, the *AASHTO LRFD Bridge Design Specifications* do not allow slotted or oversize holes.

Provide a deck placement sequence diagram in the contract plans.

C1.6.2

If phased construction is required, the differential deflection between phases due to the application of dead loads at different times can be significant. There are many ways to address this in the design and detailing of cross frames between the adjacent construction phases.

The use of independent single-member top and bottom struts without diagonals (also known as “lean-on” bracing) with a single bolt in each end is one simple and effective possible way to address this situation when the differential deflections are not too significant.

Using this type of detailing provides some bracing for the girder compression flange while still allowing for differential deflection between adjacent phases built at different times. Care should be taken to ensure that deflections are not so great that the deflected orientation of the brace is so far out of plane from the girder compression flange that it cannot continue to function as a brace.

Another approach that may be effective in cases where differential deflections are small is the use of slotted holes for the connections on one side of the cross frames in the bay between the adjacent phases. In this situation, the cross frame is installed with fully-tightened bolts in the connections to one girder, but with snug-tight bolts in slotted holes in the connections to the other girder. Once the second phase has had its deck placed and it has fully deflected, the snug-tight bolts can be tightened or field-welding can be used for the permanent connection.

For either of these two above approaches, some consideration should be given to detailing a closure pour in the deck between the two phases. Using a closure pour simplifies deck forming by avoiding the need to use a deck forming system which accommodates differential deflections.

When the differential deflections are larger, an effective approach is to omit the cross frames in the bay between the two adjacent phases until after the decks are placed on both phases and deflections have occurred. Typically, in these cases a closure pour is also detailed in the deck. The sequence of construction would then be:

1. Erect the steel for the first phase.
2. Cast the deck for the first phase.
3. Erect the steel for the second phase, separated from first phase by a portion of open deck.
4. Cast the deck for the second phase (leaving a space open for a closure pour between the first and second phase).
5. Install the cross frames in the bay between the phases. If rebar extensions (for the closure pour, unless fully mechanical)

do not permit dropping in the cross frames from above they may have to be erected from below.

6. Cast the open portion of the deck between the phases (the closure pour).

When using this approach, is it critical that both the first and second phase are wide enough and stable enough to allow each to be constructed independently without experiencing global stability problems (for further discussion, see AASHTO/NSBA G13.1, *Guidelines for Steel Girder Bridge Analysis*).

Another alternative to consider for a straight non-skewed bridge without long spans is eliminating the cross frames in the bay between construction phases. Some Owners require cross frames in every bay, so this might not be an acceptable solution in all jurisdictions. Note that permanent cross frames/diaphragms would still be needed at pier and abutment locations to transfer lateral loads (wind, EQ) to the substructures.

1.7—BEARINGS

Use elastomeric bearings wherever possible (as the design allows). To ensure proper behavior for thermal expansion and contraction on curved girder bridges, the Designer should align the guided expansion bearings on a chord to the nearest fixed bearing, or to the estimated point of thermal origin. Alternately, unguided bearings can be provided to allow the superstructure to expand in any direction and to minimize unintended restraint or bind-up of the bridge. Laterally restrained bearings should equal the minimum number required to constrain the bridge and transmit lateral loads to the substructure.

Following deck placement on curved and/or skewed bridges, consideration should be given to jacking and resetting the bearings to the proper thermal placement.

See AASHTO/NSBA Steel Bridge Collaboration G9.1, *Steel Bridge Bearing Design and Detailing Guidelines*.

C1.7

Elastomeric bearings are versatile and a very economical choice for bearings. Bearing types vary considerably from structure to structure and from state to state. The use of elastomeric bearings has increased markedly over the last several years, and Fabricators unanimously prefer them. The quality and capacity of elastomeric bearings has improved significantly. State standard specifications showing limitations of these units can easily become obsolete. Pot bearings or disc bearings should be used where loads are too large for elastomeric bearings.

Usually bridges have a defined central point of fixity (estimated point of thermal origin) which will mitigate the likelihood of the bridge "walking" due to thermal ratcheting effects or frozen bearings. Loads may be applied to the substructure which were not properly accounted for in the original design. However, designs with no true point of fixity have been used.

Additionally, refer to Texas Department of Transportation (TxDOT) research project on elastomeric bearings, FHWA/TX-98/1304-3,

1.8—DECK HAUNCH

The deck haunch is the distance from the top of the steel girder web to the bottom of the concrete deck slab (see Figure 1.8-1). The detailing of this haunch must be considered by the bridge Designer. For welded plate girders, the haunch depth is usually the same for all girders and will theoretically remain constant along the length of the girders. The dimension is usually set to accommodate variations in top flange thickness, camber tolerances, and the thickness of splice plates, along with consideration of the deck cross-slope and deck forming method. In some instances, the deck haunch may need to be set to minimize or remove sag camber in the girder web. The depth dimension of the haunch at the centerline of the girder web should be shown on the contract plans.

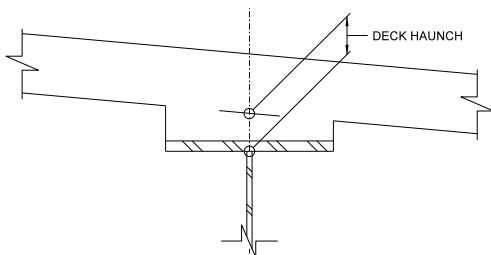


Figure 1.8-1: Concrete Deck Haunch

Figure 1.8-1 is shown for a typical welded plate girder bridge. The haunch over a rolled beam may be detailed from the top of the top flange to the bottom of the deck and will typically be a variable height due to camber and dead load deflection. Also, note that steel tub girders are typically sloped parallel to the deck cross-slope, thereby creating a constant haunch depth transversely over the top flange.

An Experimental Study of Elastomeric Bridge Bearings with Design Recommendations.

C1.8

The haunch is a dimension determined in design to accommodate the required deck thickness, flange thicknesses, steel tolerances, effects of the deck cross-slope, and the difference between predicted and actual girder deflections.

In some instances, the roadway geometry may introduce sag camber into the girder web resulting in an undesirable aesthetic appearance, where to an untrained eye there is an appearance of excessive deflection. Sag (or negative) camber can be defined as any section of the curve formed by the top of the web in its final position that falls below a working line constructed through the top of the web at the girder ends. Sag camber can result from not only a sag vertical curve on the bridge but other conditions, such as the right combination of roadway profile, alignment, and cross-slope. The Designer is cautioned to the potential of sag camber and may want to explore other methods to remove the sag camber from the girder web, such as revising the roadway geometry or potentially detailing a variable depth deck haunch. Sag camber is an aesthetic, not strength, concern, and the Designer should engage the Owner in the discussion and ultimately the decision on how to treat it when it occurs.

Although the haunch depth is theoretically constant along the length of the plate girders, the depth dimension usually needs to be adjusted in the field due to girder camber, and fabrication and erection tolerances.

The contractor will survey the top of the girder after erection and then set the deck formwork for the actual haunch (vs the theoretical) based on the actual erected camber in order to achieve the deck thickness and finished deck elevations shown on the plans.

Shear stud connectors are used to create a composite section between the girder and the concrete deck in order to transfer the horizontal shear between the top of the girder and the deck. Shear reinforcement in the concrete haunch may be required when the

1.9—ERECTION STRESSES AND CONSTRUCTABILITY

There are times when site conditions limit access to construction equipment and prohibit temporary support locations, for example, when erecting a steel bridge over railroad tracks. Erecting a single girder at a time may be the only possible method of erection. The Designer should consider flange proportions such that the girders can resist the factored loads from *AASHTO LRFD Bridge Design Specifications* Article 3.4.2 (excluding Strength IV limit state) and state specific requirements during erection.

Another factor that contributes to this situation is the maximum shipping length. Attention to *AASHTO LRFD Bridge Design Specifications*' Equation C6.10.3.4.1-1 should be given.

depth of concrete measured from the top of the top flange to the bottom of the concrete deck exceeds a certain thickness specified by agency guidelines. Shear reinforcement is also typically required if the shear stud connectors do not penetrate a minimum of 2 in. into the deck slab in order to transfer the shear adequately into the deck.

The haunch width is typically set as the same width as the top flange. The deck forming method will affect the haunch width. For example, where steel stay-in-place deck forms are used, they typically employ clip angles which are attached to the top flange, requiring the haunch to be the same width as the flange.

C1.9

In cases where long unbraced lengths may be unavoidable, such as a single girder placed with no cross frames if that is the only possible method of erection, F_{cr} as calculated using L_r , with *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD Design) Equation 6.10.8.2.3-5 or A6.3.3-5 will control the allowable erection stresses from the self-weight of the steel. The wind loads during erection will create a simultaneous lateral stress on the flanges (f_1). The dead load stresses must be taken in combination with the lateral stresses to meet the requirements of AASHTO LRFD Design Equation 6.10.8.1.1-1. Amplification of the stresses may be required as given in AASHTO LRFD Design Equations 6.10.1.6-4 and 6.10.1.6-5.

The closer the flange proportions get to the lower limit in *AASHTO LRFD Bridge Design Specifications* Equation C6.10.3.4.1-1, the less capacity the girder has to resist erection stresses. The greater b_{fc} is, the more resistance the girders will generally have to resist the stresses from self-weight, curvature effects, and wind during construction.

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SECTION 2 GIRDER DETAILS

2.1—STIFFENERS AND CONNECTION PLATES

2.1.1 Orientation With Respect to Flanges

2.1.1.1 Bearing Stiffeners

Typically, bearing stiffeners for I-girder bridges are specified to be vertical after all dead load. Alternatively, bearing stiffeners can be placed normal to the flanges.

2.1.1.2 Connection and Intermediate Stiffeners

Connection and intermediate stiffeners should be normal to the flanges. In haunched girders, the stiffeners should be normal to the top flange.

Some Fabricators prefer the stiffeners to not project past the flange edges to avoid extra handling of welding machinery.

2.1.2 Connection Details

2.1.2.1 Bearing Stiffener Connection to Flanges

Use finish-to-bear plus a fillet weld to connect bearing stiffeners to bottom flanges if a diaphragm or cross frame is connected, and use finish-to-bear if there is no cross frame. Permit welding with finish-to-bear at the Fabricator's option, even if not required for a connection.

For connection to the top flange, finish-to-bear is unnecessary. Welding the stiffener to the top flange is only necessary if there is a diaphragm or cross frame connected to the bearing stiffener.

C2.1.1.1

Many Owners prefer bearing stiffeners to be vertical after all dead load. Generally, the difference between bearing stiffeners placed to be vertical after dead load or normal to the flanges is minimal. There is no clear benefit one way or the other.

See Article 3.4 for recommendations related to box girders.

C2.1.1.2

Fabricators prefer intermediate connection plates and stiffeners to be normal to the flanges. This avoids the need to bevel the ends of the stiffeners and connection plates.

C2.1.2.1

The connection of the bearing stiffener to the bottom flange may be:

- Finish (mill or grind) to bear if no diaphragm or cross frame is connected, or
- Finish (mill or grind) to bear plus a fillet weld, or
- Complete joint penetration (CJP) weld.

Fabricators prefer finish-to-bear (allowing the option of milling or grinding) plus a fillet weld, an approach that dramatically reduces welding deformation of the bottom flange compared to a CJP weld and costs less.

2.1.2.2 Connection Stiffener Attachment to Tension Flange

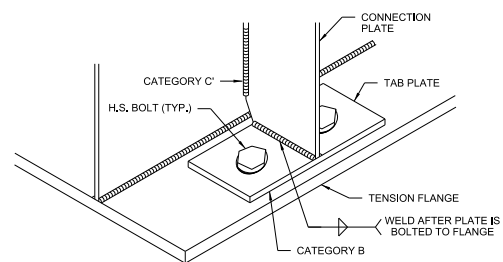
Weld the connection stiffener to the tension flange whenever justified by the economics of the design, or by a slightly increased tension flange thickness, or an adjusted location of the cross frames. Avoid bolted tab plates.

C2.1.2.2

For box girders, see Article 3.6.

The *AASHTO LRFD Bridge Design Specifications* require a positive attachment of the cross frame connection stiffener to both flanges. The connection to the compression flange is typically welded, but the connection to the tension flange is either welded or bolted through a tab plate that has been welded to the connection stiffener.

Designers may require bolting tab plates to flanges to provide improved fatigue resistance for the flange. The Category C' fatigue detail for welding a stiffener to the tension flange may require a larger flange than a Category B bolted tab. However, Designers should note that the weld attaching the connection plate to the web is of the same fatigue category (Category C') as the weld to the tension flange. (See Figure C2.1.2.2-1) The live load stress range at the surface of the flange is approximately equal to the live load stress range on the web at the termination of the weld; therefore, replacement of a welded connection with a bolted connection will not improve the fatigue resistance of the girder as a whole. Bolted tab plates are more costly than welding the stiffener directly to the flange. Additionally, it is expensive to clean and paint the contact surface before installing the tab, and will be a source of dirt build up and corrosion on the flange for weathering steel applications.



**Figure C2.1.2.2-1 Bolted Tab Plate
(NOT RECOMMENDED)**

Whether using bolts in a tab plate or elsewhere in flexural members, Designers need to be aware of the effects of holes in tension members. When checking flexural members at

the strength limit state or for constructability, the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD Design) limit the stress on the gross area of the tension flange to prevent fracture on the net section of the flange with holes. For compact composite sections in positive flexure, the nominal flexural resistance is permitted to exceed the moment at first yield at the strength limit state. However, for these sections, AASHTO LRFD Design still limits the stress in the tension flange with holes to first yield. This limit is imposed since it is not fully documented that the complete plastification of the cross-section can occur before fracture of the net section of the tension flange. Designers need to be aware of this, particularly where bolted retrofits are made on compact sections that originally did not have holes. The addition of the holes in a flange retrofit can reduce the strength of the section since the original section was allowed to exceed the moment at first yield. This can lead, for example, to a rolled beam section which was originally designed based on compact section properties now being limited to a stress below first yield—which can result in a substantial reduction in the moment capacity of the section, and ultimately the permitted load-carrying capacity of the bridge.

2.1.2.3 Intermediate Stiffeners (Not Connection Stiffener)

As per the *AASHTO LRFD Bridge Design Specifications*' requirements for straight I-girder bridges, transverse intermediate stiffeners without cross frame connections should have a “tight fit” or be cut back at the tension flange and be “tight-fit” or attached to the compression flange. The preference is to cut back the stiffener at the tension flange.

For single-sided stiffeners on curved I-girder bridges, transverse stiffeners should be attached to both flanges. For pairs of stiffeners on curved I-girder bridges, transverse stiffeners should be “tight-fit” or attached to both flanges.

C2.1.2.3

A “tight-fit” stiffener may help to straighten flange tilt without application of heat.

Attaching single-sided transverse stiffeners to both flanges helps retain the cross-sectional configuration of the girder when subjected to torsion. This also avoids the potential for locally high bending stresses within the web, especially at the top flange due to the torsional restraint of the deck slab (see *AASHTO LRFD Bridge Design Specifications* Article 6.10.11.1.1).

2.1.2.4 Longitudinal Stiffeners

Minimize the use of longitudinal stiffeners when practical. When they are needed, place them on the girder face with the fewest transverse stiffeners or cross frame connection plates. Use fillet welds and avoid complete joint penetration weld attachments between transverse and longitudinal stiffeners, unless required for fatigue.

If an intersection of longitudinal and transverse stiffeners will be entirely in a compression zone, make transverse stiffeners continuous and interrupt the longitudinal stiffeners and use a fillet weld to join the longitudinal stiffener segments to the transverse stiffeners.

If the intersection of longitudinal and transverse stiffeners will be located in a tension zone, then make the longitudinal stiffener continuous to facilitate fatigue detailing. Terminate the longitudinal stiffener (beyond the tension zone) by welding the longitudinal stiffener to the face of a transverse stiffener, using a fillet weld. Welded shop splices in longitudinal stiffeners should be complete-penetration groove welds.

Refer to *AASHTO LRFD Bridge Design Specifications* Figure 6.6.1.2.4-1 for details.

C2.1.2.4

From a design efficiency standpoint, although longitudinal stiffeners resist bend-buckling of the web with little material, they are disproportionately expensive in relation to their structural benefit. Generally longitudinal stiffeners should be avoided in routine steel girder bridges and considered only for long-span steel girders when appropriate to avoid overly thick webs because fitting and welding longitudinal stiffeners to the transverse stiffeners and connection plates involves considerable labor cost.

Consideration should be given to wrapping the weld around the end of the longitudinal stiffener for sealing.

On curved girders, the use of longitudinal stiffeners introduces additional difficulty because:

- The rigidity of the stiffener's strong axis resists fitting to previously curved webs without buckling the stiffener.
- Putting the stiffener in a tightly curved girder would require a curved stiffener, and curving longitudinal stiffeners requires either time-consuming heat curving, or cut curving with high material waste.
- If the Fabricator prefers to heat-curve the girder after welding is complete (which is typical practice), the longitudinal stiffener will resist curving and cause local distortion in the girder web.

From a fabrication standpoint, it is better to keep transverse members continuous and interrupt the horizontal stiffener (rather than the reverse) for a number of reasons:

- Handling and fitting small horizontal lengths is easier than very long pieces.
- Transverse member welds can be made with typical equipment without having to interrupt welding at the horizontal stiffener.
- Installing the horizontal stiffener first can lock in local web distortions that

become difficult to correct when adding the transverse members.

- Fabricators use transverse members to hold girder geometry, including keeping flanges square, and it is more difficult to correct geometry later.
- Longitudinal stiffeners are typically used on webs in their compression areas to resist web bend-buckling, where there is no reason to require a CJP at the longitudinal stiffener connection to a transverse stiffener or connection plate.

While the fabrication difficulties of interrupting the longitudinal stiffener at intersections with transverse stiffeners and connection plates are recognized, from a structural standpoint, this should only be done if the intersection is in a region where the web is subject to net compression in order to avoid conditions susceptible to fatigue and constraint-induced fracture (see AASHTO LRFD Design Table 6.6.1.2.4-1) due to the build-up of force in the gap that would occur if the longitudinal stiffener is interrupted. In addition, they should be attached to the intersecting transverse element because the longitudinal stiffeners are designed as continuous members to resist web bend-buckling (see AASHTO LRFD Design Article C6.10.11.3.1).

A problem can occur where the web is subject to reversal from the live load and the stress ranges can be large. In regions subjected to net tension or stress reversal under Strength Load Combination I, longitudinal stiffeners are less susceptible to fracture and fatigue if the attachment parallel to the primary stress is continuous and the transverse attachment is discontinuous. Refer to the *AASHTO LRFD Bridge Design Specifications*, Article 6.6.1.2.4 for more details. Typically in these cases, the longitudinal stiffener is extended to the first transverse stiffener beyond the region of web tension (no longer in reversal) so that the panel with the longitudinal stiffener is bounded by transverse stiffeners. Once the stiffener intersection is in a tension zone, the longitudinal stiffener conforms to strain and stress in the web and behaves like a flange. The force in the longitudinal stiffener must be transferred by

2.1.2.5 Tolerance of Fit between Bottom of Bottom Flange and Bearing Sole Plate

Use the fillet weld fit-up provisions of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* to determine the appropriate tolerance of fit between the bottom of the bottom flange and the bearing sole plate. (Also see Article 2.1.2.1.)

either a weld connecting it to the transverse stiffener with a detail producing the desired fatigue category, or run through the transverse stiffener and terminate it on the web. In this situation, the longitudinal stiffener has the same stress range as the web which has the transverse stiffener fillet welded to it. This allows use of a fillet weld to connect the transverse stiffener to the longitudinal stiffener.

Longitudinal stiffeners could be bolted to transverse stiffeners or connection plates instead of welding, but that requires additional steel elements plus drilling and shop-installed bolts, costing far more than a simple fillet-welded connection, with no fatigue or performance benefit.

C2.1.2.5

Distortion in the bottom flange from welding the web to the flange may cause a gap at the joint between the edge of the bottom flange and the sole plate. The thinner the flange, the more distortion will result. The AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* explicitly addresses this issue only for the projected area of the bearing stiffener and web on the sole plate. See Figure C2.1.2.5-1.

Away from the projected area of the bearing stiffener and web, flatness tolerances do not apply, but tolerances for fit-up to the sole plate should be taken into account. If the sole plate will be fillet-welded to the bottom flange, normal AASHTO/AWS D1.5M/D1.5 fillet weld fit-up tolerances would apply to that joint, which will have the effect of limiting the flatness of the bottom flange in the area of the sole plate periphery.

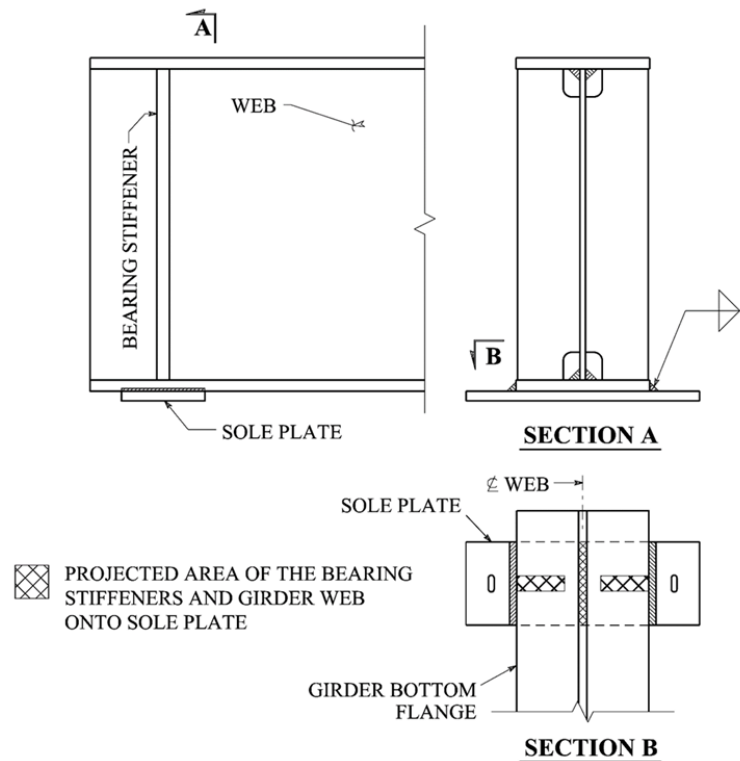


Figure C2.1.2.5-1 Projected Bearing Area

2.1.2.6 Connection of Skewed Cross Frames or Diaphragms

For cross frames at bearing connections, or in cases where intermediate cross frames must be skewed, give the Fabricator the option to use either a skewed connection or bent gusset plates. See Figures 2.1.2.6-1 and 2.1.2.6-2. Where support lines are skewed more than 20 degrees from normal, intermediate cross frames and their connection plates are to be placed normal to the girder line (see *AASHTO LRFD Bridge Design Specifications* 6.7.4.2).

C2.1.2.6

Many Fabricators prefer to bend the gusset plates rather than skew the connection plate when connecting skewed cross frames to girders. Skewed connection plates create fitting and welding problems, especially as the degree of skew increases. If the skew angle exceeds 30 degrees (from perpendicular), welds will probably have to be done manually using shielded metal arc welding (SMAW), and special qualification of the joint is required. If the skew angle exceeds 60 degrees, the weld is no longer considered a fillet weld but rather a partial joint penetration groove weld, and more time-consuming procedure and welder qualification may be required. Weld size will need to increase and stiffeners may need to be burned on a bevel. Precise fitting becomes more complex because connection plates on opposite sides of the web are different distances from the girder ends. However, cross frames with bent gussets can be fit using jigs or templates to provide accurate connections.

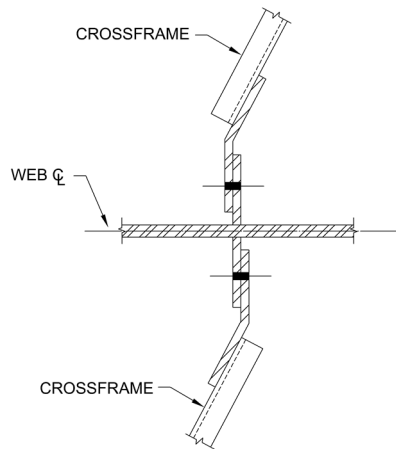
If bent gusset plates are used, they should be designed appropriately (that is, consider out-of-plane bending effects, etc.)

Another option is to use a bent connection plate. When a bent connection plate is used, the out of plane bending occurs in the bent connection plate, which is supported on three edges (by welded connection to the girder web and flanges) and the gusset plates themselves are straight. In certain circumstances (such as extreme sharp skews at end diaphragms) this detail may offer some advantages.

TxDOT has implemented a split-pipe detail, not included in the current (9th Edition, 2020) *AASHTO LRFD Bridge Design Specifications*, for use at bearing end of unit diaphragm connection plates, in which the connection plate is welded normal to a half pipe which is in turn welded to the web and flanges. This is from research project number 0-5701, “Improved Cross Frame Details.” See Figure C2.1.2.6-1. Several other considerations when using this detail are:

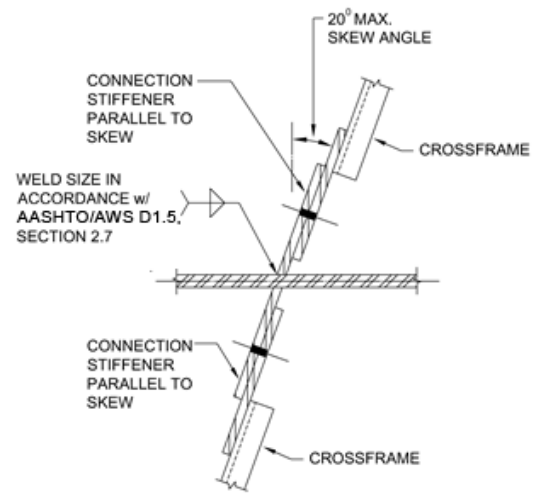
- If galvanized, venting of the split pipe should be provided during the galvanizing process.
- Pipe materials cannot be ordered as fracture critical.

Tubular sections are not covered by AASHTO/AWS D1.5M/D1.5. TxDOT has required standard fillet weld procedures.



BENT GUSSET PLATE

Figure 2.1.2.6-1



SKEWED CONNECTION STIFFENER

Figure 2.1.2.6-2

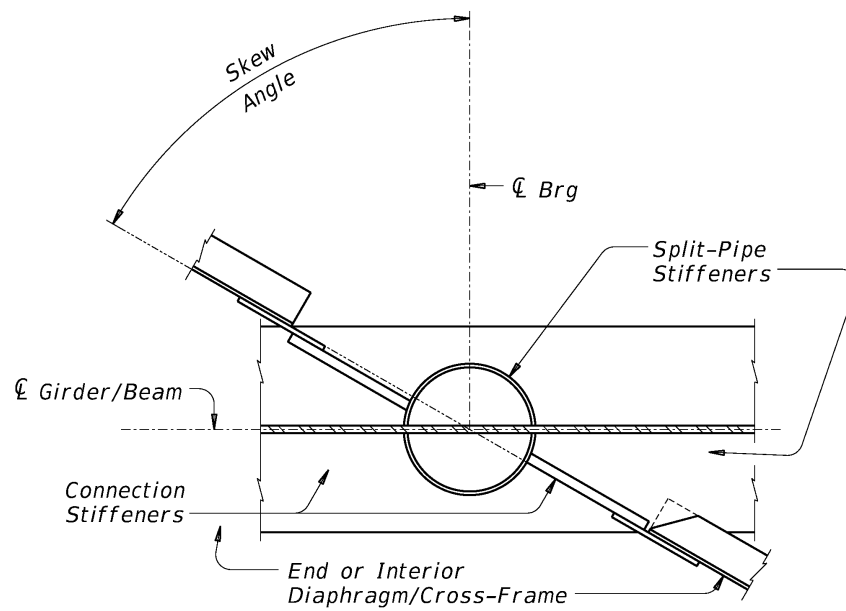
**SPLIT-PIPE STIFFENER 60° MAX SKEW ANGLE**

Figure C2.1.2.6-1

2.1.3 Minimum Spacing between Adjacent Stiffeners or Connection Plates

Provide 8 in. minimum spacing or 1½ times the plate width for welding access. In the case of skewed stiffeners or connection plates, the spacing should be measured from the closest edge of the plate and not necessarily from the plate's intersection with the web; more space will be required than for stiffeners perpendicular to the web.

C2.1.3

This recommendation allows space for machine welding procedures to be used and applies to all transverse web stiffeners and connection plates.

Check with a Fabricator for any other plates welded in very close proximity to each other to verify welding access.

Where multiple stiffeners are required over high load multi-rotation or steel bearings, the bearing sole plate size and required connection bolts may affect where stiffeners can be located. When applicable, there must be sufficient room to install bolts and replace the bearing in the future, as well as to inspect the connection during routine maintenance activities.

If jacking stiffeners are provided for future bearing replacement or adjustment, those may require close spacing to be positioned over the substructure, but access must be sufficient for stiffener to flange and stiffener to web welding.

When multiple bearing stiffeners and/or jacking stiffeners are used along with one-piece cross frames, the Designer should evaluate the ability of the Erector to install the cross frame. There may not be space for the cross frame to swing into place if multiple bearing or jacking stiffeners are present.

Sub-stiffeners transverse to main stiffeners (and parallel to the girder length) complicate welding access and create pockets prone to collect debris and corrosion, so they should be avoided.

2.2—GENERAL DETAILS

2.2.1 Field Connections

2.2.1.1 Computer Numerically Controlled (CNC) or Template-Drilled Field Splices

Computer numerically controlled (CNC) or template drilling of field splices for both I-girders and box girders should be allowed if the Fabricator consistently demonstrates the accuracy of the system and acceptability of the final product. The Fabricator should provide a written in-depth procedure to the Engineer describing operational processes and inspection and verification steps. Limited check

C2.2.1.1

CNC equipment can improve quality and economy in fabrication operations. Properly calibrated, programmed, and operated equipment provides accuracy that ensures fit of the structure in the field without requiring shop assembly for drilling or reaming. Fabricators should demonstrate that their particular methods will provide satisfactory results.

assemblies, along with continuous monitoring of the process, should assure the accuracy of the final product.

2.2.1.2 Drilling Field Splices in Longitudinal Shop Assembly

2.2.1.2.1 Extent of Assembly

Bearing to bearing assemblies, or assemblies with a specified minimum number of girders, should be typically required unless the Owner allows the Fabricator to demonstrate geometric control methods using alternative means (sequential progressive assembly for upper and lower truss components or virtual assembly using laser scans) that will assure that the fully erected structure will fit.

Drilling elements individually (without placing in assembly) using CNC equipment or templates should be permitted if satisfactory results can be demonstrated by either initial or periodic check assemblies. CNC equipment must have its calibration periodically verified, usually by its manufacturer, and personnel must be properly trained to program and locate the equipment.

Templates for multiple locations should have hardened bushings to ensure holes aren't elongated if the drill isn't properly centered. Splice plates may be employed as single use templates, but either core-type bits or sub-size holes are needed. The system to accurately locate templates should be reviewed by the Engineer and demonstrated by personnel.

C2.2.1.2.1

Historically, a minimum of three girders and/or 150 ft of length were required to be assembled to grade and sweep before any drilling or reaming was allowed to begin. At a minimum, the last of three field sections from the previous stage is "carried forward" to the current progressive girder line assembly, and the actively reamed (RA), drilled (DA) or check-fit (NC) field splices are adjusted in elevation to keep alignment within tolerance. Refer to Figure C2.2.1.2.1-1.

The same method of geometry control to get three girders in assembly is applicable to have two girders in assembly. The only requirement should be that the adjacent girders are assembled in the correct relation to one another.

Virtual assembly using computer modeling to determine exact member configuration, making upper and lower truss assemblies independently so diagonals and verticals are theoretically straight under final dead load, and allowing member splice patterns to be drilled individually without assembly are all proven methods to expedite fabrication. Software verification and periodic check assemblies are recommended to prove continued accuracy. ASTM A6 permits rolled beam webs to be up

to $\frac{3}{16}$ in. off center and AASHTO/AWS D1.5M/D1.5 permits $\frac{1}{4}$ in., so the actual web position must be considered if drilling flange splice holes without assembly.

When drilling or reaming, progressive assembly (using the historical method) allows the Fabricator to make adjustments within the (span-based) AASHTO/AWS camber tolerances (up to $1\frac{1}{2}$ in. for longer spans). This may be especially helpful when subsequent girder field sections (cambers) have already been fabricated, i.e., when camber for subsequent sections cannot be pre-corrected at NC web-cutting stage.

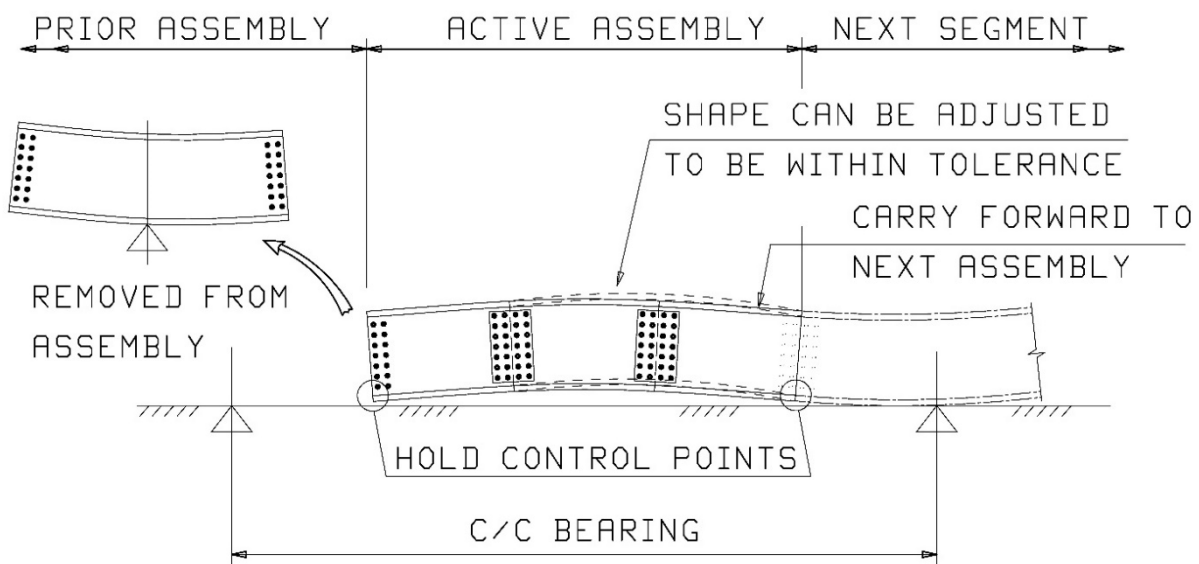


Figure C2.2.1.2.1-1 Progressive Assembly

2.2.1.2.2 Position of Assembly for Straight I-Girders

Drill or ream field splices in straight I-girders with the web horizontal or vertical (at the Fabricator's option). If webs are vertical, block or support members in the no-load position.

C2.2.1.2.2

Depending upon the Fabricator's equipment, experience and configuration of the bridge, the assembly can be made with the webs either vertical or horizontal. Either method will yield similar results if executed properly.

2.2.1.2.3 Position of Assembly for Curved I-Girders

Drill or ream field splices in curved I-girders with the web horizontal or vertical at the Fabricator's option. If webs are vertical, block or support members in the no-load condition.

C2.2.1.2.3

The geometry of the structure is typically the primary factor to determine the orientation of the web during assembly. Fabricators generally want to minimize the height of the assembly due to the additional blocking and fall protection required. The assembly can be made with the

2.2.1.2.4 Position of Assembly for Box Girders

Permit members to be rotated to use more convenient work lines or planes from the beginning to end of each assembly, or use a chord line from beginning to end of each continuous span unit. Shop drawing details must accurately show all dimensions and elevations to assemble the members properly for drilling or reaming.

2.2.1.3 Full Shop Assembly of Steel Girder Structures

Full or partial shop assembly including transverse elements should only be required by contract for appropriate cases, including: structures that are very rigid (e.g., bascule and through-girder railroad bridges); structures with small radii or complex geometry; and where girders terminate at load-carrying diaphragms or other girders. Allowing the Fabricator to use alternate schemes that will ensure proper final fit without assembly should be considered to expedite fabrication and reduce cost.

webs either vertical or horizontal. Either method will yield similar results if executed properly.

C2.2.1.2.4

Field splices in straight or curved box girders are typically assembled completely prior to shipping to the job site. The connections have traditionally been drilled or reamed while assembled, but may be drilled CNC without full assembly if the Fabricator provides the engineer with a geometry control plan.

To minimize blocking and the elevation of the assembly, the girders can be rotated to eliminate grade and cross-slope.

C2.2.1.3

Shop assembly of a single girder line (line assembly) for the purpose of drilling or reaming girder splice connections or for confirming the accuracy of pre-drilled connections is common. Inclusion of transverse elements (e.g., cross frames) in assembly (full shop assembly) is not common and is expensive and time-consuming. Therefore, if the Owner intends to require full shop assembly, this requirement should be clearly indicated in the contract documents. Extra emphasis is placed on “clearly indicated” because full shop assembly is unusual and can have a big impact on cost and schedule. It is important to make sure this requirement is clear. When full shop assembly is required, no load fit is typical for drilling splices, and fit under steel self-weight deflection is typical for simple spans like through girder railroad bridges where girders are usually in final position and not supported to prevent all deflection. Full shop assembly of a bridge that has been detailed for total dead load fit (TDLF) should not be specified.

Curved girders usually have sufficient transverse and vertical flexibility to allow relatively small horizontal and vertical displacements for installation of cross frames in the field without prior shop assembly. Heavy, deep, or rigid members will not be as flexible and may require shop assembly if the radius is tight. Important issues include longitudinal

accuracy and how the members are supported during erection.

The amount of assembly required to ensure fit in the field has changed with evolving design modeling and construction practices. It varies depending upon the Fabricator's and Erector's knowledge, methods and equipment. Current precision fabrication technologies and laser verification coupled with the ability to model and predict behavior during progressive stages of erection may eliminate the need for shop assembly. Fabricators and engineers can cooperate to determine how much assembly to perform based on experience and considering the responsibility and consequences for fit-up problems in the field.

The Owner is responsible to clearly specify any full assembly or partial transverse assembly requirements in the contract documents, which should communicate whether progressive transverse and line assembly are allowed. The Owner's instructions are best placed in the structural steel notes of the design plans.

2.2.1.4 Field Welding Considerations

In general, bolting is preferable to field welding when practical. In most instances, bolted connections can be installed more efficiently with less skilled labor than is required for field welding.

There are instances when field welding may be more practical due to load demands, the geometry of the connection or the sequence of construction. In addition, field welding is a useful tool for repairs or to strengthen an existing member. When field welding is necessary, it must be performed in conformance with the welding procedures and specifications. A properly-performed weld is acceptable whether it is performed in the shop or in the field.

Considerations to be made when specifying field welding include:

C2.2.1.4

Use field welds as needed. Some instances where field welding may be appropriate include:

- Orthotropic decks
- Bearings
- Expansion joints
- Shear stud welding
- Temporary works
- Field repairs
- Retrofits to existing structures
- Stay-in-place (SIP) deck formwork (in compression areas)
- Large sections that are impractical to ship by available means
- Cross frame connections between staged construction

- 1) Local availability of experienced, qualified welders and welding inspectors.

In geographic areas where field welds are extensively used, experienced and qualified welders will be more readily available than other areas. While most welders can be qualified, it is also necessary to employ welders who, through experience, have proven they are capable of producing welds that consistently meet the acceptance criteria of the applicable welding code.
- 2) Means to establish and maintain fit-up and dimensional control of the connection or assemblage.

The use of field welds to connect primary members (e.g., girders) or secondary members (e.g., diaphragms) introduces the potential for misalignment and loss of dimensional control. For instance, when diaphragms are shop welded, the dimensional accuracy can be verified and mistakes minimized.
- 3) Sensitivity of welding to environmental factors such as cold, wind, rain, snow, structure vibration or deflection, etc.

Ambient temperatures influence the amount of heating required to maintain appropriate pre-heat, interpass, and post-heat temperatures. Maintaining temperature can be a challenge in cold environments. Enclosures may be required to prevent wind from removing shielding gases.
- 4) Means to provide safe and stable access for welder to all parts of the weld

A stable, well-positioned platform allows the welder to focus on the weld. Access to both sides of the weld must be considered if required. When space is limited, a mock-up may be appropriate to ensure there is sufficient room for the welder to perform the weld safely. Confined spaces with elevated temperatures and shielding gases for the welding process may present safety issues that need to be addressed.
- 5) Proximity of weld location to welding machine

If field welding is required along the length of the bridge, relocation of the welding machine and leads may be required.
- 6) Position of weld

Field welding may require that the welds be performed out of position (e.g. vertical or overhead) when the same welds may have been performed in a flat or horizontal position in the shop.
- 7) Repair of coatings

When performing field welds, details for repairing paint and other coatings must be provided.
- 8) Effect of welding on construction schedule

Field welding introduces an additional operation during erection. This may be

9) Partial joint penetration (PJP) and fillet welds are preferred over complete joint penetration (CJP) welds whenever possible. Access for second side gouging, grinding and welding is often obstructed or very expensive for the contractor. If CJP welds without fused backing are necessary to satisfy fatigue criteria, safe access must be considered.

10) If a weld must be specified, a standard weld detail is preferred

11) Weldability of existing base metal for repairs or retrofits

12) Specify appropriate welding code (especially for retrofits)

13) Field NDT

2.2.2 Use of Standard, Oversize, and Slotted Bolt Holes

Standard size holes should be used for connections in straight and curved plate and box girder bridges to maintain geometry control.

overcome if the welding can be performed off the critical path of the construction schedule.

Partial joint penetration and fillet welds are generally more cost effective and quicker to perform than complete joint penetration welds, but PJP welds and CJP welds with fused backing are not allowed transverse to cyclic tensile stress, such as full depth butt joints between beams. Fillet welded splice plates have significant limits on stress range at fillet terminations or perpendicular to the fillet throat. CJP butt joints between beams or girders require weld access holes in webs at each flange to avoid areas which cannot be backgouged and ground to sound metal before the second side weld.

Using standard weld joint details reduces the time and expense of qualifying a non-standard joint. Refer to Figures 2.4 and 2.5 in the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code*.

Historical grades of steel may have properties that are not conducive to welding.

Older bridges may predate the ASTM A709/AASHTO M 270 specifications, and therefore, fall outside of the AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* Clause 1 limitations on applicability. The D1.5M/D1.5 commentary notes the chemical and mechanical similarities between A709/M 270 and common building steels (ASTM A36, A572 Gr. 50 and A588), with CVN requirements being the primary difference. The engineer must develop project specific requirements or reference other welding specifications in this instance. AWS D1.7/D1.7M *Guide for Strengthening and Repairing Existing Structures* may assist in determining the properties and weldability of older steels.

The field is not conducive to RT. May want to consider UT or PAUT.

C2.2.2

AASHTO LRFD Bridge Design Specifications Article 6.13.1 and *AASHTO LRFD Bridge Construction Specifications* Article 11.8.3.5 specify that standard-size bolt holes are to be

Oversize or slotted holes are not allowed by *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD Design) for curved plate or box girder bridges. Additionally, they are not allowed by AASHTO LRFD Design for longitudinal girder field splices. See Article 2.3 of these Guidelines.

Oversize or vertical short slotted holes can be used for cross frame connections to straight plate girders, if needed due to differential deflections due to phased construction.

Oversize holes should be used in one ply of a lateral bracing connection. The choice should be at the Fabricator's option with approval by the Owner.

2.2.3 Haunched Girders

2.2.3.1 Curved or Straight Haunch

When using haunched girders (girders with varying web depths), a straight-line or parabolic haunch can be used. Parabolic haunches are often more aesthetically pleasing. For very deep haunched girders, consideration should be given to a bolted longitudinal web splice to facilitate fabrication and shipping. See Article 2.3.

used in connections of primary load-carrying members in horizontally curved bridges, unless otherwise permitted by the contract documents.

Oversize and slotted holes can be useful for cross frame connections in widenings and phased construction of straight plate girder bridges. Another possibility in these cases is to use gusset plates as templates to field-drill standard size bolt holes in the connector plates after dead load is applied, but this requires the contractor to work under the deck, adding considerable cost and time for completing the connection, and does not control lateral movement.

Slotted holes should only be used in one of the two connected members. The other member should have a standard hole.

C2.2.3.1

Using non-parallel flanges will add effort to building a girder, as well as the diaphragms which vary in depth within the haunch area. However, the use of haunches to achieve design efficiency, vertical clearance or project aesthetics may be justified. For design considerations for sloped flanges, refer to the *AASHTO LRFD Bridge Design Specifications* and Blodgett Design of Welded Structures.

Parabolic haunches mimic the negative moment diagram, provide smooth, aesthetic transitions near contraflexure areas and take about the same effort as "straight" haunches (or tapers). Straight and parabolic tapers require the same amount of original web material, and simplify geometry for a transition to a constant depth web or field splice. Because CNC equipment for web cutting is prevalent, for most Fabricators cutting the curve of the parabola is as straightforward as cutting a straight taper. Fabricators usually fit flanges to haunched webs using typical fitting pressure, though some heat assistance may be necessary for thick flanges.

Variable-depth girders may not lend themselves to mechanized girder building equipment, so if the framing arrangement permits, it is best to

keep the entire haunch within a given field piece or offer an optional field splice at the end of the piece.

At negative moment supports, the haunch must transition to a horizontal plane for the bearing attachment. Rather than attempting to reverse the parabolic curve (recurve geometry) near the bearing, allow Fabricators the option to connect the descending flanges to a horizontal flange plate using a complete joint penetration weld, making the plate long enough to cover the bearing and accept all bearing stiffeners. If additional stiffeners are provided for future bearing replacement, they should also land on the horizontal flange as described in Article C2.2.3.2.

Longitudinal web splices facilitate use of very deep haunches, and therefore, help achieve a girder solution for long-span bridges. They can utilize vertical splice plates sandwiching the upper and lower webs or flanges welded to each web to provide a horizontal splice plane. The small, bolted flanges also provide longitudinal stiffening, which is often required for deep girders near supports, and stabilize the upper and lower girder assemblies for shipping and erection. The flanges are continuous for the full length of the girders whereas vertical splice plates might be discontinuous for handling and erection. The introduction of flanges welded to each web represents a fatigue detail that requires consideration by the engineer.

2.2.3.2 Bottom Flange at Bearing

Design of the flange transition from the flat bearing part of the girder to the curved or sloping part of the haunch should allow for either bending or welding at the transition point. When sizing the bottom flange plate at that location, consider the length of plate available from the mills and the possibility that the Fabricator will bend the plate. The dimension from the edge of the sole plate to the transition should be at least 12 in. At the point of transition, a radius should be introduced to promote the bending of the flange (5-ft minimum radius). Additionally, the Owner may wish to consider future jacking needs.

C2.2.3.2

See also Article C2.2.3.1. The transition of the flat bottom flange to the sloping part of the haunch normally uses a welded joint or a bent plate. The distance from the point of tangency to the edge of the sole plate on the bottom needs to be large enough to clear any distortion that may result from welding or bending the flange.

The 12-in. minimum dimension from the edge of the sole plate to the transition is a guiding dimension. The main issue here is to allow the steel to be flat and true to receive the sole plate while allowing the flange to be formed to follow the web geometry. It can be designed

less than 12 in., but the Engineer should be open to a request to increase the dimension.

As a guide, transitions that are less than 1 ft 0 in. in web depth change per 10 ft in length transition can be bent (e.g. 4-ft web depth change in a 40-ft transition length). This may not be able to be accommodated if the flange is too wide or thick.

Full- or partial-transverse stiffening should be provided at the point where the bottom flange becomes horizontal, unless the provisions of AASHTO LRFD Design Article D6.5.2 are satisfied for the vertical component of the inclined flange force using a length of bearing equal to zero (note: this requirement appears in the 9th Edition of AASHTO LRFD Design, published in 2020).

2.2.4 Curved Girders—Heat Curve or Cut Curve

Permit either heat curving or cut curving in accordance with the limits of the *AASHTO LRFD Bridge Design Specifications* at the Fabricator's option.

2.2.5 Girder Field Section Length

Use piece lengths that can be shipped by truck and consider contractor redesigns which allow the Fabricator to add or move splices to optimize delivery. Girders can readily be shipped in lengths up to 120 ft. Depending on route and site constraints, pieces over 150 ft and even approaching 175 ft can be delivered.

C2.2.4

Heat curving is a process in which a straight girder is heated to induce horizontal curvature. A combination of strip heating and V-heating may be utilized to achieve the proper curvature. Cut curving is a process in which the required horizontal curvature is burned into the flange, and the web is then wrapped to meet the curvature of the flange. Either method is effective when executed properly, and may be employed based on the requirements of the structure.

The *AASHTO LRFD Bridge Design Specifications* allow the use of both heat curving and cut curving procedures, with restrictions on the use of heat curving. See *AASHTO LRFD Bridge Construction Specifications* Article 11.4.12 for additional guidance.

C2.2.5

The equipment used to transport longer girders adds significantly to the length of the truck, up to 30 ft more. This will, in turn increase the amount of room that is required to make turns to deliver the girder to the job site.

Curved girders may require additional field splices due to the curvature of the girders and the location of the girder on the shipping equipment.

2.2.6 Cross Frames and Diaphragms

2.2.6.1 Intermediate Cross Frames or Diaphragms for I-Girder Bridges

Use cross frame types shown in Figure 2.2.6.1-1 or Figure 2.2.6.1-2. The Fabricator should be permitted to use parallelogram as well as rectangular configurations to keep connection plates identical. If the angle of the diagonals in an X-type frame would be less than 30 degrees, use a K-frame, otherwise use the X-Frame. The Z-frame shown in Figure 2.2.6.1-3 may be an acceptable option for girders more than 42 in. deep, and the bent plate diaphragm, or a rolled channel section as in Figure 2.2.6.1-4, is a good option for girders less than 48 in. deep.

All girder loads require permits and some jobs require the Fabricator to do a route survey to determine how the product will be delivered to the job site. Due to the extreme length, width, height, or weight of some girder loads, police escorts may be required to transport the load to the job site.

Generally, for straight field section lengths up to 120 ft, routine plate girders stability calculations will not be needed for most common hauling conditions. For lengths between 135 ft and 175 ft, depending on trailer support configurations, it may often be prudent for the Owner to request transportation stress calculations of the contractor's hauler. For field section lengths exceeding 160 ft, safe handling checks may also be warranted for girder building/loading conditions, using a software program which checks handling support limit states.

Curved girders add a different set of issues. Because the girder is curved, it must be balanced on the shipping equipment. This causes the width of the load to be more than the mid-ordinate of the girder alone.

C2.2.6.1

Cross frame types vary considerably both within and between states. AASHTO/NSBA Steel Bridge Collaboration G1.4, *Guidelines for Design Details*, should be adopted whenever possible. The following recommendations cover the more common applications.

Fabricators prefer single-angle (or when necessary, single-member, such as a WT shape) bracing. Double angles are expensive to fabricate, and painting the backs of the angles is difficult to accomplish. Fabricators prefer cross frames such as the K-frame or Z-frame that can be welded from one side only. However, where connections are subject to salt spray, they should be sealed by welding on both sides. Configuration of cross frames should allow as many identical frames as possible. Differences in elevations should be accounted for in the

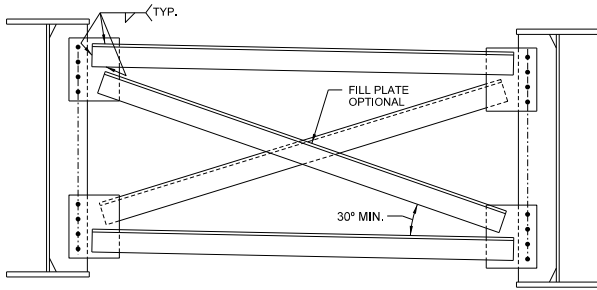
cross frames, not the connection plates. Configuring cross frames as parallelograms instead of rectangles can increase the number of identical connection plates.

Eliminating the top chord of K-frames is not recommended because it has low stiffness at the middle of the cross frame due to lack of depth. Also, it is preferred that X-frames have a top chord as well.

For single member diaphragms, using a W shape is less expensive than an MC shape even though the flanges must be coped. However, the W section is more difficult to paint. Other options are bent plates or plate girders.

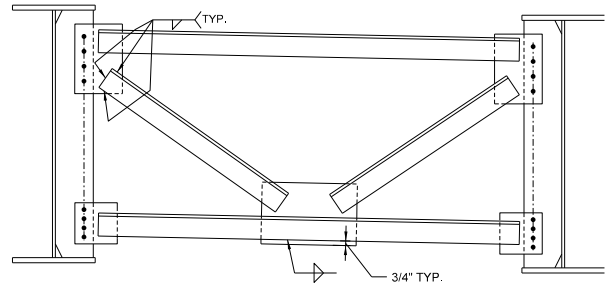
For welded gusset-to-angle connections, it is preferable to keep the welding on one side of the cross member (as shown in Figure C2.2.6.1-1) as opposed to both sides. This allows the cross frame to be fabricated without having to turn the frame assemblage over to weld the backside. Turning the cross frame over requires the availability and use of a crane in the shop, as well as the time to reset the frame to continue the welding.

If front and backside welds are desired, it is recommended that the welding is detailed as shown in Figure C2.2.6.1-2, and not detailed as weld-all-around because the weld-all-around requires the Fabricator to weld through the corner as shown in Figure C2.2.6.1-3. When welding through corners, it is very challenging to add weld metal without undercutting the base metals.



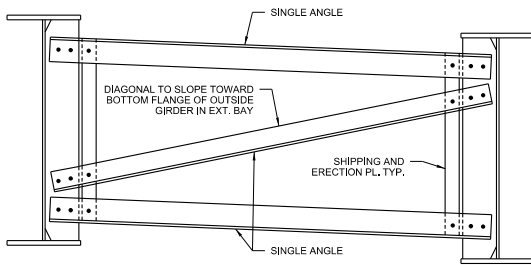
INTERMEDIATE CROSSFRAME

Figure 2.2.6.1-1



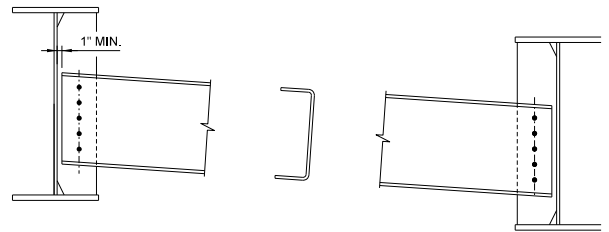
INTERMEDIATE CROSSFRAME

Figure 2.2.6.1-2



ALTERNATE FRAME
(NOT TO BE USED WITH ROLLED BEAMS OR PLATE GIRDER < 42" DEEP)
(STRAIGHT GIRDER $KL/r < 120$)

Figure 2.2.6.1-3



BENT PLATE DIAPHRAGM

Figure 2.2.6.1-4

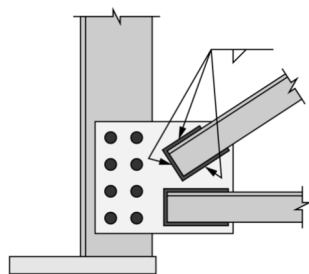


Figure C2.2.6.1-1
Welding Single-Sided is Preferred

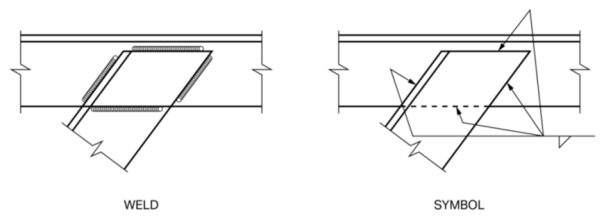


Figure C2.2.6.1-2
Welding Double-Sided
Welding Through Corners is Not Required

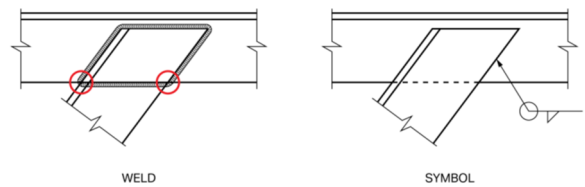


Figure C2.2.6.1-3
Welding All-Around is Not Preferred

2.2.6.2 Intermediate Cross Frames or Diaphragms for Rolled Beam Bridges

Several options are acceptable for intermediate cross frame or diaphragm types for rolled beam bridges:

- AASHTO/NSBA Steel Bridge Collaboration G1.4, *Guidelines for Design Details*.
- Rolled beam or channel with connection angles shop welded or bolted to diaphragm. Field connection bolted to beam web.
- Bent plates with a depth of $\frac{1}{2}$ the beam depth. See Figure 2.2.6.1-4.

2.2.6.3 End Cross Frames for I-Girder Bridges

For I-girder bridges, use end cross frame types shown in Figures 2.2.6.3-1 and 2.2.6.3-2.

See further guidance in AASHTO/NSBA Steel Bridge Collaboration G1.4, *Guidelines for Design Details*.

C2.2.6.2

End angles attached to rolled-beam or channel diaphragms for field bolting to stringers can eliminate intermediate connection plates.

If the stringer is painted, welding plates is a better alternative than bolting a connection angle.

C2.2.6.3

An “inverted K-type” frame is preferred as the end cross frame type. The end cross frame shown in Figure 2.2.6.3-1 requires more gusset plates and welding than the cross frame shown in Figure 2.2.6.3-2. Also, the end cross frame shown in Figure 2.2.6.3-1 has welding on the front and back, requiring the frame to be flipped over to complete the welds. Therefore, the end cross frame shown in Figure 2.2.6.3-2 may be preferable. However, if bent gusset plates are used due to a skewed condition, then the end cross frame shown in Figure 2.2.6.3-1 is preferred because the channel would be skewed to the connection plate and could not be bolted directly to it.

Some Owners prefer to have studs on the top of the top chord channel to make it composite with the deck.

End cross frames and their attachments may need to be designed for future jacking. Plate girder end diaphragms can be considered for this purpose, as well as being a useful option for severely skewed end diaphragms, where the width-to-height ratio is high.

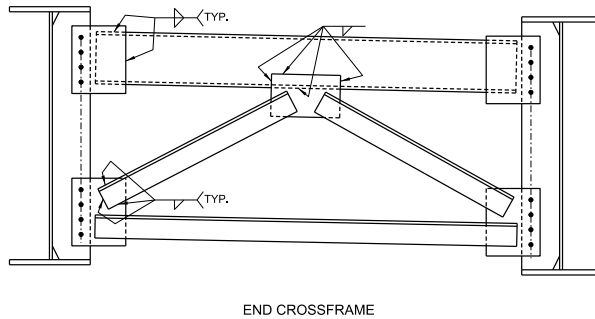


Figure 2.2.6.3-1

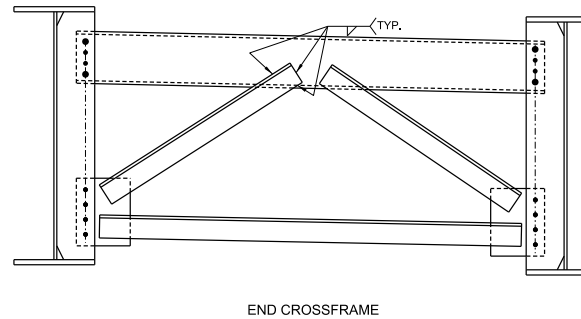


Figure 2.2.6.3-2

2.2.6.4 Gusset Plates

Designers should carefully consider the need for clipping the corners of gusset plates. If there is not a specific need for clipped or non-clipped corners, consider allowing the detailer and Fabricator to decide the specific gusset plate geometry; specify any necessary requirements (such as maximum permissible bolt pitch/gauge) associated with the optional geometry. Alternately, if there is a specific need for clipped or non-clipped corners, clearly identify the required geometry on the plans.

C2.2.6.4

There are a number of reasons for clipping the corners of gusset plates, or for avoiding the clipping of corners. Depending on the specific geometry, requiring clipped corners may result in significant additional effort in cutting the gusset plates and also may result in significant waste of material. On the other hand, detailing gusset plates with clipped corners may facilitate welding and may eliminate the need for extraneous sealing bolts. In the past, requiring clipped corners almost always resulted in increased fabrication costs associated with material waste and increased fabrication time and effort, but modern CNC cutting methods and advanced detailing software help reduce these costs. Some have also argued that clipping of corners can produce improved aesthetics, or improve safety during future in-service bridge inspections by minimizing protruding sharp corners, but such considerations are generally considered secondary for most typical bridge applications.

Consider the case shown in Figure C2.2.6.4-1. The gusset plates which connect the top chord and diagonals to the girder connection plate are clipped. Consider first the large clip, parallel to the diagonals. If this corner had not been clipped, the gusset plate would have a larger faying surface in contact with the girder connection plate, and that faying surface would require bolts for sealing the gusset plate to the connection plate. This would result in either: a) a larger vertical pitch between the three rows of bolts; or potentially, b) one or more additional rows of bolts. Adding one or more additional rows of bolts would add cost

in terms of additional bolts, additional drilling, additional field work during erection, and increased opportunity for misaligned bolt holes. The specific “best” solution in this case would depend on the exact geometry, including considerations such as whether the three rows could be spread far enough apart without violating the maximum sealing pitch or other design requirements or if additional rows would be needed, whether a large number of gusset plates could be nested and cut from a single plate in such a way as to minimize material waste, etc.

Another consideration arises from the possible need to weld the back-side connection of the diagonal to the gusset plate. Consider the small clip, perpendicular to the diagonals. If it is necessary that the back side of the diagonal be welded to the gusset plate, then clipping this corner would facilitate the making of that weld as a single straight line weld. If the corner were not clipped, then the weld would be longer and more complicated to complete as it would have to go around the unclipped corner of the gusset plate. Requiring this type of back-side weld is not generally recommended, as it adds fabrication cost (both in terms of the cost of the weld itself, and the added time and effort in the fabrication shop associated with the need to flip the cross frame to the make the backside welds), but some Owners require this weld as a sealing/corrosion protection measure.

Since some of the considerations discussed above are beyond the control of the Designer (such as determining a nesting pattern for cutting multiple gusset plates from a single piece of plate stock), Designers are encouraged to allow the detailer and Fabricator the flexibility to determine whether to clip the corners or not if other design considerations do not govern. Required criteria (such as maximum permissible bolt pitch/gauge, etc.) could be provided in a note on the plans, and the specific proposed geometry would be subject to the Engineer’s review and approval during the shop drawing review process.

Figures C2.2.6.4-2, and C2.2.6.4-3 show photos of the clipped corner gusset plates illustrated in Figure C2.2.6.4-1. Figure C2.2.6.4-4 shows a

cross frame with gusset plates which have non-clipped corners, and Figures C2.2.6.4-5 and C2.2.6.4-6 show photos of the non-clipped corner gusset plates illustrated in Figure C2.2.6.4-4.

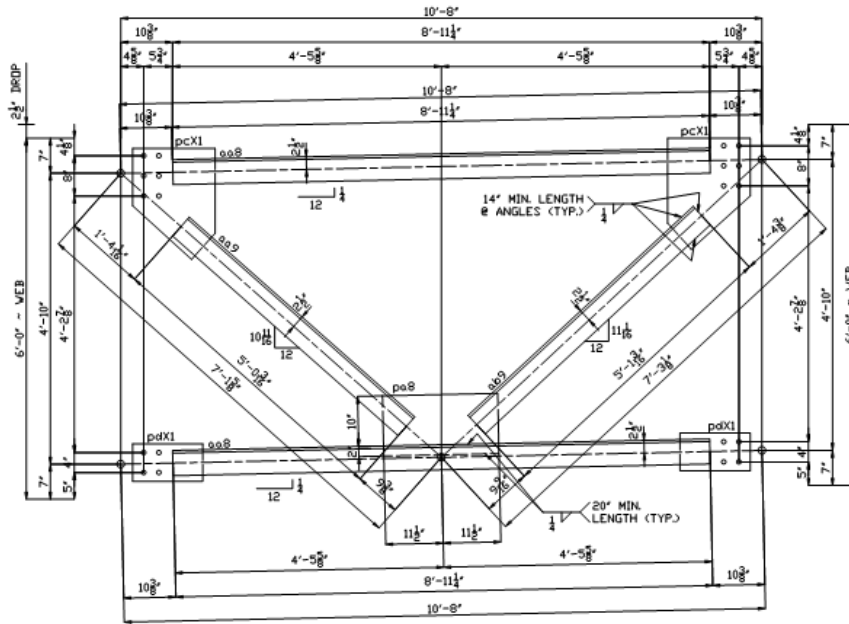


Figure C2.2.6.4-1: Shop drawing of a cross frame where the gusset plates have clipped corners.



Figure C2.2.6.4-2: Photo of a cross frame where the gusset plates have clipped corners.



Figure C2.2.6.4-3: Photo of a cross frame where the gusset plates have clipped corners.

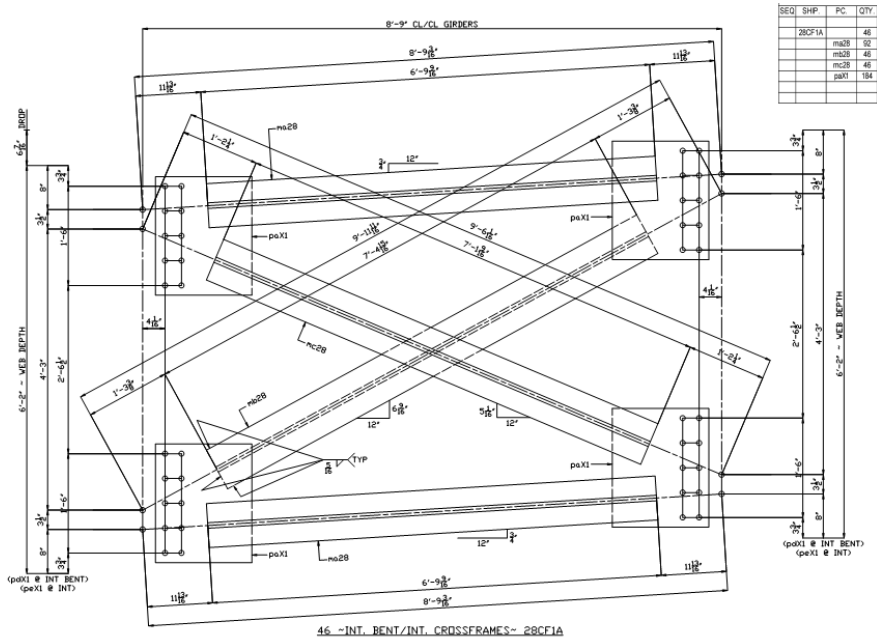


Figure C2.2.6.4-4: Shop drawing of a cross frame where the gusset plates have non-clipped corners

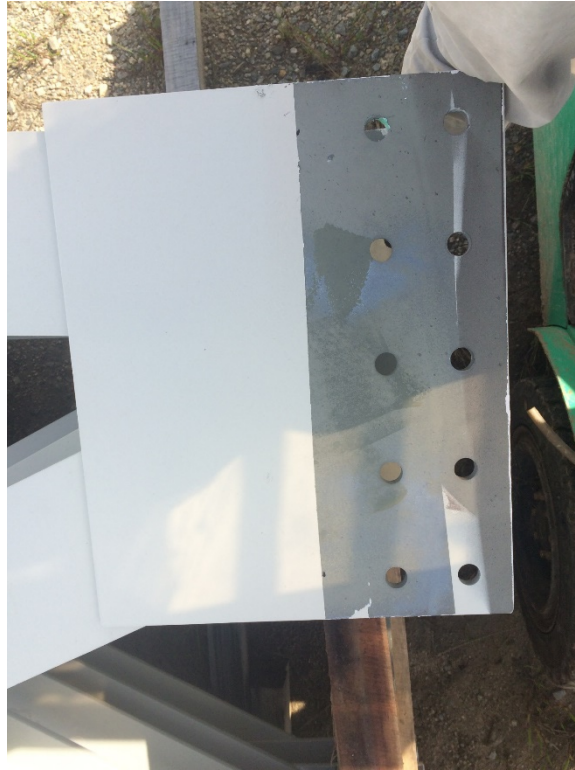


Figure C2.2.6.4-5: Photo of a cross frame where the gusset plates have non-clipped corners.



Figure C2.2.6.4-6: Photo of a cross frame where the gusset plates have non-clipped corners.

2.2.6.5 Cross Frame Member End Connections

A welded connection is the preferred connection to attach the end of a cross frame member to a gusset plate.

2.2.6.6 Reducing Demand on Cross Frames in Straight I-Girder Bridges

There are many ways to improve economy of cross frames in bridges with straight steel I-girders. Depending on the situation, these techniques can significantly reduce loads in cross frames and/or allow for smaller member sizes or fewer cross frames or cross frame members. Select techniques are listed here, including the use of lean-on bracing concepts and suggestions for economical staggered framing patterns.

In moderately- to severely-skewed bridges, significant transverse stiffness in the structural steel framing can lead to high cross frame forces. The final research report for National Cooperative Highway Research Program (NCHRP) Research Project 20-07

C2.2.6.5

A welded end connection is often preferred by Fabricators, as it is typically more cost effective than a bolted end connection.

The welds used to attach a cross frame member to a gusset plate are classified as a Category E' fatigue detail, as noted in Table 6.6.1.2.3-1 (Detail Category 7.2) in the *AASHTO LRFD Bridge Design Specifications*. Therefore the design of these connections and the members must consider the fatigue stress range in each cross frame member.

In some cases, usually when fatigue stress ranges in the welded connection are quite large, the connection of the member to the gusset plate is bolted instead of welded. Due to it being a much more expensive connection to fabricate, this should only be done when necessary and with consideration of the added cost.

However, for end cross frames only, where fit-up may be a significant concern, a bolted connection member end connection may be more desirable. The bolted end connection will allow for greater adjustment in the field during steel erection (e.g., reaming of holes within allowable dimensions).

C2.2.6.6

See Article 4.4 of the final research report for NCHRP Research Project 20-07 Task 355 (White, et. al.). The recommendations in this research report are also summarized and presented in *Skewed and Curved Steel I-Girder*

Task 355 includes many suggestions for reducing the stiffness of these transverse load paths, leading to greatly reduced cross frame forces, and fewer, lighter cross frames, without significantly compromising the strength of the girders.

Utilizing lean-on bracing concepts allows several girders to be braced across the width of the bridge by a single cross frame. Girders that lean on the cross frame brace require top and bottom struts to control girder twist (Helwig, et. al, 2015). Figure 2.2.6.6-1 depicts a lean-on bracing system, where in a given bracing line, 4 girders can lean on a single cross frame brace.

In order to develop a lean-on bracing system configuration, the erection sequence of the steel girders must be determined. The Designer is strongly encouraged to consult with local contractors and Erectors to form a solid assumption about how the bridge will be built. This ensures that the appropriate number of cross frames is placed between the first girder segments lifted, which is usually the most critical stage of girder erection.

Bridge Fit (NSBA, 2016), which can be found on the NSBA website.

A key component of a steel I-girder bridge is the bracing system. Braces at intermediate locations along the girder length provide overall stability of the girders and increase the stiffness and strength of the system during construction and in service. Intermediate braces usually consist of cross frames or diaphragms.

Cross frames represent a costly structural component, are often difficult to install due to fit-up problems, and also can attract significant live load forces which lead to fatigue problems. Minimizing the number of cross frames on the bridge can lead to better overall bridge behavior as well as reduced fabrication, erection, and maintenance costs (Helwig, et. al, 2015). The lean-on bracing concept minimizes the number of cross frames and improves the structural efficiency of the system. However, note that lean-on bracing cannot be used for curved girder bridges, where all the cross frames need to be able to carry shear, and therefore, need diagonals.

For guidance on developing a lean-on bracing framing configuration and understanding the distribution of forces across the bridge in a cross frame system with lean-on bracing, refer to the 2012 FHWA *Steel Bridge Design Handbook; Volume 13, Bracing System Design*.

The total stiffness of the lean-on bracing system is a function of the cross frame stiffness, the cross-sectional stiffness, and the in-plane stiffness of the girder. The stiffness at each line of bracing across the width of the bridge should be checked. Formulations and design assumptions from TxDOT research project 0-1772 (Helwig, et. al., 2003) can be used to maintain the general torsional bracing requirements for I-girders.

REFERENCES:

Helwig, T.A., Wang, L. *Cross Frame and Diaphragm Behavior for Steel Bridges with Skewed Supports*, TxDOT Research Report 0-1772-1, University of Texas at Austin, 2003.

Helwig, T.A., Yura, J.A. *Steel Bridge Design Handbook: Bracing System Design*, Publication No. FHWA-HIF-16-002, Vol. 13, Federal Highway Administration, Washington, D.C., 2015.

NCHRP. *National Cooperative Highway Research Report 20-07/Task 355: Guidelines for Reliable Fit-Up of Steel I-Girder Bridges, Final Report*. National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 2015.

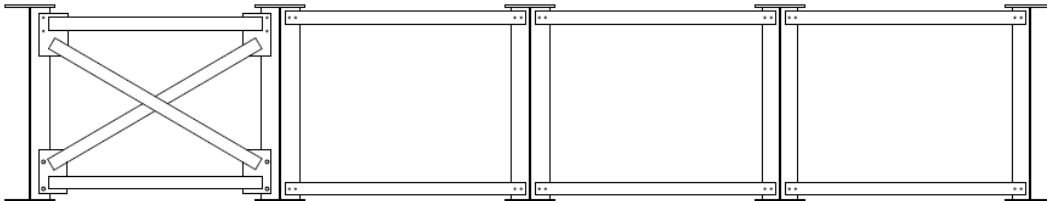


Figure 2.2.6.6-1 Lean-On Cross Frame Bracing

2.2.7 Ends of Girders

Girder end cuts should provide sufficient clearance to the back wall and expansion joint.

For bridges with severely skewed end supports, clipping of the flanges parallel to the skew angle can alleviate interference with the back wall.

2.3—LONGITUDINAL FIELD WEB SPLICES IN DEEP GIRDERS

For longitudinal field-bolted web splices in girders too deep to ship, use sub-flanges in the web splice design.

C2.2.7

Girder end cuts may need to be vertical if there is insufficient clearance to the back wall during construction due to thermal displacements, camber rotation, etc. The general consensus is that the effect on the design is minimal.

C2.3

Where deep girders are required, their depth may preclude shipping them in one piece. Longitudinal field-welded or field-bolted web splices are then required. Two possibilities for design of field-bolted splice include:

- Using a sub-flange on the top of the bottom section and on the bottom of the top section.
- Using conventional side plates, similar to a typical web splice (See Figure C2.3-1).

Both options are viable, but sub-flanges provide a stiffer section for shipping and are easier to fit up in the field. Another approach is to provide both details in the design and allow the contractor to pick his preferred option.

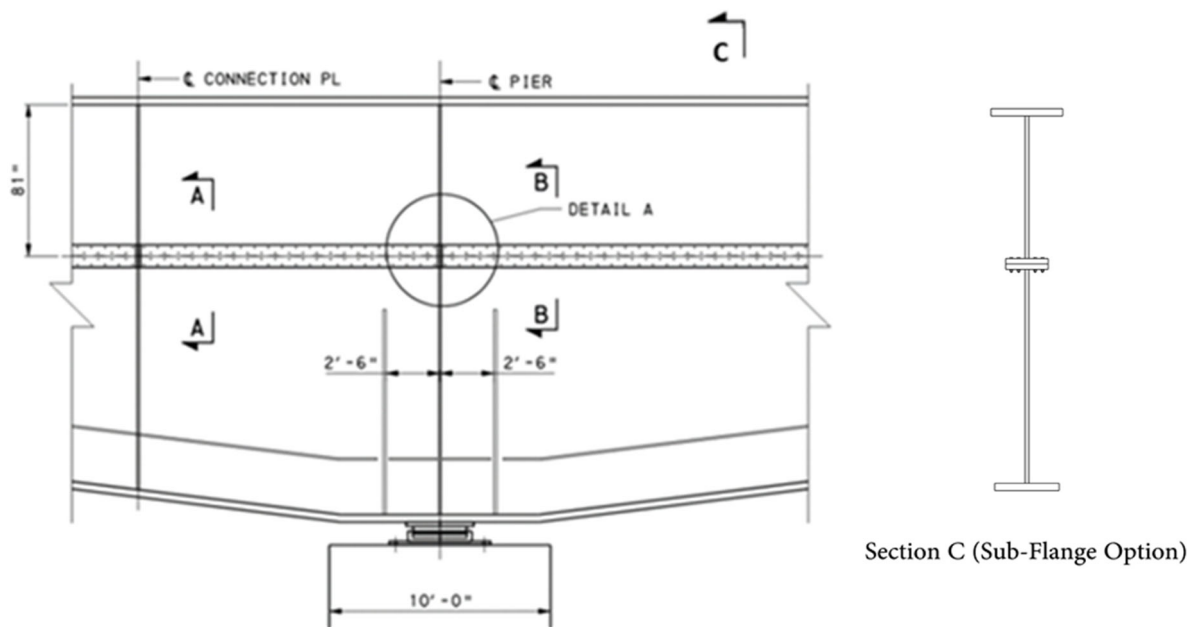


Figure C2.3-1 Field-Bolted Web Splice

2.4—BOLTED COMPRESSION JOINTS IN ARCH MEMBERS AND CHORDS OF TRUSSES

Design compression joints in arch members, truss chords, and other such members with an open joint at the splice to transmit the entire design force at the point of the splice through the bolted connection.

C2.4

AASHTO LRFD Bridge Design Specifications Article 6.13.6.1.2 allows a compression joint in arches and similar members to be designed to transmit the design forces at the splice either entirely through a bolted connection or by a combination of a mill-to-bear condition and high strength bolts sharing the load. The milled end is allowed to carry no more than 50 percent of the required force. Although the use of a connection relying entirely on the bolts to carry the design forces requires additional bolts at the splice, this design approach is considered less expensive in total due to the elimination of the mill-to-bear requirements for rib ends and presents a lower potential for problems in the field due to lack-of-fit conditions.

SECTION 3 BOXES

3.1 CLOSED BOX CONFIGURATION

Closed box corner configurations may be as shown in Figure 3.1-1, Figure 3.1-2, or Figure 3.1-3, depending on the access provided to work inside the box, design horizontal shear and transverse loads on the weld, and horizontal curvature. See Article 3.2 for welding considerations.

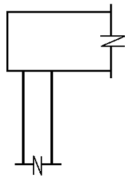


Figure 3.1-1

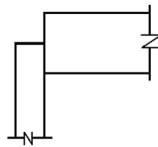


Figure 3.1-2

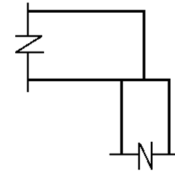


Figure 3.1-3

C3.1

Two configurations of plates for fillet-welded closed boxes are typical: terminating the web at the inside face of the flanges as shown in Figure 3.1-1 and Figure 3.1-3, and lapping the web on the edge of the flanges as shown in Figure 3.1-2. Terminating the web at the inside face of the flanges normally provides a straighter final product.

3.2 CLOSED BOX CORNER WELDS

Large Boxes—Large enough that a person can safely work inside them:

- Double fillet welds at both webs for one flange and partial joint penetration welds (allowed by AASHTO/AWS D1.5M/D1.5) for the second flange. This is a good detail for fabrication but should be evaluated by the Designer for torsion requirements, considering the number and attachment of internal diaphragms. In many cases, this configuration may be appropriate and is the preferred practice (Figure 3.2-1).
- Double fillet welds at both webs for one flange and complete joint penetration welds for the second flange. This is an expensive procedure that generally involves using backing bars that will remain in place (Figure 3.2-2).
- Double fillet welds at each of the four corners, requiring welding inside the closed box (Figure 3.2-3).
- Single fillet welds at each of the four corners. This may be appropriate for some boxes depending upon load conditions and internal diaphragms (Figure 3.2-4).

C3.2

There are several welding possibilities for welding boxes. The size of the box and its application significantly affect choice: for example, safety issues are a serious consideration if work is required inside a closed box.

If complete joint penetration welds (CJP) are required, preparation should be on the thinner plate.

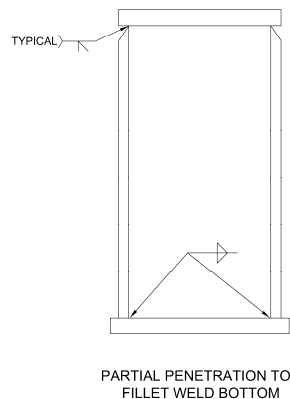


Figure 3.2-1

Small Boxes—Too small for a person to work safely inside:

- Single fillet welds at each of the four corners. This is the preferred practice. The Designer should investigate from a torsion perspective with due regard to the number of internal diaphragms and other applicable considerations. This is the best procedure for truss members (Figure 3.2-4).
- Double fillet welds at one flange and partial joint penetration welds for the second flange. This is a good detail for fabrication but should be evaluated by the Designer for torsion requirements including the number and attachment of internal diaphragms. In many cases this configuration may be appropriate (Figure 3.2-1).
- Double fillet welds at one of the flanges and complete joint penetration welds for the second flange. This is an expensive procedure and generally involves leaving backing bars in place (Figure 3.2-2).

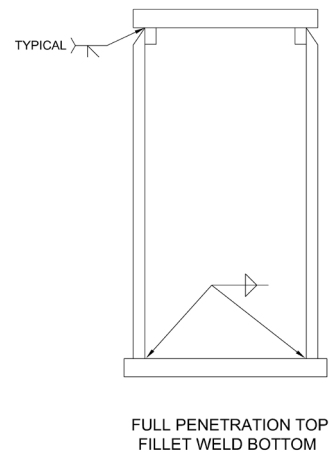


Figure 3.2-2

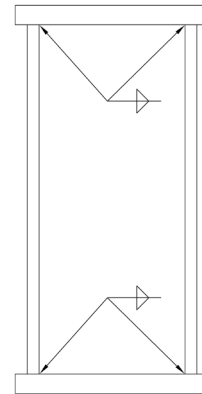


Figure 3.2-3

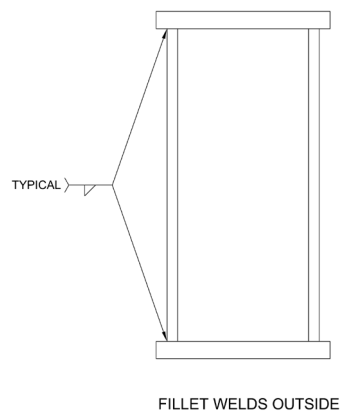


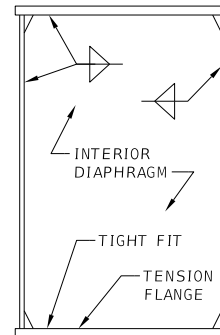
Figure 3.2-4

3.3 CLOSED BOX INTERIOR DIAPHRAGM ATTACHMENT

Weld three sides with an optional tight-fit to the tension flange.

C3.3

Attachment to the last flange installed may not be structurally necessary and can be difficult or impossible to accomplish. Most Fabricators prefer to weld only three sides. Tight fit has been historically used at the tension flange and might not be needed. If tight-fit is needed (e.g., for distortional considerations under live/thermal loading), a bolted angle may be specified in the design, to provide positive connection if the flange cannot be practically drawn down into full contact after fabrication activities & stacked tolerances. This adds worker exposure within the box and fabrication cost. See Figure C3.3-1.



CLOSED BOX INTERIOR DIAPHRAGM

Figure C3.3-1

3.4 BEARING DIAPHRAGMS IN TUB GIRDERS

For box girder bridges, place diaphragms normal to the bottom flange.

C3.4

Normal diaphragms are more economical and easier to fabricate than vertical diaphragms.

If a vertical diaphragm is specified, the diaphragm and fill plates will need to be beveled in up to two directions in order to fit the diaphragm to the top and bottom flanges of the box girder.

See Figure C3.4-1 below for an example of a typical detail for a box girder bearing diaphragm.

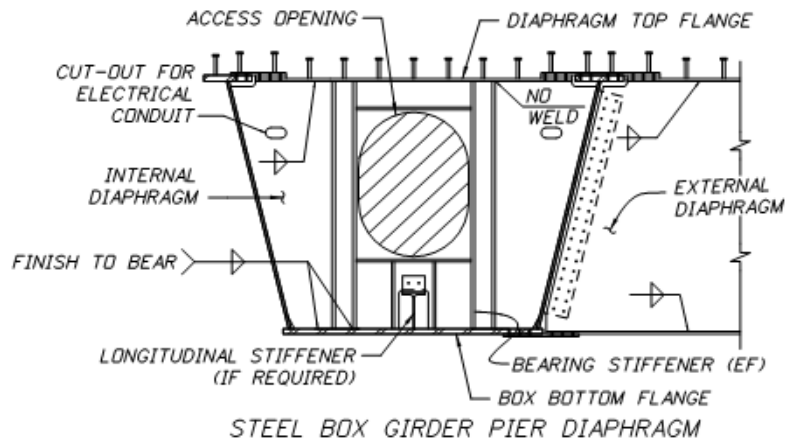


Figure C3.4-1

3.5 CLOSED BOX DIAPHRAGM MINIMUM ACCESS HOLE SIZE

Recommended size is 32 in. × 36 in. unless geometry dictates a smaller size. In which case the minimum size could be 18 in. × 24 in.

3.6 STIFFENER DETAIL NEAR BOTTOM FLANGE OF TUB GIRDERS

Acceptable details at the end of stiffeners near the bottom flange of box girders to allow for the welding of the bottom flange to web are shown in Figures 3.6-1 or 3.6-2, both of which are preferred. If a higher fatigue resistance connection is required for the tension flange, use Figure 3.6-3, when cost is justified. Figure 3.6-4 is preferred when the Fabricator welds the bottom flange to the webs prior to attaching the stiffeners.

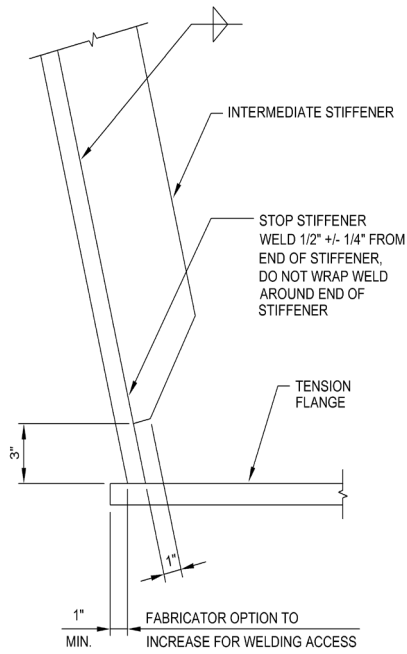
C3.5

The larger size of 32 in. × 36 in. is strongly encouraged for rescue purposes in case of an emergency during fabrication, erection, or future inspection and maintenance activities. Access openings at both ends should be shown on the design.

C3.6

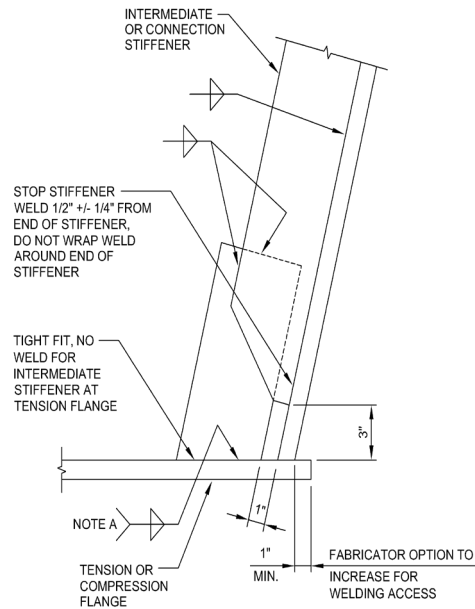
Typically, webs are joined to top flanges and transverse stiffeners installed, and then these assemblies are attached to the common bottom flange. In order to weld the web to the bottom flange continuously inside the box, details must allow the welding head to clear the bottom of the stiffener unless the Fabricator prefers to run the stiffener to the flanges.

See discussion on connection attachment to tension flanges in Article 2.1.2.2.



INTERMEDIATE STIFFENER DETAIL

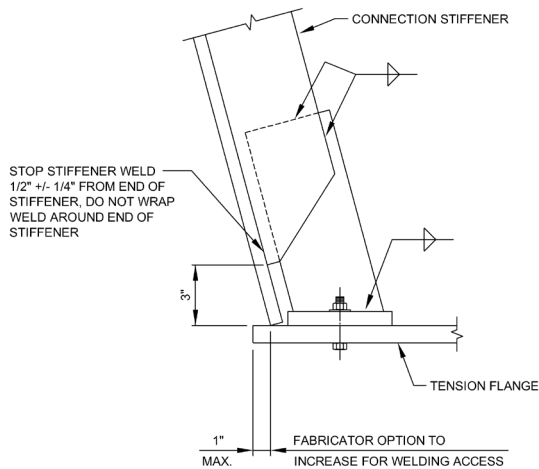
Figure 3.6-1



NOTE A:
FOR CONNECTION STIFFENER AT TENSION OR COMPRESSION FLANGE.
FOR INTERMEDIATE STIFFENER AT COMPRESSION FLANGE.

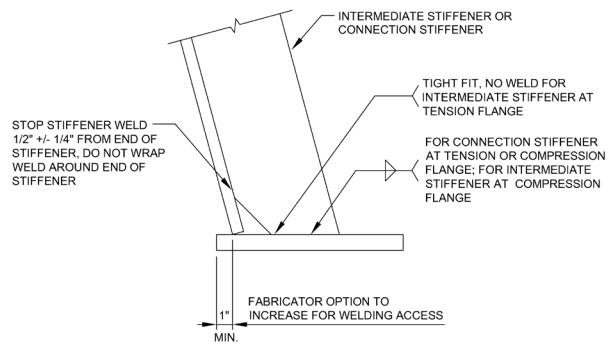
INTERMEDIATE OR CONNECTION STIFFENER DETAIL

Figure 3.6-2



ALTERNATE CONNECTION STIFFENER DETAIL AT TENSION FLANGE

Figure 3.6-3



INTERMEDIATE STIFFENER OR CONNECTION STIFFENER DETAIL

Figure 3.6-4

3.7 LONGITUDINAL STIFFENERS FOR BOTTOM FLANGE OF TUB GIRDERS

If design analysis shows that longitudinal flange stiffeners are more economical than thickening the bottom flange, use WTs.

C3.7

Fabricators prefer designs with bottom flanges that are unstiffened. If bottom flanges are stiffened, Fabricators prefer WTs to bars. If WTs are used, consider welding access when selecting the stem height.

Whether to stiffen the bottom compression flange and what type of stiffener to use are decisions that directly affect cost. If the inside of the box is to be painted and if the stiffening members are WTs, cleaning and painting on the underside of the WTs may affect the cost/benefit ratio. In addition, addressing the WTs at field splices and treatment at the bearing diaphragms complicates fabrication and field assembly. Splice plates can adequately stiffen the flange locally, so consideration can be given to interrupting the longitudinal stiffener at the field splice plates to avoid conflicts with the flange splice plates and bolts.

In most cases, the magnitude of the compressive stress in the bottom flange is low enough in the vicinity of field splices that the longitudinal stiffener may no longer be required. If the Designer wishes to avoid conflicts with the flange splice plates and bolts, terminate the longitudinal stiffener prior to the field splice plates. If the stiffener is terminated prior to the bottom flange splice plates, and the bottom flange is subject to tension or stress reversal, be aware of the poor fatigue category for the stiffener termination. A transition radius and weld termination grinding may be required to improve the fatigue category. Alternately, if the stiffener is terminated at the end of the bottom flange plate (i.e., at the centerline of the field splice), the stiffener termination itself has no fatigue implications, since the stress in the bottom flange and the stiffener have theoretically been fully transferred to the splice plates by that point; note that under this option the bottom flange inside splice plates should be split to accommodate the stiffener.

3.8 COATING THE INTERIOR OF CLOSED BOXES AND TUB GIRDERS

For typical tub girders or closed box girders, coating the interior for corrosion protection is not needed. If a light color is required for future inspection, specify a single coat of surface-tolerant light-colored paint (e.g., epoxy) with SSPC SP6 blast cleaning. Allow the Fabricator to blast and pre-coat components (e.g., top flange, web, and stiffened bottom flange of a tub girder) before final assembly so only the weld areas need to be prepared and spot-painted inside the box or tub girder. If the inside paint does not meet required slip/creep requirements for faying surfaces, mask them and leave them unpainted.

3.9 EXTERNAL CROSS FRAMES FOR MULTIPLE BOX AND TUB GIRDERS

Permanent cross frames or solid plate diaphragms between boxes and tub girders should be provided at supports. If multiple straight boxes or tub girders are adequately braced internally, external intermediate cross frames may not be required. For curved multiple box or tub girders that require cross frames between members, it is preferable to use permanent cross frames. If temporary cross frames are utilized, use temporary connections (e.g., bolt to webs instead of using welded connection plates) and leave them unpainted.

C3.8

The inside of a box girder is typically not highly susceptible to corrosion in service. Exposure to the elements is minimal in closed boxes and in tubs after the deck is poured, and adequate drainage and ventilation will prevent buildup of water inside the girder. Proper detailing of ventilation openings in the box will allow any condensation to evaporate, so painting requirements should be based on aiding inspection. The Owner may wish to consider a slip-resistant coating on the interior side of the bottom flange for traction.

C3.9

Cross frames are primarily used in bridge construction to prevent lateral-torsional buckling of the bridge girders by providing additional torsional stiffness to individual girders. For straight boxes and tub girders with sufficient internal diaphragms, the girders are torsionally stiff and thus do not require permanent cross frames except at supports. For curved boxes and tub girders, cross frames assist in load transfer between the girders. If curved bridges are individually horizontally braced and erected using sufficient falsework to prevent torsional bending until the deck is placed, then external cross frames between boxes or tubs may not be required except at supports. When temporary cross frames are utilized, painting is unnecessary because they will be removed after the deck has been poured. However, if the bridge is re-decked in the future then the process must be followed in reverse and, thus, leaving permanent cross frames may be preferred. Also, consider that external cross frames are much easier to install than they are to remove, because there is no overhead crane availability once the deck is cast.

Another consideration is that some Owners require the use of a higher redundancy load factor when external intermediate diaphragms are not provided for box girder bridges.

Texas completed a research study and provided design guidelines to check for the necessity of external cross frames in box girders (Helwig, et. al., 2007).

REFERENCES:

Helwig, Todd; Yura, Joseph; Herman, Reagan; Williamson, Eric; and Li, Dawei. *Design Guidelines for Steel Trapezoidal Box Girder Systems*. TxDOT Research Report No. 0-4307-1, University of Texas at Austin, April 2007.

SECTION 4 BOLTS

4.1 HIGH-STRENGTH BOLTS

ASTM F3125 is the specification for high-strength structural bolts. Grade A325 and A490 bolts (and their twist-off versions, F1852 and F2280), are viable options in structural steel joints. However, the use of A325/F1852 bolts is generally more prevalent in the industry.

It is preferred practice to not mix A325 and A490 type bolts in the same connection type or in the same unit of a bridge. If A325 and A490 bolts are both warranted, it is recommended that different bolt diameters be used to distinguish between the bolt grades.

C4.1

The Specification for Structural Joints Using High-Strength Bolts, published by the Research Council on Structural Connections (RCSC), addresses the design of bolted joints and the installation and inspection of the bolted assemblies. The Specification principally addresses A325 and A490 bolts and their twist-off versions, F1852 and F2280 bolts.

Additional industry references are available and discuss industry preferred practices, such as TxDOT's *Preferred Practices for Steel Bridge Design, Fabrication, and Erection* and AISC *Steel Construction Manual* Tables for Entering and Tightening Clearance.

A325 and A490 bolts are available in two types, denoting chemical composition: Type 1 and Type 3. For A325 bolts, Type 1 is a medium carbon, carbon boron, or medium carbon alloy steel, and Type 3 is weathering steel. For A490 bolts, Type 1 is a medium carbon alloy steel, and Type 3 is weathering steel. A325 Type 1 bolts can be galvanized. It is uncommon to galvanize A325 Type 3 bolts, but they also can be galvanized. A490 bolts cannot be galvanized, but in recent years alternative coatings have been developed; these are listed in ASTM F3125.

The Designer must make an informed decision when choosing which bolt grade to use. There are subtleties in the design and installation of each grade and type of bolt, including the number of times the bolt can be reused and their pre-tensioned loads.

Because the bolts look the same except for the markings on their heads, the Designer is discouraged from mixing bolt grades in the same type of connection or in the same unit on a bridge. It is preferred that one bolt grade be specified. However, if both grades are warranted by design, it is preferred practice to change the diameter of the bolt between bolt grades. This will aid the Erectors and

inspectors in distinguishing between the two in the field.

It is usually the contractor's preference to use A325 bolts. This is because, for fit up bolts, ungalvanized A325 bolts can be retightened, but A490 bolts must be replaced. Contractors prefer loosening and retightening to replacement. Also, A490 bolts are much more sensitive to tightening procedures. Switching bolt grade or size at a common field splice location for adjacent girders would require re-verification of the tightening method and, if calibrated wrenches are used, either recalibration of the tightening equipment in the middle of the erection process, or the use of separate wrenches calibrated for each bolt grade and size.

The Designer should also consider the tightening method and ensure sufficient access is provided for workers, tools and inspection. Access at the end diaphragms of skewed bridges can be problematic.

4.2 MECHANICAL OR HOT-DIPPED GALVANIZED BOLTS

Where galvanized fasteners are required, either hot-dipped or mechanically galvanized bolts can be used. However, mechanically galvanized bolts are preferred.

4.3 BLACK VERSUS GALVANIZED SHOP-INSTALLED BOLTS

Use mechanically galvanized bolts for connections in structures that will be coated, both for areas that will later be blasted and primed and also for previously primed areas.

C4.2

Hot-dipped galvanized bolts may give better corrosion protection, but mechanically galvanized bolts are often considered to have more consistent corrosion protection and fewer installation problems; hot-dipped galvanized bolts are more likely to fail rotational capacity testing. Note that galvanizing of A490 bolts is not allowed, although ASTM F3125 does list some alternative coatings available for these fasteners.

C4.3

Black bolts need to have oil removed before blasting. Also, bolts are often installed in situations where some parts of the bolt or nut may be shielded during blasting, resulting in an inadequate anchor profile. Blasting of galvanized bolts does not remove all of the galvanizing, but the prime coat will adhere to any remaining galvanized surface. Consider the use of partially lubricated nuts to decrease cleaning time and use of solvents prior to

coating. ASTM A563 Supplement 3 gives an option for lubricant placement on the bearing surface and internal threads.

See further discussion in AASHTO/NSBA Steel Bridge Collaboration document S8.1, *Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges*.

Most states have no specific requirement, assuming that Fabricators will use black bolts. Some require galvanized bolts for painted structures, especially for field connections in new steel. Field blasting of black bolts installed in shop-primed structures is likely to cause damage to adjacent primed areas. This damage may go undetected and unrepaired, shortening the life of the coating system. The use of galvanized fasteners eliminates this potential.

4.4 BOLTED FAYING SURFACES

Design plans shall specify the class of slip-critical connections as Class A, B, C, or D where:

- Class A Surface: unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings,
- Class B Surface: unpainted blast-cleaned surfaces to SSPC-SP 6 or better, and blast-cleaned surfaces with Class B coatings, or unsealed pure zinc or 85/15 zinc/aluminum thermal-sprayed coatings with a thickness less than or equal to 16 mils,
- Class C Surface: hot-dip galvanized surfaces, and
- Class D Surface: blast-cleaned surfaces with Class D coatings.

Also see Article 3.8 regarding faying surfaces inside closed boxes and tub girders.

C4.4

Slip-critical connections transfer load through friction between faying surfaces developed through the tension in the bolts clamping them together. In design, a certain surface condition factor, or slip coefficient, is assumed based on faying surface classification. Faying surface classification with higher slip coefficients may result in more economical connection designs (i.e., fewer bolts). The slip coefficient is determined by test using the RCSC *Specification for Structural Joints Using High-Strength Bolts*—Appendix A. The test regime includes a short-term test to attain the slip coefficient and a long-term creep test to ensure the surface will not compress and decrease the bolt tension. The coating type and coating thickness are considered essential variables in the test.

Certain surface types have well-established slip coefficients such as clean unpainted mill scale, unpainted blast-cleaned steel, certain unsealed thermal spray coatings, and hot-dip galvanizing. Approved coatings may be applied over blast-cleaned steel provided the coating manufacturer certifies the coating passes the RCSC test criteria for the slip classification. During the shop drawing review, the Designer verifies that the surface

preparation plus the coating type and thickness are appropriate. The submittal of the coating manufacturer's product data sheet during the shop drawing submittal may further demonstrate or confirm the coating has been tested and meets the minimum slip coefficient for the requested type of faying surface.

Historically, coatings on the faying surface have either been classified as Class A (slip coefficient=0.30) or B (slip coefficient=0.50). In 2016, a Class D (slip coefficient=0.45) was added to account for a propensity of organic zinc-rich primers that tended to just fail Class B during the RCSC test regime. Thermal-spray coatings can theoretically be applied to any thickness, although little to no slip performance data has been generated on coating thicknesses over 16 mils; hence, the restriction on coating thickness. Thermal-spray surfaces are often sealed with low-viscosity epoxy coatings, e.g. penetrating sealers, to fill the pores of the metalizing and enhance their corrosion resistance. However, tests of some sealed thermal-sprayed coatings produced very low slip coefficients, or failed the creep requirement, and therefore are not included unless proven by test.

SECTION 5 CORROSION PROTECTION

5.1 CORROSION PROTECTION SYSTEMS

Unpainted weathering steel is an inexpensive, low-maintenance corrosion-protection solution that should be considered as long as the bridge location meets the Federal Highway Administration (FHWA) *Guidelines for Uncoated Weathering Steel in Structures*. Integral unpainted weathering steel bridges require no painting. Visible surfaces of the bridge (typically fascia girders) should be blast cleaned to improve the aesthetic quality of the patina.

If painting is required for aesthetics, paint only particularly visible elements such as fascia girders.

If the environment is not suited for uncoated weathering steel, use non-weathering steel with a two-coat system such as a zinc-rich primer and a field-applied polyurethane or acrylic topcoat; topcoat only visible surfaces.

For a discussion on coating the interior of closed boxes and tub girders, see Article 3.8.

C5.1

Unpainted weathering steel is a preferred solution; the incremental cost of weathering steel and detailing to prevent staining and ponding is initially offset by avoiding the cost of coating, and continues to be offset by far less required maintenance over the years.

Guidance related to proper use of weathering steel is provided in the *Steel Bridge Design Handbook, Volume 19*. Proper details should be used to eliminate or reduce the unsightly concrete staining from the use of weathering steel. Some design strategies include drip plates (to divert runoff water and protect abutments and columns from staining), stainless steel drip pans, and details that take advantage of natural drainage. See AASHTO/NSBA G1.4 for a recommended drip bar detail.

For situations where weathering steel cannot be used, one-layer inorganic zinc (IOZ) coatings have a history of excellent performance, better than traditional 3-coat IOZ/epoxy/polyurethane systems. A two-coat system with a “breathable” topcoat such as acrylic may allow the IOZ to approach its potential as a one-layer coating.

There is a mix of practice with regard to whether topcoats are applied in the shop or the field. If appropriate care is exercised to minimize the need for touchup of a shop-applied topcoat, applying the topcoat in the shop may allow for shorter painting time in the field (but will increase fabrication time because the topcoats must cure to the point that they can be handled and the primed faying surface must be masked). However, damage during construction to shop-applied topcoats, particularly acrylics, typically results in the need for extensive field touchup; a field-applied topcoat will only need to be addressed once. In addition, field-applying the topcoat over a two-coat IOZ system will let the IOZ cure for longer and will enhance its performance. With shop-applied topcoat

or intermediate coat, there is little performance difference between IOZ and organic (epoxy) zinc primers.

Maintenance costs of coating systems can be minimized if a one-year inspection is performed. Many coating problems will be much more visible after a year of service than immediately after application, and catching and addressing them early means that overall performance of the system in subsequent decades will be greatly enhanced.

SECTION 6 OTHER

6.1 CONTRACTUAL ITEMS

6.1.1 Lump Sum versus Unit Price Bids

Use either lump sum or unit cost pricing. When using lump sum pricing, provide a breakout of weight in the design. When using unit cost pricing, use pay weights or, if using plan weights, keep the weight calculations simple by following the *AISC Code of Standard Practice Guidelines* and thereby minimizing deductions for clips, bolt holes, and other miscellaneous cut outs.

6.1.2 Partial Payment for Materials and Fabrication

Payment for mill material and typical fabricated structural steel plate girders should be based on the following:

- 50 percent of the “Fabricate and Deliver” contract price for structural steel received, documented, and stored at the fabrication plant (weight not to exceed the calculated steel quantity for the project).
- 70 percent of the “Fabricate and Deliver” contract price for members completely fabricated and stored, ready for cleaning and painting.
- 90 percent of the “Fabricate and Deliver” contract price when all steel for the contract has been fabricated, cleaned, painted and stored at the fabrication plant or other approved location.
- 100 percent when erected.

C6.1.1

When using the AISC method, calculating plan weights isn’t difficult and can provide a pre-bid check for the estimator to use against their take-off. Further, the Fabricator can use this information to notify the Owner by Request for Information (RFI) if the number the Fabricator calculates is significantly different from the engineer’s estimate. However, calculating a precise project weight by deducting cut-outs is time consuming and adds unnecessary time and expense to the project because Fabricators typically calculate their cost per unit weight by totaling their cost and dividing by the project weight (and not by use of unit weight cost data).

C6.1.2

In March 2000, the FHWA issued a memorandum authorizing and encouraging states to make payment for mill material that has been received by the Fabricator, properly stored, and appropriately documented.

The cost of financing the storage of mill material and fabricated members at the shop is high. Job site delays can add significant additional costs that affect fabricated steel prices.

Cost for mill material should be determined by weight from a schedule of values agreed to between the contractor and the Owner, rather than by invoiced cost, as cost to the Fabricator from the mill is often confidential.

6.1.3 Contractual Bid Items for Fabrication, Erection, and Field Painting

Where delivery time is critical, consider using a separate “Fabricate and Deliver Job Site” contract. For normal bridge jobs, have separate bid items for Fabrication, Erection, and Field Painting.

C6.1.3

Most Fabricators prefer a separate contract for “Fabricate and Deliver Job Site.” This works well for fast track jobs and other special situations. Where this approach is not feasible, most Fabricators prefer separate bid items for fabrication, erection, and field painting.

When a separate “Fabricate and Deliver Job Site” contract is used, the Designer must ensure the construction methods assumed for design, for example deck pouring sequence and erection scheme, are reasonable, economical, and communicated in the project documents. Any changes to the construction methods may require re-design by the Contractor.