

15. Structural Steel Superstructures

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This chapter discusses structural steel provisions of the *LFRD Bridge Design Specifications* that require amplification, clarification, and/or an enhanced application. Section 11.5 of this *Manual* provides criteria for structural steel superstructure site selection. These criteria include span lengths, girder spacing, geometrics, aesthetics, and cost. Chapter 15 addresses specific DOT&PF practices for the design and detailing of steel superstructures.

DOT&PF typically uses structural steel superstructures where prestressed concrete, decked bulb-tee girder superstructures will not work for geometric, environmental, cost, or constructability reasons. As such, steel girders are selected where span lengths exceed 145 feet, where falsework needs to be minimized, and in remote locations where bulb-tee girders prove too heavy to transport and erect. Steel box girders have increased stability during erection in comparison to steel I-girders and may be appropriate for some locations. Rolled beam sections may prove more cost-effective than welded steel plate girders for shorter spans in remote locations.

15.1. General

15.1.1. Economical Steel Superstructure Design

Factors that influence the initial cost of a steel bridge include, but are not limited to, detailing practices, the number of girders, the grade of steel, type and number of substructure units (i.e., span lengths and arrangements), steel weight, fabrication, transportation, and erection. The cost associated with these factors changes periodically in addition to the cost relationship among the factors. Therefore, evaluate and modify these guidelines as necessary for each bridge to determine the most economical type of steel girder.

Based upon market factors, the availability of steel, particularly large rolled wide flange sections, may be an issue in meeting the construction schedule. The bridge engineer must verify the availability of the

specified steel. Bridge designers should contact producers to ensure the availability of plates and rolled beams. For more detailed information on material (plates, rolled beams) availability, see Section 1.4 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructability*, G12.1.

Number of Girders/Spacing

See Section 11.4.5 for general information on the number of girders in girder bridges. See Section 11.5.3. for DOT&PF criteria on typical girder spacing for both I-girder and tub girder steel bridges. For detailed commentary on steel girder spacing, see Section 1.2 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructability*, G12.1. Typically, fewer deeper girders result in less steel weight; however, this approach may not result in the lowest superstructure cost.

Exterior Girders

The location of the exterior girder with respect to the overhang is controlled by these factors:

- Locate the exterior girder to limit the dead load and live load on girders such that the exterior girder does not control the design (i.e., the interior and exterior girder sections are identical). Overhang lengths equal to approximately 25 percent to 45 percent of the interior girder spacing typically yield moment balance between exterior and interior girders.
- The space required for deck drains may have an effect on the location of the exterior girder lines.
- Consider aesthetics when determining the location of the exterior girder lines.

Span Arrangements for Continuous Girders

Use span arrangements that balance the moments in the spans (i.e., equal maximum moments in end and interior spans). The end-span lengths are typically between 70 percent and 80 percent of the length of interior spans. As a result, the optimum proportions of the girder in all spans will be nearly the same, resulting in a more efficient and cost-effective design. To prevent uplift at the abutments, avoid end spans less than 70 percent of the interior spans.

15.1.2. Rolled Beams vs Welded Plate Girders

Welded Plate Girders

Design welded steel plate girders to optimize total cost including material costs while also considering fabrication and erection costs. Top flanges of composite plate girders are typically thinner than their bottom flanges. Vary the flange section along the length of the bridge, generally following the moment envelope to save cost by offsetting the increased fabrication costs of welded flange transitions with larger savings in material costs. Typically, the design should vary only flange thicknesses, not widths, within a field section to reduce fabrication costs. The webs of plate girders are typically deeper and thinner than the webs of rolled beams.

Due to buckling considerations, address the stability of the compression flange (i.e., the top flange in positive-moment regions and the bottom flange in negative-moment regions) by providing lateral-brace locations based upon calculations instead of the 25-foot diaphragm spacings of the *AASHTO Standard Specifications for Highway Bridges*. The traditional 25-foot diaphragm spacing, however, provides a good starting point.

On straight bridges (skewed or non-skewed), detail diaphragms as secondary members. On horizontally curved bridges, design diaphragms as primary members, because horizontally curved girders transfer a significant amount of load between girders through the diaphragms.

Rolled Beams

Rolled steel beams are doubly symmetrical sections with equal-dimensioned top and bottom flanges and relatively thick webs. Thus, these beams do not optimize the cross sections for weight savings (like a plate girder) but are cost effective due to lower fabrication and erection costs. The relatively thick webs preclude the need for intermediate web stiffeners, but full-height bearing stiffeners are required at all support locations. Unless difficult geometries or limited vertical clearances control, rolled steel beam superstructures are more cost effective for relatively shorter spans (up to about 80 feet).

Rolled steel beams are more readily available in depths up to 36 inches, with deeper beams typically rolled less frequently. Before beginning final design, verify with one or more potential fabricators that the section size and length are available.

If a large size (greater than 36-inch deep) rolled-beam design is proposed for a new bridge, the contract documents should allow the substitution of a welded plate-girder design of equal flange and web plate sizes at the Contractor's discretion.

15.1.3. Economical Girder Proportioning

General

Make girders composite with the bridge deck through shear studs and continuous over interior supports where possible. For economy, all girders in a multi-girder bridge should be identical where possible or, if necessary, use a minimal number of plate sizes.

Haunched (Variable-Depth) Girders

When practical, use constant-depth girders (i.e., girders with constant web depths). Haunched girders are generally uneconomical for interior span lengths less than 240 feet. Consider using parabolic haunched girders where aesthetics or other special circumstances are encountered, but constant-depth girders will generally be more cost effective.

Flange Plate Sizes

The minimum flange plate size for welded girders is 12 inches by 1 inch to avoid cupping of the flanges due to distortion from welding. Use as wide a flange girder plate as practical, consistent with stress and b/t (flange width/thickness ratio) requirements. The wide flange contributes to girder stability during handling and in-service. As a guide, the flange width should be approximately 20 to 30 percent of the web depth. Provide flange widths in increments of 2 inches. The maximum flange thickness is 3 inches to ensure more uniform through-thickness properties. Avoid thicker plates; they demonstrate relatively poorer properties near mid-thickness.

Within a single field section (i.e., an individual shipping piece), provide flanges of constant width. A design of multiple identical girders with constant-width flanges minimizes fabrication costs.

Proportion flanges so that the fabricator can economically cut them from steel plate between 60 inches and 120 inches wide. The most economical mill widths are 72 inches, 84 inches, 96 inches, and 120 inches. Allow $\frac{1}{4}$ inch for internal torch cutting lines and $\frac{1}{2}$ inch for exterior torch cutting lines; see Figure 15-1. Group flanges to provide an efficient use of the plates. Because structural steel plate is most economically purchased in these widths, it is advantageous to repeat plate thicknesses as much as

practical. If practical, group plates of like width by thickness to meet the minimum width-purchasing requirement, but this economical purchasing strategy may not be possible for thicker, less-used plates.

The most efficient method to fabricate flanges is to groove-weld together several wide plates of varying thicknesses received from the mill. After welding and non-destructive testing, the individual flanges are “stripped” (i.e. cut longitudinally) from the full plate. This method of fabrication reduces the number of welds, individual runoff tabs for both start and stop welds, the amount of material waste, and non-destructive testing. The objective, therefore, is for flange widths to remain constant within an individual shipping length by varying material thickness as required. Figure 15-1 illustrates one example of an efficient fabrication for girders.

Constant flange width within a field section may not always be practical in girder spans over 300 feet where a flange width transition may be required in the negative bending regions. Though not preferred, if a transition in width must be provided, shift the butt splice a minimum of 3 inches from the transition into the narrower flange plate. See Figure 15-2. This 3-inch shift makes it simpler to fit run-off tabs, weld and test the splice, and then grind off the run-off tabs.

For additional information on sizing flange plates, see Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration’s *Guidelines for Design for Constructability*, G12.1.

Field Splices

Use field splices to reduce shipping lengths, but minimize their number because they are expensive. The preferred maximum length of a field section is 80 feet; however, lengths up to 175 feet are possible, but do not use field sections greater than 120 feet without considering shipping, erection, and site constraints. For remote locations, a maximum length of 120 feet may be feasible, but should be verified. As a rule, the unsupported length in compression of the shipping piece divided by the minimum width of the flange in compression in that piece should be less than approximately 85. Good design practice is to reduce the flange cross sectional area by no more than approximately 25 percent of the area of the heavier flange plate at field splices to reduce the build-up of stress at the transition. For continuous spans, all of the field sections that cantilever over a pier should be of constant length to simplify erection.

Girder splices should be located at or near moment inflection points as practical. Splices at midspan of simple spans or at high moment regions of continuous spans are not permitted without prior approval of the Chief Bridge Engineer.

All steel girder splices must use traditional bolted splice plates for top and bottom flanges and webs. Assumed compression or bearing of girders or plates is not allowed.

Although steel girder segments are typically lighter than concrete, the bridge designer should contact the Alaska Measurement Standards and Commercial Vehicle **Compliance** (MS&CVC) to verify feasibility of transporting overweight, overheight, and overwidth bridge components.

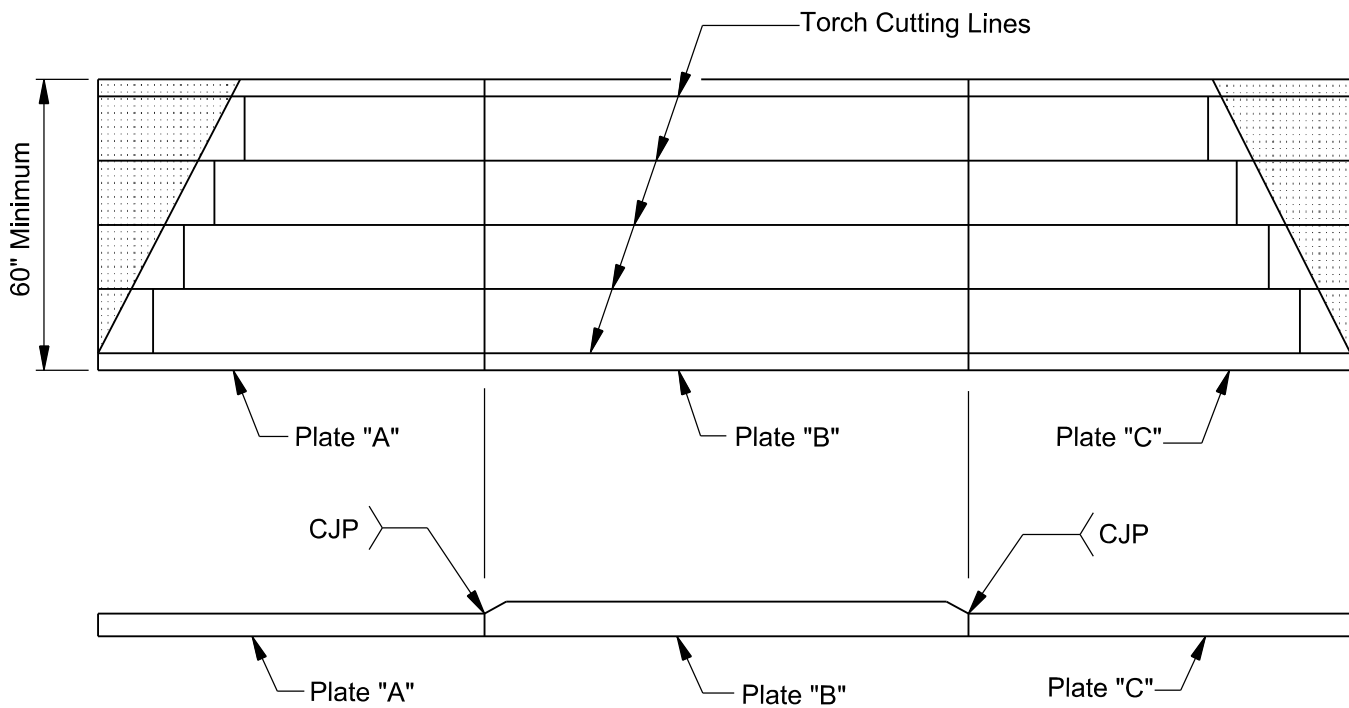


Figure 15-1
Grouping Flanges for Efficient Fabrication
 (From the AASHTO/NSBA Steel Bridge Collaboration)

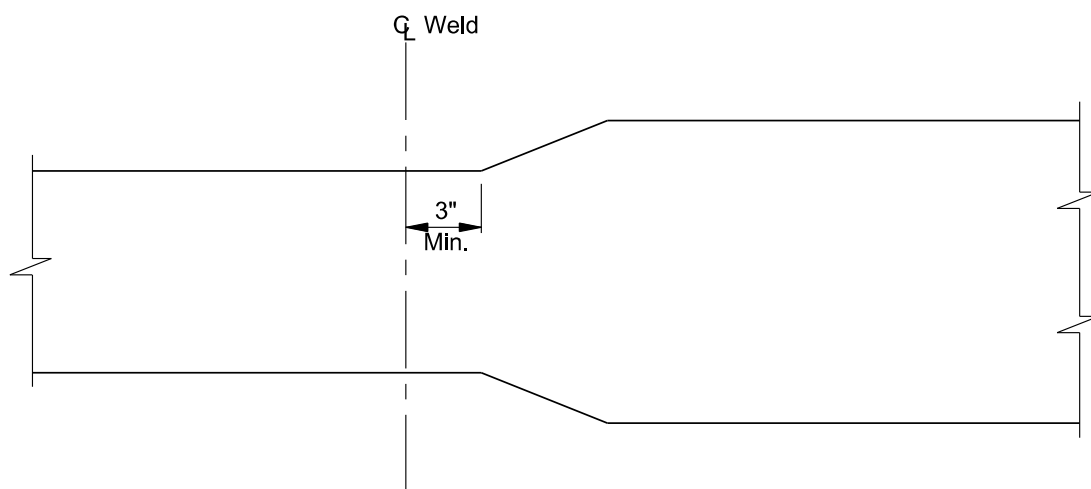


Figure 15-2
Flange Width Transition (Plan View)

Shop Splices

Include no more than two shop flange splices in the top or bottom flange within a single field section. In determining the points where changes in plate thickness occur within a field section, the designer should weigh the cost of groove-welded splices against extra plate area. Table 1.5.4-1 of the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design for Constructability*, G12.1, provides guidelines for weight savings for Grade 50 steel required to justify a flange shop splice.

In many cases, it may be advantageous to continue the thicker plate beyond the theoretical step-down point to avoid the cost of the groove-welded splice.

To facilitate testing of the weld, locate flange shop splices at least 2 feet from web splices, and locate flange and web shop splices at least 6 inches from transverse stiffeners.

Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration *Guidelines for Design for Constructability*, G12.1, provides additional guidance on shop splices.

Web Plates

Where there are no depth restrictions, optimize the web depth and girder cross section. Do not change web thickness at any splice by less than 1/16 inch. Maintain symmetry by aligning the centerlines of the webs at splices.

Web design can have a significant impact on the overall cost of a plate girder. Considering material costs in isolation, make girder webs as thin as design considerations will permit. However, this practice will not always produce the greatest economy because fabricating and installing transverse stiffeners is one of the most labor-intensive of shop operations. The following guidelines apply to the use of transverse stiffeners:

- Unstiffened webs are generally more economical for web depths approximately 48 inches or less.
- For web depths between 48 inches and 72 inches, consider options for a partially stiffened or an unstiffened web, with unstiffened webs preferred. A partially stiffened web is one where the web thickness is proportioned to be 1/16 inch less than that allowed by specification for an unstiffened web at a given depth and,

therefore, stiffeners are required only in areas of higher shear.

- Above 72 inches, consider options for partially stiffened or fully stiffened webs, with partially stiffened webs preferred. A fully stiffened web is one where stiffeners are present throughout the span.

Transverse Stiffeners

Flat bars (i.e., bar stock rolled to widths up to 8 inches at the mill) are typically more economical than plates for stiffeners. The stiffeners can be fabricated by shearing flat bars of the specified width to length. Proportion stiffeners that are intended to be fabricated from bars in ¼-inch increments in width and in ⅛-inch increments in thickness. Consult a fabricator for available flat bar sizes.

Longitudinally Stiffened Webs

Do not use longitudinally stiffened webs without the Chief Bridge Engineer's approval.

Minimum Thicknesses

The minimum thicknesses for components of structural steel plate girders are:

- webs: ½ inch
- flanges: 1 inch
- bearing stiffener plates: 1 inch
- gusset plates: ¾ inch
- angles/channels: ¼ inch

15.1.4. Falsework

Design steel superstructures without intermediate falsework during the placing of the concrete deck slab. Shored construction is not permitted.

15.1.5. AISC Certification Program

The AISC Certification Program for Structural Steel Bridge Fabricators has three levels of expertise: Standard for Steel Bridges (Simple), Intermediate Bridge, and Advanced Bridge. For all bridge members, except rolled shapes, DOT&PF requires Advanced Bridge level certification with a Fracture Critical Endorsement. For fabrication of rolled shapes DOT&PF requires Intermediate Bridge or Advanced Bridge level certification.

Currently, DOT&PF does not require certification for sign structures.

15.1.6. Buy America

Refer to the *Alaska Highway Preconstruction Manual* and *Alaska Standard Specifications* for the latest “Buy America” requirements.

15.2. Materials

Reference: LRFD Article 6.4

15.2.1. Structural Steel

Reference: LRFD Article 6.4.1

The following presents typical DOT&PF practices for the material type selection for structural steel members. Note that, for ASTM A709 and AASHTO M270, the grades are common to both designations.

Grade 36

Grade 36 steel is typically only used for secondary structural members on straight girder bridges, such as:

- transverse stiffeners,
- diaphragms,
- sole plates, and
- bearing plates.

Grade 36 steel is becoming less used and, thus, less available at times. Generally, there is little or no cost difference between Grade 50 and Grade 36 steel. Wherever possible, allow substitution of Grade 50 for Grade 36 steel.

Grade 50 and 50W

Grade 50 steel is the most commonly used grade. It is typically used for the primary and secondary members such as:

- rolled beams,
- welded girders,
- splice plates,
- diaphragms,
- bearing plates,
- sole plates, and
- stiffeners.

High-Performance Steel

Do not use Grade HPS70W and HPS100W steel without the Chief Bridge Engineer's approval.

Corrosion-Protection Coatings

The following summarizes DOT&PF policies for the use of corrosion-protection coatings:

1. **Remote Sites.** Use metalizing (spray thermal metal coating) or hot-dip galvanizing for all steel bridges to be erected in remote sites to provide a relatively maintenance-free coating. Use metalizing where field sections are greater than 50 feet in length. For field sections less

than 50 feet in length, hot-dip galvanizing may be used. See Chapter 20 for more discussion.

2. **All Other Sites.** For all other steel bridges, use galvanized steel with or without a painted top coat unless site conditions suggest that unpainted weathering steel will work. See FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures," October 3, 1989. Historically, DOT&PF has only considered the use of unpainted weathering steel (i.e., Grade 50W) in the interior Alaska environment, which is dry.

The use of unpainted weathering steel is not appropriate in all environments and at all locations. The most prominent disadvantage of weathering steel is that it does not work well in coastal maritime environments. The inevitable staining of the substructure prior to the placement of the bridge deck due to weathering of the unpainted steel creates a poor image (i.e., one of lack of proper maintenance) to the traveling public. In addition, do not use weathering steel at locations where the following conditions exist:

1. **Environment.** Do not use in industrial areas where concentrated chemical fumes may drift onto the structure, or where the nature of the environment is otherwise undesirable.
2. **Water Crossings.** Do not use over bodies of water where the clearance over the ordinary high water is 10 feet or less.
3. **Grade Separations.** Do not use for highway grade separation structures.
4. **Coastal.** Do not use over saltwater or within 100 miles of the coast.

For additional guidance on the appropriate application and detailing of unpainted weathering steel, see the AISI publication "Performance of Weathering Steel in Highway Bridges: A Third Phase Report."

Charpy V-Notch Fracture Toughness

Reference: LRFD Article 6.6.2

Apply Temperature Zone 3 when using LRFD Table 6.6.2.1-2 for the State of Alaska.

All steel members require charpy V-notch fracture-toughness verification, and must be tested for non-fracture-critical (T) impact testing requirements unless fracture-critical (F) impact testing requirements are specified.

15.2.2. Bolts

Reference: LRFD Article 6.4.3

Type

For normal construction, high-strength bolts will be:

- **Galvanized or Metalized Steel:** Use 7/8-inch or 1-inch ASTM F3125, Grade A325, Type 1 mechanical galvanized with galvanized direct tension indicating washers (DTI).
- **Weathering Steel:** Use 7/8-inch or 1-inch ASTM F3125, Grade A325, Type 3 with weathering DTIs.

Twist-off bolts may be used, but DTIs are still required.

Hole Size

Typically, use a standard hole size. Do not use oversized or slotted holes, except in unusual circumstances. For galvanized girders, use standard holes and ream to the original size after galvanizing.

15.2.3. Splice Plates

In all cases, for all splice and filler plates, use the same steel as used in the web and flanges of plate girders.

15.3. Horizontally Curved Members

Reference: LRFD Articles 6.10 and 6.11

15.3.1. General

Wherever practical use chorded straight girders for curved alignments. Do not use curved girders, unless approved by the Chief Bridge Engineer.

The *LRFD Specifications* includes horizontally curved girders as a part of the provisions for proportioning I-shaped and tub girders at both the Strength and Service limit states. In addition, the *LRFD Specifications* specify analysis methodologies that detail various required levels of analysis.

15.3.2. Diaphragms, Bearings and Field Splices

The use of horizontally curved steel members requires the consideration of many factors that differ from straight girders including but not limited to:

1. **Cross Frames/Diaphragms.** All curved steel simple-span and continuous-span bridges must have diaphragms directed radially except end diaphragms, which should be placed parallel to the centerline of bearings. Cross frames and diaphragms should be as close as practical to the full depth of the girders.
2. **Load Considerations.** Design all diaphragms, including their connections to the girders, to carry the total transferred load at each diaphragm location. Design cross frame and diaphragm connections for the 75 percent and average load provisions of LRFD Article 6.13.1, unless actual forces in the connections are determined from an appropriate structural model. Using the provisions of LRFD Article 6.13.1 may result in very large connections that are difficult to detail.
3. **Charpy V-Notch Testing.** Consider cross frames and diaphragms to be primary members.
4. **Expansion/Contraction.** Bridges expand and contract in all directions. For typical bridges that are long in relationship to their width (say $2\frac{1}{2}$ times the width), ignore the transverse expansion. For ordinary geometric configurations where the bridge length is long relative to the bridge width and the degree of curvature is moderate (those satisfying the requirements of LRFD Article 4.6.1.2.4b), no additional consideration is necessary for the

unique expansion characteristics of horizontally curved structures. Wide, sharply curved, or long-span structures may require the use of high-load, multi-rotational bearings. Consider providing restraint either radially and/or tangentially to accommodate the transfer of seismic forces and the thermal movement of the structure because the bridge tries to expand in all directions.

5. **Flange Splices.** Design the splices in flanges of curved girders to carry lateral bending stresses and vertical bending stresses in the flanges.

15.4. Fatigue Considerations

Reference: LRFD Article 6.6

LRFD Article 6.6.1 categorizes fatigue as either “load induced” or “distortion induced.” Load induced is a “direct” cause of loading. Distortion induced is an “indirect” cause in which the force effect, normally transmitted by a secondary member, may tend to change the shape of or distort the cross section of a primary member.

15.4.1. Load-Induced Fatigue

General

LRFD Article 6.6.1.2 provides the framework to evaluate load-induced fatigue, which is determined by:

- the stress range induced by the specified fatigue loading at the detail under consideration;
- the number of repetitions of fatigue loading a steel component will experience during its 75-year design life. For lower truck-traffic volume bridges, such as those that occur in Alaska, this is determined by using the actual anticipated truck volumes; and
- the nominal fatigue resistance for the Detail Category being investigated.

DOT&PF Policy

Where practical, design for infinite life. Avoid the use of steel bridge details with fatigue resistances lower than Detail Category C' (i.e., Detail Categories D, E, and E'). Do not provide empty bolt holes.

The Chief Bridge Engineer must approve any exceptions to DOT&PF policy on load-induced fatigue.

15.4.2. Distortion-Induced Fatigue

LRFD Article 6.6.1.3 provides specific detailing practices for transverse and lateral connection plates intended to reduce significant secondary stresses that could induce fatigue crack growth. The provisions of the *LRFD Specifications* are prescriptive and require no mathematical computation of stress range.

15.4.3. Other Fatigue Considerations

In addition to the considerations in Sections 15.4.1 and 15.4.2, investigate the fatigue provisions in other Articles of Chapter 6 of the *LRFD Specifications*. These include:

- Fatigue and Fracture Limit State — LRFD Articles 6.5.3, 6.10.5, and 6.11.5.
- Fatigue at shear connectors — LRFD Articles 6.10.10.1.2 and 6.10.10.2.
- Bolts subject to axial-tensile fatigue — LRFD Article 6.13.2.10.3.

15.5. General Detail Requirements

Reference: LRFD Article 6.7

15.5.1. Deck Haunches

See Section 16.2.3 for design and detailing of haunches.

15.5.2. Camber

Where dead load deflection and vertical curve offset are greater than the minimum of 3/4 inch or 1/8 inch per 10 feet of segment length, provide girders with a compensating camber. Camber the entire girder length as required by the loading and profile grade. Calculate camber to the nearest 0.01 inch, with ordinates at variable locations as needed throughout the length of the girder. Show the required camber values from a chord line that extends from point of support to point of support.

Provide a camber diagram in all contract documents with structural steel girders. Include a camber diagram with ordinates as follows:

- steel DL deflection (DC),
- non-composite externally applied DL deflection (DC),
- superimposed DL deflection (DC),
- wearing surface deflection (DW) and
- total DL deflection,
- geometric camber.

The designer may show any additional information if desired (e.g., ordinates at field splices).

15.5.3. Diaphragms and Cross Frames

Reference: LRFD Articles 6.7.4 and 6.6.1.3.1

Diaphragms and cross frames are vitally important in steel girder superstructures. They stabilize the girders in the positive-moment regions during construction and in the negative-moment regions after construction. Diaphragms and cross frames also serve to distribute gravitational, centrifugal, and wind loads. Determine the spacing of diaphragms and cross frames based upon the provisions of LRFD Article 6.7.4.1. As with most aspects of steel girder design, the design of the spacing of diaphragms and cross frames is iterative. A good starting point is the traditional diaphragm and cross frame spacing of 25 feet with cross frames at the maximum and minimum movement locations. Most economical steel girder designs will use spacings typically greater than 25 feet in the positive-moment regions.

The following applies to diaphragms and cross frames:

1. **Location.** Place diaphragms or cross frames at each support and throughout the span at an appropriate spacing. Plan the location of the field splices to avoid conflict between the connection plates of the diaphragms or cross frames and any part of the splice material.
2. **Skew.** For up to a skew angle of 20 degrees, all intermediate diaphragms and cross frames may be placed perpendicular to the girders. Locating diaphragms and cross frames near girder supports on bridges with high skews requires careful consideration. When locating a diaphragm and cross frame between two girders, the relative stiffness of the two girders must be similar. Otherwise, the diaphragm and cross frame will act as a primary member supporting the more flexible girder. This may be unavoidable on bridges with exceptionally high skews where a rational analysis of the structural system will be required to determine actual forces.
3. **End Diaphragms.** Place end diaphragms along the centerline of bearing. Set the top of the diaphragm below the top of the girder to accommodate the joint detail and the thickened slab at the end of the superstructure deck, where applicable. Design the end diaphragms to support the edge of the slab including live load plus impact.
4. **Interior Support Diaphragms and Cross Frames.** Generally, place interior support diaphragms and cross frames along the centerline of bearing. They provide lateral stability for the bottom flange and bearings.
5. **Curved-Girder Structures.** Design diaphragms or cross frames connecting horizontally curved girders as primary members and orient them radially.
6. **Detailing.** Typically, detail diaphragms and cross frames to follow the cross slope of the deck; i.e., the diaphragm or cross frame is parallel to the bottom of the deck. This allows the fabricator to use a constant drop on each connection plate (i.e., the distance from the bottom of the flange to the first bolt hole on the connection plate is constant). The contract

documents should allow the contractor to use diaphragms or cross frames fabricated as a rectangle (as opposed to a skewed parallelogram). In this case, the drops vary across the bridge.

The following identifies typical DOT&PF practices on the selection of diaphragms and cross frames:

1. **Intermediate Diaphragms.** Preferably, use X-frames as intermediate diaphragms in I-girder bridges as shown in Figure 15-3. For relatively wide girder spacings relative to the girder depth (i.e., aspect ratio ≥ 1.5), a K-frame (as shown in Figure 15-4) may be more appropriate than an X-frame for intermediate diaphragms. Figure 15-5 shows a typical intermediate diaphragm for a box girder bridge, where a channel and the transverse stiffeners act as an intermediate diaphragm, which aids in fabrication and construction.
2. **End Diaphragms.** Preferably, use solid diaphragms as end diaphragms in I-girder bridges where possible, as shown in Figure 15-6. Figures 15-7 and 15-8 illustrate typical end diaphragms for box girder bridges for a section at an abutment and a section over a pier, respectively.

15.5.4. Lateral Bracing

Reference: LRFD Article 6.7.5

Lateral bracing typically consists of trussing members oriented in the horizontal plane placed along the bottom portion of steel I-girder bridges. The *LRFD Specifications* requires that the need for lateral bracing be investigated for all stages of assumed construction procedures. If the bracing is included in the structural model used to determine force effects, then design the lateral bracing for all applicable limit states.

In general, lateral bracing is not required in the vast majority of steel I-girder bridges (short through medium spans); however, the designer must check this. Typical diaphragms and cross frames will transfer lateral loads adequately to eliminate the need for lateral bracing.

For box girders, internal top lateral bracing is more typical. Employ steel stay-in-place forms attached with a structural weld as lateral bracing, with approval. See Figure 15-8.

LRFD Article 4.6.2.7 provides various alternatives relative to lateral wind distribution in multi-girder bridges.

15.5.5. Inspection Access (Box Girders)

Detail all new steel box girder bridges with access openings to allow inspection of the girder interior. Provide one access opening at each end of all box girders. Do not locate access openings over travel lanes or railroad tracks and, preferably, not over shoulders. Locate these such that the general public cannot gain easy entrance.

Access openings are typically placed in the webs of box girders; however, they may be placed in the bottom flange provided the total factored flange stress is less than 10 ksi. The dimensions of the access opening should be a minimum 30 inches by 30 inches square.

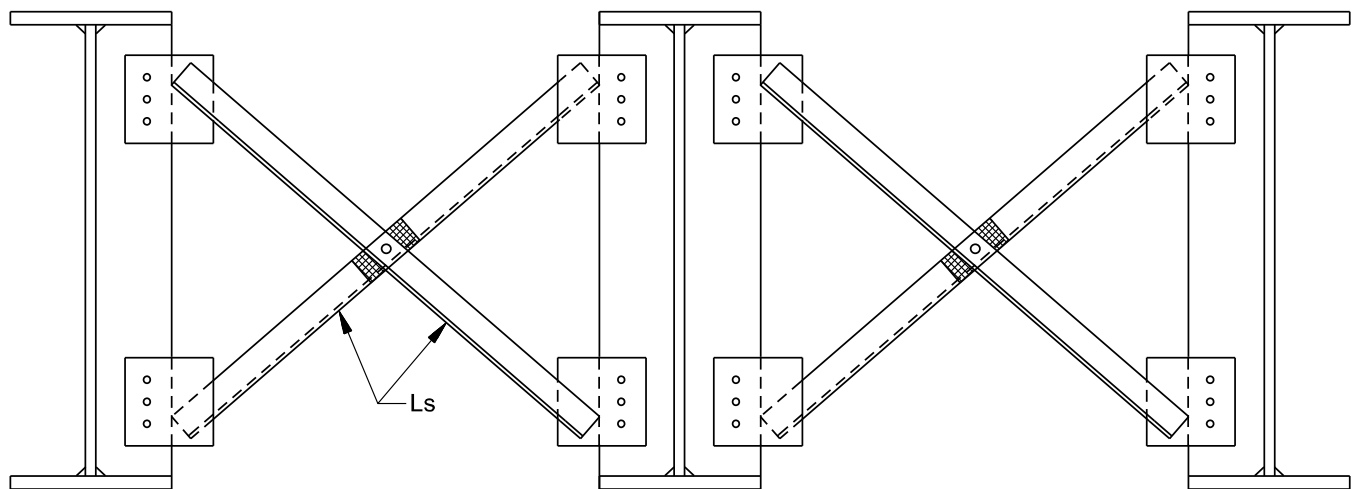


Figure 15-3
Typical Intermediate Diaphragm for an I-Girder Bridge (X-Frame)

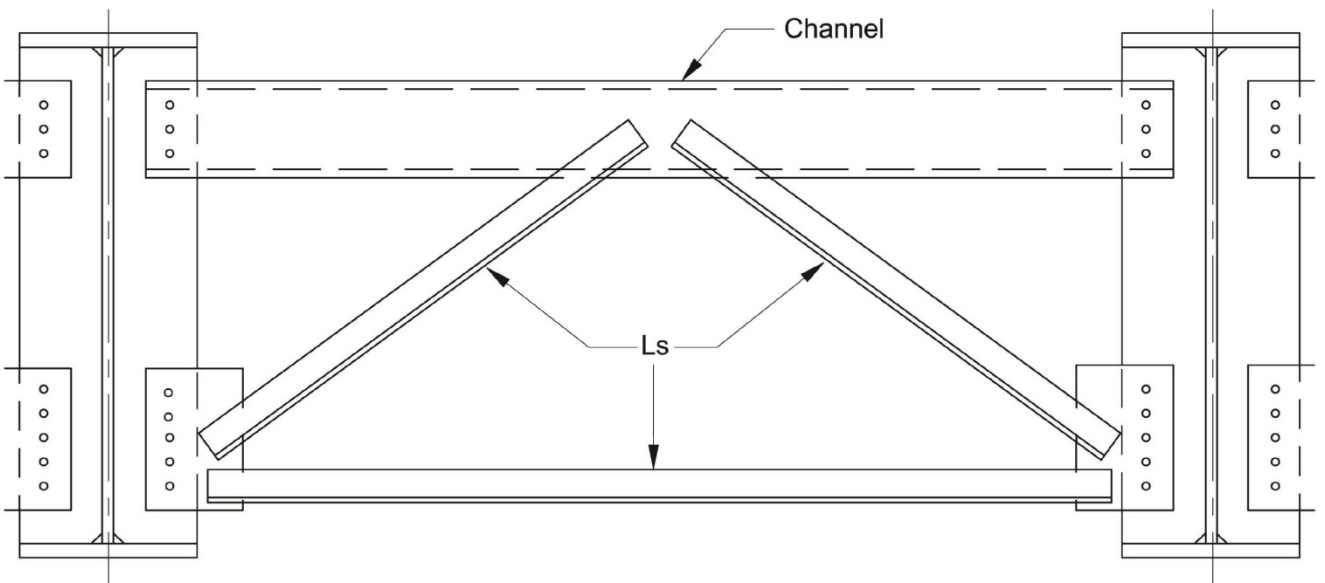


Figure 15-4
Typical Intermediate Diaphragm for an I-Girder Bridge (K-Frame)

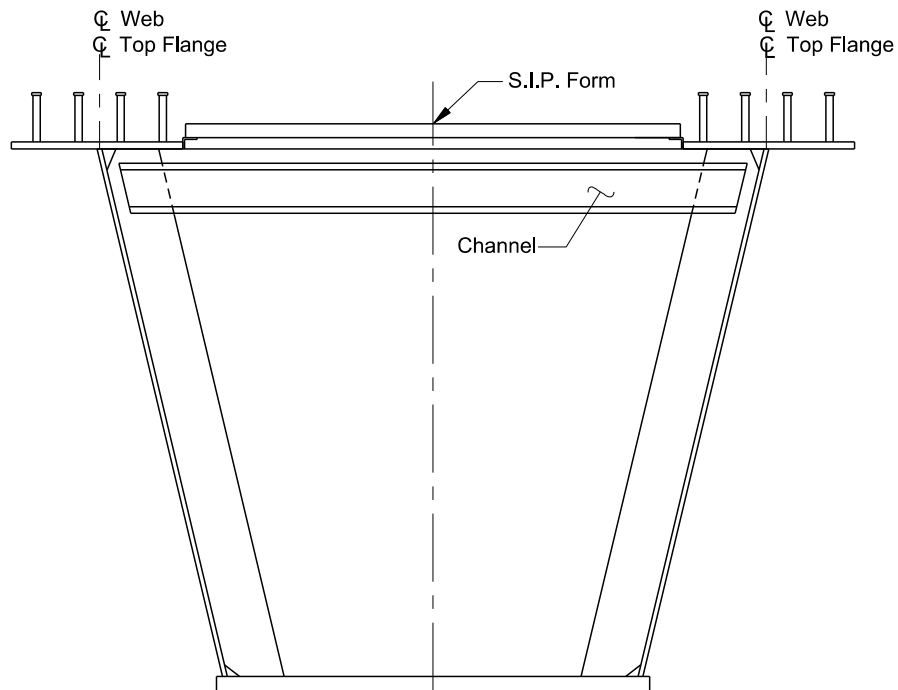


Figure 15-5
Typical Intermediate Diaphragm for a Straight, Square Box-Girder Bridge

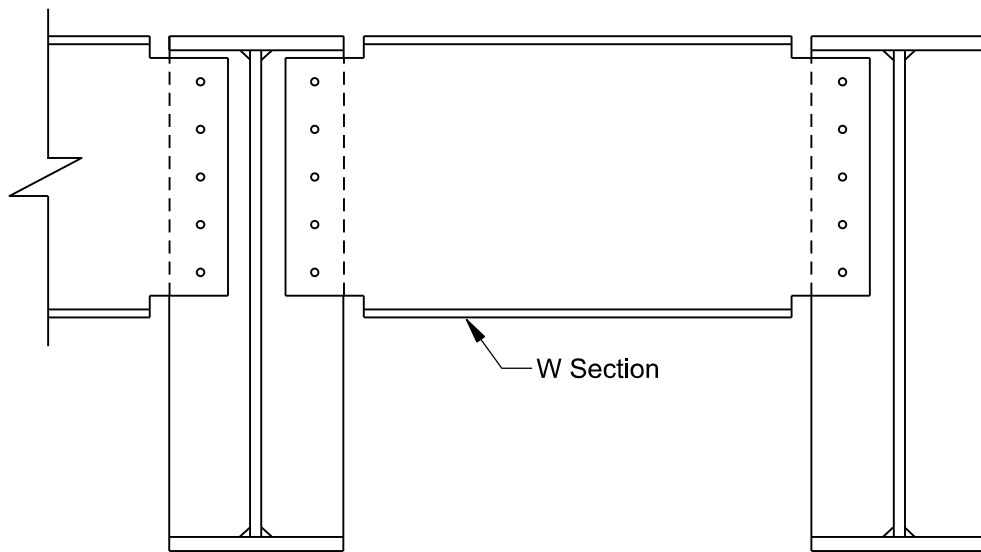


Figure 15-6
Typical End Diaphragm for a Rolled Beam or Shallow Plate I-Girder Bridge

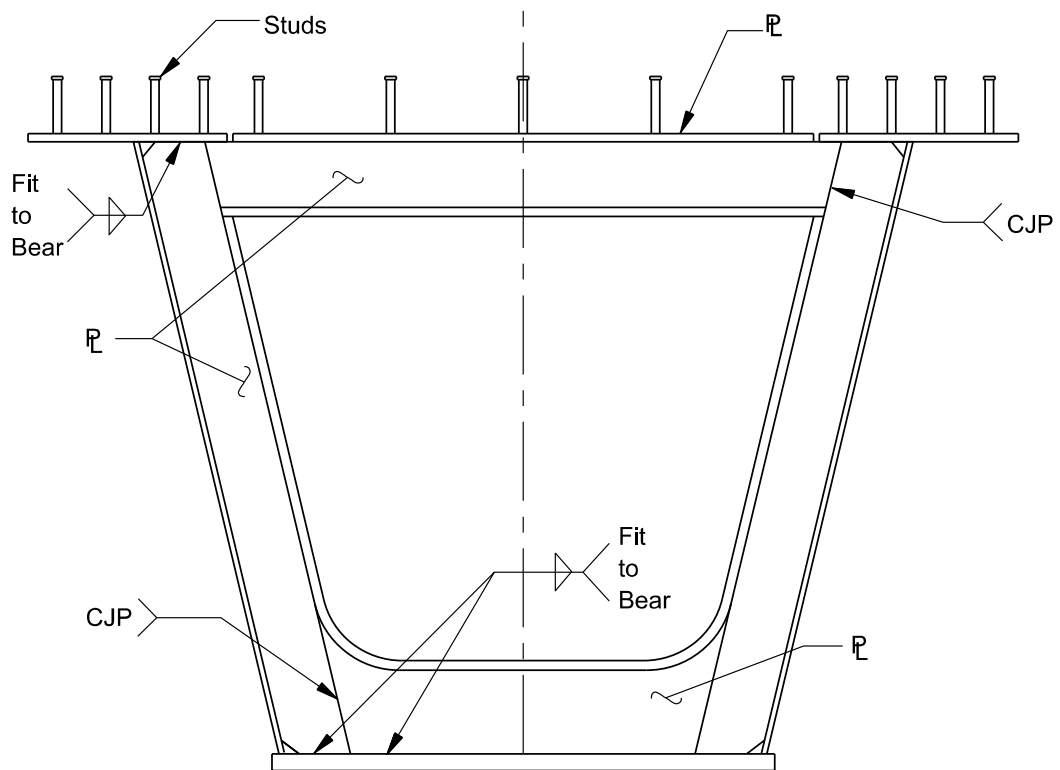


Figure 15-7
Typical End Diaphragm for a Box-Girder Bridge at an Abutment

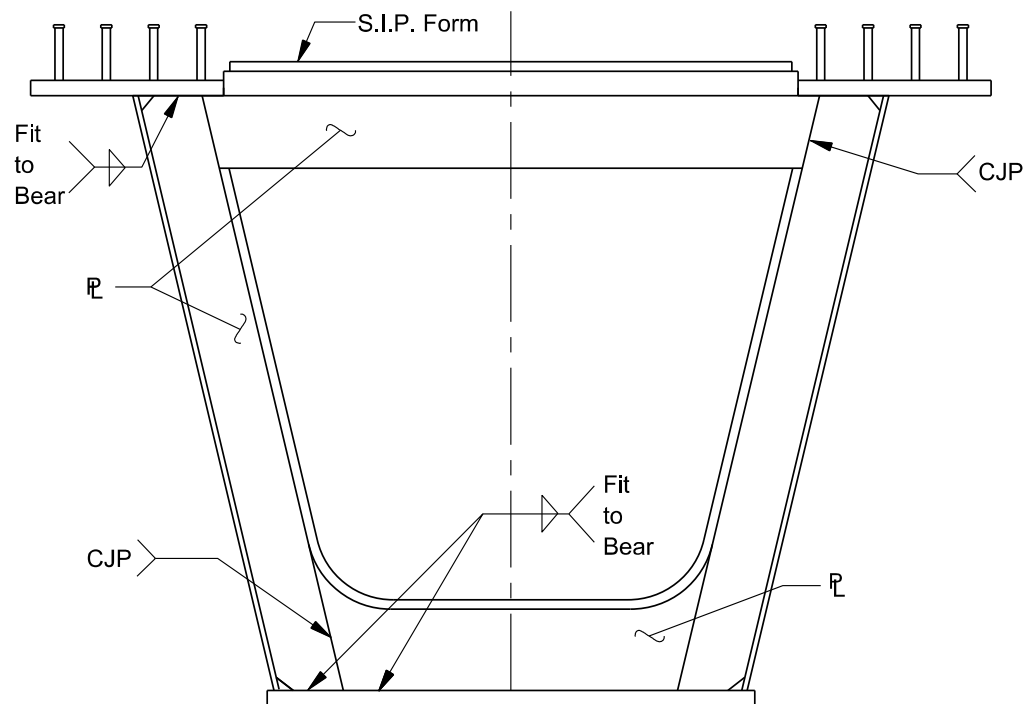


Figure 15-8
Typical End Diaphragm for a Box-Girder Bridge over a Pier

15.6. I-Sections in Flexure

Reference: LRFD Article 6.10

15.6.1. Limit States

Reference: LRFD Article 6.10.1

Positive-Moment Region Maximum-Moment Section

For a composite girder, consider the positive-moment region maximum-moment section to be compact in the final condition (see LRFD Article 6.10.7.1). The cured concrete deck in the positive-moment region provides a large compression flange that laterally braces the top flange. Very little, if any, of the web is in compression, but it should be checked.

Top Flange (Compression Flange). In the final condition after the deck has cured, the top flange adds little to the resistance of the cross section. During curing of the concrete deck, however, the top flange is very important. The Strength limit state during construction when the concrete is not fully cured may govern the design of the top flange in the positive-moment region, as specified in LRFD Article 6.10.3.4.

Bottom Flange (Tension Flange). The bottom flange, if properly proportioned, is not governed by the construction phase. The bottom flange is typically governed by the final condition. The Service II load combination permanent deformation provisions of LRFD Article 6.10.4.2 may govern and must be checked.

Negative-Moment Region Pier Section

The negative-moment region pier section will most likely be a non-compact section during all conditions. The concrete deck over the pier is in tension in the negative-moment region and, thus, considered cracked and ineffective at the nominal resistance (i.e., Strength load combinations). Thus, a good portion of the steel cross section is in compression. To qualify as compact, the web usually must be too thick to be cost effective. Thus, the cost-effective section will typically be a non-compact section.

Both top and bottom flanges in the negative moment region are typically governed by the Strength limit state in the final condition. Furthermore, the bottom flange in compression is typically governed by the location of the first intermediate diaphragm off the pier because it provides the discrete bracing for the flange.

15.6.2. Shear Connectors

Reference: LRFD Article 6.10.10

The preferred size for shear studs for use on the flanges of girders and girders is $\frac{7}{8}$ inch diameter. Additional requirements are in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

15.6.3. Stiffeners

Reference: LRFD Article 6.10.11

Transverse Intermediate Stiffeners

Reference: LRFD Article 6.10.11.1

Design straight girders without intermediate transverse stiffeners, if economical. If stiffeners are required, provide fascia girders with stiffeners only on the inside face of the web for aesthetics. Due to the labor intensity of welding stiffeners to the web, the unit cost of stiffener by weight is approximately nine times that of the unit cost of the web by weight. It is seldom economical to use the thinnest web plate permitted; therefore, investigate the use of a thicker web and fewer intermediate transverse stiffeners, or no intermediate stiffeners at all. If the design requires stiffeners, the preferred width of the stiffener is one that can be cut from commercially produced bar stock.

Weld intermediate transverse stiffeners on both sides of the compression flange. Do not weld transverse stiffeners to tension flanges. The distance between the end of the web-to-stiffener weld and the near toe of the web-to-flange fillet weld should be between $4t_w$ and $6t_w$.

The width of the projecting stiffener element, moment of inertia of the transverse stiffener, and stiffener area must satisfy the requirements of LRFD Article 6.10.11.1.

Orient transverse intermediate stiffeners normal to the web. However, where the angle of crossing is between 70 degrees and 90 degrees, skew the stiffeners so that the diaphragms or cross frames may be connected directly to the stiffeners.

Bearing Stiffeners

Reference: LRFD Article 6.10.11.2

Provide full-height bearing stiffeners for all plate girders to prevent the possibility of web buckling for both temporary and permanent supports. Provide bearing stiffeners on both web faces and at the bearing points of rolled beams and plate girders. Design the weld connecting the bearing stiffener to the web to

transmit the full bearing force from the stiffener to the web due to the factored loads.

15.6.4. Deck-Overhang Cantilever Brackets

Reference: LRFD Article 6.10.3

During construction, the deck overhang brackets may induce twist in the exterior girder. The contract documents must require the contractor to check the twist of the exterior girder and bearing of the overhang bracket on the web. See Figure 15-9.

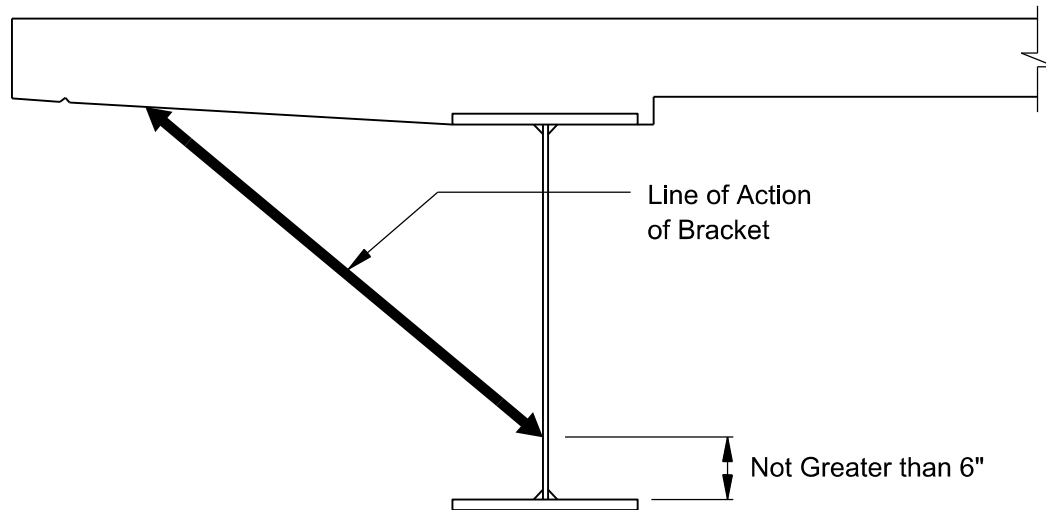


Figure 15-9
Schematic of Location for Deck Overhang Bracket

15.7. Connections and Splices

Reference: LRFD Article 6.13

15.7.1. Bolted Connections

Reference: LRFD Article 6.13.2

Design

Design all bolted connections as slip-critical at the Service II limit state, except for secondary bracing members.

15.7.2. Welded Connections

Reference: LRFD Article 6.13.3

Welding Process

The governing specification for welding new steel bridge girders is the ANSI/AASHTO/AWS *Bridge Welding Code D1.5*. However, this specification does not provide control over all of the welding issues that may arise on a project. Consult the following additional reference specifications as needed:

- AWS D1.1 for welding of tubular members and strengthening or repair of existing structures, and
- AWS D1.4 if the welding of reinforcing steel must be covered by a specification.

Include testing frequency requirements in the contract documents when not already specified by AWS or the *Alaska Standard Specifications for Highway Construction*.

Coordinate with the Chief Bridge Engineer for any issues related to the interpretation of, application for, etc., on the welding specifications. Use prequalified welds as much as possible.

Welding Types and Symbols

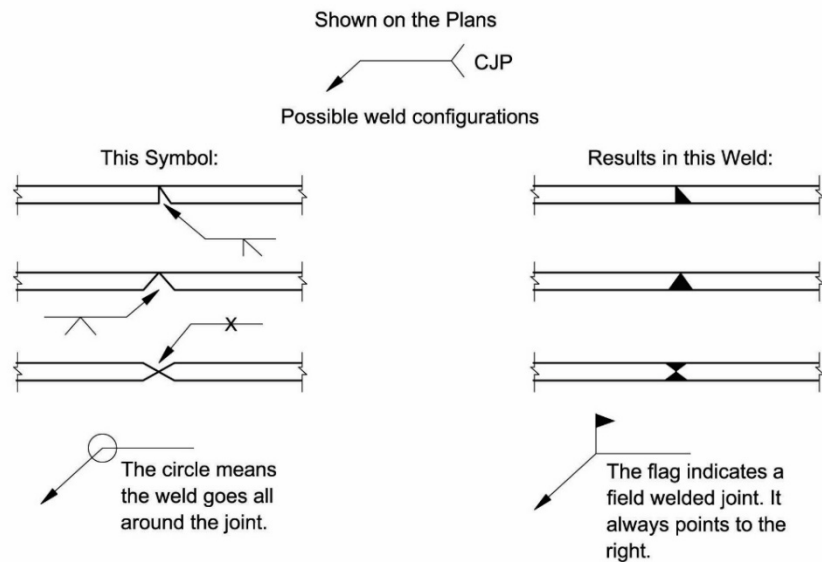
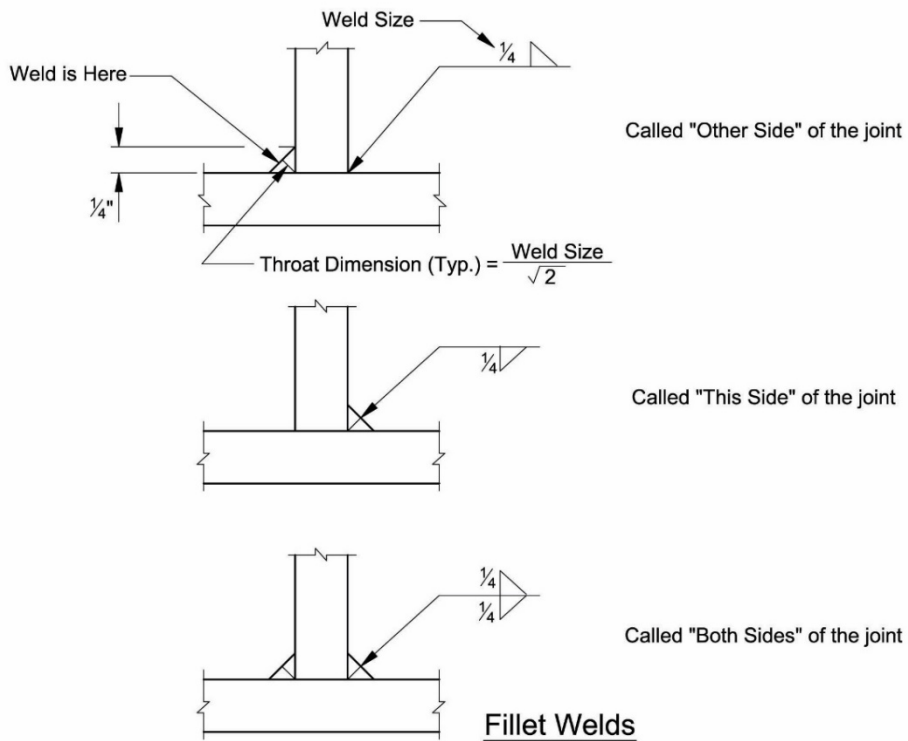
The primary types of welds used in bridge fabrication are fillet welds and groove welds. Welding symbols must comply with AWS A2.4 “Standard Symbols for Welding, Brazing, and Non-Destructive Examination.” Welding symbols provide an instruction on the type, size, and other characteristics of the desired weld. When these symbols are properly used, the meaning is clear and unambiguous. If not used exactly as prescribed, the meaning may be ambiguous, leading to problems for all involved. The *AISC Steel Construction Manual* and most steel design textbooks have examples of welding symbols that, although technically correct, are more complicated than the typical bridge designer needs. With minor modifications, the examples in Figure 15-10 will suffice for the majority of bridge fabrication circumstances.

Field Welding

Prohibit field welding for all but a few special applications. These permissible applications are:

- welded splices for piles
- connecting pile tips to piles
- shear stud connections
- sole plates to bottom flange plates

Do not permit direct welding of stay-in-place (SIP) deck forms to girder flanges. Weld metal forms to a strap or angle that is placed over the flange.



"This Side" and "Other Side" welds are the same size unless specified otherwise.

Symbols apply between abrupt changes in direction of welding unless governed by the "All-Around Symbol" or otherwise dimensioned.

CJP Groove Welds

Figure 15-10
Welding Symbols

Weld-Metal Strength and Electrode Nomenclature

The strength of the weld filler metal is known from the electrode designation. Figure 15-11 illustrates the standard nomenclature to identify electrodes. The figure represents more than a bridge designer typically needs to know but, as an illustration, is informative. This applies to electrodes with a weld-metal tensile strength of 70,000 psi and the indicated welding procedures for all positions of welding or only flat and horizontal positions, respectively.

To make a weld of sufficient strength, the designer must consider three variables — weld length, weld throat, and weld-metal strength. Because weld strength is a function of these three variables, many possible combinations can yield sufficient weld strength. Full-penetration groove welds in tension require matched welds because the length and throat of the weld also match the dimensions of the base metal.

Design of Welds

The design of fillet welds is integral to LRFD Section 6 on Steel Design. The *LRFD Specifications* addresses topics such as resistance factors for welds, minimum weld size, and weld details to reduce fatigue susceptibility.

The weld-strength calculations of LRFD Section 6 assume that the strength of a welded connection is dependent only on the weld metal strength and the area of the weld. Weld metal strength is a self-defining term. The area of the weld that resists load is a product of the theoretical throat multiplied by the length. The theoretical weld throat is the minimum distance from the root of the weld to its theoretical face. See Figure 15-10.

These digits indicate the following:	
Exx1z	All positions of welding
Exx2z	Flat and horizontal positions
Exx3z	Flat welding positions only
These digits indicate the following:	
Exx10	DC, reverse polarity
Exx11	AC or DC, reverse polarity
Exx12	DC straight polarity, or AC
Exx13	AC or DC, straight polarity
Exx14	DC, either polarity or AC, iron powder
Exx15	DC, reverse polarity, low hydrogen
Exx16	AC or DC, reverse polarity, low hydrogen
Exx18	AC or DC, reverse polarity, iron powder, low hydrogen <i>This is most common.</i>
Exx20	DC, either polarity, or AC for horizontal fillet welds; and DC either polarity, or AC for flat position welding
Exx24	DC, either polarity, or AC, iron powder
Exx27	DC, straight polarity, or AC for horizontal fillet welding; and DC, either polarity, or AC for flat position welding, iron powder
Exx28	AC or DC, reverse polarity, iron powder, low hydrogen

Note: The “xx” shown above is a two-digit number indicating the weld metal tensile strength in 1000 psi increments. For example, E7018 is 70,000 psi.

Figure 15-11
Electrode Nomenclature

Fillet welds resist load through shear on the throat, while groove welds typically resist load through tension, compression, or shear depending upon the application.

Often, it is best to only show the type and sizes of the weld required and leave the details for the specific weld geometry to the fabricator.

When considering design options, note that the most significant factor in the cost of a weld is the volume of the weld material that is deposited. Over-sizing a welded joint is unnecessary and uneconomical. A single-pass weld is one made by laying a single weld bead in a single move of the welder along the joint. A multiple-pass weld is one in which several beads are laid one upon the other in multiple moves along the joint. Welds sized in a single pass are preferred because these are most economical and least susceptible to resultant discontinuities. The maximum weld size for a single-pass fillet weld is 5/16 inch. The AWS D1.1 *Structural Welding Code*, Table 3.7, provides more specific maximum single-pass fillet-weld sizes for various welding processes and positions of welding. Design the weld economically, but its size should not be less than 1/4 inch and, in no case, less than the requirements of LRFD Article 6.13.3.4 for the thicker of the two parts joined. Show the weld terminations.

The following types of welds for girders are prohibited:

- field-welded girder splices
- intersecting welds
- intermittent fillet welds
- partial penetration groove welds

Provide careful attention to the accessibility of welded joints. Provide sufficient clearance to enable a welding rod to be placed at the joint. Often, a large-scale sketch or an isometric drawing of the joint will reveal difficulties in welding or where critical weld stresses must be investigated.

Inspection and Testing

A systematic program of inspection and testing is indispensable to the reliable use of welding. Inspection is done at the shop and at the field site. The function of the inspection is to guarantee the use of specified materials and procedures under conditions where proper welding is possible. If the sequence of welding has been specified, the inspector should be able to certify conformance.

Despite careful inspection, weld discontinuities may escape detection unless all or part of the work is subjected to tests. There are two broad categories of testing — destructive testing, which is used very sparingly for big problems or forensic studies, and nondestructive testing, which is used routinely to guarantee the quality of the welds. DOT&PF uses the following types of non-destructive evaluation (NDE):

1. **Radiographic Testing (RT).** Used to find cracks and inclusions after a weld is completed. The process involves placing film on one side of the weld and a source of gamma or x-rays on the other side of the weld. Shadows on the exposed film indicate cracks or inclusions in the welds or adjacent areas. Use RT for groove-weld inspection. It is most effective when both sides of the weld are accessible.
2. **Ultrasonic Testing (UT).** Relies on the reflection patterns of high-frequency sound waves, which are transmitted at an angle through the work. Cracks and defects interrupt the sound transmission, altering the display on an oscilloscope. UT can reveal many defects that the other methods do not, but it relies very heavily on the interpretative skill of the operator. Use UT for the inspection of groove welds and is preferred when a permanent record is desired.
3. **Magnetic Particle Testing (MT).** Performed by covering the surface of a weld with a suspension of ferromagnetic particles and then applying a strong magnetic field. Cracks interrupt the magnetic force lines, causing the particles to concentrate near the crack in patterns easily interpreted by the inspector. Use MT for inspecting fillet welds.
4. **Dye Penetrant Testing (DP).** Use a dye in liquid form to detect cracks. Capillary tension in the liquid causes the dye to penetrate into the crack, remaining behind after the surface is clean. Use DP to locate surface flaws in and around fillet welds.
5. **Visual Testing (VT).** Required on all welds.

To aid the inspector, the contract documents for continuous structures must identify the location of tension regions along both the top and bottom girder flanges. Show the length of each stress region and

reference these regions to the point of support. Figure 15-12 illustrates the information required.

15.7.3. Splices

Reference: LRFD Article 6.13.6

Shop Splices

In addition to the provision of LRFD Article 6.13.6, the following applies to shop splices:

1. **Location.** Numerous groove welds and/or groove welds located in high stress regions are not desirable. Locate flange shop splices away from high moment regions and web splices away from high shear regions. This is simple for flange splices in negative moment regions but more difficult with positive moment regions. In positive moment areas, the magnitude of moment does not change quickly along the girder compared to the negative moment.

The location of shop groove splices is normally dependent upon the length of plate available to the fabricator. This length varies depending upon the rolling process. Most plates are available (e.g., Grades 36 and 50) in lengths up to 80 feet depending on thickness. Consider the cost of adding a shop-welded splice instead of extending a thicker plate when designing members. Discuss these issues with a fabricator or the NSBA during design.

2. **Welded Shop Splice.** Figure 15-13 illustrates welded flange splice details. At flange splices, the thinner plate should not be less than one-half the thickness of the thicker plate. See LRFD Article 6.13.6.2 for more information on splicing different thicknesses of material using groove welds.

Field Splices

In addition to the provisions of LRFD Article 6.13.6, the following applies to field splices:

1. **Welded Field Splices.** These are prohibited.
2. **Location.** In general, locate field splices in main girders at low-stress areas and near the points of dead-load contraflexure for continuous spans. Long spans may require that field splices be located in high moment areas.
3. **Bolts.** Calculate design loads for bolts by an elastic method of analysis. Provide at least two

lines of bolts on each side of the web splice and four lines in flange splices.

4. **Composite Girder.** If a compositely designed girder is spliced at a section where the moment can be resisted without composite action, design the splice as non-composite. If composite action is necessary to resist the loads, design the splice for the forces due to composite action.
5. **Design.** Design bolted splices to satisfy both the slip-critical criteria under Service II loads and the bearing-type connection criteria under the appropriate Strength limit states.
6. **Swept Width (or shipping width) for Horizontally Curved Girders.** The swept width is the horizontal sweep in a curved girder plus its flange width. Locate field splices such that the maximum swept width for a horizontally curved girder is 10 feet within a single field section. Otherwise special hauling/shipping requirements apply at a substantial cost.

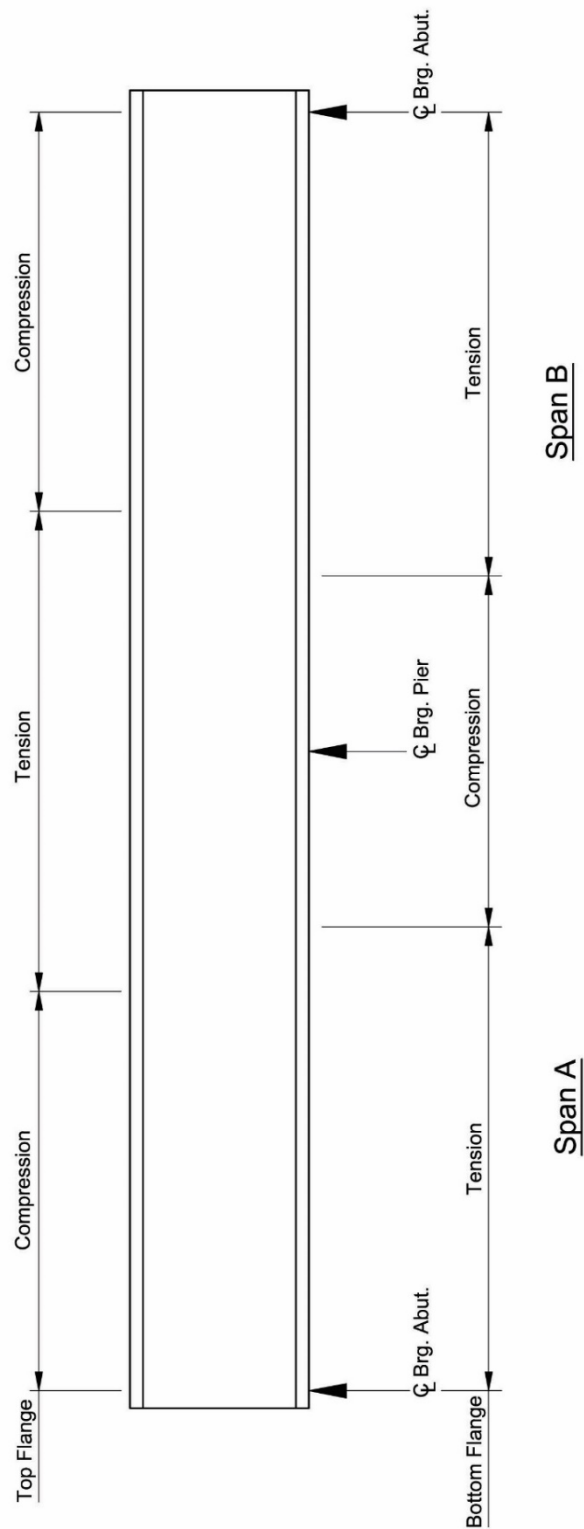


Figure 15-12
Schematic of Flange Tension Regions

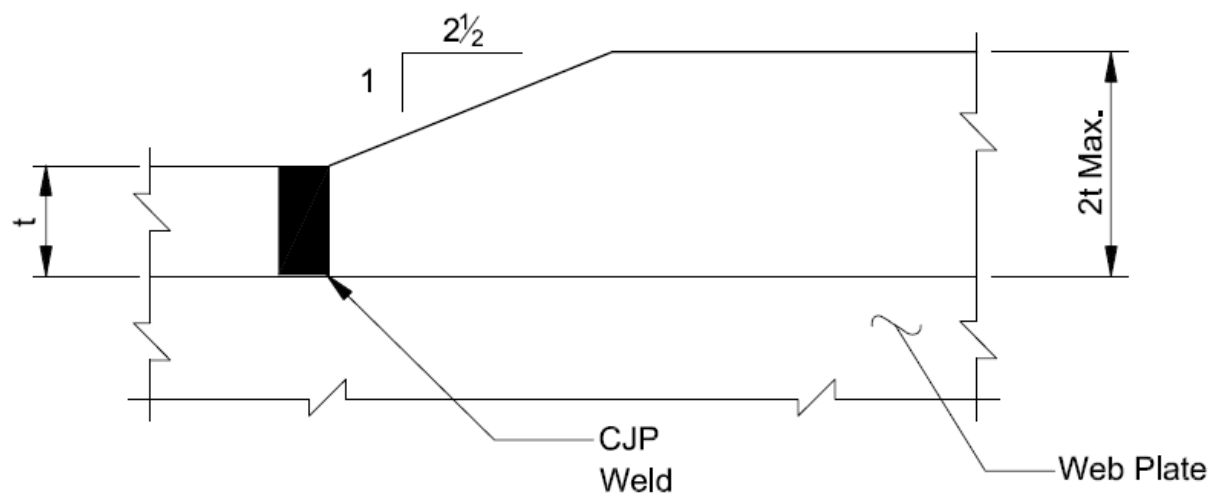


Figure 15-13
Typical Welded Splice Details