



Guidelines for Design for Constructibility

AASHTO/NSBA Steel Bridge Collaboration

Guidelines for Design for Constructibility

Preface

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

As consensus documents, the Collaboration standards represent the best available current approach to the processes they cover. It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization, but it is understood that local statutes or preferences may prevent full adoption for some. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

Disclaimer

All data, specifications, suggested practices presented herein, are based on the best available information and delineated in accordance with recognized professional engineering principles and practices, and are published for general information only. Procedures and products, suggested or discussed, should not be used without first securing competent advice respecting their suitability for any given application.

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Foreword

This effort started as a research project sponsored by the Education Foundation of the American Institute of Steel Construction under a grant by Stupp Bros. Bridge & Iron Co. Foundation. The primary focus of the study was to publish a document that addressed many of the questions that have been and are continually asked concerning the constructibility of bridges.

The concept was discussed at one of the Collaboration meetings and the collaboration members decided to participate in the effort. Task Group 12 was established to take on the project.

The Task Group developed two separate questionnaires relating to constructibility issues; one from the state or designers perspective and one from the fabricator, detailer and erector view point. The questionnaires were published on the Collaboration's web site and were also mailed to 22 States and 16 Fabricators and Detailers. Responses were received from 12 States and 8 Fabricator/Detailers.

The results of the survey were then summarized and reviewed by the Task Group, recommendations agreed upon and the document prepared and balloted.

Many of the recommended details are from the FHWA Mid-Atlantic States Region, SCEF Standard Details, while others came from Collaboration Standards under preparation and individual state standards that were returned with the original questionnaire.

The document has been prepared as a guide and thus much of the information is general in nature, representing a consensus of various state positions as well as various fabricator positions. Recommendations should not be considered as hard and fast rules to be followed by any contracting engineer, authority, fabricator, and/or contractor.

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Section 1 Design

1.1 Rolled Beams Versus Welded Plate Girders

Issue:

What considerations should be evaluated when choosing between the use of rolled beams or welded plate girders?

Recommendation:

Where choice is possible between rolled beams and welded plate girders for a structure, specify rolled beams and ensure the selected sections are available.

Include an optional welded plate girder in the design if the structure has a radius less than 1200 feet, if span lengths exceed the capacity of Group 3 rolled sections, or if the camber is excessive.

Allow the fabricator to substitute plate girders that satisfy design requirements for rolled beams.

Commentary:

For typical situations, fabricators usually prefer rolled beams. States generally consider rolled beams more economical than welded girders and may consider alternatives if delivery or specific requirements (camber, curvature, length between splices) are a problem.

Consider the following factors when either rolled beams or welded plate girders could be designed for a structure:

- For curved structures and structures requiring severe camber, consider using welded girders.
- Fatigue criteria essentially limit cover plating to in-kind replacements.
- The *AISC Manual of Steel Construction, LRFD, 3rd Edition* Section 16, A3.1c and associated commentary provides guidance on the use of Group 4 and 5 rolled shapes in tension and flexure.
- Fabricators that do not routinely heat-curve typically consider 1200 feet as a minimum radius for rolled beams.
- Availability varies among the standard shapes.
- When delivery of rolled shapes is a problem, fabricators like having the option to substitute an equivalent welded girder.

1.2 Girder Spacing

Issue:

What considerations need to be evaluated to when choosing the number of girders in the bridge cross-section?

Recommendation:

Consider the following:

- Owner preferences and limitations
- Cost of steel fabrication and erection
- Vertical clearances

Commentary:

Studies by HDR Engineering, University of Missouri, and NSBA have shown that for spans up to 140 feet long, 10-foot to 11-foot girder spacing is more economical, and for spans over 140 feet in length, spacing of 11 to 14 feet is more economical. Comparisons between 9 foot and 12 foot girder spacing designs on two-span bridges by HDR show that girder weights were lighter in the more widely spaced layouts. Similar work done at the University of Missouri and by NSBA shows that the use of fewer girders in the bridge cross section resulted in more cost-effective designs.

Many states have rules for girder spacing. When surveyed, states supplied the following variety of limitations:

- no maximum limit, allowing spacing to be a question of design and economy (three states)
- 10 foot max spacing, and use deck design as the determining factor (three states)
- 8 foot max spacing
- 9 foot max spacing when precast deck forms are used
- 12 foot max spacing, and design based on future decking

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and 11-foot lanes

- 15 foot max spacing, and a minimum of four girders
- minimum of six girders if stage construction is necessary when redecking
- use a minimum number of girders but avoid large flanges and thick webs
- if future redecking or phased construction is planned, use a minimum of three girders to ensure lateral stability
- design for future overloads

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 crossframes 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:

- Additional concrete and reinforcing steel in the deck
- Methods for forming the deck
- Stability and redundancy of structure during future redecking
- Weight of individual girder pieces

Girder depth limitations based on vertical clearance demands may restrict optimizing the number of girders. Further, it is usually not economical to increase vertical clearance by raising the bridge profile when approaches must also be raised.

1.3 Minimum Thickness for Stiffeners, Webs, and Flanges

Issue:

For welded girder construction, certain minimum requirements for material thicknesses are normally recommended to reduce deformation and the potential for weld defects.

Recommendation:

Thickness for stiffeners, connection plates, and webs; $\frac{7}{16}$ inch minimum, $\frac{1}{2}$ inch preferred. For flanges, $\frac{3}{4}$ inch minimum thickness.

Commentary:

Preferred minimum thicknesses depend on the welding equipment used.

See **Table 1.3.A** for survey preferences.

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Table 1.3.A
Minimum Thickness for Stiffeners, Webs, and Flanges

Description	Fabricators	States
Stiffeners and Connection Plates	$\frac{7}{16}$ inch min., $\frac{1}{2}$ inch preferred	$\frac{3}{8}$, $\frac{7}{16}$, or $\frac{1}{2}$ inch
Girder Webs	$\frac{7}{16}$ inch min., $\frac{1}{2}$ inch preferred	$\frac{3}{8}$, $\frac{7}{16}$, or $\frac{1}{2}$ inch
Flanges	$\frac{3}{4}$ inch min.	$\frac{3}{4}$ inch

1.4 Material Size Availability

1.4.1 Plate Material Size Availability

Issue:

When sizing girder flanges, what are the maximum lengths available for the various plate widths and thicknesses?

Commentary:

The availability of material sizes varies from mill to mill. The minimum width available from one mill is 48 inches and from two others is 60 inches.

Recommendation:

For the design, select material that is readily available. Table 1.4.1.A and Table 1.4.1.B show one mill's dimensions of typically available plates, but contact a mill or fabricator for current plate availability information.

Plates are generally available in widths to 150 inches and one mill has a limit of 190 inches.

Table 1.4.1.A
Example Maximum Plate Length Availability
ASTM A709 Grades 36, 50, 50W (all dimensions in inches)

Plate Thickness	Plate width								
	48	60	72	84	96	108	120	132	144
$\frac{1}{2}$	1440	1500	1525	1525	1500	1350	1250	1100	850
$\frac{3}{4}$	1440	1500	1525	1525	1371	1219	1097	997	900
1	1440	1500	1371	1175	1028	914	823	748	686
$1\frac{1}{2}$	1440	1440	914	784	686	609	549	499	457
2	1000	1144	686	588	514	457	411	374	343
$2\frac{1}{2}$	936	915	549	470	411	366	329	299	274
3	780	763	457	392	343	305	274	249	229
$3\frac{1}{2}$	669	654	392	336	294	261	235	214	196
4	585	572	343	294	257	229	206	187	171

Notes:

- Information was provided by Bethlehem Steel (11/1/01) and is intended to represent typical availability. Greater dimensions may be available from this and other mills. Contact a steel mill or fabricator for current information.
- The shaded areas represent certain materials that are available in a maximum length of 600 inches. These materials when CVN Testing is required are: FCM Gr. 50 and 50W, over 3" through 4" thick, all Zones; FCM Gr. 50, over 2" through 3" thick, Zone 3; non FCM Gr. 50, over 3" through 4" thick, Zones 2 and 3; and non FCM Gr. 50, over 3" through 4" thick Zone 3.
- Widths and thicknesses are grouped for convenience. Other widths and thicknesses available in similar lengths. Interpolate between adjacent values for other size plates.

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Table 1.4.1.B
Example Maximum Plate Length Availability
ASTM A709 Grades HPS 70W (all dimensions in inches)

Plate thickness	Plate width								
	48	60	72	84	96	108	120	132	144
½	1500	1500	1500	1500	1500	1500	1410	600	600
¾	1500	1500	1500	1520	1350	1200	1070	600	600
1	1500	1500	1360	1440	1250	1110	1000	600	600
1½	1500	1500	1270	1090	850	840	760	499	457
2	1240	1240	950	810	710	630	411	374	343
2½	600	600	549	470	411	366	329	299	274
3	600	600	457	392	343	305	274	249	229
3½	600	600	392	338	294	261	236	214	196
4	585	572	343	294	257	229	206	187	171

Notes:

- Information was provided by Bethlehem Steel (11/1/01) and is intended to represent typical availability. Greater dimensions may be available from this and other mills. Contact a steel mill, fabricator, or the NSBA for current information.
- Plate sizes shown in the shaded areas are quenched and tempered.
- Widths and thicknesses are grouped for convenience. Other widths and thicknesses available in similar lengths. Interpolate between adjacent values for other size plates.

1.4.2 Wide Flange Beam Length Availability

Issue:

What are the maximum lengths available for wide flange rolled beams?

Commentary:

The table shows one mill's length capacity.

Recommendation:

The maximum length available from one mill is shown in Table 1.4.2.A. Contact a fabricator or mill for additional information.

Table 1.4.2.A
Example Maximum Wide Flange Beam Length Availability

Prime Section	Footweight	Max Length (Feet)	Prime Section	Footweight	Max Length (Feet)
W40 x 16	431	80	W30 x 15	391	110
W40 x 16	397	110	W30 x 15	173 - 357	120
W40 x 16	199 - 372	120	W30 x 10-1/2	90 - 148	120
W40 x 12	149 - 327	120			
W36 x 16-1/2	393	110	W27 x 14	368	92
W36 x 16-1/2	230 - 359	120	W27 x 14	307 - 336	100
W36 x 12	135 - 256	120	W27 x 14	146 - 281	120
			W27 x 10	94 - 129	120
W33 x 15-3/4	387	110			
W33 x 15-3/4	201 - 354	120			
W33 x 11-1/2	118 - 169	120			

Note: Information provided by Nucor-Yamato Steel (December, 2002).

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1.5 Flange Sizing

1.5.1 Flange Plate Thickness

Issue:

When locating shop flange splices, what is the optimum number of splices per flange and what thickness increments are preferred?

Recommendation:

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.

Limit the number of different plate thicknesses for a project. Select flange thicknesses in at least $\frac{1}{8}$ -inch increments up to $2\frac{1}{2}$ inches and $\frac{1}{4}$ -inch increments over $2\frac{1}{2}$ inches.

At flange splices, the thinner plate should not be less than one-half the thickness of the thicker plate.

1.5.2 Shop-Welded Splices

Issue:

What fabrication considerations should you evaluate when determining whether to use a shop-welded flange thickness transition splice or extend the thicker plate?

Recommendation:

Specify a shop-welded splice when the savings in flange material and when plate length limitation or special circumstances dictate. Table 1.5.2.A provides a method to make the evaluation.

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

Commentary:

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 splicing with or without a thickness change. Availability of material sizes varies from mill to mill; see Table 1.4.1.A and Table 1.4.1.B for one mill's sizes. Minimizing the number of flange plate thicknesses for a project reduces mill quantity extras and simplifies fabrication and inspection operations. See Table 1.5.2.A for information on when thickness transitions are economically justified.

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 optimize with 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 $2\frac{1}{2}$ inch, consider reducing to four plate thicknesses of $1\frac{1}{4}$, $1\frac{1}{2}$, $1\frac{7}{8}$ and $2\frac{1}{2}$ inch.

Commentary:

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 eliminate a shop splice by 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 "rules of thumb" (e.g., Texas DOT estimates saving 800 to 1000 pounds may justify a butt splice). Table 1.5.2.A 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 the table:

Evaluate splicing a plate 16" x 1" x 35' to a plate 16" x $1\frac{1}{2}$ " x 35'

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versus using a plate 16" x 1½" x 70'. The weight saved by adding the splice is equivalent to the weight of a plate 16" x ½" x 35' (16" x 0.5" x 3.4 pounds/inch² x 35' = 952 pounds) 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 1.5.2.A, times the plate width of 16 inches, resulting in a value of 1,120 pounds. Because the actual saving is 950 pounds, Table 1.5.2.A indicates that it is more economical to extend the 1½ inch plate for the full 70 feet than to add the shop splice.

Table 1.5.2.A
Weight Saving Factor Per Inch of Plate Width
for ASTM A709-Gr 50 Non-Fracture Critical Flanges Requiring Zone 1 CVN Testing

Multiply weight savings/inch x 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

Source: compiled from various fabricators, November 2001.

Notes:

- 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%.
- For compression flanges where CVN testing is not required, the factors should be increased by about 10%, except the bottom two rows should increase by about 30%.
- For fracture critical material, the factors should be reduced by values between 10% and 25% 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.

1.5.3 Flange Plate Width

Issue:

What fabrication considerations should designers be aware of when sizing girder flanges?

Recommendation:

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

Commentary:

The most economical size plate to buy from a mill is between 72 and 96 inches wide. For size availability see 1.4. Fabricators order plate with additional width and length to account for cutting (? inch per cut between plates and along sides), plate sweep tolerance, and waste (about ½ inch on each outside edge). For example, a fabricator might order a plate 74 inches wide to cut five 14-inch-wide plates.

Straight Girders

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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 1.5.3.A.

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 1.5.3.B.
- 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 of on a plate to optimize material use.) See Figure 1.5.3.B.
- When in doubt, consult a fabricator.

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 as well as having considerably fewer weld starts and stops. Changes in thickness rather than width in a field section save as much as 35% 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 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 1.5.3.A as an example.

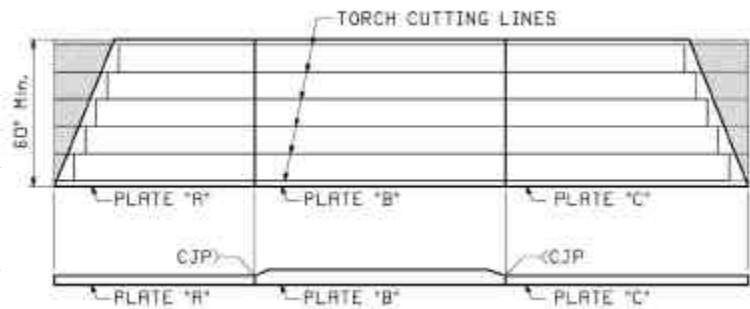


Figure 1.5.3.A

Curved Girders

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 1.5.3.B was 700 feet and the center plate was 2 inches thick by 60 feet 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

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consider how material might be ordered and spliced. See Figure 1.5.3.B as an example.

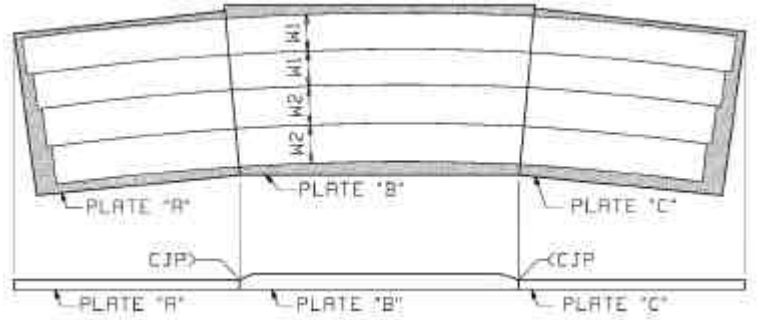


Figure 1.5.3.B

1.6 Differential Deflections

1.6.1 Deflections for Straight Structures on Skewed Piers and Abutments

Issue:

Girder deflections under dead load (girder self weight, deck weight) for skewed and curved bridges are not equal across the width of the bridge. For example, neglecting bracing effects on a simple span skewed bridge, the maximum point of deflection for each girder will be at midspan of each girder, but these points do not align across the width of the bridge. On curved bridges, the girder on the outside of the curve and the girder on the inside of the curve, have different deflections. The differential issue is further complicated if the bents are not radial. It may be difficult to install crossframes at locations with significant differential deflection. Once crossframes are installed in these situations, they may restrain deflections causing girders to rotate out of plumb and lateral stresses to be introduced into the flanges. How should the designer, fabricator, erector, and contractor address differential deflection for straight structures including slab placement issues on skewed piers and abutments, especially the final out-of-plumb condition of the girders?

Recommendation:

The performance of thousands of steel bridges has demonstrated that

Commentary:

Differential deflections on skewed bridges make it difficult to fit crossframes and result in transverse girder rotations and out-of-plumb girders. States vary in their requirements or expectations relative to the dead-load condition in which crossframes should fit without appreciable stresses: survey responses were almost equally divided among no-load, steel dead load, full dead load, and not specified. Regarding web verticality, two states indicated web vertical under full dead load, with no other responses on this topic. On the question of whether crossframes should be skewed or normal, one state preferred normal, five states set crossframes parallel up to skews of 30°, two states up to 20°, one state up to 15°, and one state up to 10°.

Fabricators also vary in their approach. Some fabricators preferred that crossframes be detailed to fit in the full dead-load condition, but others had no opinion except that the design should indicate the preferred method. On the web verticality question, several fabricators recommended slotted holes and tightening bolts after the deck is poured, acknowledging that this can be expensive. Others indicated that for significant differential deflections the designer should show whether webs are to be vertical after erection, after deck pour, or under some other condition. Most fabricators believe a standard should be established addressing this issue. On the issue of whether crossframes are normal or parallel to skewed substructures, most fabricators prefer that they be placed normal to the girder lines. Oversize holes are not a solution to the issues of differential deflections. Although oversize holes will help with fit up; serious alignment problems, rotations and additional lateral stresses will still result.

The following issues arise during design relative to

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differential deflections typically do not cause problems (although out-of-plane bending on webs can cause fatigue problems if diaphragms are attached to webs instead of flanges). Differential movement between adjacent members becomes a significant concern in certain types of structures (small radius curves, high skews, cantilevers, etc.), and greater deflections may occur with the use of smaller members in conjunction with new design code criteria.

In design, evaluate the effects of differential deflections and girder rotations (transverse and longitudinal) that may result for skewed bridges, curved bridges, or staged construction. Consider such effects in bearing design.

Consider the need to show on the plans how members are to be detailed and fabricated, including the condition under which the diaphragms should fit (for no-load fit, full dead-load fit, or some condition in between, such as under the steel dead load but without the deck).

If the girders are required to be plumb under full dead load or steel dead load, address the expected rotations anticipated under the applied dead load. Recognize, however, that the anticipated rotations are only a prediction and that actual rotations will likely be different, and therefore girders will be somewhat out-of-plumb.

If the full dead-load fit is specified, address in the erection framing plans or erection procedures, the magnitude of the rotations anticipated at cross-frame installation and the temporary and final condition at the bearings.

Crossframes normal to girders may be placed in-line for girders that have similar differential deflections between each pair of girders at the crossframe location, provided none of the crossframes with significant differential deflections connect close to a bearing location.

For straight bridges, place crossframes

differential deflections on skewed bridges:

- For skewed piers and abutments, should nearby crossframes be placed along the skew or normal to the girder?
- Should the intermediate crossframes be skewed parallel to the piers and abutments or placed normal to the girders and staggered, and at what angle should the change be made?
- If the differential deflection is significant, the webs of the girders will rotate in a transverse direction. The webs could either be erected in the vertical position and rotate out-of-plumb after the dead load is applied, or be erected out of plumb and rotate to a vertical position after the dead load is applied. What is an acceptable out-of-plumb tolerance if transverse rotation is taken into account? What about permissible bearing rotations, especially for steel-on-steel conditions?

Crossframes at Skewed Piers or Abutments

The problem for crossframes at skewed piers or abutments is the rotation of the girders at those locations. In a square bridge, rotation of the girders at the bearings is in the same plane as the girder web. If supports are skewed, girder rotation due to non-composite loads will be normal to the piers or abutments. This rotation displaces the top flange transversely from the bottom flange and causes the web to be out of plumb. For Figure 1.6.1.A, one simplistic set of computations would show that if the centerline deflections for Girders G4 through G1 are 8 inches, 6½ inches, 5 inches, and 3½ inches respectively, the computed movement of the top flange relative to the bottom flange at the piers due to girder rotation for those girders is between $\frac{5}{8}$ inch and 1 inch.

Because rotation is normal to the centerline of bearings, lateral displacement of the top flange relative to the bottom flange normal to the theoretical girder centerline will be about $\frac{5}{8}$ inch for the 1 inch movement. The web at this location will, therefore, be out of plumb by $\frac{5}{8}$ inch over the 72 inch depth. With the 20 inch wide flange as an example, the bottom flange would be out of level by about $\frac{3}{16}$ inch. The type of bearings used determines whether this is a problem. Any items connected to the top flange at these locations may also be a problem. If a similar span exists on the other side of the pier, problems for the expansion joint are magnified. Curb alignment may also shift slightly because of this condition.

For the same girders but with a skew angle of 20°, the movement normal to the pier over the girder depth would be about $\frac{5}{16}$ inch and the out-of-level of the bottom flange would be about $\frac{3}{32}$ inch.

If the bearings can tolerate the rotation and no other problems are present, the situation may be acceptable, but the design

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either parallel to the skew or normal to the girders if the deflection between girders is constant at cross-frame connections and the skew angle is equal to or less than 20°. Otherwise place crossframes normal to the girders.

Cooperate with the fabricator, detailer, and erector to ensure proper geometry is achieved, especially if members are to be detailed in a condition other than “no-load” or if the girders are required to be plumb after erection.

Definitions:

- No-load fit – members detailed to fit in the field as though no dead load, including self weight, is on the structure, with the girder webs vertical when the crossframes are installed.
- Steel dead load fit or steel-load fit – members detailed to fit in the field as though the steel dead load is on the structure, but not the deck load, with the girder webs vertical when the crossframes are installed.
- Full dead-load fit - members detailed to fit in the field as though the webs are vertical after the full non-composite dead load of steel and concrete is applied.
- Differential deflection – The difference in relative displacement between either ends of a member (e.g., crossframe) or adjacent members (e.g., girders) at a common location in a structure (e.g., midspan). This usually refers to vertical movement but could also include lateral or angular motion.

should look at the size and capacity of end crossframe members for the degree of skew and the out-of-plumb bearing. Another alternative is to detail the girder and end crossframes so that the webs will be vertical in their final position.

Intermediate Crossframes Skewed or Normal

Crossframes set parallel to the skew angle will have less differential deflection than those set normal. Rotations similar to those noted above may require the designer to reevaluate the size of the crossframe members.

Skewed interior crossframes will have the same affect as end frames due to the rotation of girders when loads are applied. Granted the vertical deflection may be the same or only slightly different but the girder still rotates top to bottom, or longitudinally. This becomes less severe closer to the center of the spans.

AASHTO LRFD (Section 6.7.4.2) requires intermediate crossframes to be normal to main members for bridges with skews greater than 20 degrees.

Accounting for Effects of Differential Deflection

When dead-load deflection differs between adjacent girders that are connected by crossframes, the girders will rotate transversely when the non-composite dead load is applied (see Figure 1.6.1.B). If girders with large differential deflections are plumb under their own weight when erected and crossframes are fully connected in this condition, the girders will be out-of-plumb after the deck is poured. If the girders must be plumb after the dead load is applied, they must be erected out of plumb and the crossframes detailed and installed based on the anticipated behavior of the system. The amount of rotation depends on the difference in deflections between adjacent girders and the spacing between the girders. Erecting the girders and crossframes to accommodate these deflection differences will become more complex in that girders will need to be lifted and/or raised in order to connect the crossframes. Normally this condition may not have significant consequences as demonstrated by the performance of bridges over the years. The issue has become increasingly significant with greater deflections resulting from the use of lighter sections.

For small skews, fabrication tolerances specified by the AASHTO/AWS Bridge Welding Code for flange straightness, camber, and other dimensions are often larger than variation that might be expected from rotations due to differential deflections.

The design should evaluate how far out of plumb a girder can be based on the anticipated deflections and erection parameters specified. A web inclination of $\frac{1}{8}$ inch in 12 inches would cause an 8-foot-deep girder to be 1 inch out of

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plumb from top flange to bottom flange, and each flange would theoretically move laterally $\frac{1}{2}$ inch, which is consistent with AWS fabrication tolerances. An inclination of $\frac{1}{4}$ inch in 12 inches would produce 2 inches out of plumb for the 8 foot deep girder, and the transverse component of the dead load would be about 2% of the dead load applied to the top flange of the girders at that location. A transverse load equal to 2% of the vertical load applied to a flange that might have a transverse moment-of-inertia of 1% of the longitudinal moment-of-inertia adds stresses to the top and bottom flanges. These stresses need to be evaluated by the Designer.

A final issue is staggered crossframes, normal rather than skewed to the girder lines, as opposed to skewed or normal in-line crossframes full width. Crossframes can be placed in-line if the girder spacing and the relationship of the differential deflections across the structure vary linearly. However, enough distance must be left between the bearing and the crossframe at the end of the last crossframe line nearest a pier to allow the girder to rotate transversely without damage. This assumes that pier crossframes provide restraint and that AASHTO requirements are met.

In situations where there may be significant dissimilar deflections, staggering crossframes may be acceptable. Crossframes are often staggered near sharply skewed supports.

Consider the effects of out-of-plane bending on the webs and lateral flange bending if the crossframes are staggered. Where larger transverse loadings are expected, crossframes may have to be kept in line or stiffeners may be required behind crossframe connections to minimize the possibility of web cracking and high localized distortion.

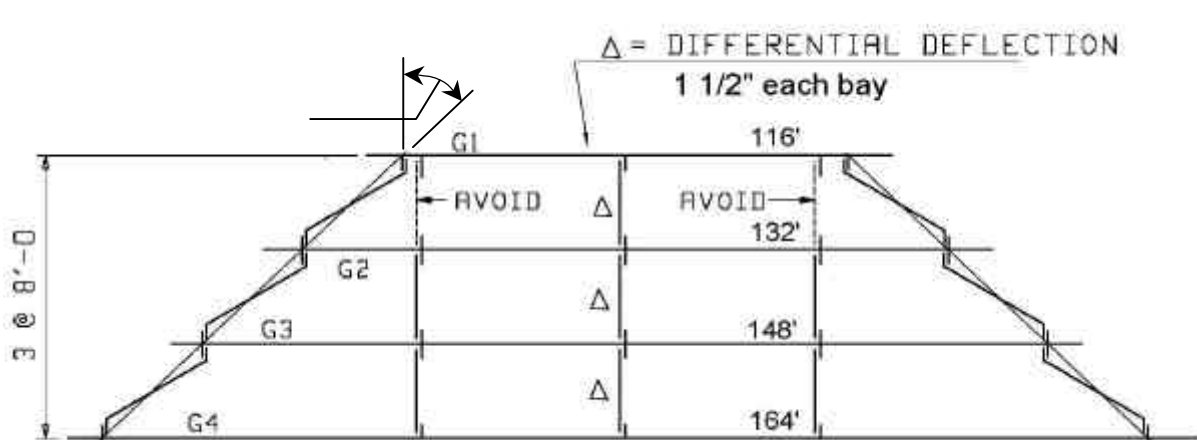


Figure 1.6.1.A

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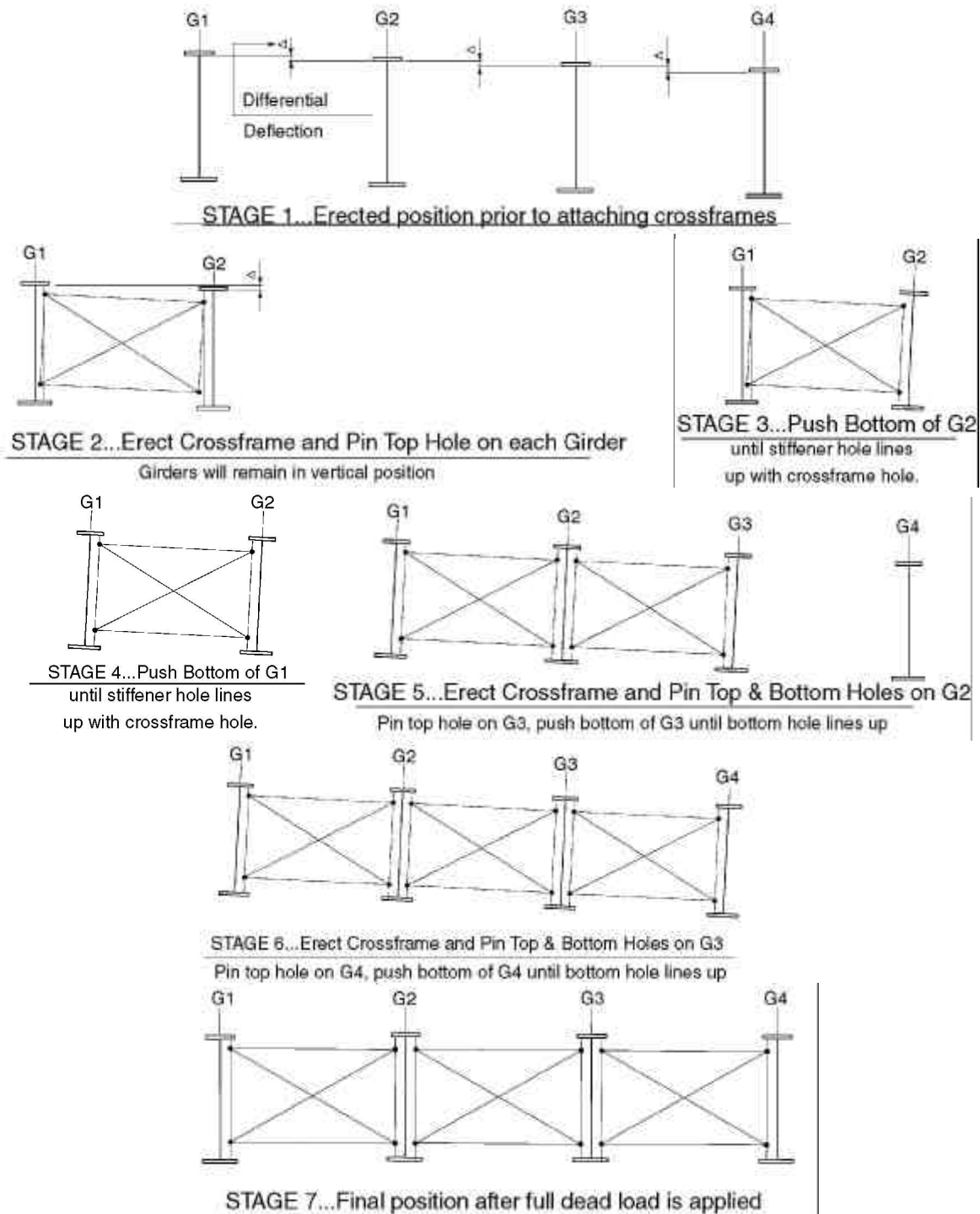


Figure 1.6.1.B

1.6.2 Deflection for Curved Structures

Issue:

How should the designer, fabricator, and erector address issues related to deflection for curved structures?

Commentary:

Deflection of curved girders is complicated, and differential deflections are appreciably more so. Both of these issues are beyond the scope of this document except that the

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Recommendation:

This question is beyond the scope of this document.

recommendations for differential deflections for straight girders in 1.6.1 should be considered when evaluating curved girders.

1.6.3 Deflection Due to Phased Construction

Issue:

How should the designer, fabricator, and erector address issues related to deflection due to phased construction?

Commentary:

If phased construction is required, the differential deflection between units due to the application of dead loads at different times can be significant. Field-drilling holes or using slotted holes and installing bolts after both phases are completed is expensive and time consuming, especially if the crossframes are not necessary. The use of independent single-angle top and bottom struts with a single bolt in each end is a simple and effective substitute. Omitting the crossframes between units avoids the problem. The design must detail the deflections anticipated and provide guidance to the contractor for deck forming, reinforcement displacements and concrete placement sequences to avoid damage.

Recommendation:

Use single-angle top and bottom struts or omit the crossframes or diaphragms between units where phased construction would cause significant differential deflection unless special conditions require crossframes.

Provide guidance for proper deck placement.

1.7 Bearings

Issue:

Elastomeric bearings are versatile and a very economic choice for bearings. When should they be specified?

Commentary:

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. Where loads are too large for elastomeric bearings, industry prefers pot bearings.

Recommendation:

Use elastomeric bearings wherever possible (as the design allows).

See Collaboration G9.1 "Guide Specification for Steel Bridge Bearing Design and Detailing", and the "Steel Bridge Bearing Selection and Design Guide," Chapter 4, Volume 2 of the "Highway Design Handbook".

All responding states use elastomeric bearings where possible, imposing limitations such as thermal expansion of 2 inches or less, spans less than 120 feet or curve limitations, and thickness less than 4 inches. Beyond those limit states, use pot bearings and Teflon/stainless or other sliding bearings. One state uses weldments and another uses weldments on bronze plates.

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Section 2 Girder Design

2.1 Stiffeners, Connection Plates, and Box Girder Bearing Diaphragms

2.1.1 Bearing Stiffeners

Issue:

Bearing stiffeners and connection plates may be normal to the top flange or to a detail working line connecting girder ends or vertical, either correcting or not correcting for dead-load rotation. Skewed bridges, particularly with crossframes or diaphragms that are significantly skewed or skewed bridges with curved girders, introduce complications. Should bearing stiffeners be vertical or normal?

Recommendation:

Permit bearing stiffeners to be vertical or normal at the fabricator's option. Girder end cuts should provide sufficient clearance to the back wall.

Commentary:

Most fabricators prefer bearing stiffeners normal. Girder end cuts may need to be vertical if there is insufficient clearance to the back wall. Five states require bearing stiffeners to be vertical, one state requires them to be normal, and one state allows either. Generally, states agree that the effect on the design is minimal.

2.1.2 Bearing Diaphragms in Box Girders

Issue:

Should bearing diaphragms in box girders be vertical or normal?

Recommendation:

Permit bearing diaphragms in box girders to be either vertical or normal at the fabricator's option.

Commentary:

If an internal and external diaphragm are connected to the top flange, vertical bearing diaphragms in box girders present particular problems. Fill plates machined to a bevel in either one or two (if the box is skewed) directions may be required if diaphragms and connection material are detailed to be vertical. Vertical diaphragms may also be designed and detailed with the flange in the same plane as the box girder flanges. Normal diaphragms are more economical and easier to fabricate and erect.

2.1.3 Connection and Intermediate Stiffeners

Issue:

Should connection and intermediate stiffeners be vertical or normal?

Recommendation:

Permit connection and intermediate stiffeners to be normal unless unusual conditions require the design to detail them otherwise.

Commentary:

All fabricators prefer both connection and intermediate stiffeners to be normal. One state requires both connection and intermediate stiffeners to be vertical. Three states use normal, four states allow either normal or vertical, and two states have no requirement.

Curved girder bridges present problems similar to boxes because the crossframes, if they are designed as main load-carrying members, must be connected to the top and bottom flanges either directly or through a connection plate. If the connection stiffeners are vertical, machined bevel fills may be required, adding significantly to the complexity of the project.

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2.2 Welding and Related Details

2.2.1 Bearing Stiffener Connection to Bottom Flange

Issue:

How should bearing stiffeners be connected to bottom flanges?

Recommendation:

Use finish-to-bear plus a fillet weld to connect bearing stiffeners to bottom flanges if a diaphragm or crossframe is connected, and use finish-to-bear if there is no connection.

Commentary:

The connection of the bearing stiffener is either:

- finish (mill or grind) to bear if no diaphragm or crossframe is connected, or
- finish (mill or grind) to bear plus a fillet weld, or
- complete joint penetration (CJP) weld.

Fabricators all 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. Two states and many railroads require CJP; however, one state recognizes a need to change. All other states use finish to bear plus a fillet weld or finish to bear if no diaphragm or crossframe is connected.

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

Issue:

How should the tolerance of fit between bottom of bottom flange and bearing sole plate be determined?

Recommendation:

Use the provisions of AASHTO/AWS D1.5 Bridge Welding Code to determine the appropriate tolerances. This may be an appropriate topic for discussion at the prefabrication conference. (Also see 2.2.1)

Commentary:

Distortion in the bottom flange from welding normally causes 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 D1.5 addresses this issue and offers appropriate tolerances, including tight tolerances between the bottom of the bottom flange and the sole plate over the projected area of the bearing stiffener and web on the sole plate. Bearing design should be based on these conditions.

All fabricators follow appropriate provisions of the AASHTO/AWS D1.5, as do all states responding except one that has special requirements for flatness.

2.2.3 Minimum Spacing between Adjacent Stiffeners or Connection Plates

Issue:

What should be the minimum spacing between adjacent stiffeners or connection plates?

Recommendation:

Provide 8 inch minimum spacing or 1½ times the plate width for welding access.

Commentary:

State requirements vary for minimum spacing between adjacent stiffeners or connection plates, including these values: 4 inches clear, 6 inches clear, 30° access, 45° access, and "use judgment." To allow access for Dart Welder or other equipment, fabricator responses were 7 inches, 10 inches, and 1½ times the plate width.

2.2.4 Connection Stiffener Attachment to Tension Flange (For Bottom Flange of Box Girders See 3.5)

Issue:

How should the connection stiffener be attached to the tension flange?

Recommendation:

Do not use tab plates. Weld the connection

Commentary:

AASHTO specifications require a positive attachment of the crossframe connection stiffener to both flanges. The connection to the compression flange is always welded, but the connection to the tension flange is

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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 crossframes.

either welded or bolted through a tab plate that has been welded to the connection stiffener. The fatigue category for the tension flange is lower for the welded detail than for the bolted tabs; however, the bolted tabs cost upwards of \$150 (in 2002 dollars) each to furnish and install. Tabs should only be used when absolutely required. Sometimes adjustments to the location of the crossframe connection plates can eliminate the need for tab plate (bolted) connections.

Designers commonly bolt tab plates to flanges to provide improved fatigue resistance for the flange. Designers should note that the weld attaching the connection plate to the web is of the same fatigue category as the tension flange weld. 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 welded connection with a bolted connection will not improve the fatigue resistance of the girder as a whole.

2.3 General Details

2.3.1 Intermediate Stiffeners (Not Connection Stiffener) at Tension Flange

Issue:

At tension flanges should intermediate stiffeners be tight fit or cut short?

Recommendation:

Call for a tight fit (as defined in AWS D1.5) for the intermediate stiffeners to the tension flange.

Commentary:

Intermediate stiffeners not connected to crossframes or diaphragms are generally welded to the compression flange and either fit tight or cut short of the tension flange. Fabricators are divided in their preference; however, several point out that a tight fit helps to straighten flange tilt without application of heat. States prefer tight fit by a margin of 7 to 4.

2.3.2 Bolted Compression Joints in Arch Members and Chords of Trusses

Issue:

Should compression joints in arch members and chords of trusses be designed using open joints with enough bolts to carry all of the load, or using milled joints and 50% bolts?

Recommendation:

Design compression joints in arch members, truss chords, and other such members with an open joint at the splice with 100% bolts and the appropriate splice plate thickness.

Commentary:

AASHTO allows a compression joint in arches and similar members to be designed as either a milled joint plus 50% of the bolts that are required to carry the load or an open joint with 100% bolts.

Most states have no specific policy. Several states have used milled joints and would consider a value engineering proposal from the fabricator. Fabricators responding prefer to use open joints with the extra bolts to carry load, an approach that is less expensive and presents lower potential for problems in the field.

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2.3.3 Connection of Skewed Intermediate Crossframes

Issue:

Should bent gusset or skewed connection plates be used to connect skewed intermediate crossframes to girders?

Recommendation:

Give the fabricator the option to use either skewed connection or bent gusset plates. Limit skewed crossframe to a maximum angle of 20° (see also 1.6.1). See Figure 2.3.3.A and Figure 2.3.3.B.

Commentary:

Except for one survey respondent, fabricators preferred to bend the gusset plates rather than skew the connection plate when connecting skewed intermediate crossframes to girders. Skewed connection plates create fitting and welding problems, especially as the degree of skew increases. If the skew angle exceeds 20° , welds will probably have to be done manually using SMAW. 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, crossframes with bent gussets can be fit using jigs or templates to provide accurate connections.

Six states use skewed connection plates, with some having maximum limits varying from 15° to 30° . One or two states have provisions for using offset plates when the skew exceeds 30° . Four states use bent plates with two states having limits of 30° and 45° .

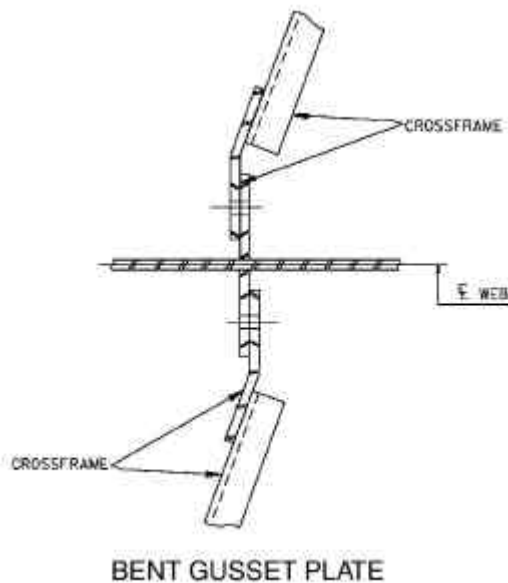


Figure 2.3.3.A

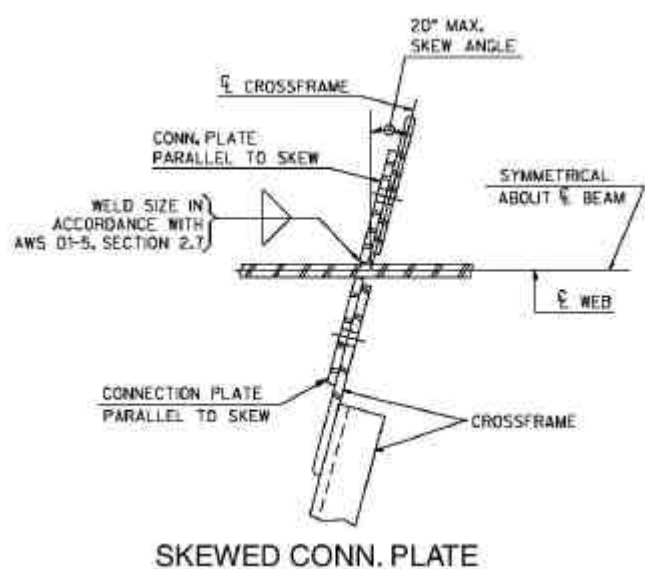


Figure 2.3.3.B

2.3.4 Field Splices

2.3.4.1 Straight I Girders

Issue:

Shop assembly requirements for field splices in straight I girder bridges.

Recommendation:

Drill or ream field splices in straight I-girders with the web horizontal or vertical (at the

Commentary:

Field splices in main members of rolled beams, plate girders, and tub and box girders are typically required to be shop-assembled and drilled or reamed while the members are supported in the no-load condition. Field

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fabricator's option) with members assembled bearing to bearing unless the Engineer approves another method of sequential geometry control or if CNC drilling is approved (see 2.3.4.4). If webs are vertical, block or support members in the no-load position.

2.3.4.2 Curved I Girders

Issue:

Shop assembly requirements for field splices in curved I girder bridges.

Recommendation:

Drill or ream field splices in curved I girders with the web horizontal or vertical at the fabricator's option. Assemble bearing to bearing unless the Engineer is satisfied that the fabricator's geometry control methods permit otherwise. If webs are vertical, block or support members in the no-load condition.

2.3.4.3 Box Girders

Issue:

Shop assembly requirements for field splices in straight or curved tub girder bridges.

Recommendation:

Drill or ream field splices for straight or curved box girders assembled bearing to bearing unless the Engineer approves another method of sequential geometry control or if CNC drilling is used (see 2.3.4.4). Members may 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 details must accurately show all dimensions and elevations to assemble the members properly for drilling or reaming.

2.3.4.4 CNC Drilled Field Splices

Issue:

CNC drilling requirements for field splices in straight and curved girder bridges.

Recommendation:

CNC 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

splices in straight I girders have traditionally been reamed or drilled with webs horizontal with at least three members in assembly. Collaboration document S2.1 requires minimum assembly to be bearing-to-bearing.

Commentary:

Field splices in curved I girders are drilled or reamed with the webs horizontal or vertical depending on shop practice and capabilities. The radius of curvature is a major factor in determining which method is appropriate for the individual shop. For girders with a small radius, more members can normally be placed in an assembly with the webs vertical than with the webs horizontal.

Commentary:

Field splices in straight or curved box girders are usually drilled or reamed while assembled. The girders can be rotated to eliminate grade and cross slope.

Commentary:

Computer numerically controlled (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.

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verification steps. Limited check assemblies, along with continuous monitoring of the process, should assure the accuracy of the final product.

2.3.5 Shop Assembly of Curved Girder Structures

Issue:

When should full or partial shop assembly of curved structures be required?

Recommendation:

Full or partial shop assembly with crossframes should only be required by contract for structures that are very rigid (e.g., bascule and through-girder railroad bridges) with small radii or complex geometry, or where girders terminate at load-carrying diaphragms or other girders. It may also be appropriate to allow the fabricator to use alternate schemes to ensure proper final fit without assembly.

Commentary:

Both partial and full shop assembly of curved girder structures, including crossframes, are expensive and time-consuming operations. Nearly all surveyed fabricators believe that shop assembly should be limited to short stiff girders with small radii. However, partial shop assembly may be appropriate when curved members frame into a header or another girder and when crossframe or full-depth diaphragm connections are complicated, and full assembly may be warranted when the overall geometry is very complex, such as a skewed, curved and flared structure. Three fabricators noted their obligation to ensure that members will fit during erection. Three states require full or partial assembly. Seven states require assembly for time-sensitive projects, for curved ramps terminating at load-carrying diaphragms, and for complex structures. One state echoed the fabricator's position that final fit remains the fabricator's responsibility.

Curved girders often have sufficient transverse and vertical flexibility to allow relatively small horizontal or vertical displacement for installation of crossframes either in the shop or in the field. Heavy rigid members will not be as flexible and may require shop assembly. The more important issues are longitudinal accuracy and how the members are supported during erection.

2.3.6 Haunched Girders

2.3.6.1 Bottom Flange at Bearing

Issue:

How should the flange transition from the flat bearing part of the girder to the curved or sloping part of the haunch be made?

Recommendation:

Design 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

Commentary:

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.

One fabricator prefers welding if the flange is over 1¼ inch thick. Other fabricators prefer bending depending on considerations such as the radius and

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dimension from the edge of the sole plate to the transition should be at least 12 inches. Additionally, the owner may wish to consider future jacking needs.

the length of plate available from mill. Fabricator preferences for clearance from bend line or weld joint to sole plate vary from 3 to 12 inches. Most states recorded no preferences. Those responding to the survey were evenly split between welding and bending, and several that prefer welding would allow bending. Dimensions from the sole plate to the bend or weld line vary and include 4, 6, and 24 inches and $\frac{1}{2}$ the sole plate width and may provide horizontal jacking surfaces at piers for future bearing maintenance.

2.3.6.2 Curved or Straight Haunch

Issue:

For haunched girders, is the curved or straight haunch preferred?

Recommendation:

Avoid haunched girders by using parallel flanges instead. If a haunch is needed for clearance, aesthetics, etc., the straight taper is more cost effective.

Commentary:

Surveyed fabricators all prefer the straight haunch. It is easier to cut webs, locate and fit stiffeners, and weld splices for straight haunches and also reduces fit-up problems at the web to flange joint.

Few states still use haunched girders. Three states prefer the curved flange for appearance and stress flow.

2.3.7 Curved Girders – Heat-Curve or Cut-Curve

Issue:

For curved girders, is heat-curving or cut-curving preferred?

Recommendation:

Permit either heat-curving or cut-curving in accordance with AASHTO specification limits at the fabricator's option.

Commentary:

AASHTO specifications allow the use of both heat-curving and cut-curving procedures, with restrictions on their use. One surveyed fabricator cut-curves any curved girder with less than a 2000-foot radius. Other fabricators prefer to heat-curve when allowed by AASHTO. Cut-curving is required by three states, one of which will allow heat curving if so requested. Six states allow either method.

2.3.8 Crossframes and Diaphragms

2.3.8.1 Intermediate Crossframes or Diaphragms for I-Girder Bridges

Issue:

What type of intermediate crossframe or diaphragm should be used for I-girder bridges?

Recommendation:

Use crossframe types shown in Figure 2.3.8.1.A or Figure 2.3.8.1.B. The fabricator should be permitted to use parallelogram as well as rectangular configurations to keep connection plates identical. The X-frame is the recommended typical detail; however, if the angle of the diagonals is less than 30° , use the K-frame. The Z-frame, Figure

Commentary:

Crossframe types vary considerably both within and between states. Standards developed and endorsed by such groups as FHWA SCEF and the Collaboration TG1 should be adopted whenever possible. The following recommendations cover the more common applications.

Fabricators unanimously prefer single-angle (or when necessary, single-member, such as WT) bracing. Double angles are expensive to fabricate, and painting the backs of the angles creates unnecessary problems. Fabricators prefer crossframes that can be welded from one side

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2.3.8.1.C (Tennessee DOT) may be an acceptable option for girders more than 42 inches deep, and the bent plate diaphragm, Figure 2.3.8.1.D (Kansas DOT) is a good option for girders less than 48 inches deep.

If girder spacing, girder depths or deck overhangs so warrant, consider adding a top strut to X-frames.

only. Configuration of crossframes should allow as many identical frames as possible. Differences in elevations should be accounted for in the crossframes, not the connection plates. Configuring crossframes as parallelograms instead of rectangles will often increase the number of identical connection plates. Several fabricators endorse bent plate channel diaphragms as an economical member for shallow girders.

In general states also prefer single-angle or single-member bracing. Simplicity is a common requirement. Several states use bent plates or rolled beam diaphragms for girders less than 48 inches deep. Most states prefer X-frames for deeper girders, but several states also use K-frames.

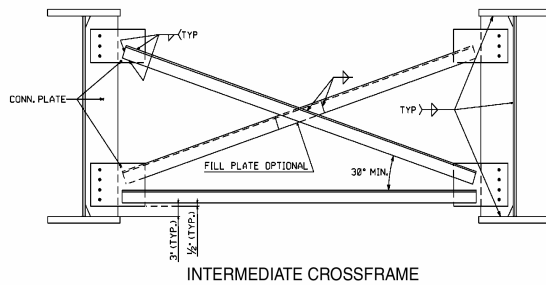


Figure 2.3.8.1.A

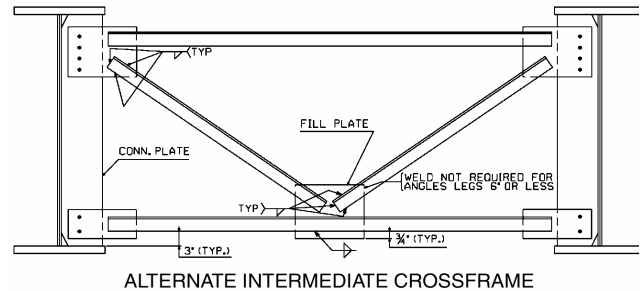


Figure 2.3.8.1.B

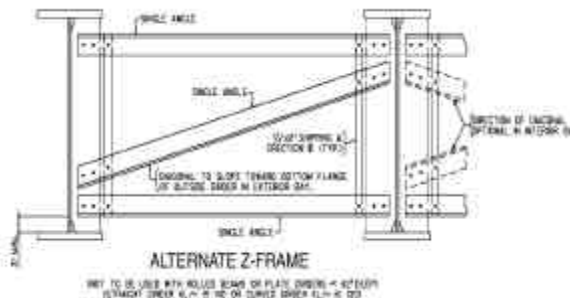


Figure 2.3.8.1.C

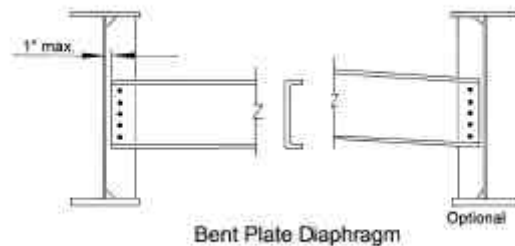


Figure 2.3.8.1.D

2.3.8.2 Intermediate Crossframes or Diaphragms for Rolled Beam Bridges

Issue:

What intermediate crossframe types or diaphragms should be used for rolled beam bridges?

Recommendation:

Several options are acceptable:

Commentary:

One surveyed fabricator prefers bent plates. Another prefers end angles attached to rolled-beam or channel diaphragms for field bolting to stringers, thus eliminating intermediate connection plates. States use

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- 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 2.3.8.1.D.
- SCEF and Collaboration Task Group 1 standards.

2.3.8.3 End Crossframes or Diaphragms for I-Girder Bridges

Issue:

What end crossframe types or diaphragms should be used for I-girder bridges?

Recommendation:

Use end crossframe types shown in Figure 2.3.8.3.A and Figure 2.3.8.3.B.

Commentary:

One state requires that end diaphragms and their attachments be designed for future jacking.

Optimally, most states will adopt common details, improving efficiency and economy.

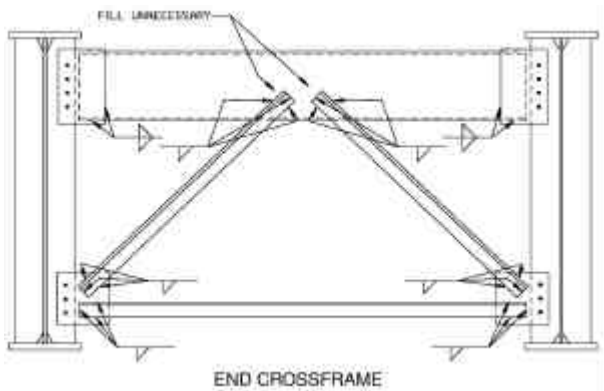


Figure 2.3.8.3.A

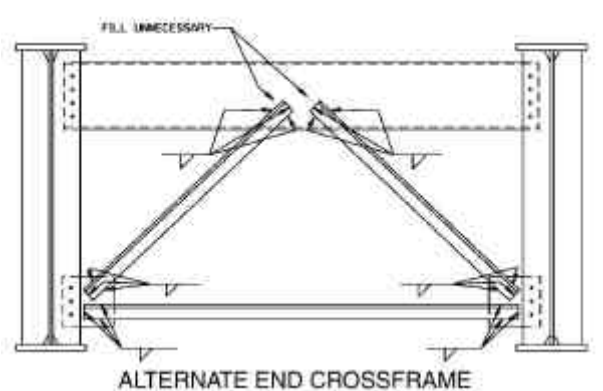


Figure 2.3.8.3.B

2.4 Longitudinal Field Web Splices in Deep Girders

Issue:

For longitudinal field web splices in deep girders, what type of field bolted splice should be designed?

Recommendation:

For longitudinal field-bolted web splices in girders too deep to ship, use conventional side plates in the web splice design.

Commentary:

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.

All fabricators preferred the conventional side plates. Most states have not used either. One state uses conventional side plates.

Section 3 Boxes

3.1 Closed Box Configuration

Issue:

What configurations of plates should be used for fillet-welded closed boxes?

Recommendation:

Use configurations shown in Figure 3.1.A for large boxes and curved boxes. For truss chords use Figure 3.1.A if practical. If not, consider Figure 3.1.B or Figure 3.1.C depending on web or flange thickness for truss-chord as well as other truss members.

Commentary:

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.A and Figure 3.1.C, and lapping the web on the edge of the flanges as shown in Figure 3.1.B. Fabricator preferences are evenly split between the options. Surveyed states had few comments, except that several were concerned about enough support for flux and other welding considerations. Terminating the web at the inside face of the flanges would normally provide a straighter final product.

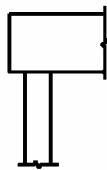


Figure 3.1.A

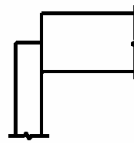


Figure 3.1.B

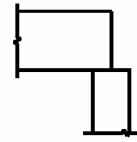


Figure 3.1.C

3.2 Closed Box Corner Welds

Issue:

What are the appropriate weld types for attaching the webs and flanges of boxes?

Recommendations:

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 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.A).
- 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.B).
- Double fillet welds at each of the four corners,

Commentary:

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 full penetration welds are required, preparation should be on the thinner plate.

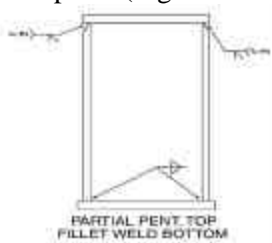
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requiring welding inside the closed box (Figure 3.2.C).

- Single fillets welds at each of the four corners. This may be appropriate for some boxes depending upon load conditions and internal diaphragms (Figure 3.2.D).

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

- Single fillet welds at each of the four corners. The preferred practice. The designer should investigate from a torsion perspective with due regard to the number of internal diaphragms and other such applicable considerations. This is the best procedure for truss members (Figure 3.2.D).
- 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.A).
- 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.B).



Preferred
Figure 3.2.A



Figure 3.2.B

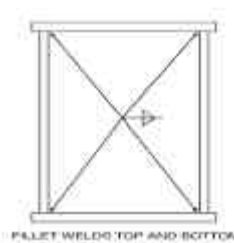
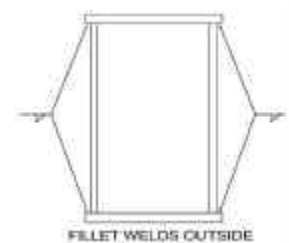


Figure 3.2.C



Preferred
Figure 3.2.D

3.3 Closed Box Diaphragm Attachment

Issue:

How should interior diaphragms of closed boxes be attached?

Recommendation:

Weld three sides and tight-fit to the tension flange. Also allow an optional bolted connection to all or only one side. Nuts may be welded to angles attached to the diaphragms, permitting bolt installation without entering the box, essential if

Commentary:

The issue is how the diaphragms attach to the box, particularly whether attachment to the last flange installed is really necessary. Most states had no response or no standard although two states weld three sides and tight-fit at the tension flange, and one state typically welds all four sides. Most fabricators prefer to weld three sides and tight-fit to the tension flange; one fabricator

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solid diaphragms or small boxes are used.

prefers to bolt the diaphragms.

3.4 Closed-Box Interior Diaphragm Minimum Access Hole Size

Issue:

What size should the access opening be for closed-box interior diaphragms?

Recommendation:

The minimum access hole size should be 18 by 24 inches and, where practical, 32 by 36 inches.

Commentary:

State requirements vary significantly from 14 by 26 inches to 34 by 43 inches. Fabricator preferences vary from 18 by 24 inches to 32 by 36 inches. The larger size of 32 by 36 inches 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.

3.5 Stiffener Detail Near Bottom Flange of Tub Girders

Issue:

What are acceptable details at the end of stiffeners near the bottom flange to allow for the welding of the bottom flange to web of the girder?

Recommendation:

Use details shown in Figure 3.5.A or Figure 3.5.B, both of which are preferred, or use Figure 3.5.C. Figure 3.5.D is preferred when the fabricator welds the bottom flange to the webs prior to attaching the stiffeners.

Commentary:

Typically, webs are jointed 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. States have no standard for this particular situation but most will allow modified details to accommodate automatic or semi-automatic welding of the flange to the web. Fabricators have proposed details that states have accepted on individual jobs.

See discussion on connection attachment to tension flange in 2.2.4.

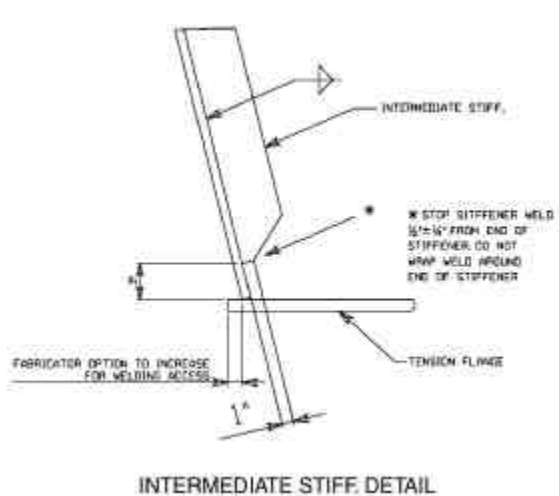


Figure 3.5.A

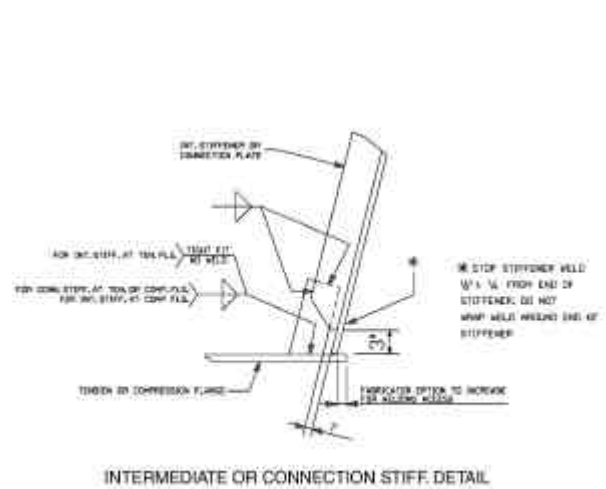


Figure 3.5.B

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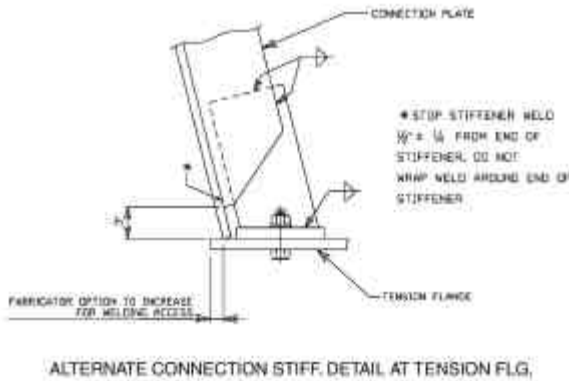


Figure 3.5.C

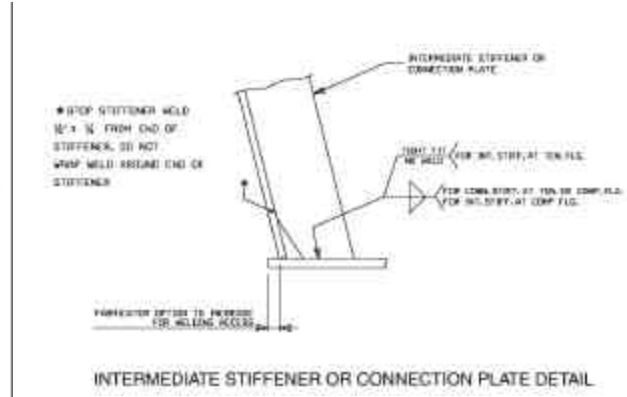


Figure 3.5.D

3.6 Stiffened or Unstiffened Bottom Compression Flange of Tub Girders

Issue:

Should bottom compression flanges be stiffened?

Recommendation:

If design analysis shows that longitudinal flange stiffeners are more economical than thickening the flange, use WTs but stop the stiffener short of the field splice (splice plates should adequately stiffen the flange).

Commentary:

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, splicing the WTs at field splices and treatment at the bearing diaphragms complicates fabrication and field assembly.

States generally use WTs when stiffening the bottom compression flange of these members. Fabricators prefer designs with bottom flanges that are unstiffened. If bottom flanges are stiffened, fabricators prefer WTs to bars.

3.7 Coating the Interior of Closed Boxes and Tub Girders

Issue:

Should the interior of closed boxes and tub girders be coated?

Recommendation:

For typical tub girders or closed box girders, coating for corrosion protection is discouraged. If future inspection mandates cleaning and painting, specify a single coat of surface-tolerant light-colored paint (e.g., epoxy) with 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.

Commentary:

Three states paint the interior of these boxes using two coats, with the second light coat for inspection. Five states use only one coat, also light, and one state uses a full three-coat system.

The most economical solution is to use unpainted weathering steel for both external and internal surfaces. Many states, however, require painting the inside of box for inspection purposes even if weathering steel is used and the external surfaces are unpainted.

Most states use an inorganic zinc primer with or without a second coat that is light in color so that the inside of the box is inspectable after erection. Some states use a second coat that is white or very light in color. At least one state allows a single light coat (not a primer) for inspection purposes only. Several states indicated that they have experienced no corrosion

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problems inside unpainted boxes. One state cites constant condensation on the inside of boxes as a reason to paint.

3.8 Relative Costs of Closed Boxes and Tub Girders

Issue:

Are closed boxes or tub girders preferred?

Recommendation:

No recommendation - there have been no reported studies of boxes of similar weight and span lengths.

Commentary:

Fabricators expect that closed boxes would cost at least 20% to 30% more due to welding, painting, and safety considerations.

3.9 External Crossframes for Multiple Box and Tub Girders

Issue:

Should external crossframes, temporary or permanent, be required for straight or curved multiple box or tub girder bridges?

Recommendation:

Permanent crossframes between boxes and tub girders should be provided at supports. If multiple straight boxes or tub girders are adequately braced internally, external intermediate crossframes are not required. For curved multiple box or tub girders that require crossframes between members, use permanent crossframes. Temporary crossframes should use temporary connections (e.g., bolt to webs instead of using welded connection plates) and be unpainted.

Commentary:

Fabricators believe that crossframes should be used primarily for curved structures. If individual girders are adequately horizontally braced, external crossframes between boxes or tubs are not needed except at supports for straight bridges. If curved bridges are individually horizontally braced and erected using sufficient falsework to prevent torsional bending until the deck is placed, then external crossframes between boxes or tubs may not be required except at supports. For other environments, install unpainted temporary crossframes and remove them after the deck has been poured.

Most states that use boxes or tubs require temporary external crossframes that are removed after the deck is poured. One state does not require them on straight bridges, and one state requires crossframes only at supports. Several states mentioned the issue of stability during re-decking. Texas is doing research to assess the need for the frames.

Section 4 Bolts

4.1 Metric or U.S. Customary Units for Bolts

Issue:

Should bolts be specified in metric or US Customary units?

Recommendation:

Specify US Customary sizes for bolts and holes for new designs, and allow their substitution on shop drawing details for metric designs. Do not mix US Customary bolts and metric hole sizes.

Commentary:

Domestically produced metric bolts are not readily available except in very large quantities. Fabricators point out that hard metric bolts are cost-prohibitive for the sizes and quantities typically required for bridges. In fact, very few fabricators have reported furnishing hard metric bolts. Where designs call for hard metric bolts, shop details are generally prepared substituting US Customary sizes (inch).

Based on FHWA policies, states began detailing with hard metric during the 1990's, but most have now reverted to US Customary sizes. However, as states return to US Customary sizes, many designs prepared in metric will remain, so states should allow the fabricator to use U.S. customary bolts and hole sizes on those jobs.

4.2 Mechanical or Hot-Dipped Galvanized Bolts

Issue:

Which type of galvanizing is better?

Recommendation:

Where galvanized fasteners are required, use mechanically galvanized bolts, except that when galvanized A325 bolts are used on weathering steel projects, use hot-dipped galvanized bolts. Note that galvanizing of A490 bolts is not allowed.

Commentary:

States are divided on the issue of mechanical versus hot-dip galvanizing, with many of them allowing either but preferring one. Some states believe that the hot-dipped bolts give better corrosion protection, but other states believe that the mechanically galvanized bolts have more consistent coating thickness and have fewer tightening problems. Fabricators unanimously recommend using mechanically galvanized bolts because of better product consistency, better availability, and fewer installation problems.

4.3 Black Versus Galvanized Shop-Installed Bolts

Issue:

Will galvanized bolts that have been blasted and painted after installation provide better protection than black bolts that receive the same process, and is it worth the additional cost?

Recommendation:

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

Commentary:

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. The compatibility of the nut lubricant and its accompanying dye may also be an issue. Tests on some lubricants and dyes show no detriment to the adhesion of the shop primer on the galvanized bolt. Consider paint manufacturer recommendations for any pretreatment of the galvanized nuts prior to shop priming. Shop-painted galvanized bolts will almost certainly perform better than shop-coated black bolts at very little additional cost. The Collaboration guide specification for

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inorganic zinc paint systems (S8.1) endorses galvanized bolts.

Most states have no specific requirement, assuming that fabricators will use black bolts. Some states specify black bolts for such connections. All responding fabricators but one prefer black bolts. One prefers to use galvanized bolts to avoid contamination of adjacent surfaces from the oil on the bolts.

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Section 5 Corrosion Protection

5.1 Corrosion Protection Systems

Issue:

What is the recommended corrosion protection system?

Recommendation:

Unpainted weathering steel is the least expensive, lowest-maintenance solution. Integral unpainted weathering steel bridges require no painting. Visible surfaces of the bridge should be blast cleaned to improve the aesthetic quality of the patina. Follow the state's established corrosion protection practices, if any.

Commentary:

Three major systems of corrosion protection used on steel bridges are:

- Weathering steel
- Painted steel, with one shop prime coat followed by field coats
- Painted steel with all coats applied in the shop

Fabricators favor weathering steel whenever possible. When weathering steel is used and blast cleaning required, fabricators prefer that only the fascia girders require blast cleaning.

States typically establish corrosion protection practices based on local performance demands and preferences.

Many states use weathering steel based on the current FHWA guidelines ("FHWA Guideline for Uncoated Weathering Steel Bridges"). For painted structures, five states use the three-shop coat system, and most others use the one shop prime coat plus two field coats or two shop-coats and one field-coat. One state lets the contractor decide.

5.2 Bolted Faying Surfaces

Issue:

Should bolted connection design be based on Class A or Class B surfaces?

Recommendation:

For painted girders, paint faying surfaces and design for Class B surfaces if the state's primer meets those requirements. For weathering steel jobs, clean the surfaces that have slip-critical connections and design for an unpainted Class B connection. Design plans should specify the class of the connection.

Commentary:

Two classes of bolted connection are generally used in design. The Class A and Class B connections require different preparation of the faying surfaces. The friction provided by these preparations is the basis for the value of the design capacity of the bolts in the connection. To reach the higher design values, the fabricator must either blast the faying surfaces for weathering steel or blast and paint the faying surfaces for painted structures with a suitable primer. These higher design values result in a reduction in bolt count in the connection.

When faying surfaces are to be painted, the primed surface must provide enough friction for the connection as designed, either Class A or Class B. Coating manufacturers test and certify their zinc-rich primers for the class the primer meets. They also test to ensure the primer will not creep. Test methods are described in the AISC "Manual of Steel Construction, Specification for Structural Joints Using ASTM A325 or A490 Bolts, Appendix A, Testing Method to determine the Slip Coefficient for coatings used in bolted joints.

Section 6 Other

6.1 Contractual Items

6.1.1 Lump Sum versus Unit Price Bids

Issue:

Should contract bid items be lump sum or unit price?

Recommendation:

Contract bid items for structural steel may be lump sum, except that repair and other jobs with indefinite quantities may be an exception. Also, some storage items may need to be bid as “per-ton per-day.”

Commentary:

All fabricators prefer lump sum bids. One state uses unit price bids; one state uses unit price but treats as lump sum unless there is a design change; all others use lump sum bids.

6.1.2 Partial Payment for Materials and Fabrication

Issue:

What are the recommended parameters for partial payment for materials and fabricated members?

Recommendation:

Payment for mill material and fabricated structural should be based on the following:

- 100% of the invoiced cost for mill material received, documented, and stored at the fabrication plant (weight not to exceed the calculated steel quantity for the project).
- 70% of the “Fabricate and Deliver” contract price for members completely fabricated and stored, ready for cleaning and painting.
- 90% 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% when erected.

Commentary:

In March 2000, The Federal Highway Administration, 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. Additionally many states allow payment to 90% of the contract price if all of the steel for the contract has been fabricated, blasted, painted and properly stored. Payment for partially completed members (all fabrication on the member complete leaving only cleaning and painting) at 70% of the contract price is an intermediate step that is missing.

The cost of financing the stored of mill material and fabricated members at the shop is high. Job site delays can add significant additional costs that affect fabricated steel prices. Some states pay for raw material (steel plate) on hand once the steel is delivered to the shop.

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

Issue:

What is the preferred way to handle contractual bid items for fabrication, erection, and field painting?

Commentary:

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

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Recommendation:

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

fabrication, erection, and field painting.

Several states have used separate “Fabricate and Deliver” contracts on special jobs and have found them satisfactory. State survey responses did not submit their normal contractual requirements.