Chapter 15

STEEL STRUCTURES

NDOT STRUCTURES MANUAL

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Chapter 15
STEEL STRUCTURES

This Chapter discusses structural steel provisions in Section 6 of the *LFRD Bridge Design Specifications* that require amplification or clarification for NDOT-specific application. Section 11.5 provides criteria for the general site conditions for which structural steel is appropriate. This includes span lengths, girder spacing, geometrics, aesthetics and cost.

15.1 GENERAL

15.1.1 <u>Economical Steel Superstructure Design</u>

15.1.1.1 General

Factors that influence the initial cost of a steel bridge include, but are not limited to, detailing practices, the number of girders (for a girder bridge), the grade of steel, type and number of substructure units (i.e., span lengths), steel tonnage, fabrication, transportation and erection. The cost associated with these factors changes periodically in addition to the cost relationship among them. Therefore, the guidelines used to determine the most economical type of steel girder on one bridge must be reviewed and modified as necessary for future bridges.

Based upon market factors, the availability of steel may be an issue in meeting the construction schedule. It is the responsibility of the bridge designer to verify the availability of the specified steel and beam section. Bridge designers must contact producers and fabricators to ensure the availability of plates and rolled beams. For more detailed information on availability, see Section 1.4 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.1.1.2 Number of Girders/Girder Spacing

See Section 11.4.5.2 for general information on the number of girders in girder bridges. See Section 11.5.3.4 for NDOT criteria on typical girder spacing for 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 Constructibility*, G12.1-2003.

15.1.1.3 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 the exterior girder such that the exterior girder does not control the design (i.e., the interior and exterior girders are identical).
- Consider the minimum and maximum overhang widths that are specified in Section 16.2.9.
- The space required for deck drains may have an effect on the location of the exterior girder lines.

15.1.1.4 Span Arrangements

Where pier locations are flexible, the bridge designer should optimize the span arrangement. In selecting an optimum span arrangement, it is critical to consider the cost of the superstructure, substructure, foundations and approaches together as a total system.

To provide a balanced span arrangement for continuous steel bridges, the end spans should be approximately 80% of the length of interior spans. This results in the largest possible negative moments at the piers and smaller resulting positive moments and girder deflections. As a result, the optimum proportions of the girder in all spans will be nearly the same, resulting in an efficient design. End spans less than 50% of the interior span lengths should be avoided to mitigate uplift concerns.

15.1.2 <u>Rolled Beams vs Welded Plate Girders</u>

15.1.2.1 General

Typical NDOT practice is to use rolled beams for spans up to approximately 90 ft. Welded plate girders are used for spans from approximately 90 ft to 400 ft.

When rolled beams are specified, ensure that the selected sections are available consistent with the construction schedule. The *NDOT Standard Specifications* allows the contractor to substitute welded plate girders comprised of plates having a thickness equal to those of the flange and web of the specified rolled beams. For more detailed information, see Section 1.1 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.1.2.2 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 smaller than their bottom flanges. The flange section is varied 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, only flange thicknesses, not widths, are varied within a field section to reduce fabrication costs. The webs of plate girders are typically deeper and thinner than the webs of rolled beams. To save in total costs, the designer should increase minimum web thicknesses to avoid the use of stiffeners.

Due to buckling considerations, the stability of the compression flange (i.e., the top flange in positive-moment regions and the bottom flange in negative-moment regions) must be addressed by providing lateral-brace locations based upon LRFD Equation 6.10.8.2.3 instead of the 25-ft diaphragm spacing of the AASHTO *Standard Specifications for Highway Bridges*. The traditional 25-ft diaphragm spacing, however, provides a good minimum preliminary value.

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

15.1.2.3 Rolled Beams

Rolled steel beams are characterized by doubly symmetrical, as-rolled cross sections with equal-dimensioned top and bottom flanges and relatively thick webs. Thus, the cross sections are not optimized for weight savings (as is a plate girder) but are cost effective due to lower fabrication and erection costs. The relatively thick webs preclude the need for web stiffeners. Unless difficult geometrics or limited vertical clearances control, rolled steel beam superstructures are more cost effective in relatively shorter spans.

Rolled steel beams are available in depths up to 36 in, with beams 24 in and greater rolled less frequently. Before beginning final design, verify with one or more potential fabricators and/or producers that the section size is available. Beams up to 44 in in depth are available but are usually not of domestic origin.

15.1.3 <u>Economical Plate Girder Proportioning</u>

The AASHTO/NSBA Steel Bridge Collaboration has published the *Guidelines for Design for Constructibility*, G12.1-2003. This document presents cost-effective details for steel bridges from the perspective of the steel fabricator. The following Section presents information from the AASHTO/NSBA *Guidelines* that is of interest.

15.1.3.1 General

Plate girders and rolled beams shall be made composite with the bridge deck through shear studs and should be continuous over interior supports where possible. To achieve economy in the fabrication shop, all girders in a multi-girder bridge should be identical where possible. When using plate girders, a minimal number of plate sizes should be used.

15.1.3.2 Haunched Girders

When practical, girders with constant web depths shall be used. Haunched girders are generally uneconomical for spans less than 300 ft. Parabolic haunched girders may be used where aesthetics or other special circumstances are involved, but constant-depth girders will generally be more cost effective.

15.1.3.3 Flange Plate Sizes

The minimum flange plate size for plate girders is 12 in by 1 in to avoid cupping of the flanges due to distortion from welding. Designers should 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, and it reduces the number of passes and weld volume at flange butt welds. As a guide, flange width should be approximately 20% to 25% of web depth. Flange widths should not be sized in any set increments but based on mill plate widths minus the waste from torch cutting. Limit the maximum flange thickness to 3 in to ensure more uniform through-thickness properties. Thicker plates demonstrate relatively poor material properties near mid-thickness.

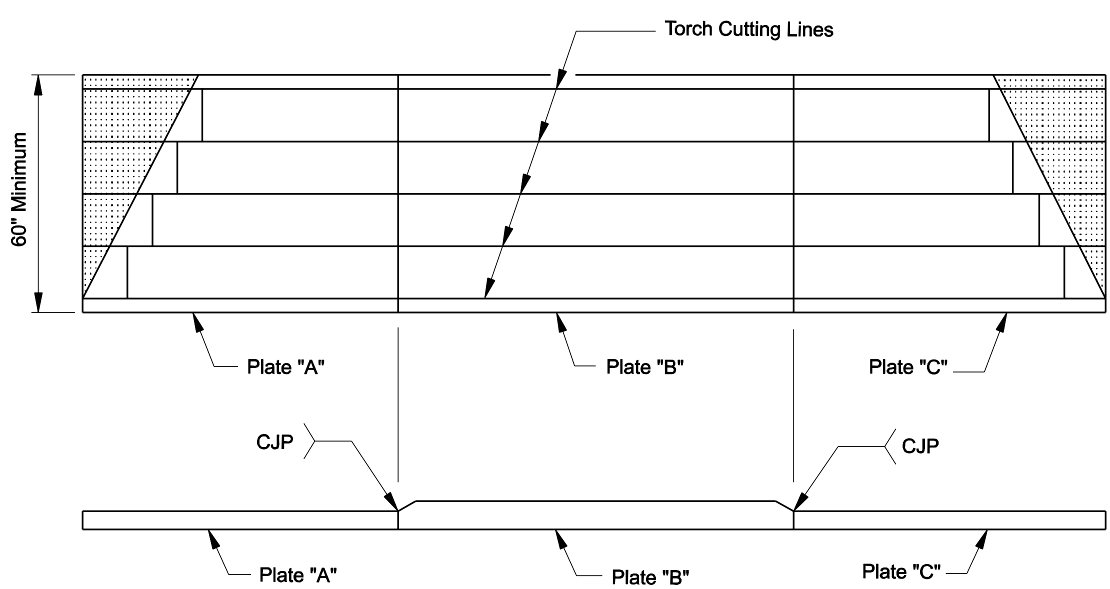
Within a single field section (i.e., an individual shipping piece), the flanges should be of constant width. A design using multiple identical girders with constant-width flanges minimizes fabrication costs.

Proportion flanges so the fabricator can economically cut them from steel plate between 60 in and 120 in wide. The most economical mill widths are 72 in, 84 in, 96 in and 120 in. Allow ¼ in for internal torch cutting lines and ½ in for exterior torch cutting lines; see Figure 15.1-A. Flanges should be grouped 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. Many of the plates of like width can be grouped 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" 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 the number of X-rays for 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-A illustrates one example of an efficient fabrication for girders.

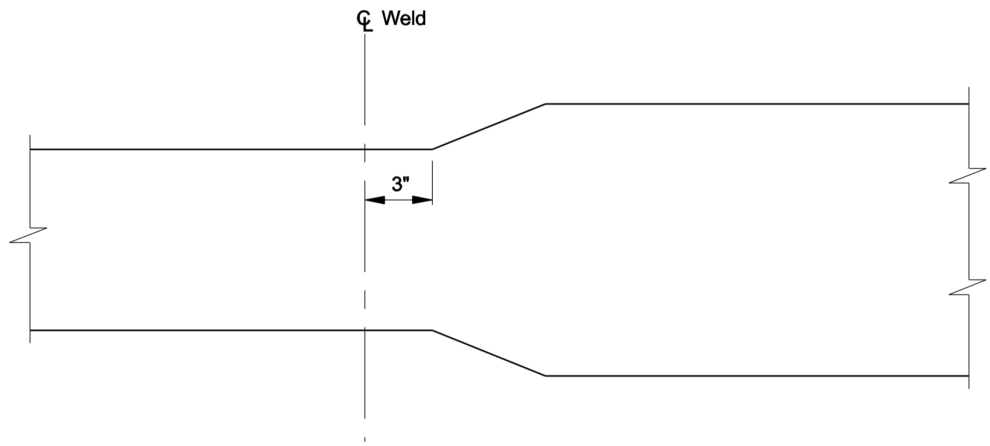
Constant flange width within a field section may not always be practical in girder spans over 300 ft where a flange width transition may be required in the negative bending regions. Though not preferred, if a transition in width must be provided, shift the butt splice a minimum of 3 in from the transition into the narrower flange plate. See Figure 15.1-B. This 3-in 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 Constructibility*, G12.1-2003.



GROUPING FLANGES FOR EFFICENT FABRICATION
(From the AASHTO/NSBA Steel Bridge Collaboration)

Figure 15.1-A



FLANGE WIDTH TRANSITION
(Plan View)

Figure 15.1-B

15.1.3.4 Field Splices

Field splices are used to reduce shipping lengths, but they are expensive and their number should be minimized. The preferred maximum length of a field section is 120 ft; however, lengths up to 150 ft are allowed, but field sections greater than 120 ft should not be used without considering shipping, erection and site constraints. As a general 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% of the area of the heavier flange plate at field splices to reduce the build-up of stress at the transition. For continuous spans, the field sections over a pier should be of constant length to simplify erection.

NDOT does not specify a maximum weight for field sections.

15.1.3.5 Shop Splices

Include no more than two shop flange splices in the top or bottom flange within a single field section. The designer should maintain constant flange widths within a field section for economy of fabrication as specified in Section 15.1.3.3. 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 1.5.2.A of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003, 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. The contract documents should allow this alternative.

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

Section 1.5 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003, provides additional guidance on shop splices.

15.1.3.6 Web Plates

Where there are no depth restrictions, the web depth should be optimized. NSBA provides a service to bridge owners to assist in optimizing web depths. Other sources may also be used if they are based upon material use and fabrication unit costs. The minimum web thickness shall be ½ in. Web thickness at any splice should not be changed by less than ⅛ in. 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 alone, it is desirable to 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:

1. Unstiffened webs are generally more economical for web depths approximately 48 in or less.

2. Between 48-in and 72-in depths, consider options for a partially stiffened and unstiffened web, with unstiffened webs preferred. A partially stiffened web is defined as one whose thickness is 1/16 in less than allowed by specification for an unstiffened web at a given depth and where stiffeners are required only in areas of higher shear.

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

15.1.3.7 Transverse Stiffeners

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

15.1.3.8 Longitudinally Stiffened Webs

Longitudinally stiffened webs are typically not used. In addition to being considered uneconomical, the ends of longitudinal stiffeners are fatigue sensitive if subject to applied tensile stresses. Therefore, where used, they must be ended in zones of little or no applied tensile stresses.

15.1.4 Falsework

Steel superstructures shall be designed without intermediate falsework during the placing and curing of the concrete deck slab.

15.1.5 AISC Certification Program

The AISC certification program for structural steel fabricators includes several categories:

1. SBR – Simple Steel Bridge Structures. Includes highway sign structures, parts for bridges (e.g., cross frames) and unspliced rolled beam bridges. NDOT does not require this certification for sign structures. The high-mast lighting standard plan, however, incorporates this certification.

2. CBR – Major Steel Bridges. All bridge structures other than unspliced rolled beam bridges.

3. CSE and ACSE. These steel bridge erector certifications are for simple bridges (CSE) and complex bridges (ACSE). NDOT does not typically use these certifications.

15.1.6 Buy America

23 CFR Part 635.410 presents the "Buy America" provisions for Federal-aid projects. These provisions require that manufacturing processes for steel and iron products and their coatings must occur in the United States. A minimal amount of foreign material can be used, if it does not exceed 1/10 of 1% of the total contract price or $2,500, whichever is more. These "Buy America" provisions are included in the *NDOT Standard Specifications*. Note that "Buy American" provisions are different and do not apply to Federal-aid highway projects.

15.2 MATERIALS

Reference: LRFD Article 6.4

15.2.1 Structural Steel

Reference: LRFD Article 6.4.1

The following presents typical NDOT practices for the material type selection for structural steel members.

15.2.1.1 Grade 36

Grade 36 steel is typically used for the following structural members:

- transverse stiffeners,
- diaphragms, 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.

15.2.1.2 Grade 50

Grade 50 steel is typically used for the following structural members:

- rolled beams,
- plate girders,
- splice plates,
- diaphragms,
- steel piles, and
- bearing plates.

15.2.1.3 High-Performance Steel

15.2.1.3.1 *Grade HPS70W*

For some plate-girder bridges, a good choice of steel may be Grade HPS70W. In addition to increased strength, the high-performance steels exhibit enhanced weathering, toughness and weldability properties. The premium on material costs is offset by a savings in tonnage. The most cost-effective design solutions tend to be hybrid girders with Grade 50 webs with HPS70W tension and compression flanges in the negative-moment regions and tension flanges only in the positive-moment regions.

HPS70W may be painted for aesthetic reasons.

15.2.1.3.2 *Grade HPS100W*

A new high-performance steel with a minimum specified yield strength of 100 ksi has been recently introduced. It has yet to be proven cost-effective for girder bridge applications and should not be used.

15.2.1.4 Unpainted Weathering Steel

15.2.1.4.1 *General*

In general, NDOT discourages the use of unpainted weathering steel (i.e., Grades 50W, HPS70W) because of aesthetic considerations.

Unpainted weathering steel is often the more cost-effective choice for structural steel superstructures. The initial cost advantage when compared to painted steel can range up to 15%. When future repainting costs are considered, the cost advantage is more substantial. This reflects, for example, environmental considerations in the removal of paint, which significantly increases the life-cycle cost of painted steel. The application of weathering steel and its potential problems are discussed in depth in FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures," October 3, 1989. Also, the proceedings of the "Weathering Steel Forum," July 1989, are available from the FHWA Office of Implementation, HRT-10.

Despite its cost advantage, the use of unpainted weathering steel is not appropriate in all environments and at all locations. The most prominent disadvantage of weathering steel is aesthetics. The inevitable staining of the steel where susceptible to water leakage from above (e.g., below deck joints) creates a poor image (i.e., one of lack of proper maintenance) to the traveling public. Therefore, NDOT policy is to only consider the use of weathering steel for highway bridges over railroads and over stream crossings that are not adjacent to highways; i.e., where the girders are not visible to the traveling public. In addition, weathering steel shall not be used at locations where the following conditions exist:

1. Environment. Unpainted weathering steel shall not be used in industrial areas where concentrated chemical fumes may drift onto the structure, or where the nature of the environment is questionable.

2. Water Crossings. Unpainted weathering steel shall not be used over bodies of water where the clearance over the ordinary high water is 10 ft or less.

3. Grade Separations. Unpainted weathering steel shall not be used for highway grade separation structures.

The staining potential can be addressed by applying a silane treatment conforming to Section 646 of the *NDOT Standard Specifications* to the substructure elements. The silane treatment should be applied to mature concrete in accordance with the manufacturer's recommendations.

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

15.2.1.4.2 *Design Details for Weathering Steel*

Where weathering steel girders are used, the bearing plates shall be the same steel as the girders they support. The bolts, nuts, washers and Direct Tension Indicators (DTIs) shall be Type 3 as specified in ASTM A325/ASTM A563 and ASTM F 959.

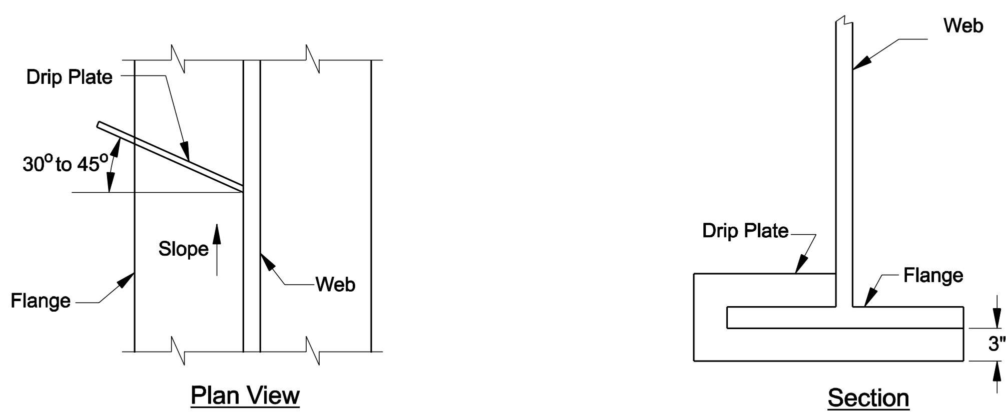
Paint weathering steel at the ends of girders, at expansion joints and over piers for a distance of 10 ft or 1.5 times the web depth, whichever is greater. Use only the prime coat of the approved bridge paint systems.

When using unpainted weathering steel, the following drainage treatments shall be incorporated:

1. Minimize the number of bridge deck drains and extend the drainage outlets below the steel bottom flange.

2. Eliminate details that serve as water and debris “traps.” Seal or paint overlapping surfaces exposed to water. This sealing or painting applies to non-slip-critical bolted joints. Slip-critical bolted joints or splices should not produce “rust-pack” when the bolts are spaced according to the *LRFD Specifications* and, therefore, do not require special protection.

3. Place a drip plate or other material transverse across the top of the bottom flange in front of the substructure elements to prevent water from running off the flange onto the concrete. Ensure that these attachments meet all fatigue requirements. Figure 15.2-A shows a typical drip plate detail.



DRIP PLATE DETAIL

Figure 15.2-A

15.2.1.5 Charpy V-Notch Fracture Toughness

Reference: LRFD Article 6.6.2

The temperature zone appropriate for using LRFD Table 6.6.2-1 for the State of Nevada is Temperature Zone 2.

15.2.2 Bolts

Reference: LRFD Article 6.4.3

15.2.2.1 Type

For normal construction, high-strength bolts shall be:

1. Painted Steel: Use ⅞-in A325 (Type 1).
2. Weathering Steel: Use ⅞-in A325 (Type 3).

15.2.2.2 Hole Size

Typically, do not use oversized or slotted holes; these may be used only in unusual circumstances with approval. Use appropriate design considerations when oversized or slotted holes are approved.

15.2.3 Splice Plates

In all cases, steel for all splice and filler plates shall be the same material 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

Use a curved girder on curved alignments, unless otherwise approved.

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, analysis methodologies that detail various required levels of analysis are also specified.

15.3.2 Diaphragms, Bearings and Field Splices

Cross frames and diaphragms shall be considered primary members. However, due to the difficulty of obtaining a Charpy specimen from a rolled shape such as an angle, Charpy V-notch impact-energy testing of the cross frames is not required. All curved steel simple-span and continuous-span bridges shall have diaphragms directed radially except end diaphragms, which should be placed parallel to the centerline of bearings.

Design all diaphragms, including their connections to the girders, to carry the total load to be transferred at each diaphragm location. Cross frames and diaphragms should be as close as practical to the full depth of the girders. Design cross frame and diaphragm connections for the 75% 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.

Bridges expand and contract in all directions. For typical bridges that are long in relationship to their width, the transverse expansion is ignored. For ordinary geometric configurations where the bridge length is long relative to the bridge width (say, 2½ times the 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. The designer must 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.

Design the splices in flanges of curved girders to carry flange bending or 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 <u>Load-Induced Fatigue</u>

Reference: LRFD Article 6.6.1.2

15.4.1.1 General

LRFD Article 6.6.1.2 provides the framework to evaluate load-induced fatigue. This Section provides additional information on the implementation of LRFD Article 6.6.1.2.

Load-induced fatigue is determined by the following:

- 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 higher truck-traffic volume bridges, this is taken as infinite. For lower truck-traffic volume bridges, this is determined by using anticipated truck volumes; and
- the nominal fatigue resistance for the Detail Category being investigated.

15.4.1.2 NDOT Policy

NDOT policy on load-induced fatigue is as follows:

1. <u>New Bridges</u>. For new steel bridges, it is mandatory to design for infinite life. In addition, all details must have a fatigue resistance greater than or equal to Detail Category C (i.e., Detail Categories A, B, B′, C and C′).
2. <u>Existing Bridges</u>. Section 22.4.3.5 presents NDOT policy for load-induced fatigue for work on existing bridges (e.g., bridge rehabilitation, bridge widening).

Any exceptions to NDOT policy on load-induced fatigue require the approval of the Chief Structures Engineer.

15.4.1.3 Fatigue Stress Range

The following applies:

1. <u>Regions</u>. Fatigue should only be considered in those regions of a steel member that experience a net applied tensile stress, or where the compressive stress of the

unfactored permanent load is less than twice the maximum fatigue tensile stress. Twice the maximum fatigue tensile stress represents the largest stress range that the detail should experience. This requirement checks to determine if the detail will go into tension. If not, fatigue is not a consideration.

2. Range. The fatigue stress range is the difference between the maximum and minimum stresses at a structural detail subject to a net tensile stress. The stress range is caused by a single design truck that can be placed anywhere on the deck within the boundaries of a design lane. If a refined analysis method is used, the design truck shall be positioned to maximize the stress in the detail under consideration. The design truck should have a constant 30-ft spacing between the 32-kip axles. The dynamic load allowance is 0.15 and the fatigue load factor is 0.75.

3. Analysis. Unless a refined analysis method is used, the single design lane load distribution factor in LRFD Article 4.6.2.2 should be used to determine fatigue stresses. These tabularized distribution-factor equations incorporate a multiple presence factor of 1.2 that should be removed by dividing either the distribution factor or the resulting fatigue stresses by 1.2. This division does not apply to distribution factors determined using the lever rule.

15.4.1.4 Fatigue Resistance

LRFD Article 6.6.1.2.3 groups the fatigue resistance of various structural details into eight categories (A through E′). Experience indicates that Detail Categories A, B and B′ are seldom critical. Investigation of details with a fatigue resistance greater than Detail Category C may be appropriate in unusual design cases. For example, Category B applies to base metal adjacent to slip-critical bolted connections and should only be evaluated when thin splice plates or connection plates are used. For Detail Categories C, C′, D, E and E′, the *LRFD Specifications* requires that the fatigue stress range must be less than the specified fatigue resistance for each of the respective Categories.

The fatigue resistance of a category is determined from the interaction of a Category Constant "A" and the total number of stress cycles "N" experienced during the 75-year design life of the structure. This resistance is defined as $(A/N)^{1/3}$. A Constant Amplitude Fatigue Threshold $((\Delta F)_{TH})$ is also established for each Category. If the applied fatigue stress range is less than ½ of the threshold value, the detail has infinite fatigue life.

For bridges designed for infinite life, the applied fatigue stress range shall be less than ½ of the threshold value, $½(\Delta F)_{TH}$. This practice provides a theoretical design life of infinity. For all other bridges, the fatigue resistance shall be calculated in accordance with LRFD Article 6.6.1.2.3.

Fatigue resistance is independent of the steel strength. The application of higher grade steels causes the fatigue stress range to increase, but the fatigue resistance remains the same. This independence implies that fatigue may become more of a controlling factor where higher strength steels are used.

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Example 15.4-1

Given: State Highway System bridge
Two-span continuous bridge, 150-ft each

Area investigated is located 13 ft from interior support
Unfactored DL stress in the top flange = 7.9 ksi Tension
Unfactored fatigue stresses in the top flange using unmodified single lane distribution factor = 5.6 ksi Tension and 0.8 ksi Compression

Find: Determine the fatigue adequacy of the top flange with welded stud shear connectors in the negative moment region.

Solution:

Step 1: The LRFD Specifications classifies this connection as Detail Category C. Therefore:

- A = Detail Category Constant = 44.0 x 10^8 ksi^3 (LRFD Table 6.6.1.2.5-1)

- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 10.0 ksi (LRFD Table 6.6.1.2.5-3)

Step 2: Compute the factored live-load fatigue stresses by applying dynamic load allowance and fatigue load factor and removing the multiple presence factor:

Tension: 5.6(1.15)(0.75)/1.2	=	4.0 ksi
Compression: 0.8(1.15)(0.75)/1.2	=	0.6 ksi
Fatigue Stress Range	=	4.6 ksi

Step 3: Check for infinite life:

First, check the infinite life term (see Commentary C6.6.1.2.5 of the *LRFD Specifications* for a table of single-lane ADTT values for each detail category above which the infinite life check governs). This infinite-life term will typically control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = ½(\Delta F)_{TH} = 0.5(10.0) = 5.0$ ksi. Because the fatigue stress range (4.6 ksi) is less than the infinite life resistance (5.0 ksi), the detail should exhibit infinite fatigue life and, therefore, the detail is satisfactory.

Provisions for investigating the fatigue resistance of shear connectors are provided in LRFD Article 6.10.10.2.

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15.4.2 Distortion-Induced Fatigue

Reference: LRFD Article 6.6.1.3

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 concise and direct and require no mathematical computation of stress range.

15.4.3 Other Fatigue Considerations

Reference: Various LRFD Articles

The designer is responsible for ensuring compliance with fatigue requirements for all structural details (e.g., stiffeners, connection plates, lateral bracing) shown in the contract documents.

In addition to the considerations in Section 15.4.1, the designer should investigate the fatigue provisions in other Articles of Chapter 6 of the *LRFD Specifications*. These include:

- Fatigue due to out-of-plane flexing in webs of plate girders — LRFD Article 6.10.6.
- 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 DIMENSION AND DETAIL REQUIREMENTS

Reference: LRFD Article 6.7

15.5.1 Deck Haunches

A deck haunch is an additional thickness of concrete between the top of the girder and the bottom of the deck to provide adjustability between the top of the cambered girder and the roadway profile. The haunch is detailed at the centerline of bearing and varies in the span, if necessary, to accommodate variations in camber, superelevation ordinate and vertical curve ordinate. The maximum positive camber allowed in excess of that specified at mid-span is ¾ in for spans less than 100 ft and 1½ in for spans more than 100 ft. A 2-in haunch is recommended for spans of less than 100 ft, and a 3-in haunch is recommended for spans of more than 100 ft. The haunch is neglected when determining the resistance of the section. See Section 16.2.2.

15.5.2 Minimum Thickness of Steel

Reference: LRFD Article 6.7.3

For welded plate girder fabrication, minimum thickness requirements are mandated to reduce deformations and defects due to welding. The thickness of steel elements should not be less than:

- Plate girder webs: ½ in
- Stiffeners, connection plates: $\frac{7}{16}$ in, ½ in preferred
- Plate girder flanges: 1 in
- Bearing stiffener plates: 1 in
- Gusset plates: ⅜ in
- Angles/channels: ¼ in

For more detailed information, see Section 1.3 of the AASHTO/NSBA Steel Bridge Collaboration's *Guidelines for Design for Constructibility*, G12.1-2003.

15.5.3 Dead-Load Deflection

15.5.3.1 Deflections from Deck Shrinkage

In addition to the deflection due to dead load for simple-span bridges, the deflection from shrinkage of the concrete deck shall be computed by:

$$\Delta = \frac{0.00002L^2}{Y_{ts}}$$

Where: Δ = centerline span deflection, ft

L = girder span length, ft

Y_{ts} = distance in ft from CG of steel girder section only to top flange at centerline of span

15.5.3.2 Camber

The entire girder length shall be cambered as required by the loading and profile grade. The loading includes the consideration for shrinkage of the concrete deck as presented in Section 15.5.3.1. In addition, where dead load deflection and vertical curve offset are greater than ¼ in, the girders shall have a compensating camber. Camber will be calculated to the nearest 0.01 ft with ordinates at 0.1 points 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. The camber shall be parabolic.

A camber diagram is required in all contract documents with structural steel girders.

15.5.4 Diaphragms and Cross Frames

Reference: LRFD Articles 6.7.4 and 6.6.1.3.1

Diaphragms on rolled-beam bridges and cross frames on plate-girder bridges 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. Cross frames also serve to distribute gravitational, centrifugal and wind loads. The spacing of diaphragms and cross frames should be determined 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 maximum diaphragm and cross frame spacing of 25 ft. Most economical steel girder designs will use spacings typically greater than 25 ft in the positive-moment regions.

15.5.4.1 General

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. The location of the field splices should be planned to avoid conflict between the connection plates of the diaphragms or cross frames and any part of the splice material.

2. Skew. Regardless of the angle of skew, place all intermediate diaphragms and cross frames perpendicular to the girders. Locating cross frames near girder supports on bridges with high skews requires careful consideration. When locating a cross frame between two girders, the relative stiffness of the two girders must be similar. Otherwise, the 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 and Cross Frames. End diaphragms and cross frames should be placed 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. The end diaphragms should be designed to support the edge of the slab including live load plus impact.

4. Interior Support Diaphragms and Cross Frames. Generally, interior support diaphragms and cross frames should be placed along the centerline of bearing. They provide lateral stability for the bottom flange and bearings.

5. Curved-Girder Structures. Diaphragms or cross frames connecting horizontally curved girders are considered primary members and shall be oriented radially.

6. Detailing. Diaphragms and cross frames are typically detailed 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 NDOT practices on the selection of diaphragms and cross frames:

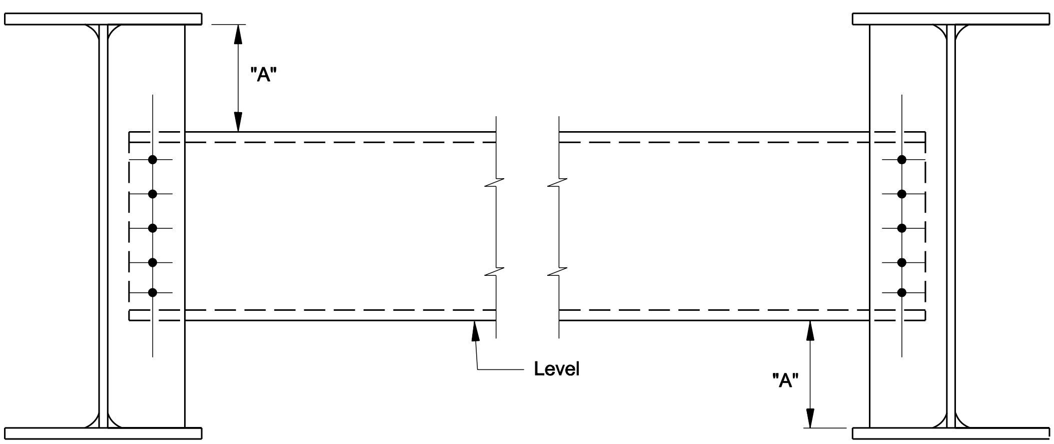
1. Solid Diaphragms. These are preferred for rolled beams. For rolled-beam bridges with seat abutments, the end diaphragms shall be full depth to provide sufficient lateral restraint.

2. K-Frames. These are preferred for plate girder bridges.

3. X-Frames. In the case of relatively narrow girder spacings relative to the girder depth, an X-frame may be more appropriate than a K-frame.

15.5.4.2 Diaphragm Details

On spans composed of rolled beams, diaphragms at interior span points may be detailed as illustrated in Figure 15.5-A. Figure 15.5-B illustrates the typical abutment support diaphragm connection details for rolled beams. Plate girders with web depths of 48 in or less should have similar diaphragm details. For plate girder webs more than 48 in deep, use cross frames as detailed in Figures 15.5-C and 15.5-D.



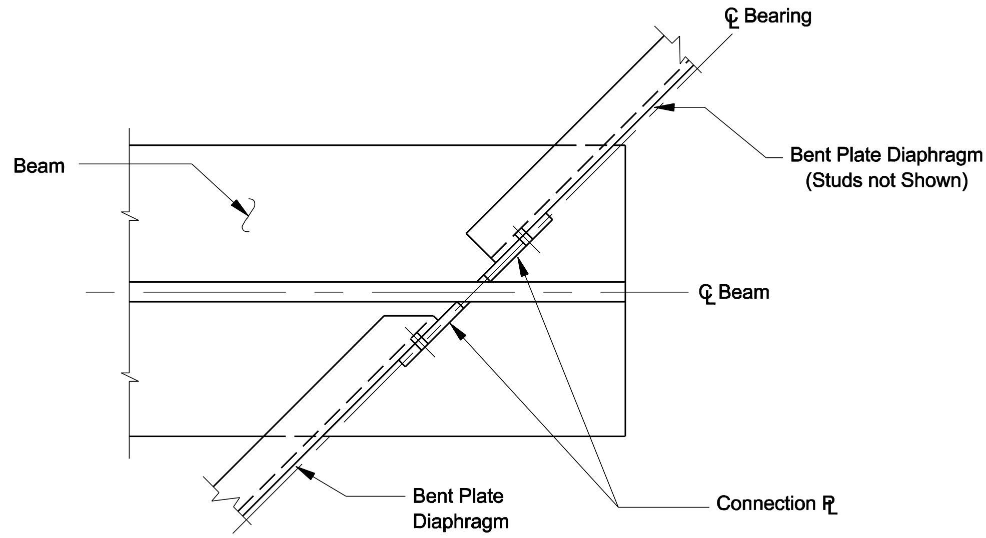
Note: "A" dimensions should be approximately equal.

ELEVATION

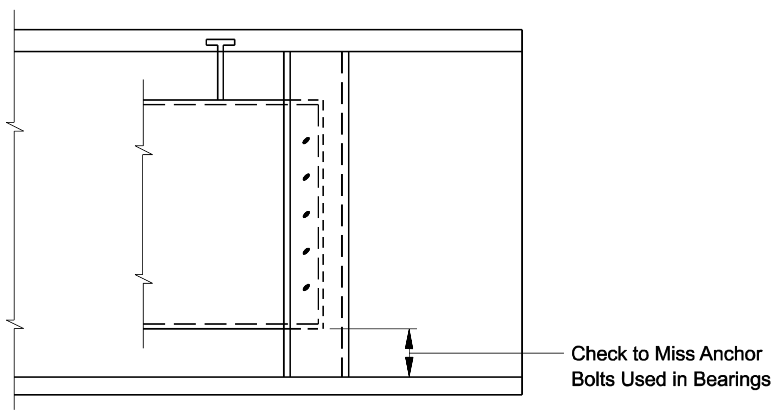
Note: Select a channel depth approximately ½ of the girder depth.

TYPICAL PIER AND INTERMEDIATE DIAPHRAGM CONNECTION
(Rolled Beams)

Figure 15.5-A



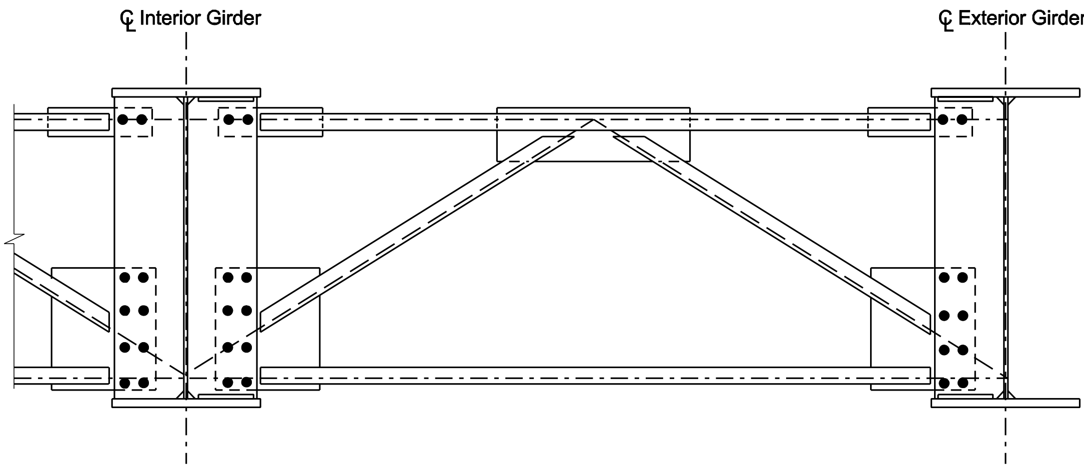
PLAN



ELEVATION

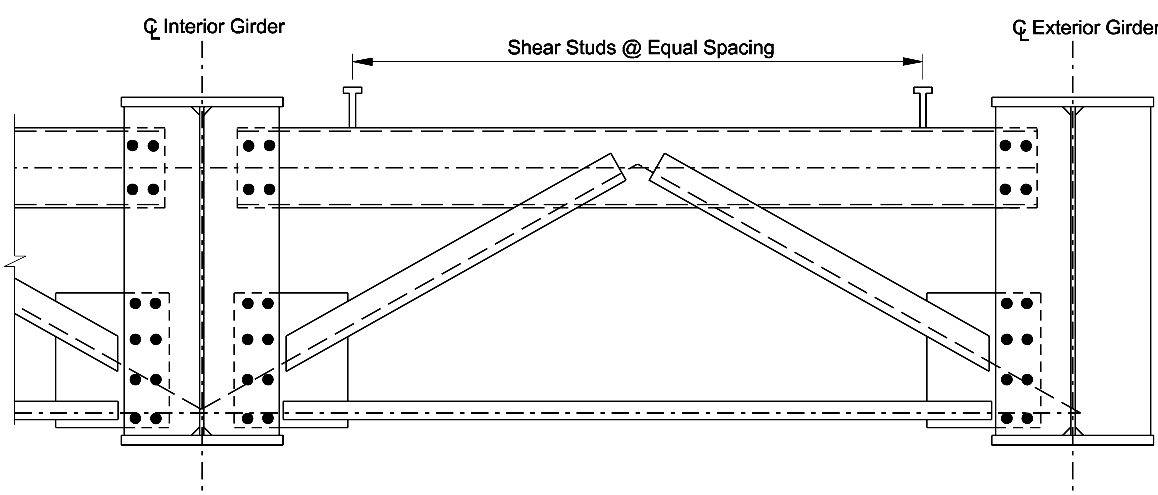
TYPICAL ABUTMENT DIAPHRAGM CONNECTION
(Rolled Beams)

Figure 15.5-B



TYPICAL PIER AND INTERMEDIATE CROSS FRAMES
(Plate Girder Web > 48 in)

Figure 15.5-C



TYPICAL ABUTMENT CROSS FRAMES
(Plate Girder Web > 48 in)

Figure 15.5-D

Pier and intermediate diaphragms for rolled-beam spans shall be detailed with a 3-in minimum clearance between the top of the diaphragm and the bottom of the top beam flange. For bridges having a normal roadway crown, the diaphragms shall be level. For bridges having a superelevated roadway, the diaphragms shall be placed parallel to the slab.

Intermediate diaphragms should be designed and detailed as non-load bearing. Diaphragms at points of support should be designed as a jacking frame as specified in Section 15.5.5.

15.5.4.3 Cross Frame Details

Figure 15.5-C illustrates typical pier and intermediate cross frame details for plate girder webs more than 48 in deep. In general, an X-frame is more cost effective than a K-frame; however, with a relatively wide girder spacing, the X-frame becomes shallow and less effective. The K-frame should be used instead of the X-frame when the girder spacing becomes much greater than the girder depth (e.g., where the girder spacing is greater than 1.75 of the girder depth) and the “X” becomes too shallow. A solid bent-plate diaphragm with a depth equal to 75% of the girder depth is a good option for plate girders less than 48 in deep.

Figure 15.5-D illustrates the typical abutment cross frame connection details for plate girder webs more than 48 in deep.

The rolled angles that comprise the cross frames are minimum sizes based upon the limiting slenderness ratios of LRFD Articles 6.8.4 and 6.9.3.

Current NDOT practice requires that cross frame transverse connection plates be welded or bolted to the compression flange and bolted to the tension flange. The welded and bolted connections to the flanges should be designed to transfer the cross frame forces into the flanges.

The width of connection plates should be sized to use bar stock and be not less than 5 in. When the connection plate also acts as a transverse stiffener, it shall meet the requirements of LRFD Article 6.10.8.1.

15.5.5 <u>Jacking</u>

Reference: LRFD Article 3.4.3

The contract documents shall include a jacking plan for all bearing supported structures. The bridge designer should include live load in the jacking plan for bridges with moderate to high traffic volumes or those with no readily available detour. Contact the District Office for concurrence on any jacking plan that does not include live load. The bearing type shall determine the level of detail shown for the jacking plan.

Include only bearing stiffeners at all points of jacking for plain or reinforced elastomeric bearings. Provide a conceptual jacking plan showing the jack location and clearances, required factored reactions and modifications to cross frames and diaphragms. Also, show conceptual requirements for falsework and jacking frames if required.

Include a complete jacking plan for high-load multi-rotational, isolation or other specialty bearings. The jacking plan must include necessary bearing stiffeners, jack locations and clearances, factored reactions and additional modifications to cross frames and diaphragms.

Also, include a detailed design of the jacking frame if required, but do not include its fabrication as part of the contract documents. Provide only conceptual falsework requirements.

In general, jacking frames will not be required at the supports unless there is insufficient clearance between the bottom of girder and top of cap to place a jack. If less than 7 in of clearance for the jack, the designer must decide whether the jack can be supported by temporary falsework. If temporary falsework is not feasible, provide details for a jacking frame or widen the cap and place the bearings on pedestals to provide sufficient space for a jack to be placed under the girder. Other locations where jacking may be required are:

- at supports under expansion joints where joint leakage could deteriorate the bearing areas of the girders; and
- at expansion bearings with large displacements where deformation-induced wear-and-tear is possible.

If no jacking frame is provided, the cross frame at the support must transfer lateral wind and seismic forces to the bearings.

15.5.6 Lateral Bracing

Reference: LRFD Article 6.7.5

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 it should be designed 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, it must be checked by the designer. Typical diaphragms and cross frames will transfer lateral loads adequately to eliminate the need for lateral bracing. For tub girders, internal top lateral bracing is more typical. Tub girders can rack as much as 0.5 ft in one day due to the thermal effects of the sun. To counteract this effect, provide temporary lateral bracing between adjacent boxes at ¼ points of spans. Remove after the deck has been placed.

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

15.5.7 Inspection Access (Tub Girders)

All new steel tub girder bridges shall be detailed with access openings to allow inspection of the girder interior. Do not locate access openings over travel lanes or railroad tracks and, preferably, not over shoulders or maintenance roads. They should be located such that the general public cannot gain easy entrance.

Provide access openings in the bottom flange plate of all steel tub girders. Provide one access opening at each end of the bridge when the total span length is 100 ft or more.

Access plates shall be connected to the bottom flange with high-strength bolts. If the general public has access to the openings, provide bolts with special head configurations. Contact the Assistant Chief Structures Engineer – Inventory/Inspection for specifications on these special heads. The dimensions of the access opening should be a minimum 2 ft by 2 ft square.

15.6 I-SECTIONS IN FLEXURE

Reference: LRFD Article 6.10

15.6.1 General

Reference: LRFD Article 6.10.1

15.6.1.1 Positive-Moment Region Maximum-Moment Section

For a compositely designed girder, the positive-moment region maximum-moment section may also be considered compact in the final condition. The cured concrete deck in the positive-moment region provides a large compression flange and it laterally braces the top flange. Very little, if any, of the web is in compression.

15.6.1.1.1 *Top 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 governs the design of the top flange in the positive-moment region as specified in LRFD Article 6.10.3.4.

15.6.1.1.2 *Bottom Flange (Tension Flange)*

The bottom flange, if properly proportioned, is not governed by the construction phase. The bottom flange is governed by the final condition. The Service II load combination permanent deformation provisions of LRFD Article 6.10.4.2 govern.

15.6.1.2 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., ultimate). Thus, a good portion of the steel cross section is in compression. To qualify as compact, the web usually needs to 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 governed by the Strength limit state in the final condition. Furthermore, the bottom flange in compression is governed by the location of the first intermediate diaphragm off of the pier because it provides the discrete bracing for the flange.

15.6.1.3 Negative Flexural Deck Reinforcement

Reference: LRFD Article 6.10.1.7

In the negative-moment region where the longitudinal tensile stress in the slab, due to factored construction loads or the Service II load combination, exceeds the factored modulus of rupture, LRFD Article 6.10.1.7 specifies a minimum area of steel. The total cross sectional area of the longitudinal steel should not be less than 1% of the total cross sectional area of the deck slab (excluding the wearing surface) in these regions. However, the designer shall also ensure that sufficient negative-moment steel is provided for the applied loads.

15.6.1.4 Rigidity in Negative-Moment Regions

Reference: LRFD Articles 6.10.1.5 and 6.10.1.7

LRFD Article 6.10.1.5 permits the assumption of uncracked concrete in the negative-moment regions for member stiffness. This stiffness is used to obtain continuity moments due to live load, future wearing surface and barrier weights placed on the composite section.

For the Service limit state control of permanent deflections under LRFD Article 6.10.4.2 and the Fatigue limit state under LRFD Article 6.6.1.2, the concrete slab may be considered fully effective for both positive and negative moments for members with shear connectors throughout their full lengths and satisfying LRFD Article 6.10.1.7.

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 shall be ⅞ in diameter by 5 in; the minimum is ¾ in diameter by 5 in. The minimum number of studs in a group shall consist of three in a single transverse row. Skew the studs parallel to the bottom slab reinforcing steel. Increase the stud length in 1-in increments when necessary to maintain a 2-in minimum penetration of the stud into the deck slab. Studs placed on relatively thin elements such as girder webs should be detailed as ¾-in diameter.

15.6.3 Stiffeners

Reference: LRFD Article 6.10.11

15.6.3.1 Transverse Intermediate Stiffeners

Reference: LRFD Article 6.10.11.1

Straight girders may be designed without intermediate transverse stiffeners, if economical, or with intermediate transverse stiffeners placed on one side of the web plate. If stiffeners are required, fascia girders should only have stiffeners 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, the use of a thicker web and fewer intermediate transverse stiffeners, or no intermediate stiffeners at all, should be investigated. If the bridge designer proceeds with a design that requires stiffeners, the preferred width of the stiffener is one that can be cut from commercially produced bar stock.

Intermediate transverse stiffeners should be welded near side and far side to the compression flange. Transverse stiffeners should not be welded 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 $6t_w$.

Transverse stiffeners, except when used as diaphragm or cross frame connections, should be placed on only one side of the web. The width of the projecting stiffener element, moment of inertia of the transverse stiffener and stiffener area shall 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° and 90°, the stiffeners may be skewed so that the diaphragms of cross frames may be connected directly to the stiffeners.

Longitudinal stiffeners should be avoided but, if used in conjunction with transverse stiffeners on spans with deeper webs, should preferably be placed on the opposite side of the web from the transverse stiffener. Where this is not practical (e.g., at intersections with cross frame connection plates), the longitudinal stiffener should be continuous and not be interrupted for the transverse stiffener.

15.6.3.2 Bearing Stiffeners

Reference: LRFD Article 6.10.11.2

Provide bearing stiffeners for all plate girders to prevent the possibility of web buckling at temporary supports. They only require placement on one side and, on the fascia girders, they shall be placed on the inside.

Bearing stiffeners are required at the bearing points of rolled beams and plate girders. Bearing stiffeners at integral abutments may be designed for dead and construction loads only.

Design the bearing stiffeners as columns and extend the stiffeners to the outer edges of the bottom flange plates. The *LRFD Specifications* does not specify an effective column length for the design of bearing stiffeners. Because the reaction load applied at one end of the stiffener pair is resisted by forces distributed to the web instead of by a force concentrated at the opposite end, as in columns, it is not necessary to consider the stiffeners as an end-hinged column even where the flanges are free to rotate. Use an effective column length of ¾ of the web depth.

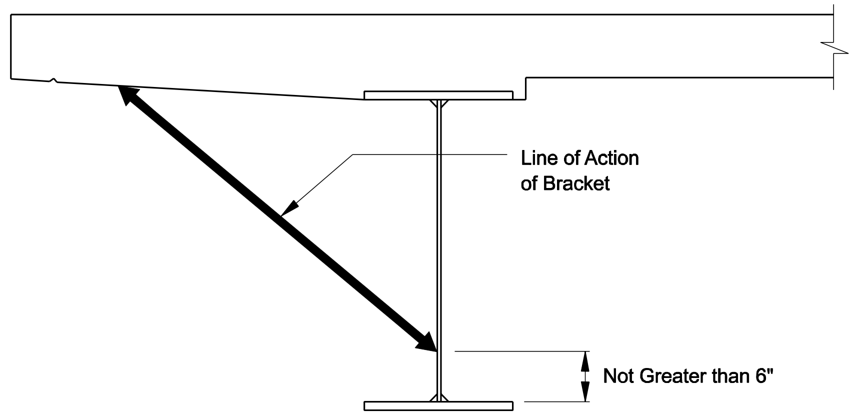
The weld connecting the bearing stiffener to the web should be designed to transmit the full bearing force from the stiffener to the web due to the factored loads.

Detail bearing stiffeners with the stiffener ends bearing on the loaded flange being milled to bear, or weld with a full penetration butt weld. The opposite end will be tight fit only to the flange. Where bearing stiffeners are also used as diaphragm or cross frame connection plates, the stiffeners shall also be fillet welded to the girder flanges if they are milled to bear or tight fit.

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. Include in the contract documents the requirement for the contractor to check the twist of the exterior girder and bearing of the overhand bracket on the web. See Figure 15.6-A.



SCHEMATIC OF LOCATION FOR DECK OVERHANG BRACKET

Figure 15.6-A

15.7 CONNECTIONS AND SPLICES

Reference: LRFD Article 6.13

15.7.1 Bolted Connections

Reference: LRFD Article 6.13.2

The following applies to bolted connections:

1. Type. For painted steel, ⅞-in A325 (Type1) bolts should be used. For unpainted weathering steel, A325 (Type 3) bolts should be used.

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

3. Slip Resistance. LRFD Table 6.13.2.8-3 provides values for the surface condition. Use Class B surface condition for the design of slip-critical connections. Class B is applicable to unpainted, blast-cleaned surfaces and to blast-cleaned surfaces with a Class B coating. All specified coatings must be tested to ensure a slip resistance equal to or exceeding Class B. NDOT policy is to paint the faying surfaces of all slip critical connections with the prime coat of the approved paint systems shown in the Qualified Products List (QPL). Systems on the QPL must meet the minimum requirements of the Research Council on Structural Connections, June 2004 version of "Testing Method to Determine the Slip Coefficient for Coatings Used in Bolted Joints."

15.7.2 Welded Connections

Reference: LRFD Article 6.13.3

15.7.2.1 Welding Process

The governing specification for welding 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. Additional reference specifications that may need to be consulted are:

- 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.

The *Bridge Welding Code* accepts as *prequalified* (i.e., acceptable without further proof of suitability if applied under specified conditions) four electric arc welding processes:

- shielded metal arc welding (SMAW). This process is also known as stick welding and is what is commonly considered "welding";

- submerged arc welding (SAW);

- gas metal arc welding (GMAW). This process is also called metal inert gas welding or MIG; and
- flux-cored arc welding (FCAW).

Gas metal arc and flux-cored arc welding shall not be used except with written approval of the Chief Structures Engineer. Electro-slag welding and electro-gas welding (a development of electro-slag welding) shall not be permitted due to potential problems with the ductility of electro-slag welds.

GMAW should not be used in windy conditions due to the potential loss of shielding gas. The *Welding Code* states that "GMAW, GTAW, EGW or FCAW-G shall not be done in a draft or wind unless the weld is protected by a shelter. Such shelter shall be of material and shape appropriate to reduce wind velocity in the vicinity of the weld to a maximum of five miles per hour" (AWS D1.1-2000, para. 5.12.1).

SMAW is the principal method for hand welding; the others are automatic or semi-automatic processes. Shop practice on most weldments is automatic, offering the advantages of much higher speed and greater reliability. Hand welding is mostly limited to short production welds or tack welds during the fitting up of components prior to production welding.

Acceptable procedures for using these processes or others require the testing of the welding operations and of welds, using a filler metal that is compatible with the base metal, proper preparation of the joints, controlling the temperature and rate of welding, and control of the welding process.

15.7.2.2 Field Welding

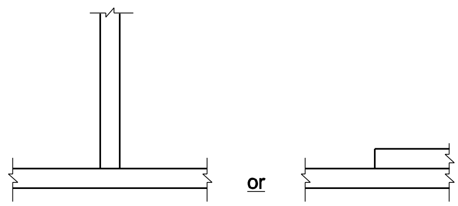
The primary types of welds used in bridge fabrication are fillet welds and groove welds. Figure 15.7-A illustrates a typical cross section where specification of a fillet weld is appropriate, and Figure 15.7-B illustrates a typical cross section where specification of a groove or butt weld is appropriate.

Field welding is prohibited for all but a few special applications. These permissible applications are welded splices for piles, connecting pile tips to piles, bearing plates to bottom flange plates and connector plates between new and existing portions of widened bridges at ends of simply supported spans (though bolted connections are preferred for this application).

Direct welding of stay-in-place (SIP) deck forms to girder flanges is not permitted. Metal forms are welded to a strap that is placed over the flange.

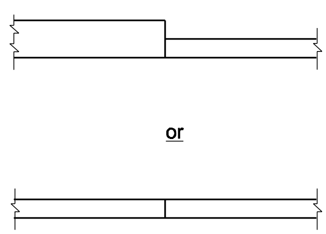
15.7.2.3 Welding Symbols

Welding symbols are used as an instruction on the type, size and other characteristics of the desired weld. The forms of the symbols are precisely defined by AWS A2.4. 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. The *AISC 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 15.7-C will suffice for the majority of bridge fabrication circumstances.



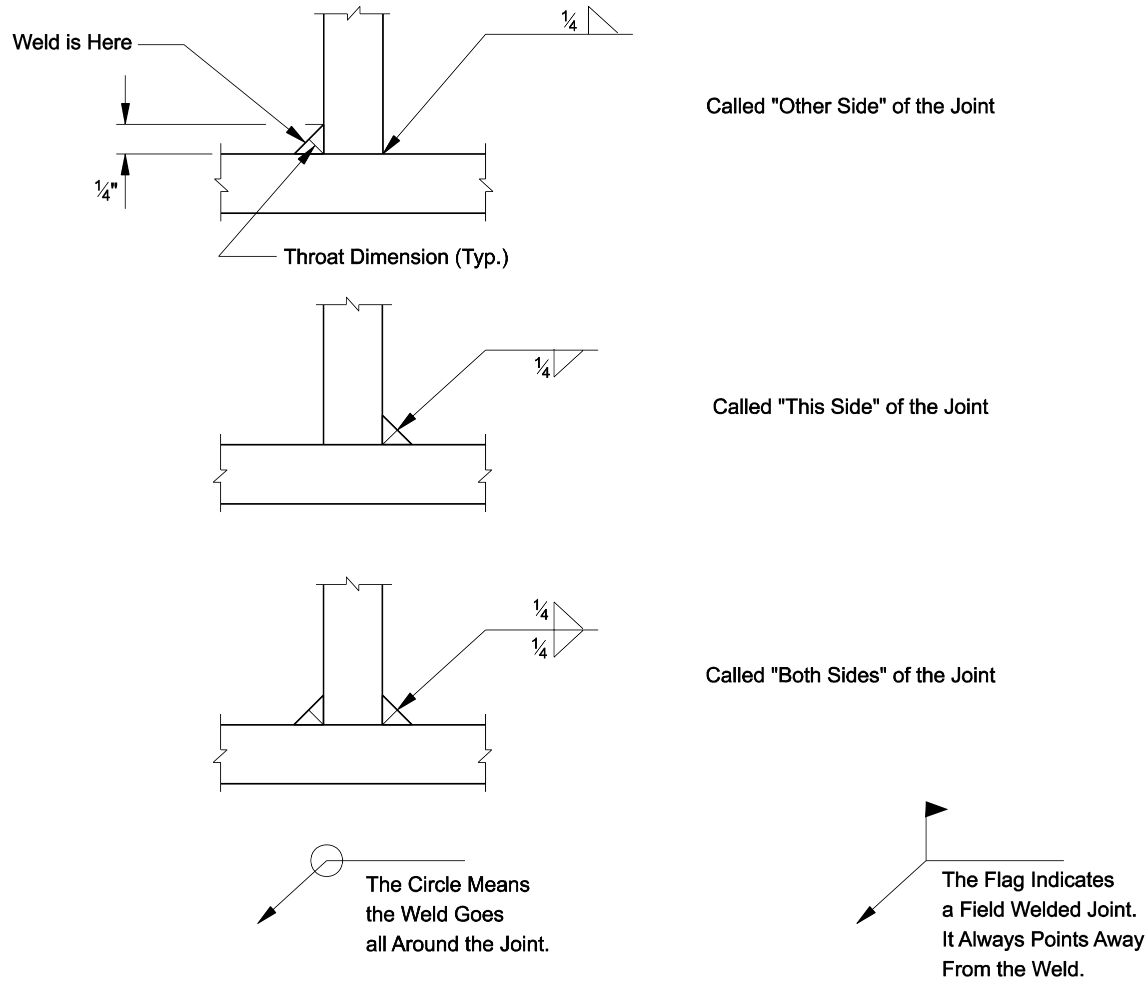
JOINT WITH FILLET WELD

Figure 15.7-A



JOINT WITH GROOVE OR BUTT WELD

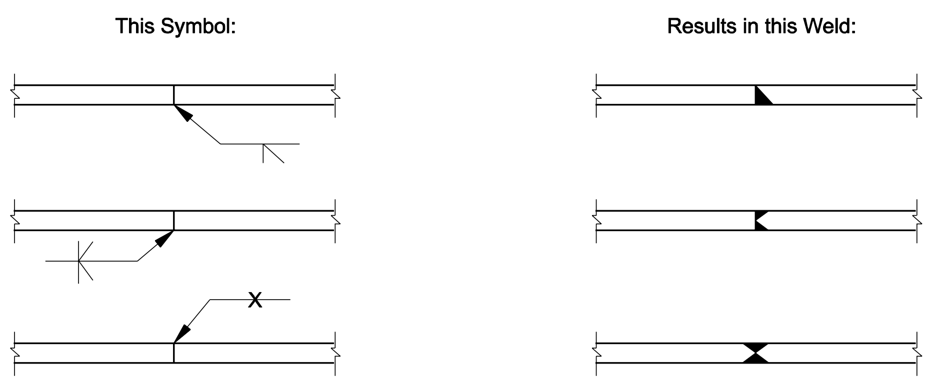
Figure 15.7-B



"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.

FILLET WELDS



BUTT WELDS

WELDING SYMBOLS

Figure 15.7-C

15.7.2.4 Weld-Metal Strength and Electrode Nomenclature

The strength of the weld filler metal is known from the electrode designation. Figure 15.7-D illustrates the standard nomenclature to identify electrodes. The figure represents more than a bridge designer typically needs to know but, as an illustration, a typical pile weld note may say use E7018 or E7028 series electrodes. This means that electrodes with a weld-metal 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. Fillet welds joining higher strength steels are a good application of undermatching. Undermatching weld metal (i.e., specifying a weld-metal strength less than the base metal) can be an attractive alternative when joining higher strength steels (e.g., F_y = 70 ksi and 100 ksi) because undermatching can decrease distortion, residual stresses and cracking tendencies.

15.7.2.5 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 fairly 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 15.7-C. Fillet welds resist load through shear on the throat, while groove welds resist load through tension, compression or shear depending upon the application.

Often, it is best to only show the type and size of the weld required and leave the details 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 specifying 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 to be made in a single pass are preferred because these are most economical and least susceptible to resultant flaws. The maximum weld size for a single-pass fillet weld applicable to all weld types is 5/16 in. 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. The weld should be designed economically, but its size should not be less than ¼ in 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.

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
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

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.

ELECTRODE NOMENCLATURE

Figure 15.7-D

The following types of welds are prohibited:

- field-welded splices,
- intersecting welds,
- intermittent fillet welds (except for the connection of stop bars at expansion joints), and
- partial penetration groove welds (except for the connection of tubular members in hand rails).

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.

15.7.2.6 Inspection and Testing

Indispensable to the reliable use of welding is a systematic program of inspection and testing. Inspection is done at the shop and at the field site. The function of the inspection is to guarantee that specified materials and procedures are used 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 defects 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 extensively to guarantee the quality of the welds. NDOT uses the following types of non-destructive testing (NDT):

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. RT is most effective on full-penetration groove joints with ready access to both sides.

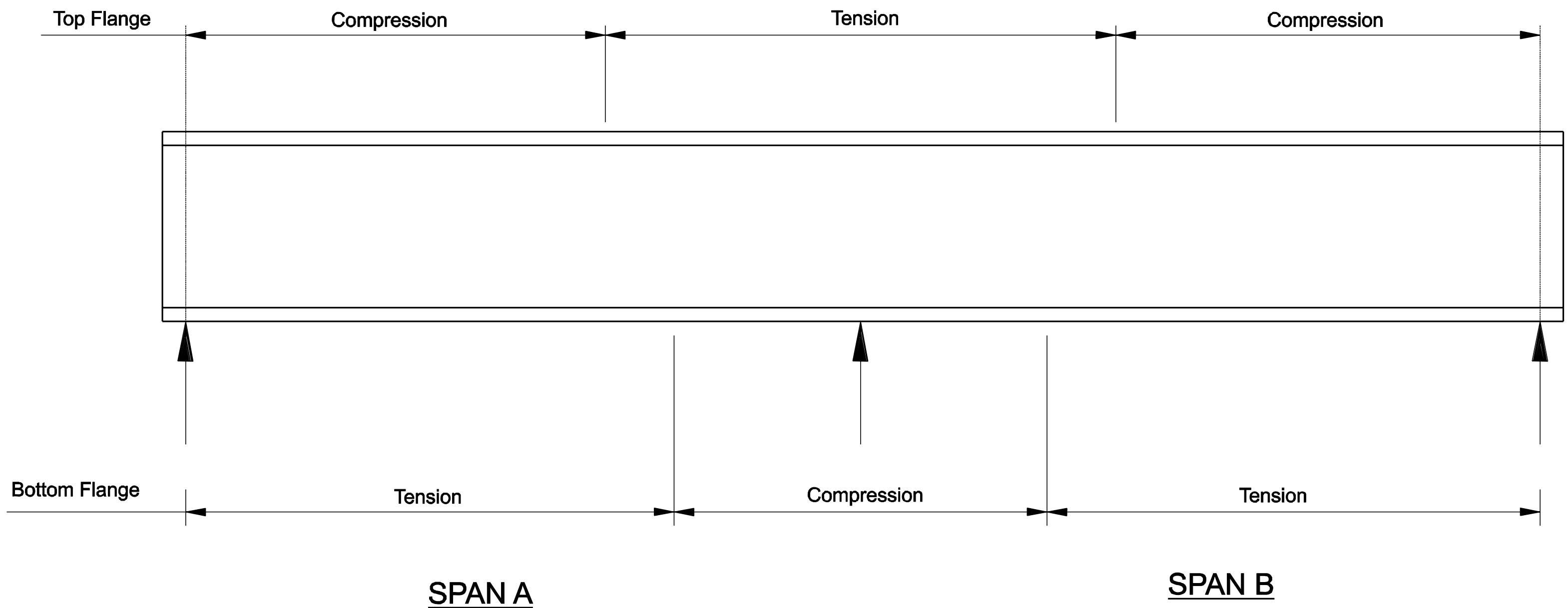
2. Ultrasonic Testing (UT). Relies on the reflection patterns of high-frequency sound waves, which are transmitted at an angle through the work. Cracks and defects interrupt the sound transmission, altering the display on an oscilloscope. UT can reveal many defects that the other methods do not, but it relies very heavily on the interpretative skill of the operator. UT is used on full-penetration groove welds.

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 in the weld interrupt the magnetic force lines, causing the particles to concentrate in the vicinity of the crack in patterns easily interpreted by the inspector. MT is used on fillet welds.

4. Dye Penetrant Testing (DP). Uses 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 cleaned. DP is used to locate surface flaws in and around fillet welds.

5. Eddy Current Testing (ET). Eddy Current testing uses a phenomenon called electromagnetic induction to detect flaws in conductive materials. This form of testing detects flux leakage emanating from a discontinuity in metal when an eddy current is passed through the material. Eddy Current testing can detect very small flaws in or near the surface of the material, the surfaces need minimal preparation, and physically complex geometries can be quickly investigated.

To aid the inspector, the contract documents for continuous structures shall include a sketch showing 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 15.7-E illustrates the information required.



SCHEMATIC OF FLANGE TENSION REGIONS

Figure 15.7-E

15.7.3 Splices

Reference: LRFD Article 6.13.6

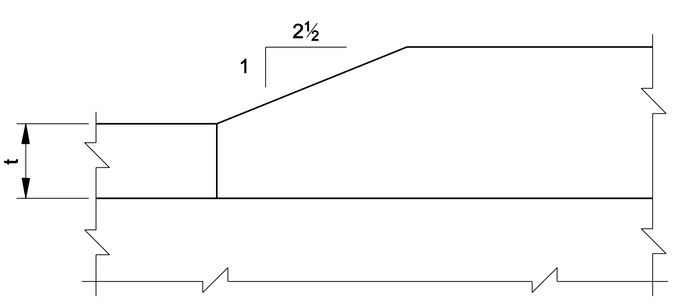
15.7.3.1 Shop Splices

In addition to the provisions of LRFD Article 6.13.6, the following will apply to 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. As such, shop splices on longer span bridges must be located in fairly high positive moment regions.

   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. The maximum length of plates that are normalized, quenched and tempered (70 HPS) is 50 ft. Other plates (e.g., 36 and 50) can be obtained in lengths greater than 80 ft depending on thickness. The cost of adding a shop-welded splice instead of extending a thicker plate should be considered when designing members. Discussions with a fabricator or the NSBA during the design is suggested.

2. Welded Shop Splice. Figure 15.7-F 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 butt welds.



FLANGE SPLICE DETAILS

TYPICAL WELDED SPLICE DETAILS

Figure 15.7-F

15.7.3.2 Field Splices

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

1. Location. In general, field splices in main girders should be located 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.

2. Bolts. Design loads for bolts shall be calculated by an elastic method of analysis. Provide at least two lines of bolts on each side of the web splice.

3. Composite Girder. If a compositely designed girder is spliced at a section where the moment can be resisted without composite action, the splice may be designed as noncomposite. If composite action is necessary to resist the loads, the splice should be designed for the forces due to composite action.

4. Design. Bolted splices must be designed to satisfy both the slip-critical criteria under Service II loads and the bearing-type connection criteria under the appropriate Strength limit states.

5. Swept Width (or shipping width) for Curved Girders. The swept width is the horizontal sweep in a curved girder plus its flange width. Field splices should be located such that the maximum swept width for a horizontally curved girder is 10 ft within a single field section.