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Chapter Eighteen
STRUCTURAL STEEL SUPERSTRUCTURES

Chapter Eighteen discusses structural steel provisions in Section 6 of the **LFRD Bridge Design Specifications** that require amplification, clarification and/or an improved application. The Chapter is structured as follows:

1. Section 18.1 provides general information, mostly relating to cost-effective design practices, for which there is not a direct reference in Section 6 "Steel Structures" of the LRFD Specifications.

2. Sections 18.2 through 18.7 provide information which augments and clarifies Section 6 of the Specifications. To assist in using these Sections, references to the Specifications are provided, where applicable.

Chapter Thirteen of the **Montana Structures Manual** provides criteria for the general site considerations for which structural steel is appropriate. This includes span lengths, depth of superstructure, girder spacing, geometrics, seismic, aesthetics and cost. Chapter Eighteen addresses the detailed design of steel superstructures. Furthermore, the discussion in Chapter Eighteen is restricted to multi-girder steel superstructures. This reflects the popularity of these systems because of their straightforward design, ease of construction and aesthetically pleasing appearance. In addition, with some exceptions, the State of Montana lacks major waterways and/or large ravines that would require trusses, arches or suspension systems. Rigid frames have also been omitted because of their expensive fabrication.

18.1 GENERAL

18.1.1 Economical Steel Superstructure Design

Factors that influence the cost of a steel girder bridge include, but are not limited to, the number of girders, the type of material, type of substructure, amount of material, fabrication, transportation and erection. The cost of 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 the next bridge.

18.1.1.1 Number of Girders

Generally, the fewest number of girders in the cross section as compatible with deck design requirements provides the most economical bridge.

MDT typically uses a minimum of four girders in a bridge cross section. The use of three girders may be considered on low-volume facilities where bridge closure during re-decking is not a problem and the superstructure is not susceptible to impact from overheight vehicles below.

18.1.1.2 Girder Spacing

The optimum girder spacing for a bridge will generally be determined by dividing the distance between the exterior girders into the least number of equal spaces that meet the structural requirements of the design. MDT uses girder spacings between 1.5 m and 4.5 m for most typical multi-girder steel bridges.

The optimum location of the exterior girder is controlled by these factors:

1. Because it is more cost effective to use the same girder design for interior and exterior girders to minimize fabrication costs, the exterior girder should be located to yield similar total moments (dead load plus live load, etc.) as the interior girders. This is typically achieved with an overhang width of approximately 35% to 40% of the girder spacing.

2. On bridges carrying barrier rails, the space required for deck drains may have an effect on the location of the exterior girder lines.

Note: These are general rules of thumb, and some judgment should be exercised to allow for an even beam spacing or specific design requirements.

18.1.1.3 Steel Weight Curves

AISC has prepared Steel Weight Curves based on data obtained over several years for cost-effective girder designs; see Figure 18.1A. The curves are based on the use of the Load Factor Design provisions of the **Standard Specifications for Highway Bridges** and should be considered as an approximation for superstructures designed with the LRFD Specifications when the modification indicated for HS25 loading is made. The Steel Weight Curves should only be used to provide a preliminary estimate of steel weight and a rough check on the economy of new designs.

18.1.1.4 Arrangements

Where pier locations are flexible, optimize the span arrangement. Steel design should not necessarily be associated with the use of longer spans. In selecting an optimum span arrangement, it is critical in all cases to consider the cost of the superstructure and substructure together as a total system.

A balanced span arrangement for continuous spans, with end spans approximately 0.8 of the length of interior spans, results in the largest possible negative moments at the piers, and smaller resulting positive moments and girder deflections. As a result, the optimum depth of the girder in all spans will be nearly the same resulting in a much more efficient design.

18.1.1.5 Rolled Beams versus Plate Girders

For shorter bridges, rolled beams will be more economical than welded plate girders. The major steel producers no longer routinely produce rolled WF sections over 1 m in depth. If rolled beams over 1-m deep must be used, check with the National Steel Bridge Alliance (NSBA) to determine their availability and cost. Regardless, allow the fabricator the option of replacing the rolled beam with an equivalent welded plate girder; i.e., with the same plate thicknesses as the flange and the web of the rolled beam.

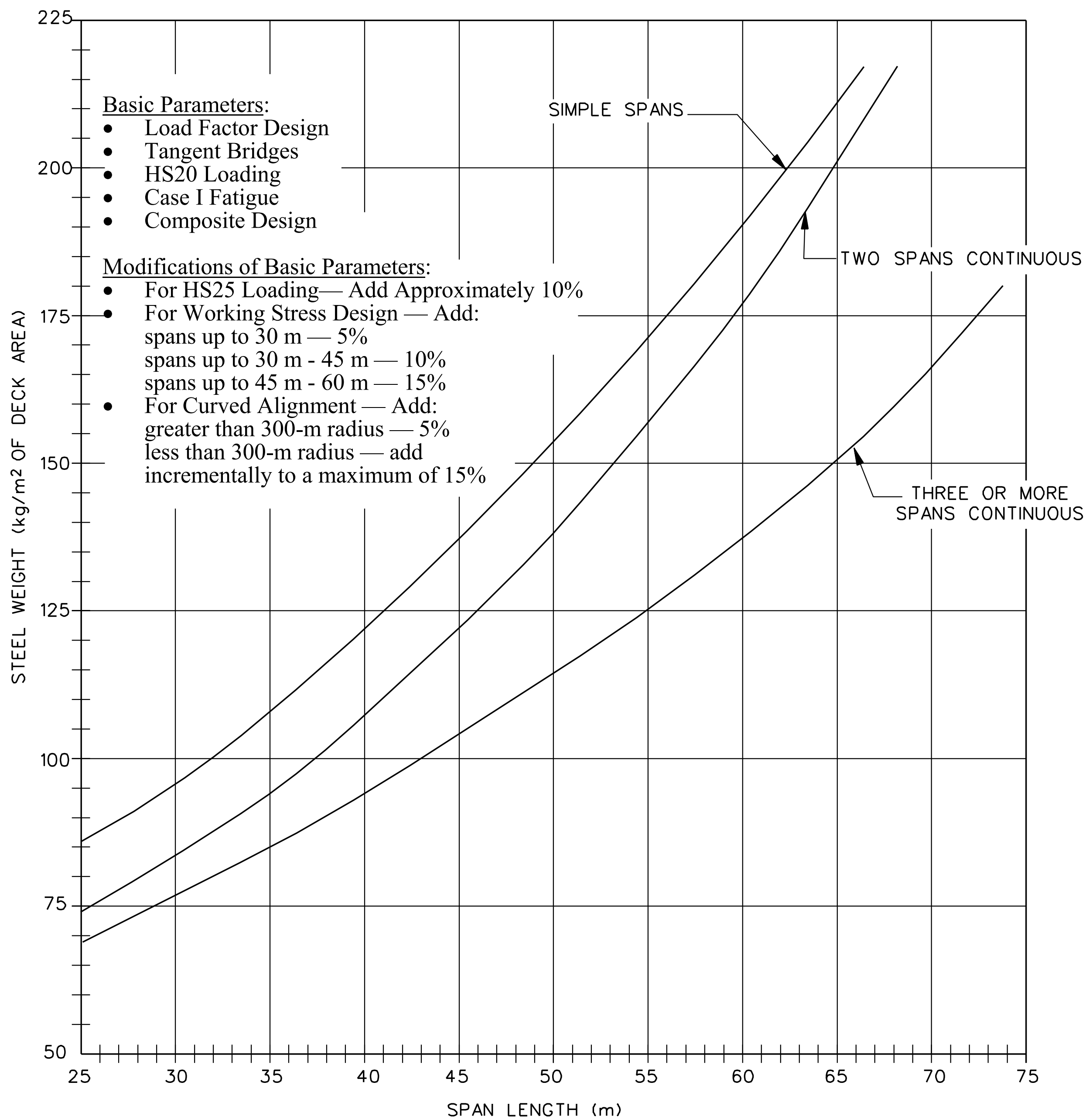
18.1.2 <u>Economical Plate Girder Design</u>

In addition to the information in the LRFD Specifications, the following applies to the design of structural steel plate girders.

18.1.2.1 General

Plate girders should be made composite with the bridge deck and continuous over interior supports where applicable.

To achieve economy in the fabrication shop, all girders in a multi-girder bridge should be identical with the critical girder, usually the interior girder, governing all girder designs. Identical girders, both interior and exterior, can be achieved by limiting overhang lengths as suggested in Section 18.1.1.2.



Notes: 1. *These curves apply to I-girders only.*

2. *For the curve labeled "Three or More Spans Continuous," a balanced span arrangement is assumed (end span equal to approximately 0.8 of the interior spans), and the interior span length should be used with the curve.*

STEEL WEIGHT CURVES

Figure 18.1A

18.1.2.2 Haunched Girders

When practical, constant-depth girders shall be used. Haunched girders are generally uneconomical for spans less than 120 m. They may be used where aesthetics or other special circumstances are involved, but constant-depth girders will generally be more cost-effective.

18.1.2.3 Longitudinally Stiffened Webs

Longitudinally stiffened webs are generally uneconomical for spans less than 90 m. The ends of longitudinal stiffeners are fatigue sensitive if subject to applied tensile stresses. Therefore, they must be ended in zones of little or no applied tensile stresses. In general, do not use longitudinally stiffened webs.

18.1.2.4 Field Splices

Field splices are used to reduce shipping lengths, but they are expensive and their number should be minimized. Field sections should not exceed 38 m in length and 82 000 kg in weight without investigation of permits and shipping 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. It is a good design practice 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.

18.1.2.5 Size of Flange

The minimum flange plate size for built-up girders is 300 mm x 20 mm. Figure 18.1B presents commonly specified metric plate thicknesses. Designers may use a flange width of approximately 20% to 25% of the web depth as a rule of thumb for an initial trial design. Designers should use as wide a flange girder plate as practical consistent with stress and b/t requirements. This contributes to girder stability and reduces the number of passes and weld volume at flange butt welds. Flange widths should always be an even number of mm to avoid 0.5-mm widths when working to flange centerlines; preferably, they should be in increments of 50 mm. The desirable maximum flange thickness is 80 mm.

<table>
<tr><th>Metric Units
(mm)</th><th>Equivalent
English Units
(inches)</th><th>Metric Units
(mm)</th><th>Equivalent
English Units
(inches)</th></tr>
<tr><td>5</td><td>0.1969</td><td>28</td><td>1.1024</td></tr>
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<tr><td>9</td><td>0.3543</td><td>38</td><td>1.4961</td></tr>
<tr><td>10</td><td>0.3937</td><td>40</td><td>1.5748</td></tr>
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<tr><td>16</td><td>0.6299</td><td>60</td><td>2.3622</td></tr>
<tr><td>18</td><td>0.7087</td><td colspan="2" rowspan="4">60 mm - 200 mm, use 10-mm increments.
> 200 mm, use 50-mm increments.
Based on ANSI Standard B32.3. Mills can produce any metric plate size upon request.</td></tr>
<tr><td>20</td><td>0.7874</td></tr>
<tr><td>22</td><td>0.8661</td></tr>
<tr><td>25</td><td>0.9843</td></tr>
</table>

METRIC PLATE THICKNESSES

Figure 18.1B

18.1.2.6 Shop Splices

Use no more than two shop splices (three plate thicknesses) 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. In determining the points where changes in plate thickness occur within a field section, the designer should weigh the cost of butt-welded splices against extra plate area. In many cases, it may be advantageous to continue the thicker and/or wider plate beyond the theoretical step-down point to avoid the cost of the butt-welded splice. Locate shop splices at least 150 mm away from web splices or transverse stiffeners to facilitate testing of the weld.

An understanding of the most economical way of producing flanges in the shop makes this easier to understand. The most efficient way to construct flanges is to butt-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 (Figure 18.1C). This reduces the number of welds, individual runoff tabs to both start and stop welds, the amount of material waste and the number of X-rays for non-destructive testing. The obvious objective, therefore, is for flange widths to remain constant within an individual shipping length by varying material thickness as required. Constant flange width within a field section may not always be practical in girder spans over 100 m where a flange width transition may be required in the negative bending regions.

Because structural steel plate is most economically purchased in widths of at least 1200 mm, it is advantageous to repeat plate thicknesses as much as practical. In the example shown in Figure 18.1C, many of the plates of like width could be grouped by thickness to meet the minimum 1200-mm width purchasing requirement, but the thicker plates do not allow this. In addition, all but the 75-mm plates shown are unique, thereby requiring additional material costs when purchasing plates. Furthermore, each splice must be individually, rather than gang, welded.

The discussion of flange design leads to the question of how much additional flange material can be justified to eliminate a width or thickness transition. Based on the experience of fabricators, some rules of thumb have been developed. Approximately 590 kg of steel should be saved to justify the cost of a transition in a flange plate.

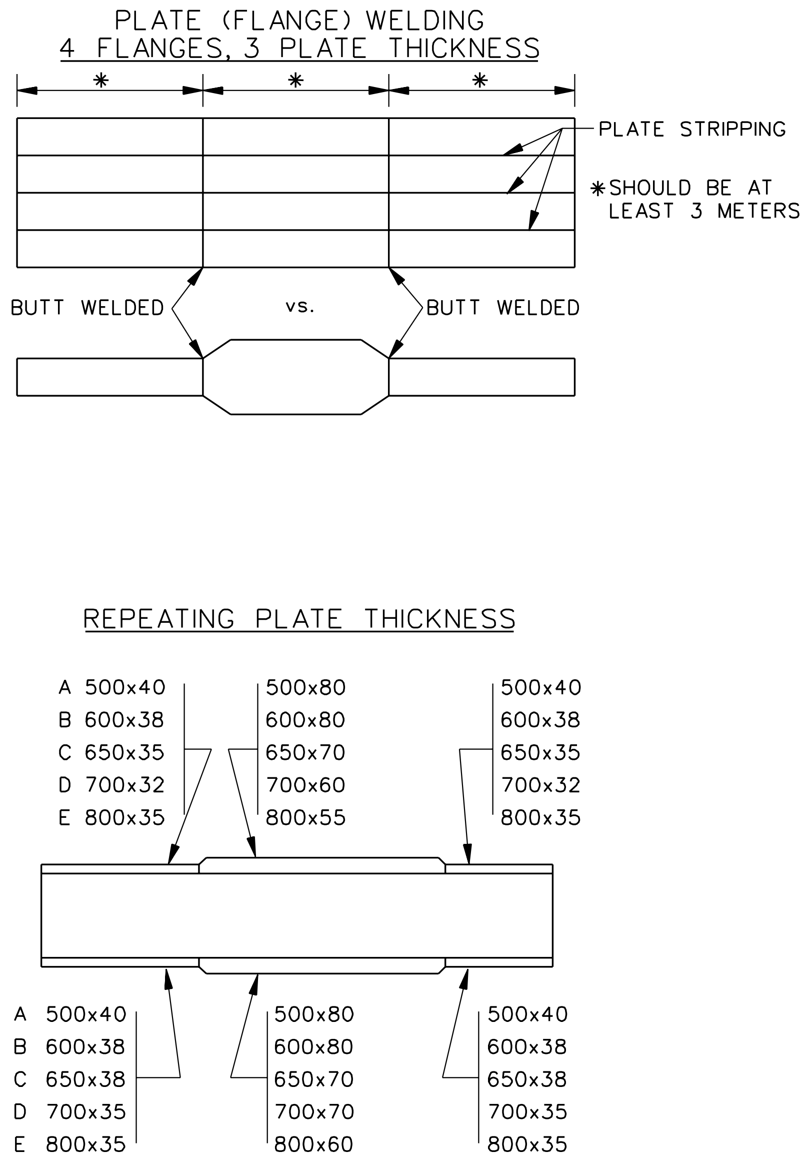
Though not preferred, if a transition in width must be provided, shift the butt splice a minimum of 75 mm from the transition as shown in Figure 18.1C. This makes it much easier to fit run-off tabs, weld and test the splice and then grind off the run-off tabs.

18.1.2.7 Web Plates

Where there are no depth restrictions, the web depth should be optimized. Designers may use preliminary design services available through the NSBA and Bethlehem Steel for the optimization of the web depth. Other programs or methods may also be used if they are based upon material use and fabrication unit costs. Web depths should always be an even number of mm, preferably in increments of 50 mm. The minimum web thickness should be 11 mm. Web thickness should not be changed by less than 5 mm.

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 will not always produce the greatest economy because fabricating and installing stiffeners is one of the most labor-intensive of shop operations. The following guidelines apply to the use of stiffeners:

1. Transversely unstiffened webs are generally more economical for web depths approximately 1250 mm or less.



DETAILS FOR FLANGE TRANSITIONS

Figure 18.1C

2. Between 1250-mm and 1800-mm depths, consider options for a partially stiffened and unstiffened web. A partially stiffened web is defined as one whose thickness is 1.5 mm less than allowed by specification for an unstiffened web at a given depth. Above 1800 mm, consider options for partially stiffened and fully stiffened webs.

18.1.3 Continuous Structures

Span-by-span continuity enhances both the strength and rigidity of the structure. One important benefit of structural continuity is the reduction in the number of deck expansion joints. Open or leaking deck joints may cause extensive damage to girder ends, diaphragms, bearings, bent caps and pier caps. See Chapter Fifteen for more discussion on bridge deck expansion joints.

18.1.4 Horizontally Curved Members

The design of horizontally curved structural steel beams shall be based on the AASHTO **Guide Specifications for Horizontally Curved Highway Bridges**. The following provides additional information.

18.1.4.1 General

The effect of curvature must be accounted for in the design of all steel superstructures where the components are fabricated on horizontal curves. The magnitude of the effect of horizontal curvature is primarily a function of the curve radius, girder spacing, span, diaphragm spacing and, to a lesser degree, depth and flange proportions. The effect of curvature develops in two ways that are either nonexistent or insignificant in tangent bridges. First, the general tendency is for each girder to overturn, which has the effect of transferring both dead and live load from one girder to another transversely. The net result of this load transfer is that some girders carry more load and others less. The load transfer is carried through the diaphragms and the deck. The second effect of curvature is the concept of flange bending caused by torsion in curved components being almost totally resisted by horizontal shear in the flanges. The horizontal shear results in moments in the flanges. The stresses caused by these moments either add to or reduce the stresses from vertical bending. The torsion also causes warping of the girder webs.

The LRFD Specifications currently do not include design provisions for horizontally curved steel bridges. Until such time as LRFD curved girder provisions are developed, any bridge containing a curve segment must be designed by load factor design using the current HS loadings.

18.1.4.2 Methods of Analysis

All curved systems should be analyzed for design by a rigorous structural method or by the V-load method that was published as Chapter 12 of the **USS Highway Structures Design Handbook**, except where curvature is within the limits as specified in Article 4.6.1.2.1 of the LRFD Specifications.

18.1.4.3 Diaphragms, Bearings and Field Splices

Cross frames and diaphragms shall be considered primary members. All curved steel simple-span and continuous-span bridges should have their 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 preferably shall be as close as practical to the full depth of the girders.

For ordinary geometric configurations, no extra consideration need be given to the unique expansion characteristics of curved structures. On occasion, in some urban metropolitan areas (but rarely in Montana), wide sharply curved structures are required. In these circumstances, the designer must consider multi-rotational bearings and selectively providing restraint either radially and/or tangentially to accommodate the thermal movement of the structure.

Design the splices in flanges of curved girders to carry flange bending or lateral bending stresses and vertical bending stresses in the flanges.

Flange tip stresses shall be governed by the Load Factor Design Specifications for allowable stresses and fatigue allowables.

18.1.5 Integral End Bents

Chapter Nineteen discusses the design of integral end bents. The following applies to the use of integral end bents in combination with steel superstructures:

1. Bridge Length. Integral end bents empirically designed may be used with structural steel bridges where the movement does not exceed 50 mm at the abutment and the skew does not exceed 20°. Longer expansion lengths may be used if rational analysis of induced pile stresses indicates that the piles are not overstressed.

2. Deck Pour. Place an interior diaphragm within 3 m of the end support to provide beam stability during the deck pour.

3. Anchorage. Steel beams and girders should be anchored to the concrete cap. A minimum of three holes should be provided through the webs of steel beams or girders to allow #19 bars to be inserted to anchor the beam to the backwall.

18.1.6 Falsework

Steel superstructures should generally be designed with no intermediate falsework during placing and curing of the concrete deck slab.

18.2 MATERIALS

Reference: LRFD Article 6.4

18.2.1 Structural Steel

Reference: LRFD Article 6.4.1

18.2.1.1 Material Type

The most cost-effective choice of steel is unpainted M270 Grade 345W. Recently, HPS 485W steel has been developed that may prove to be as cost effective as 345W. Determine availability, fabrication and estimated cost comparisons to 345W and secure the Bridge Design Engineer's approval before proceeding with HPS 485W design. The initial cost advantage of 345W compared to painted, high-strength steel (e.g., M270 Grade 345) 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 can make the use of painted steel cost prohibitive.

For long-span girder bridges, the more cost-effective solution may be M270 Grade HPS 485W. The premium on material costs is offset by a savings in tonnage. The most cost-effective HPS 485W design solutions tend to be hybrid girders with Grade 345 webs with 485W tension and compression flanges in the negative-moment regions and tension flanges only in the positive-moment regions.

Despite its cost advantage, the use of unpainted weathering steel is not appropriate in all environments and at all locations. The application of weathering steel and its potential problems are discussed in depth in FHWA Technical Advisory "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. Unpainted weathering steel should not be used where any of the following adverse conditions exist:

1. Environment. Unpainted weathering steel should not be used in industrial areas where concentrated chemical fumes may drift onto the structure. If in doubt, its suitability should be determined by a corrosion consultant from the steel industry.

2. Location. Unpainted weathering steel should not be used at grade separations in "tunnel" conditions, which are produced by depressed roadway sections with narrow shoulders between vertical retaining walls, with shallow vertical clearances and with deep abutments adjacent to the shoulders. This "tunnel" effect prevents roadway spray from being dissipated and spread by air currents. Note that there is no evidence of salt spray corrosion where the longitudinal extent of the vertical walls is limited to the abutment itself, and roadway spray can be dissipated on both approaches.

3. Water Crossings. Sufficient clearance over bodies of water should be maintained so that water vapor condensation does not result in prolonged periods of wetness on the steel. In Montana, these situations may occur over stagnant overflow channels or irrigation canals that run full during the irrigation season. If freeboard is minimal and the bridge is wide, do not use weathering steel unless there are geometric limitations that preclude the use of other sections.

Where unpainted weathering steel is inappropriate, and a concrete alternative is not feasible, the most economical painted steel is AASHTO M270 Grade 345 steel in both webs and flanges. These are less expensive than Grade 250 designs. Hybrid designs, such as Grade 345 in flanges and Grade 250 in webs, seldom result in significant economy. The theoretical economy is not achievable because the nominal shear resistance of homogeneous sections is computed by summing the contributions of beam action and the post-buckling, tension-field action. Tension-field action is not currently permitted for hybrid sections.

The FHWA Technical Advisory "Uncoated Weathering Steel in Structures" is an excellent source of information, but its recommendation for partial painting of the steel in the vicinity of deck joints should not be considered the first choice. The best solution is to eliminate deck joints. If a joint is used, consideration should be given to painting all superstructure steel within 3 m of the joint. In shorter bridges, the end joint should be replaced by an integral end bent (see Chapter Nineteen).

18.2.1.2 Details for Unpainted Weathering Steel

When using unpainted weathering steel, the following drainage treatments should be considered to avoid premature deterioration:

1. A groove should be provided at the end of the deck overhang.

2. The number of bridge deck drains should be minimized, the drainage pipes should be generous in size, and they should extend below the steel bottom flange.

3. Eliminate details that serve as water and debris "traps." Seal or paint overlapping surfaces exposed to water. This 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.

4. Consider protecting pier caps and abutment walls with a concrete sealer to minimize staining.

5. Consider wrapping the piers and abutments during construction to minimize staining while the steel is exposed to rainfall.

6. Place a bead of caulk or other material transverse across the top of the bottom flange in front of the substructure elements to prevent water from running off of the flange onto the concrete.

18.2.1.3 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 Montana is Temperature Zone 3.

18.2.2 <u>Bolts</u>

Reference: LRFD Article 6.4.3

For normal construction, high-strength bolts shall be:

1. <u>Unpainted Weathering Steel</u>: 22 mm A325 M (Type 3); Open Holes: 25 mm

2. <u>Painted Steel</u>: 22 mm A325M (Type 1); Open Holes: 25 mm

18.2.3 <u>Other Structural Elements</u>

Reference: None

Grade 250 steel shall not be used for secondary members when unpainted weathering steel is used in the web and flanges. In all cases, steel for all splices shall be the same material as used in the web and flanges of built-up girders.

For steel bridges that will be painted, the designer must specify in a special provision which color will be used. Even when using unpainted weathering steel, a color must be specified if any of the steel will be painted.

18.3 LOADS

18.3.1 Limit States

See Section 14.1.3 for a discussion on the load side of the basic LRFD equation that represents all limit states.

18.3.2 Distribution of Dead Load

See Section 14.2.4 for a discussion on the distribution of dead load.

18.3.3 Live-Load Deflection

Reference: LRFD Articles 2.5.2.6.2 and 3.6.1.3.2

Limitations on live-load deflections are optional in the LRFD Specifications. However, MDT has elected to apply the traditional limitation on live load plus dynamic load allowance deflections of 1/800 of the span length for the design of steel rolled beam and plate girder structures of simple or continuous spans. For structures in urban areas used by pedestrians and/or bicyclists, live-load deflections should be limited to 1/1000 of the span length. The span lengths used to determine deflections shall be assumed to be the distance between centers of bearings or other points of support.

Live-load deflections should be evaluated using the provisions of Articles 2.5.2.6.2 and 3.6.1.3.2 in the LRFD Specifications. In calculating live load deflections, start with a composite section that uses the gross cross-sectional area of the deck. Assume that section applies for the length of the girder. Divide the width of the concrete section by the ratio of the elastic moduli (E_s/E_c) to transform the section before calculating deflections. In effect, the distribution of live loads is the number of loaded lanes divided by the number of girders. The concrete deck should be considered to act compositely with the girder even though sections of the girder may not be designed as composite.

For horizontally curved girders, uniform participation of the girders should not be assumed. Instead, the live load should be placed to produce the maximum deflection in each girder individually in the span under consideration. When multiple lanes are loaded, multiple presence factors should be applied.

18.4 FATIGUE CONSIDERATIONS

Reference: LRFD Article 6.6

In Article 6.6.1, the LRFD Specifications categorizes fatigue as either "load induced" or "distortion induced." Actually, both are load induced, but the former is a "direct" cause of loading, and the latter 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.

18.4.1 Load-Induced Fatigue

Reference: LRFD Article 6.6.1.2

Article 6.6.1.2 provides the framework to evaluate load-induced fatigue. Section 18.4.1 provides additional information on the implementation of Article 6.6.1.2 and defines MDT's interpretation of the LRFD provisions.

Load-induced fatigue is determined by the following:

1. the stress range induced by the specified fatigue loading at the detail under consideration;

2. the number of repetitions of fatigue loading a steel component will experience during its 75-year design life. This is determined by anticipated truck volumes; and

3. the nominal fatigue resistance for the Detail Category being investigated.

18.4.1.1 Fatigue Stress Range

The following applies:

1. Range. The fatigue stress range is the difference between maximum and minimum stresses at a detail subject to a net tensile stress caused by a single design truck which 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 9-m spacing between the 145-kN axles. The dynamic load allowance is 0.15, and the fatigue load factor is 0.75.

2. Regions. Fatigue should only be considered in those regions of a steel member either having a net applied tensile stress, or where the unfactored permanent loads produce a compressive stress less than twice the maximum fatigue tensile stress.

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. This tabularized distribution-factor equations incorporate a multiple presence factor of 1.2, which 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.

18.4.1.2 Stress Cycles

Article 6.6.1.2.5 of the LRFD Specifications defines the number of stress cycles (N) as:

$$N = [(365)(75)(ADTT)(p)]\ (n) \qquad \text{(Eq. 18.4.1)}$$

Where:

ADTT = Average Daily Truck Traffic = the number of trucks per day headed in one direction "averaged" over the design life of the structure. The Department's method of "averaging" is described in the following example problems.

p = the maximum fraction of the total ADTT which will occupy a single lane. As defined in Article

3.6.1.4.2, if one direction of traffic is restricted to:

- 1 lane p = 1.00
- 2 lanes p = 0.85
- 3 or more lanes p = 0.80

n = number of stress range cycles per truck passage. As defined in Article 6.6.1.2.5, for simple and continuous spans not exceeding 12 m, n = 2.0. For spans greater than 12 m, n = 1.0, except at locations within 0.1 of the span length from a continuous support, where n = 1.5.

The term in the brackets of Equation 18.4.1 represents the total accumulated number of truck passages in a single lane during the 75-year design life of the structure. Traffic volumes will, of course, increase over time. See the Project File for traffic growth numbers for a given project. If there is any doubt on the annual growth rates, contact the Rail, Transit and Planning Division.

Examples 18.4.1 and 18.4.2 illustrate the application of traffic growth rates to determine the total fatigue live-load cycles over the 75-year design life of the structure.

18.4.1.3 Fatigue Resistance

Article 6.6.1.2.3 of the LRFD Specifications 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 Category C is appropriate only 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. The Specifications requires that the fatigue stress range for Detail Categories C through E′ must be less than the fatigue resistance for each respective Category.

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 a 75-year design life of the structure. This resistance is defined as $(A/N)^{1/3}$. A Constant Amplitude Fatigue Threshold 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.

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. These imply that fatigue may become a more controlling factor where higher strength steels are used.

* * * * * * * * *

Example 18.4.1

Given: 2-lane rural arterial
Current AADT = 3000 vpd (1500 vpd each direction)
Annual traffic growth rate = 1.5%
Percent trucks = 13%
Two spans, 50-m each
Connection plate located 10 m from the interior support
Unfactored DL stress in bottom flange = 28 MPa compression
Unfactored fatigue stresses in bottom flange using unmodified single-lane distribution factor = 27 MPa tension and 34 MPa compression

Find: Determine the fatigue adequacy at the toe of a transverse connection plate to bottom flange weld.

Solution:

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

- Constant A = 14.4 x 10^{11} MPa^3 (LRFD Table 6.6.1.2.5-1)

- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 82.7 MPa (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: 27(1.15)(0.75)/1.2 =19.4 MPa
- Compression: 34(1.15)(0.75)/1.2 =24.4 MPa
  Fatigue Stress Range =43.8 MPa

Step 3: Determine if fatigue must be evaluated at this location:

- Net tension = (DL stress) − (Fatigue stress)
- Net tension = 28 MPa (Compressive) − 19.4 MPa (Tensile) = 8.6 MPa (Compressive)

Although there is no net tension at the flange, the unfactored compressive DL stress (28 MPa) does not exceed twice the tensile fatigue stress (38.8 MPa). Therefore, fatigue must be considered.

Step 4: Check for infinite life:

First, check the infinite life term. This will frequently control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = ½(\Delta F)_{TH}$ = 0.5(82.7) = 41.35 MPa. Because the fatigue stress range (43.8 MPa) exceeds the infinite life resistance (41.35 MPa), the detail does not have infinite fatigue life.

Step 5: Compute the total number of truck passages over the design life of the structure for both directions of travel:

$$V_T = \frac{365\,V_o\,((1+r)^t - 1)}{r} \qquad \text{(Eq. 18.4.2)}$$

Where:

V_T = Total number of truck passages (both directions)

V_o = Current average daily number of trucks (both directions)

r = Annual traffic growth rate, decimal

t = Design life of structure, years

For this Example:

V_o = (3000) (0.13) = 390
r = 1.5% = 0.015
t = 75 years

Equation 18.4.2 becomes:

$$V_T = \frac{(365)(390)((1+0.015)^{75} - 1)}{0.015}$$

$$V_T = 19.50 \text{ x } 10^6$$

Step 6: Compute the total number of truck passages per direction during the 75-year design life:

V_T / Direction = (19.50 x 10^6)(0.5) = 9.75 x 10^6

Step 7: Determine "p" and "n" for Equation 18.4.1:

- Because this is a two-lane structure carrying bi-directional traffic, all truck traffic headed in one direction can occupy only one lane. Therefore, p = 1.0. See LRFD Article 3.6.1.4.2.

- The span exceeds 12 m and the point being considered is located more than 0.1 of the span length away from the interior support. Therefore, n = 1.0. See Equation 18.4.1.

Step 8: Using Equation 18.4.1, compute the number of stress cycles:

N = [(9.75 x 10^6)(p)] (n)
N = [(9.75 x 10^6)(1.0)] (1.0)
N = 9.75 x 10^6

Step 9: Compute the nominal fatigue resistance:

Fatigue Resistance		75-Year Life	Infinite Life
$(\Delta F)_n$	=	$(A/N)^{1/3}$	$½(\Delta F)_{TH}$

Step 10: Check to see if the detail will have at least a 75-year fatigue life:

$$(\Delta F)_n = (A/N)^{1/3} = [(14.4 \times 10^{11})/(9.75 \times 10^6)]^{1/3} = 52.86 \text{ MPa}$$

The 75-year fatigue resistance (52.86 MPa) exceeds the fatigue stress range (43.8 MPa); therefore, the detail is satisfactory.

Example 18.4.2

Given: 2-lane freeway bridge carrying westbound traffic only
Current AADT = 15,000 vpd (7,500 vpd in WB lanes)
Percent trucks = 22%
Two-span continuous bridge, 50-m each
Area investigated is located 4 m from interior support
Unfactored DL stress in the top flange = 55 MPa Tension
Unfactored fatigue stresses in the top flange using unmodified single lane distribution factor = 39 MPa Tension and 6 MPa 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:

- Constant A = 14.4 x 10^{11} MPa^3 (LRFD Table 6.6.1.2.5-1)
- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 69.0 MPa (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: 39(1.15)(0.75)/1.2 = 28.0 MPa
- Compression: 6(1.15)(0.75)/1.2 = 4.3 MPa

Fatigue Stress Range = 32.3 MPa

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 will frequently control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = ½(\Delta F)_{TH} = 0.5(69.0) = 34.50$ MPa. Because the fatigue stress range (32.3 MPa) is less than the infinite life resistance (34.50 MPa), the detail has infinite fatigue life and there is no need to check the 75-year fatigue life. The detail is satisfactory.

Provisions for investigating the fatigue resistance of shear connectors are provided in LRFD Articles 6.10.7.4.2 and 6.10.7.4.3.

* * * * * * * * * *

18.4.2 Distortion-Induced Fatigue

Reference: LRFD Article 6.6.1.3

Article 6.6.1.3 of the LRFD Specifications provides specific detailing practices for transverse and lateral connection plates intended to reduce significant secondary stresses which could induce fatigue crack growth. The provisions of the Specifications are concise and direct and require no mathematical computation; therefore, no further elaboration on distortion-induced fatigue is necessary.

18.4.3 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 on the plans.

In addition to the considerations in Section 18.4.1, the designer should be aware of the fatigue provisions in other Articles of Chapter 6 of the LRFD Specifications. They include:

1. Fatigue due to out-of-plane flexing in webs of plate girders — LRFD Article 6.10.6.

2. Fatigue at shear connectors — LRFD Articles 6.10.7.4.2 and 6.10.7.4.3.

3. Bolts subject to axial-tensile fatigue — LRFD Article 6.13.2.10.3.

18.5 GENERAL DIMENSION AND DETAIL REQUIREMENTS

Reference: LRFD Article 6.7

18.5.1 Design Information Table

For continuous structures, the drawings shall include a Design Information Table.

18.5.2 Dead-Load Camber

Reference: LRFD Article 6.7.2

18.5.2.1 General

Plate girders must be cambered to compensate for the vertical curve offset combined with the sum of load deflections due to composite and non-composite dead loads. Camber will be calculated to the nearest 1 mm. Dead load should include the weight of the steel, the deck and railing, and the future wearing surface. The effects of vertical curvature and superelevation should be considered. All beams and girders should be assumed to equally contribute to flexural resistance. Unfactored force effects should be used to determine the deflections. Calculated deflections are increased by 10% before entry into the Design Information Table to account for shrinkage and creep in the concrete.

18.5.2.2 Diagram

When the maximum total camber exceeds 5 mm, the plans must include a diagram and a table showing camber coordinates resulting from the effects listed in Section 18.5.2.1. Figure 18.5A provides an example of a camber table and an explanatory diagram.

The diagram shows vertical deflections with respect to a straight line ("string line") connecting the top of the girder web at the centerline of bearing at abutments and the top of the web at the centerline of bearing at intermediate bents. Bridges with variable superelevation must also have a String Line Slope Table that shows the slope of the string line for each span of each girder.

The camber table provides ordinates from the string line in millimeters at span tenth points and at girder field splices. At each of these locations, it provides an ordinate for the deflection due to the dead load of the girder and diaphragms alone and for the deflection due to total dead load. The table also provides an ordinate to match the roadway vertical curvature and the variation in the deck cross slope, then totals these ordinates at each location. The fabricator uses the total camber to shape the girder web. The contractor uses the two dead load ordinates to set deck forms.

Figure 18.5B shows the calculation of girder throw at the top of the web and at the top of the slab. These quantities provide the perpendicular offset between a vertical line and a line perpendicular to a tangent to the roadway surface. The angle separating the two lines matches the slope of the roadway vertical curvature at that location. The two lines intersect at the bottom of the web, called out as the Working Point in the Figure. The fabricator places bearing stiffeners along the line perpendicular to the profile grade tangent. Under load, the stiffeners will rotate to a vertical or more nearly vertical position. The plans show these throw quantities at each bearing in a table. Figure 18.5C contains an example of how this information appears on the plans.

18.5.3 Minimum Thickness of Steel

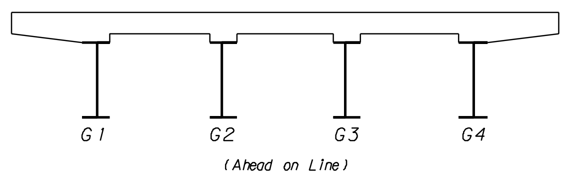
Reference: LRFD Article 6.7.3

The thickness of steel elements should not be less than:

1. Webs Cut From Plates: 11 mm.

2. Plate Girder Flanges: 20 mm.

TABLE OF DESIGN INFORMATION



D.L. = Dead Load
L.L. = Live Load
I. = Impact
T = Truck Loading
L = Lane Loading

(+) = Compression in Top Flange
(-) = Compression in Bottom Flange
Shears & Reactions in Kilonewtons
Moments in Kilonewton-Meters
Deflections in Millimeters

		SPAN 1 AND SPAN 4		SPAN 2 AND SPAN 3		BENT 1 AND BENT 5		BENT 2 AND BENT 4		BENT 3	
		INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER
Moment	D.L.	3135	2591	2162	1796	—	—	-7423	-6153	-7087	-5899
	L.L.+I.	3143 (T)	2494 (T)	3257 (L)	2581 (L)	—	—	-4024 (L)	-3218 (L)	-4355 (L)	-3483 (L)
Reaction or Shear	D.L.	779	647	748	624	416	344	1527	1271	1471	1229
	L.L.+I.	473 (L)	376 (L)	488 (L)	387 (L)	378 (L)	300 (L)	816 (L)	649 (L)	834 (L)	663 (L)
Deflection	D.L.	73	60	62	51	—	—	—	—	—	—
	L.L.+I.	36	30	51	43	—	—	—	—	—	—

NOTES

Fabricate intermediate diaphragm stiffeners and web splices perpendicular to top girder flange.

D.L. deflections are directly proportional to D.L. weights which are as follows:
Total D.L. = 23.5 kN/m of Exterior Girder (Average)
Total D.L. = 28.2 kN/m of Interior Girder (Average)
D.L. Structural Steel = 5.99 kN/m of Girder (Average)

Total dead load deflections have been increased by 10% to allow for deflections due to shrinkage and creep of the concrete slab.

Deflections in the table preceded by a minus sign indicate upward deflections.

TABLE OF CAMBER INFORMATION

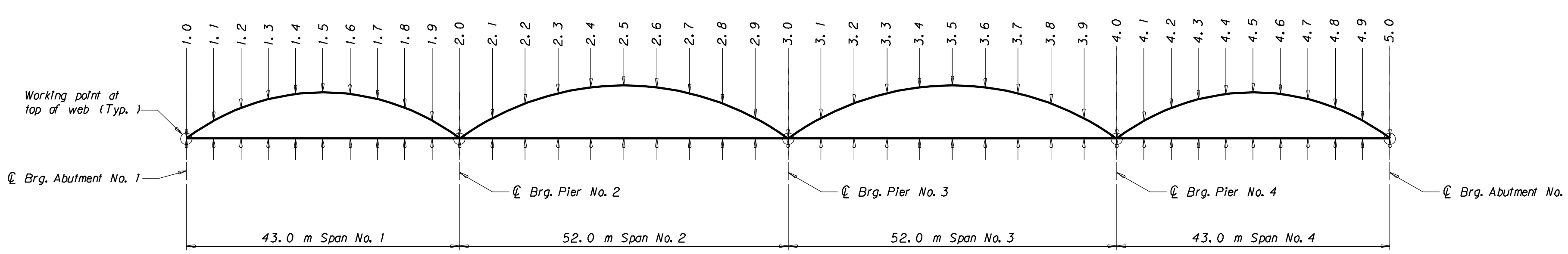
Location		SPAN NO. 1									
		1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9
Interior Girder	Diaphragm and Girder Deflection	0	5	10	13	14	13	10	6	3	2
	Total D.L. Deflection	0	29	52	68	73	68	56	38	19	6
Exterior Girder	Diaphragm and Girder Deflection	0	5	10	11	13	11	10	6	3	2
	Total D.L. Deflection	0	24	44	59	64	59	48	33	17	5

Location		SPAN NO. 2									
		2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9
Interior Girder	Diaphragm and Girder Deflection	0	2	5	8	11	11	11	8	5	2
	Total D.L. Deflection	0	6	21	38	52	59	54	40	22	6
Exterior Girder	Diaphragm and Girder Deflection	0	2	5	8	10	11	10	8	5	2
	Total D.L. Deflection	0	5	17	33	46	51	46	35	19	6

Location		SPAN NO. 3									
		3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9
Interior Girder	Diaphragm and Girder Deflection	0	2	5	8	11	11	11	8	5	2
	Total D.L. Deflection	0	6	22	40	54	59	52	38	21	6
Exterior Girder	Diaphragm and Girder Deflection	0	2	5	8	10	11	10	8	5	2
	Total D.L. Deflection	0	6	19	35	46	51	46	33	17	5

Location		SPAN NO. 4									
		4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.7	4.8	4.9
Interior Girder	Diaphragm and Girder Deflection	0	2	3	6	10	13	14	13	10	5
	Total D.L. Deflection	0	6	19	38	56	68	73	68	52	29
Exterior Girder	Diaphragm and Girder Deflection	0	2	3	6	10	11	13	11	10	5
	Total D.L. Deflection	0	5	17	33	48	59	64	59	44	24

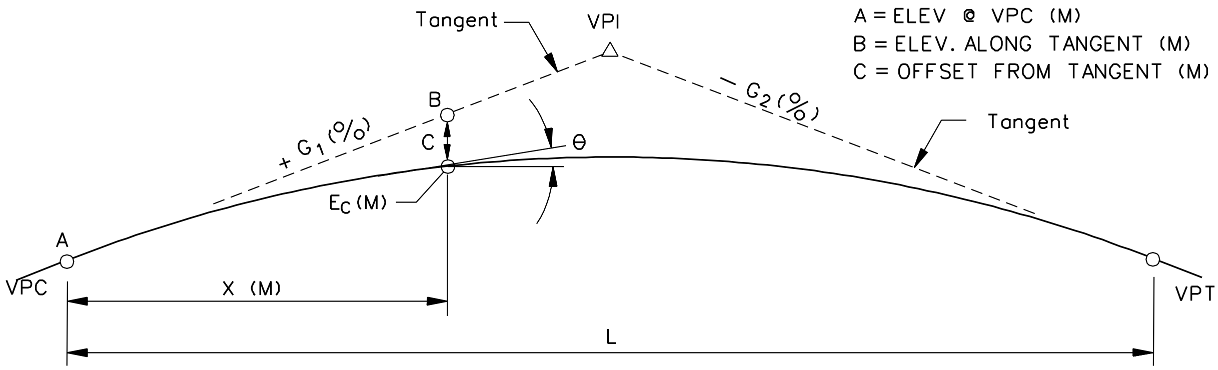
FIGURE 18.5A



GIRDER CAMBER DIAGRAM
No Scale

BRIDGE OVER MIDDLE FORK FLATHEAD RIVER
AT STA. 433+45.0
FEDERAL AID PROJECT NO. BR 1-2(86)180
FLATHEAD COUNTY

CAMBER DETAILS, DESIGN INFORMATION AND NOTES
No Scale

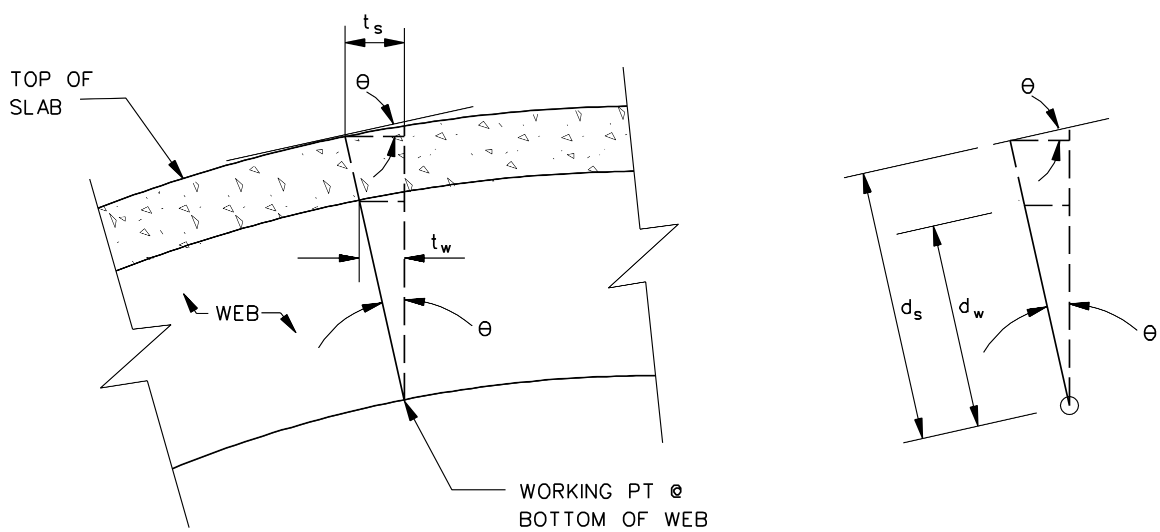


Curve Equation: $E_c = A + B + C$

$$E_c = E_{VPC} + G_1 \frac{X}{100} + \left(\frac{G_2 - G_1}{200L}\right) X^2$$

Slope of Curve @ Point X (Radians): $\theta = \frac{dE_c}{dx} = \frac{G_1}{100} + \frac{G_2 - G_1}{100L} X$

where: E_c = Elevation of Curve at Point X (m)



t_s=THROW @ TOP OF SLAB

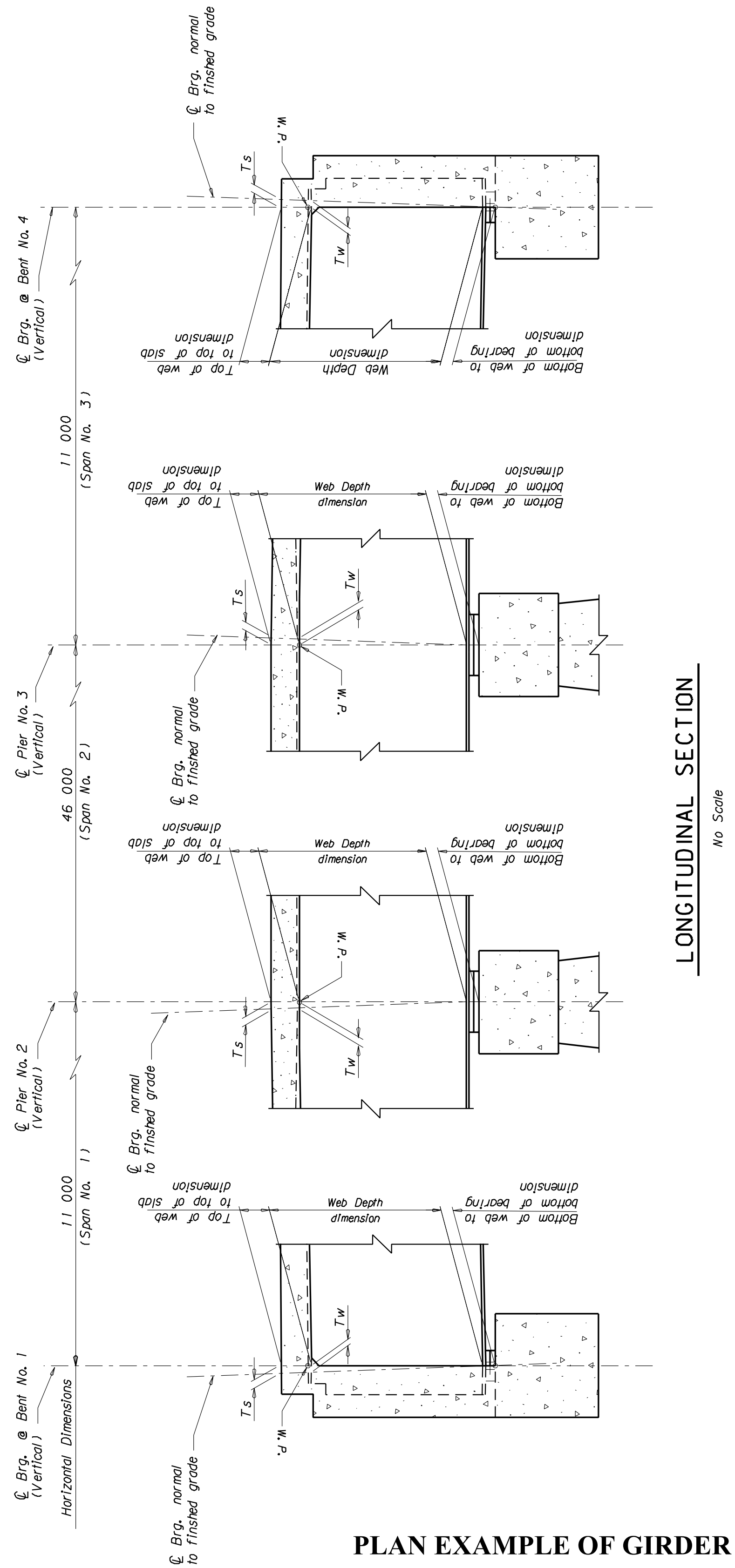
t_w=THROW @ TOP OF WEB

FOR SMALL θ, SINθ ≅ θ (RADIANS)
THEREFORE:

$\boxed{t_s = d_s\theta}$ & $\boxed{t_w = d_w\theta}$

THROW CALCULATIONS

Figure 18.5B



GIRDER THROW CALCULATIONS

	Bent No. 1		Bent No. 2		Bent No. 3		Bent No. 4	
	Ts	Tw	Ts	Tw	Ts	Tw	Ts	Tw
Girder 1	21	18	0	0	0	0	17	15
Girder 2	14	12	0	0	0	0	13	11
Girder 3	7	6	0	0	0	0	7	6
Girder 4	1	1	0	0	0	0	0	0

Ts = Throw @ top of slab
Tw = Throw @ top of web
W.P. = Working Point

STRINGLINE SLOPE TABLE (%)

	Span No. 1	Span No. 2	Span No. 3
Girder 1	1.13	0	-0.89
Girder 2	0.74	0	-0.67
Girder 3	0.36	0	-0.35
Girder 4	0.04	0	-0.01

PLAN EXAMPLE OF GIRDER THROW

Figure 18.5C

18.5.4 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 during and after construction and 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 7.6 m. Most economical, modern steel girder designs will use spacings typically greater than 7.6 m.

18.5.4.1 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 any conflict between the connection plates of the diaphragms or cross-frames and the splice material.

2. Skew. Regardless of the angle of skew, place all intermediate diaphragms and cross-frames perpendicular to the girders. The intermediate diaphragms and cross-frames should be continuous across the cross section and not staggered.

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

18.5.4.2 Diaphragm Details

On spans composed of rolled beams, diaphragms at continuous supports and at intermediate span points may be detailed as illustrated in Figure 18.5D. Figure 18.5E illustrates the typical end diaphragm connection details for rolled beams. Plate girders with web depths of 1060 mm or less should have the same diaphragm details. For plate girder webs more than 1060 mm deep, use cross-frames as detailed on Figure 18.5F.

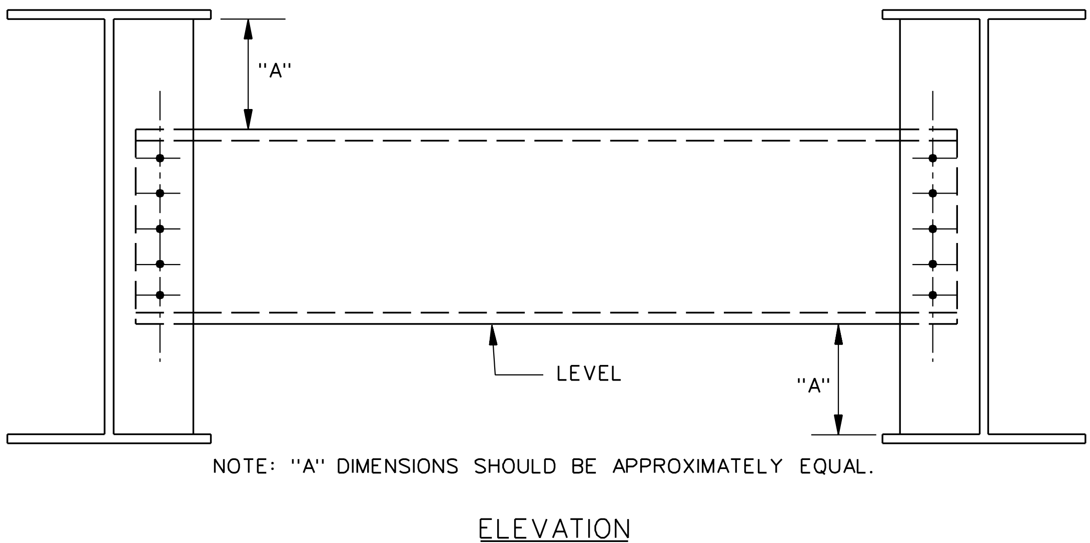
Intermediate diaphragms should be designed and detailed as nonload bearing. Diaphragms at points of support should be designed as a jacking frame, if needed, to support dead load only. See Section 18.5.5 to determine if the need exists. Jacking diaphragms should be considered at all supports when it is impractical to jack the girders directly.

18.5.4.3 Cross-Frame Details

Figure 18.5F illustrates typical cross-frame details. In general, the X-frame at the top of the Figure is more cost effective than the K-frame at the bottom. However, the K-frame should be used instead of the X-frame when the girder spacing becomes much greater than the girder depth and the "X" becomes too shallow.

Montana practice requires that cross-frame transverse connection plates, where employed, be welded to both the tension and compression flanges. The connection plate welds to the flanges should be designed to transfer the cross-frame forces into the flanges.

Connection plates should be fillet welded near side and far side to flanges. The flange welds should conform to the details shown in Figure 18.5G.

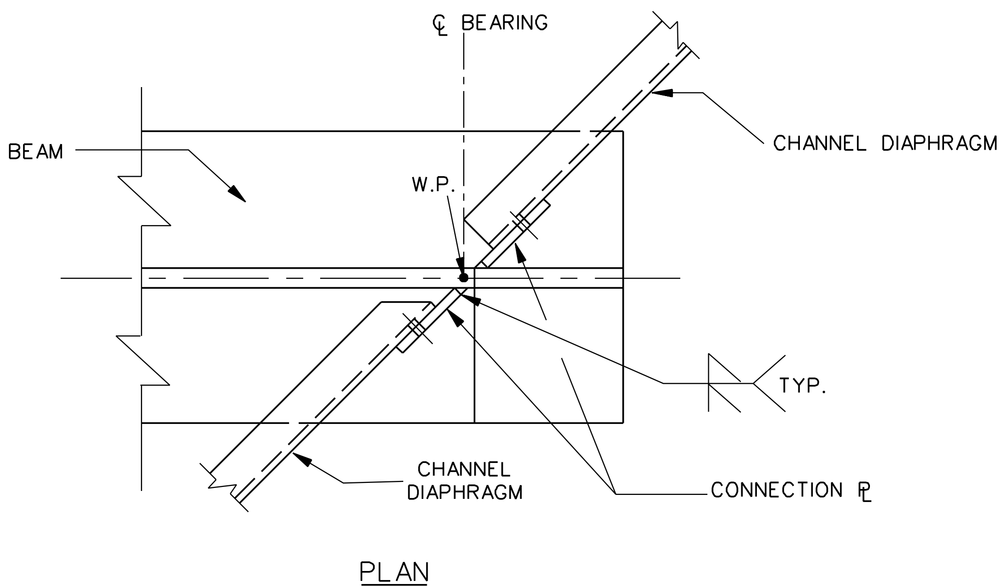


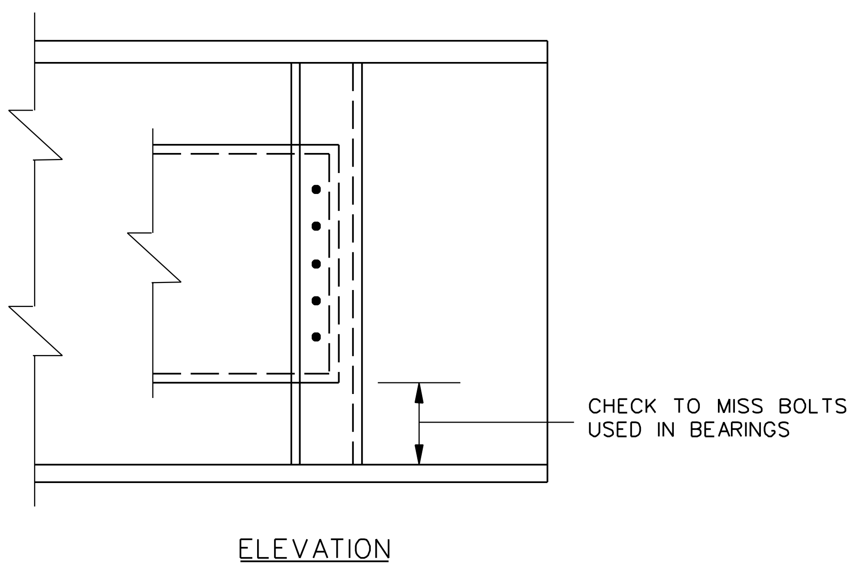
Beam	Diaphragm*	High-Strength Bolts
W920	C 460 x 63.5	5-M22
W840	C 460 x 63.5	5-M22
W760	C 380 x 50.4	4-M22
W690	C 380 x 50.4	4-M22
W610	C 310 x 30.8	3-M20

**Select a channel depth approximately one-half of the web depth.*

TYPICAL INTERMEDIATE DIAPHRAGM CONNECTION
(Rolled Beams)

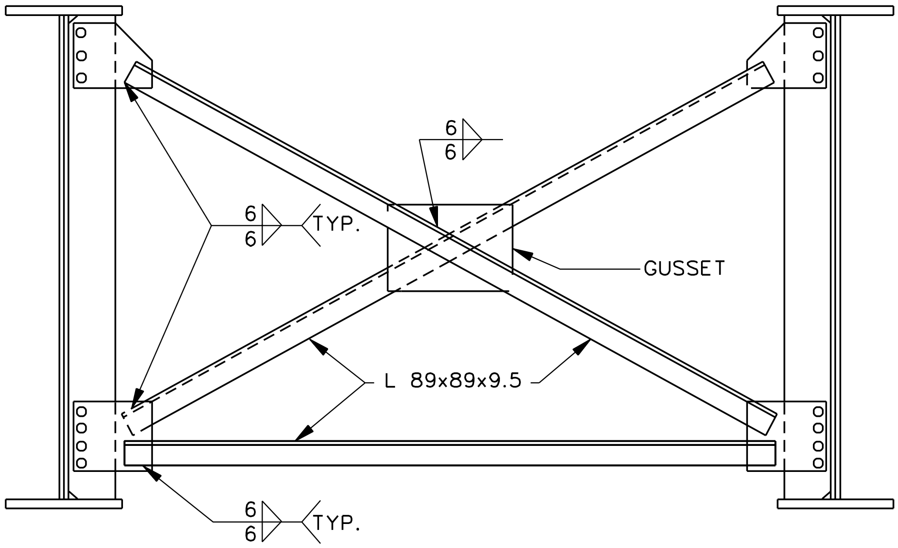
Figure 18.5D

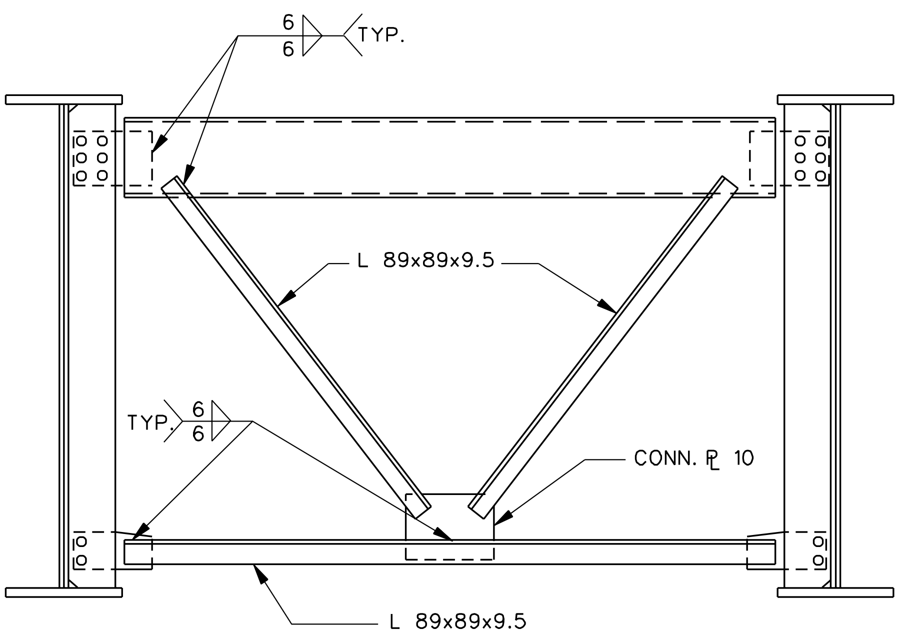




TYPICAL END DIAPHRAGM CONNECTION
(Rolled Beams)

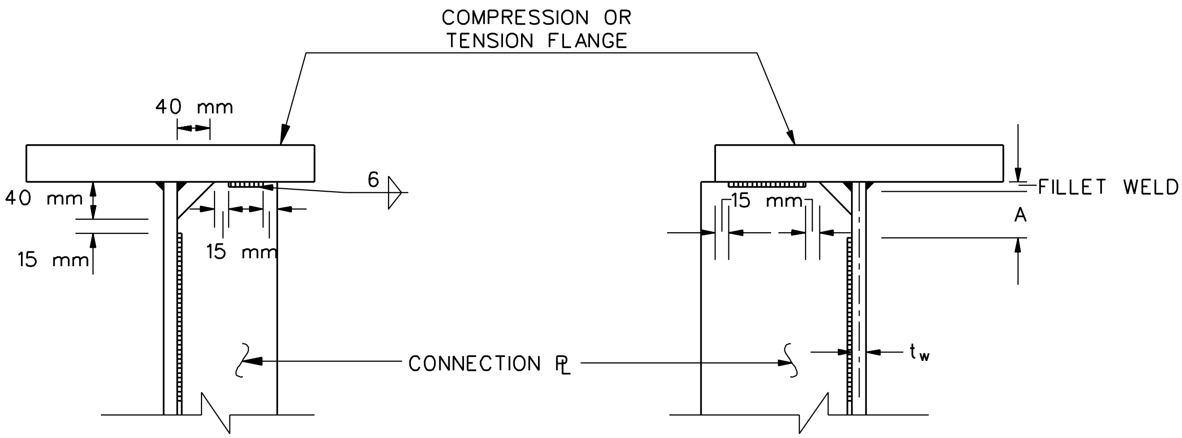
Figure 18.5E





TYPICAL CROSS-FRAMES

Figure 18.5F



WELDED CONNECTION PLATE ATTACHMENT AT COMPRESSION OR TENSION FLANGE

Figure 18.5G

The width of connection plates should be sized to use bar stock and be not less than 125 mm. When the connection plate also acts as a transverse stiffener, it shall meet the requirements of LRFD Article 6.10.8.1.

18.5.5 Jacking

Reference: LRFD Article 3.4.3

The proper interpretation of the LRFD Specifications is that the plans should indicate designated points of jacking and whether or not the structure is capable of resisting 1.3 times the dead load reactions at those points. Slender beams may require web stiffeners at the jacking points. These stiffeners may either be part of the construction plans or fastened to the girder when and if the jacking is required. In general, jacking frames will not be required at the supports unless there is insufficient clearance between the bottom of beam and top of cap to place a jack. If less than 180 mm clearance is available for the jack, then the designer must decide whether the jack can be supported by temporary falsework. If temporary falsework is not feasible, then a jacking frame should be provided or the cap widened and the bearings placed on pedestals to provide sufficient space for a jack to be placed under the beam. Other locations where jacking may be required are:

1. at supports under expansion joints where joint leakage could deteriorate the girder bearing areas; and

2. at large displacement expansion bearings where deformation induced wear-and-tear is possible.

If no jacking frame is provided, then the cross-frame at the support still must be capable of transferring lateral wind forces to the bearings. For continuous structures using integral end bents, providing jacking frames at interior supports should not be considered.

18.5.6 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 and, if the bracing is included in the structural model used to determine force effects, it should be designed for all applicable limit states.

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

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

18.6 I-SECTIONS IN FLEXURE

Reference: LRFD Article 6.10

18.6.1 General

Reference: LRFD Article 6.10.1

18.6.1.1 Negative Flexural Deck Reinforcement

Reference: LRFD Article 6.10.3.7

Article 6.10.3.7 of the LRFD Specifications specifies that, in the negative moment area, 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) 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. However, the designer shall also ensure that sufficient negative moment steel is provided for the applied loads.

18.6.1.2 Rigidity in Negative Moment Areas

Reference: LRFD Articles 6.10.3.4 and 6.10.5

Article 6.10.3.4 of the LRFD Specifications permits assuming uncracked concrete in the negative moment areas for member stiffness. This 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 6.10.5 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 6.10.3.7.

18.6.2 Strength Limit States

Reference: LRFD Article 6.10.4

Moment redistribution according to elastic procedures (i.e., a 10% reduction in elastic negative support moments accompanied by a statically equivalent increase in the positive moments in adjacent spans) will be permitted for continuous spans if $F_y \leq 345$ MPa and if the negative moment support sections are compact.

18.6.3 Service Limit State Control of Permanent Deflection

Reference: LRFD Article 6.10.5

Moment redistribution is permitted for the investigation of permanent deflections.

18.6.4 Shear Connectors

Reference: LRFD Article 6.10.7.4

Shear connectors should be welded studs of which 22-mm diameter welded studs are preferred; the minimum diameter of studs is 19 mm. Shear connectors should have a minimum 64-mm concrete cover and should penetrate at least 50 mm above the bottom of the deck slab. The minimum longitudinal shear connector pitch is six stud diameters, and the maximum pitch is 600 mm.

The minimum number of studs in a group is two in a single transverse row. The transverse spacing, center to center, of the studs should be not less than four stud diameters. The minimum clear distance between the edge of the beam flange and the edge of the nearest stud shall be 25 mm. Details and spacing of stud shear connectors shall be detailed on the plans.

18.6.5 Stiffeners

Reference: LRFD Article 6.10.8

18.6.5.1 Transverse Intermediate Stiffeners

Reference: LRFD Article 6.10.8.1

Straight girders may be designed without intermediate transverse stiffeners, if economical, or with intermediate transverse stiffeners placed on one side of the web plate. In fact, if required, fascia girders should only have stiffeners on the inside face of the web. 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 web. 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 it is decided to proceed 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 need not and, for economical reasons, 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 $6t_W$. See Figure 18.6A for details.

Transverse stiffeners, except at 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 6.10.8.1.

Longitudinal stiffeners used in conjunction with transverse stiffeners on spans over 80 m 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.

18.6.5.2 Bearing Stiffeners

Reference: LRFD Article 6.10.8.2

Bearing stiffeners are required at the bearing points of rolled beams and plate girders. Bearing stiffeners at integral end bents may be designed for dead load only. Design the stiffeners as columns and extend stiffeners to the outer edges of the bottom flange plates. The weld connecting the stiffener to the web should be designed to transmit the full bearing force from the stiffener to the web due to the factored loads. The bearing stiffeners may be either milled to fit against the flange through which they receive their reaction or welded to the flange with full penetration groove welds. See Figure 18.6B for details.

18.6.6 Cover Plates

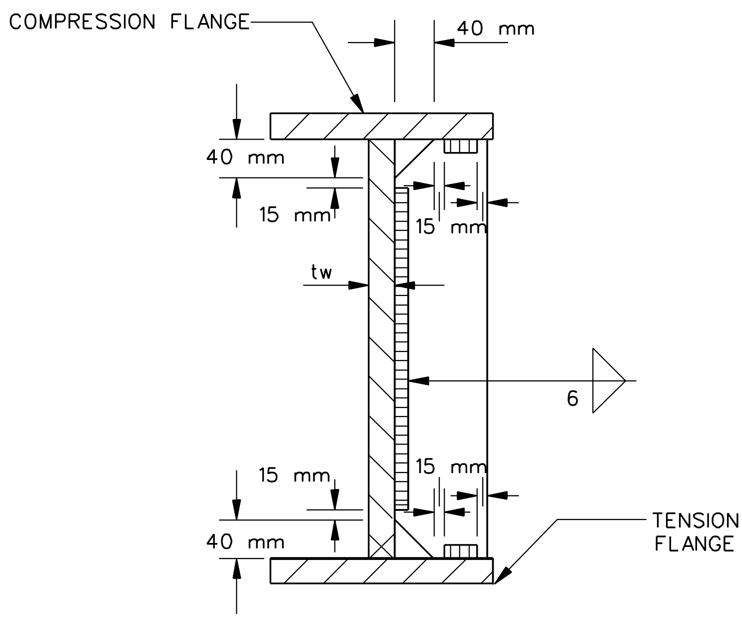
Reference: LRFD Article 6.10.9

Article 6.10.9.1 of the LRFD Specifications specifies that partial length cover plates should not be used with flange plates whose thickness exceeds 20 mm in non-redundant load path structures. According to LRFD Article 1.3.4, those elements and components whose failure is not expected to cause collapse of the bridge should not be designated as failure-critical and the associated structural system as redundant. The thickness of a single cover plate should not exceed twice the thickness of the flange plate. Multiple cover plates should not be employed. The width of the cover plate should be different from that of the flange plate to allow proper placement of the weld.

18.6.7 Constructibility

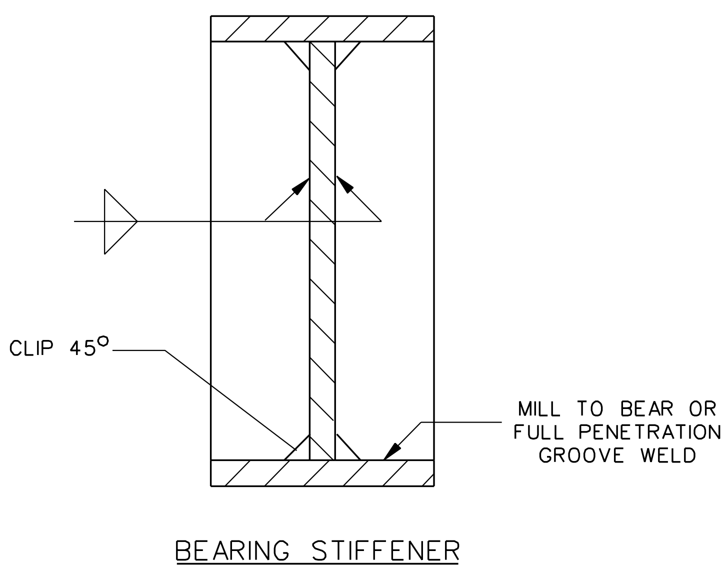
Reference: LRFD Article 6.10.3.2

Wind load, before the deck is placed, is transmitted to the piers by the structure acting as a lateral beam. Because of the diaphragms or



TRANSVERSE INTERMEDIATE STIFFENER DETAILS

Figure 18.6A



BEARING STIFFENER

Figure 18.6B

cross-frames present, the girders equally share the wind load. Normally, the structure can sustain this wind load without overstress.

18.6.8 Inelastic Analysis Procedures

Reference: LRFD Article 6.10.10

More thorough investigations using inelastic analysis procedures are generally not warranted and should not be used unless approved in advance by the Bridge Design Engineer.

18.7 CONNECTIONS AND SPLICES

Reference: LRFD Article 6.13

18.7.1 Bolted Connections

Reference: LRFD Article 6.13.2

The following applies to bolted connections:

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

2. Design. All bolted connections shall be designed 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 (K_s). Use Class A surface condition for the design of slip-critical connections.

18.7.2 Welded Connections

Reference: LRFD Article 6.13.3

18.7.2.1 Welding Process

Welding is performed by fusing two pieces of metal together so that they become continuous. The process of melting and cooling resembles the conditions under which the steel was originally made, and the characteristics of the weld can be very much like the adjacent steel.

Welded connections are the most common types of connections used in shop fabrication today. During the 1950s, welding fabrication replaced riveted fabrication as steel specifically formulated for weldability replaced A7 and other older steel chemistries.

The governing specification for welding is the ANSI/AASHTO/AWS **Bridge Welding Code D1.5**. This single specification contains almost all requirements for the welding fabrication of bridges. The engineer needs to be aware, however, that 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:

1. AWS D1.1 for welding of tubular members and strengthening or repair of existing structures, and

2. AWS D1.4 if a situation arises where the welding of reinforcing steel must be covered by a specification.

This Code accepts as *prequalified* (i.e., acceptable without further proof of suitability if applied under specified conditions) four welding processes using electric arcs. These are:

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

2. submerged arc welding (SAW);

3. gas metal arc welding (GMAW). This process is also sometimes called *metal inert gas welding* or MIG; and

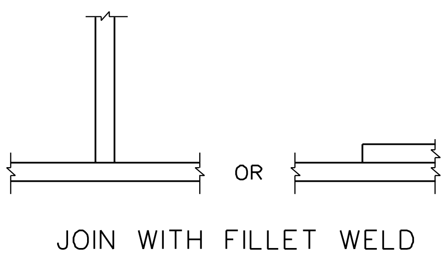
4. flux-cored arc welding (FCAW).

Of these, 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 fitting up components prior to production welding.

Acceptable procedures for using these processes or others requires 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.

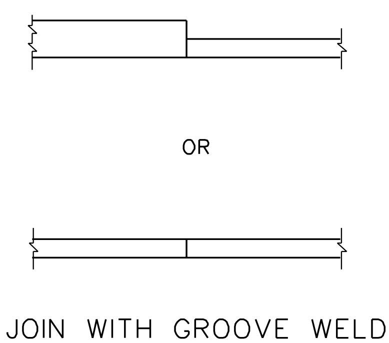
18.7.2.2 Welds for Bridges

The primary types of welds used in bridge fabrication are fillet welds and butt (or groove) welds. One of these two types accounts for approximately 80% of all bridge welding. A typical cross section where specification of a fillet weld is appropriate is shown in Figure 18.7A, and a typical cross section where specification of a butt or groove weld is appropriate is shown in Figure 18.7B.



FILLET WELD

Figure 18.7A



BUTT WELD

Figure 18.7B

18.7.2.3 Welding Symbols

Welding symbols are used as an instruction on the form, 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 for all involved. The **AISC Manual** and most steel design textbooks have examples of welding symbols which, although technically correct, are more complicated than the typical engineer needs. With minor modifications, the examples in Figure 18.7C will suffice for approximately 80% of bridge fabrication circumstances.

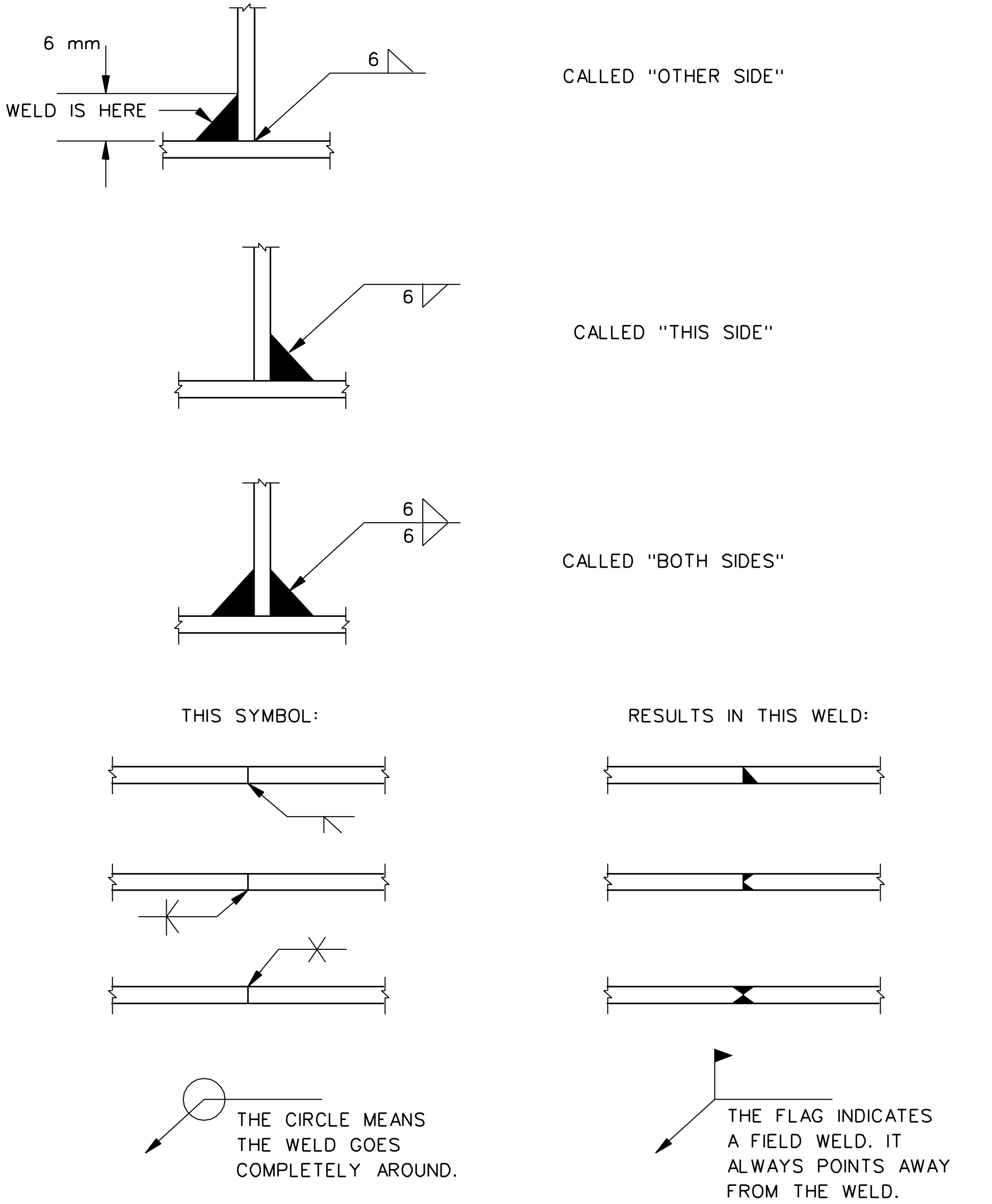
18.7.2.4 Electrode Nomenclature

The strength of the weld filler metal is known from the electrode designation. Figure 18.7D illustrates the standard nomenclature to identify electrodes. The figure represents more than a designer typically needs to know but, as an illustration, the standard MDT pile weld note says 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.

18.7.2.5 Design of Welds

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

To proceed with the calculations required by the specification, the concept is that the strength of a welded connection is dependent on the weld metal strength and the area of the weld that



"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.

WELDING SYMBOLS

Figure 18.7C

These digits indicates 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 (6.9 MPa) increments. For example, E7018 is 70,000 psi (483 MPa).

ELECTRODE NOMENCLATURE

Figure 18.7D

resists the load. 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. 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 only to show the type and size of weld required and leave the details to the fabricator.

When considering design options, note that the most significant driving 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. Welds sized to be made in a single pass are preferred. The weld should be designed economically, but its size should not be less than 6 mm and, in no case, less than the requirements of LRFD Article 6.13.3.4 for the thicker of the two parts joined. Weld terminations should be shown.

The following prohibitions should be observed:

1. <u>Field Welding</u>. Field welding is prohibited for all splices.

2. <u>Intersecting Welds</u>. These should be avoided, if practical.

3. <u>Intermittent Fillet Welds</u>. These are prohibited.

4. <u>Partial Penetration Groove Welds</u>. These are prohibited except as permitted for orthotropic steel decks under LRFD Article 9.8.3.7.2.

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.

18.7.2.6 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.

In spite of 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. The Department routinely uses the following types of non-destructive testing (NDT):

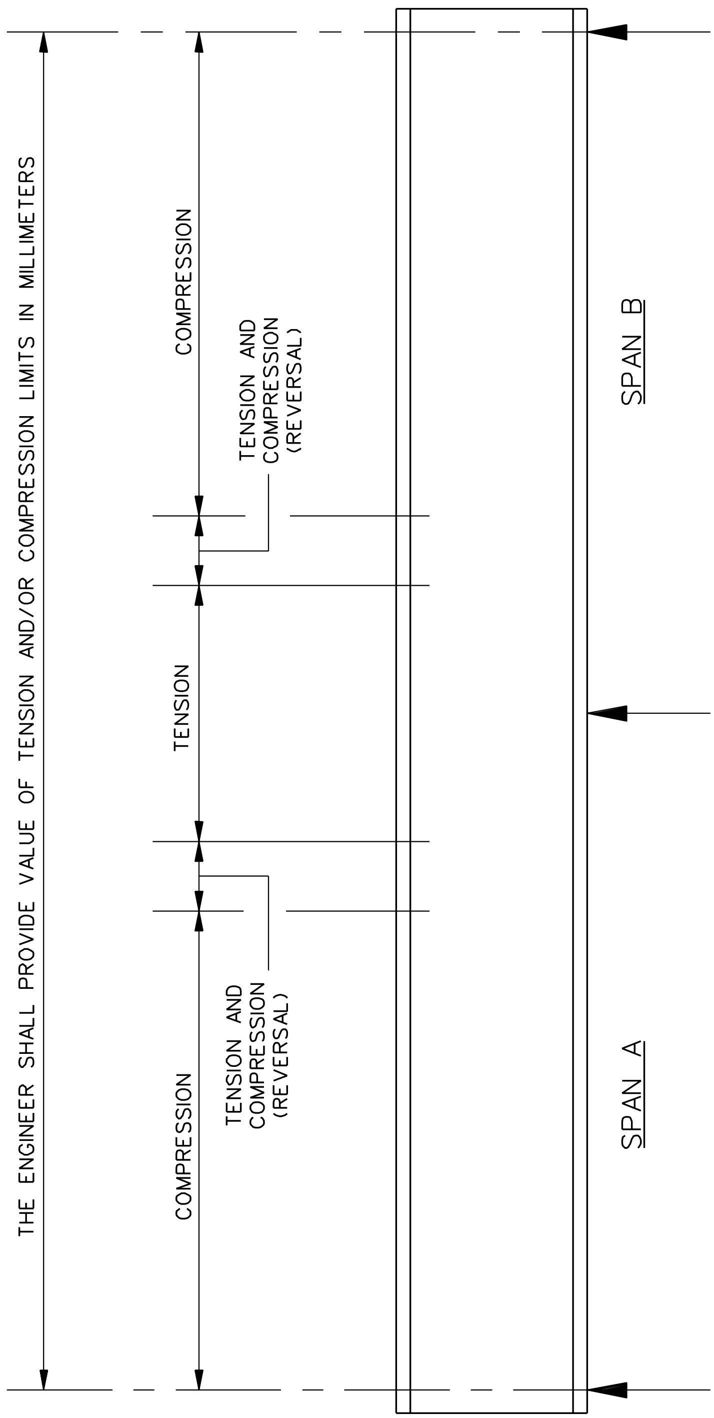
1. <u>Radiography (RT)</u>. Used to find cracks and inclusions after a weld is completed. The process involves placing film on one side of the weld and a source of gamma or x-rays on the other side of the weld. Shadows on the exposed film indicate cracks or inclusions in the welds or adjacent areas. RT is most effective on full penetration butt joints with ready access to both sides.

2. <u>Ultrasonic Testing (UT)</u>. 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. This method can reveal many defects that the other methods do not, but it relies very heavily on the interpretative skill of the operator.

3. <u>Magnetic Particle (MT)</u>. Done by covering the surface of the 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 in patterns easily interpreted by the inspector.

4. <u>Dye Penetrant (DP)</u>. 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.

To aid the inspector, the plans for continuous structures should include a sketch showing the location of compression, reversal and tension regions along the girder top flange. Show the length of each stress region and reference them to the point of support. Figure 18.7E illustrates the information required.



SCHEMATIC OF TOP FLANGE STRESS

Figure 18.7E

18.7.3 Splices

Reference: LRFD Article 6.13.6

Significant revisions have been made to the provisions for the design of bolted field splices (LRFD Article 6.13.6.1) in the 1999 Interims to the LRFD Specifications. The American Iron and Steel Institute (AISI) has developed software called AISI splice incorporating the revisions. The program is available from AISI for the cost of handling. These revisions were made to:

1. ensure a more consistent interpretation for the design of splices in flexural members at all limit states;

2. better handle the design of splices for composite flexural members, especially in areas of stress reversal;

3. provide a more consistent and reasonable design shear for splices in flexural members; and

4. better determine the effective flange area to be used for flexural members with holes.

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

1. Location. In general, field splices should be located at low-stress areas and near the points of dead-load contraflexure for continuous spans. Numerous butt welds and/or butt welds located in high stress regions are not desirable. The location of shop butt 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 normalized and quenched and tempered plates is 15 m. Other plates can be obtained in lengths greater than 25 m depending on thickness. The cost of adding a shop welded splice instead of extending a thicker plate should be considered when designing members. Discussion with a fabricator or the NSBA during the design is suggested.

   To assist the fabricator and contractor, the designer should provide an indication of where splices are acceptable and not acceptable. Figure 18.7F presents a typical detail that may be included in the contract plans. In addition, the plans should note that a plate as long as practical should be used, eliminating as many shop splices as possible.

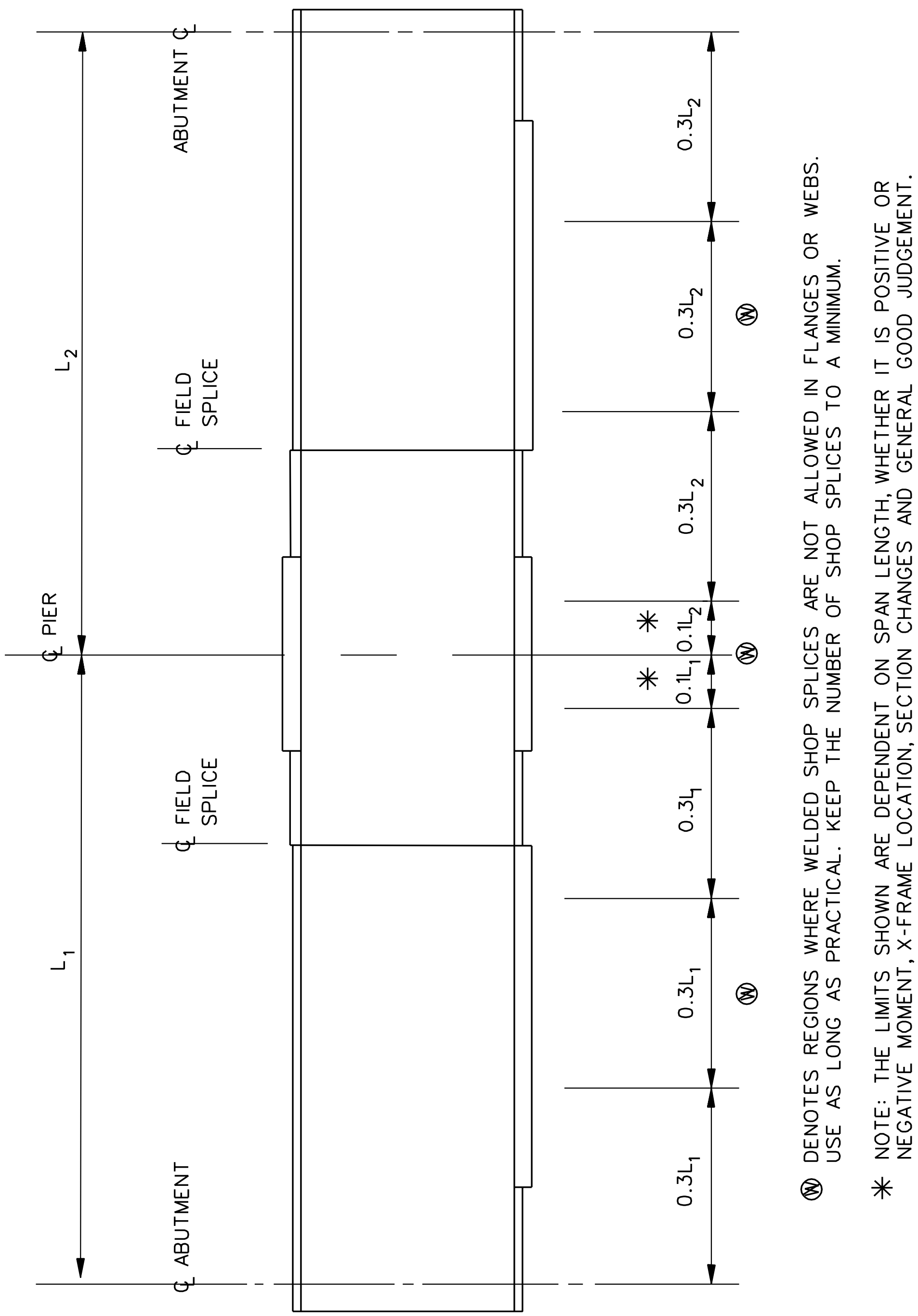
2. Swept Width. The swept width is equal to the sweep in a curved girder plus the flange width. On curved girders, the swept width between splices should generally be limited to 3 m to accommodate the shipment of the steel.

3. Bolts. Bolt loads shall be calculated by an elastic method of analysis. Provide no less than two lines of bolts on each side of the web splice.

4. Composite Girder. If a composite 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.

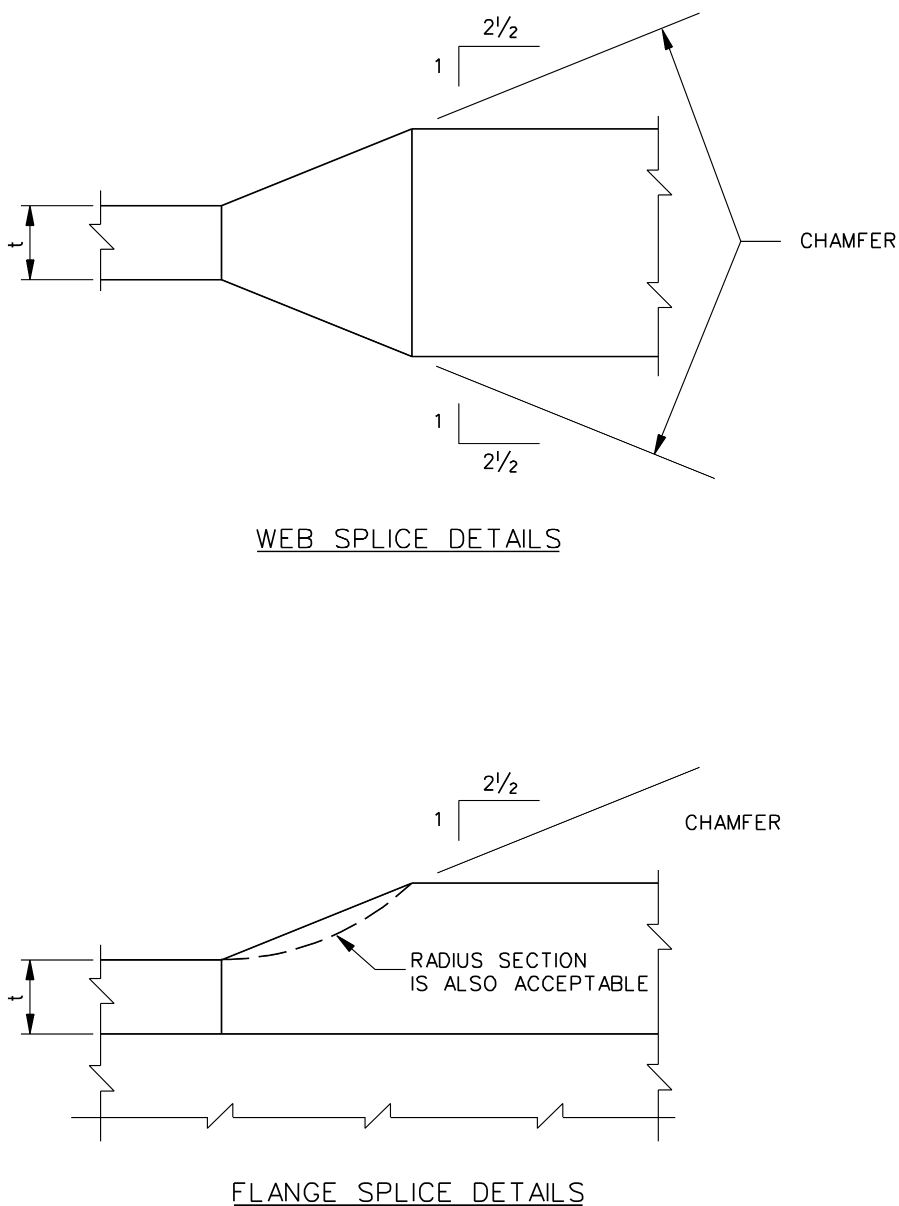
5. Design. Bolted splices must be slip-critical under Service II loads and must be designed as a bearing type connection under strength limit states.

6. Welded Shop Splice. Figure 18.7G illustrates welded splice details. See LRFD Article 6.13.6.2 for more information regarding splicing different thicknesses of material using butt welds.



LOCATION OF SPLICES

Figure 18.7F



TYPICAL SUBMERGED ARC WELD DETAILS

Figure 18.7G