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Chapter 6

6.0 Structural Steel

6.0.1 Introduction

This chapter primarily covers design and construction of steel plate and box girder bridge superstructures. Because of their limited application, other types of steel superstructures (truss, arch, cable stayed, suspension, etc.) are not addressed.

Plate girder bridges are commonly used for river crossings and curved interchange ramps. Typical span lengths range from 150 to 300 feet. Steel girders are also being used where limited vertical clearance requires shallow superstructure depth. They may be set over busy highway lanes with a minimum of disruption and falsework, similar to precast concrete elements. Longitudinal launching of steel framing and transverse rolling of completed steel structures has been done successfully.

English units are the current standard for detailing. Widening or rehabilitation plan units should be consistent with the original.

6.0.2 Special Requirements for Steel Bridge Rehabilitation or Modification

As part of steel bridge rehabilitation or modification, calculations shall be made to demonstrate the adequacy of existing members and connections, with special attention given to fracture critical components such as truss gusset plates. When structural modifications or other alterations result in significant changes in stress level, deficiencies shall also be corrected. A thorough survey of impacted components shall be made to determine section loss due to corrosion or prior modification.
6.1 Design Considerations

6.1.1 Codes, Specification, and Standards

Steel highway bridges shall be designed to the following codes and specifications:

- AASHTO LRFD Bridge Design Specifications (LRFD), latest edition
- AASHTO Guide Specifications for LRFD Seismic Bridge Design (SEISMIC)
- ANSI/AWS A2.4-98 Standard Symbols for Welding, Brazing, and Nondestructive Examination

The following codes and specifications shall govern steel bridge construction:

- AASHTO/AWS D1.5M/D1.5: Bridge Welding Code, latest edition

The following AASHTO/NSBA Steel Bridge Collaboration publications are available to aid in the design and fabrication of steel bridges. These publications can be downloaded from the AASHTO website or a copy can be obtained from the Steel Specialist:

- G1.2-2003, Design Drawing Presentation Guidelines
- G12.1-2016, Guidelines for Design for Constructability
- G1.3-2002, Shop Detail Drawing Presentation Guidelines
- S2.1-2016, Steel Bridge Fabrication Guide Specification
- G4.2-2006, Recommendations for the Qualification of Structural Bolting Inspectors
- G4.4-2006, Sample Owners Quality Assurance Manual
- G13.1-2014, Guidelines for Steel Girder Bridge Analysis
- G9.1-2004, Steel Bridge Bearing Design and Detailing Guidelines
- S10.1-2014, Steel Bridge Erection Guide Specification
- G1.4-2006, Guidelines for Design Details
- G1.1-2000, Shop Detail Drawing Review/Approval Guidelines
- G2.2-2016, Guidelines for Resolution of Steel Bridge Fabrication Errors

The following FHWA Steel Bridge Design Handbook, which includes 19 volumes of steel bridge design aids and 6 design examples, are also available as design aids and can be downloaded from the FHWA website at: www.fhwa.dot.gov/bridge/steel/pubs/if12052/. These documents are current with the AASTHO Bridge Design Specifications, 5th Edition.

Hard copies of these publications can be obtained from the Steel Specialist.

- Bridge Steels and Their Mechanical Properties—Volume 1
- Steel Bridge Fabrication—Volume 2
- Structural Steel Bridge Shop Drawings—Volume 3
- Structural Behavior of Steel—Volume 4
6.1.2 **WSDOT Steel Bridge Practice**

Unshored, composite construction is used for most plate girder and box girder bridges. Shear connectors are placed throughout positive and negative moment regions, for full composite behavior. A minimum of one percent longitudinal deck steel, in accordance with AASHTO LRFD Article 6.10.1.7, shall be placed in negative moment regions to ensure adequate deck performance. For service level stiffness analysis, such as calculating live load moment envelopes, the bridge deck shall be considered composite and uncracked for the entire bridge length, provided the above methods are used. For negative moment at strength limit states, the bridge deck shall be ignored while reinforcing steel is included for stress and section property calculations. Where span arrangement is not well balanced, these assumptions may not apply.

Plastic design may be utilized as permitted in the AASTHO LRFD Bridge Design Specifications.

Currently, economical design requires simplified fabrication with less emphasis on weight reduction. The number of plate thicknesses and splices should be minimized. Also, the use of fewer girder lines, spaced at a maximum of about 14 to 16 feet, saves on fabrication, shipping, painting, and future inspection. Widely spaced girders will have heavier flanges, hence, greater stability during construction. Normally, eliminating a girder line will not require thickening remaining webs or increasing girder depth. The increased shear requirement can be met with a modest addition of web stiffeners or slightly thicker webs at interior piers.
For moderate to long spans, partially stiffened web design is the most economical. This method is a compromise between slender webs requiring transverse stiffening throughout and thicker, unstiffened webs. Stiffeners used to connect cross frames shall be welded to top and bottom flanges. Jacking stiffeners shall be used adjacent to bearing stiffeners, on girder or diaphragm webs, in order to accommodate future bearing replacement. Coordinate jack placement in substructure and girder details.

Steel framing shall consist of main girders and cross frames. Bottom lateral systems shall only be used when temporary bracing is not practical. Where lateral systems are needed, they shall be detailed carefully for adequate fatigue life.

Standard corrosion protection for steel bridges is the *Standard Specifications* four-coat paint system west of the Cascades and where paint is required for appearance. Unpainted weathering steel shall only be used east of the Cascades. WSDOT does not allow the use of steel stay-in-place deck forms.

### 6.1.3 Preliminary Girder Proportioning

The superstructure depth is initially determined during preliminary plan development and is based upon the span/depth ratios provided in Chapter 2. The depth may be reduced to gain vertical clearance, but the designer should verify live load deflection requirements are met. See AASHTO LRFD Table 2.5.2.6.3-1. Live load deflections shall be limited in accordance with the optional criteria of AASHTO LRFD Articles 2.5.2.6.2 and 3.6.1.3.2.

The superstructure depth is typically shown as the distance from the top of the bridge deck to the bottom of the web. Web depths are generally detailed in multiples of 6 inches.

On straight bridges, interior and exterior girders shall be detailed as identical. Spacing should be such that the distribution of wheel loads on the exterior girder is close to that of the interior girder. The number of girder lines should be minimized, with a maximum spacing of 14–16 feet. Steel bridges shall be redundant, with three or more girders lines for I-girders and two or more boxes for box girders, except as otherwise approved by the Steel Specialist and Bridge Design Engineer.

### 6.1.4 Estimating Structural Steel Weights

For the preliminary quantities or preliminary girder design, an estimate of steel weights for built-up plate composite I-girders can be obtained from Figure 6.1.4-1. This figure is based upon previous designs with AASHTO HS-20 live loads with no distinction between service load designs and load factor designs. This chart also provides a good double check on final quantities.

The weights shown include webs, flanges, and all secondary members (web stiffeners, diaphragms, cross frame, lateral systems, and gusset plates) plus a small allowance for weld metal, bolts, and shear connectors.

Both straight and curved box girder quantities may be estimated with the chart, using a 10 to 20 percent increase.

The chart should only be used for a lower bound estimate of curved I-girder weight. Roadway width and curvature greatly influence girder weight, including cross frames. An additional resource for estimating structural steel weights is the *NSBA Steel Span Weight Curves* published in 2016, which can be obtained off of their website.
Composite Welded Steel Plate “I” Girder

Figure 6.1.4-1
6.1.5 Bridge Steels

Use AASHTO M 270/ASTM A 709 grades 50 or 50W for plate girders and box girders. AASHTO M 270 grades HPS 50W, HPS 70W and HPS 100W may only be used if allowed by the Bridge Design Engineer. HPS 70W can be economical if used selectively in a hybrid design. For moderate spans HPS 70W could be considered for the bottom flanges throughout and top flanges near interior piers.

For wide flange beams, use AASHTO M 270/ASTM A 709 Grade 50S or ASTM A 992. For ancillary members such as expansion joint headers, utility brackets, bearing components or small quantities of tees, channels, and angles, ASTM M 270/ASTM A 709 bridge steels are acceptable but are not required. In these cases, equivalent ASTM designated steels may be used.

The following table shows equivalent designations. Grades of steel are based on minimum yield point.

<table>
<thead>
<tr>
<th>ASTM</th>
<th>ASTM A 709/ AASHTO M 270</th>
</tr>
</thead>
<tbody>
<tr>
<td>A 36</td>
<td>Grade 36</td>
</tr>
<tr>
<td>A 572 gr 50</td>
<td>Grade 50</td>
</tr>
<tr>
<td>A 992 (W and rolled sections)</td>
<td>Grade 50S</td>
</tr>
<tr>
<td>A 588</td>
<td>Grade 50W</td>
</tr>
<tr>
<td>---</td>
<td>Grade HPS 50W**</td>
</tr>
<tr>
<td>---</td>
<td>Grade HPS 70W</td>
</tr>
<tr>
<td>---</td>
<td>Grade HPS 100W*</td>
</tr>
</tbody>
</table>

*Minimum yield strength is 90 ksi for plate thickness greater than 2½".  
**Avoid unless project has a large order with long lead times available.

A 992 or 50S steel is most commonly available in W shapes but is also available in other rolled sections including beams (S and M shapes), H-piles, tees, channels, and angles. All the materials in the table are prequalified under the Bridge Welding Code.

All main load-carrying members or components subject to tensile stress shall be identified in the plans and shall meet the minimum Charpy V-notch (CVN) fracture toughness values as specified in AASHTO LRFD Table 6.6.2-2, temperature zone 2. Fracture critical members or components shall also be designated in the plans.

Availability of weathering steel can be a problem for some sections. For example, steel suppliers do not stock angles or channels in weathering steel. Weathering steel wide flange and tee sections are difficult to locate or require a mill order. A mill order is roughly 10,000 pounds. ASTM A 709 and AASHTO M 270 bridge steels are not stocked by local service centers. The use of bridge steel should be restricted to large quantities such as found in typical plate or box girder projects. The older ASTM specification steels, such as A 36 or A 572, should be specified when a fabricator would be expected to purchase from local service centers.

Structural tubes and pipes are covered by other specifications. See the latest edition of the AISC Manual of Steel Construction for selection and availability. These materials are not considered prequalified under the Bridge Welding Code. They are covered under the Structural Welding Code AWS D1.1. Structural tubing ASTM A 500 shall
not be used for dynamic loading applications. ASTM A 1085 is a newer cold formed and welded HSS section specification that is a Gr 50 steel. Supplements for heat treating and CVNs are included and may also be specified. CVN tests are typically performed in the flats of the HSS square or rectangular tube sections. CVN values in the bend radius of the tubes are lower than values obtained in the flats. Heat treating of the sections can improve the values, but no data is currently available. ASTM A 1085 should also not be specified for dynamic loading applications. The designer should check with suppliers to ensure the size and quantities are readily available. In some cases minimum tonnage is required in order to obtain HSS sections in ASTM A 1085. Consult with the Steel Specialist for more information.

6.1.6 Plate Sizes

Readily available lengths and thicknesses of steel plates should be used to minimize costs. Tables of standard plate sizes have been published by various steel mills and should be used for guidance. These tables are available through the Steel Specialist or online.

In general, an individual plate should not exceed 12′-6″ feet in width, including camber requirements, or a length of about 60 feet. If either or both of these dimensions are exceeded, a butt splice is required and should be shown or specified on the plans. Some plates may be available in lengths over 90 feet, so web splice locations should be considered optional. Quenched and tempered plates are limited to 50 feet, based on oven size.

Plate thicknesses of less than 5/16 inches shall not be used for bridge applications. Preferred plate thicknesses, English units, are as follows:

- 5/16″ to 7/8″ in 1/16″ increments
- 7/8″ to 1 1/4″ in 1/8″ increments
- 1 1/4″ to 4″ in 1/4″ increments

6.1.7 Girder Segment Sizes

Locate bolted field splices so that individual girder segments can be handled, shipped, and erected without imposing unreasonable requirements on the contractor. Crane limitations need to be considered in congested areas near traffic or buildings. Transportation route options between the girder fabricator and the bridge site can affect the size and weight of girder sections allowed. Underpasses with restricted vertical clearance in sag vertical curves can be obstructions to long, tall segments shipped upright. The Region Project Office should help determine the possible routes, and the restrictions they impose, during preliminary planning or early in the design phase.

Segment lengths should be limited to 150 feet, depending upon cross section. Long, slender segments can be difficult to handle and ship due to their flexibility. Horizontal curvature of girder segments may increase handling and shipping concerns. Out-to-out width of curved segments, especially box girders, should not exceed 14 feet without additional travel permits and requirements. Weight is seldom a controlling factor for I-girders. However, 40 tons is a practical limit for some fabricators. Limit weight to a maximum of 100 tons if delivery by truck is anticipated.
Consider the structure’s span length and the above factors when determining girder segment lengths. In general, field splices should be located at dead load inflection points. When spans are short enough, some field splices can be designated optional if resulting segment lengths and weights meet the shipping criteria.

6.1.8 Computer Programs

The designer should consult the Steel Specialist to determine the computer program best suited for a particular bridge type.

Office practice and good engineering principles require that the results of any computer program or analysis be independently verified for accuracy. Also, programs with built-in code checks must be checked for default settings. Default settings may reflect old code or office practice may supersede the code that the program was written for.

6.1.9 Fasteners

All bolted connections shall be friction type (slip-critical). Assume Class B faying surfaces where inorganic zinc primer is used. If steel will be given a full paint system in the shop, the primed faying surfaces need to be masked to maintain the Class B surface.

Properties of High-Strength Bolts

<table>
<thead>
<tr>
<th>Material</th>
<th>Bolt Diameter</th>
<th>Tensile Strength ksi</th>
<th>Yield Strength ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM F 3125 GR A325 &amp; GR F1852</td>
<td>½–1½ inch</td>
<td>120</td>
<td>92</td>
</tr>
<tr>
<td></td>
<td>Over 1½</td>
<td>Not Available</td>
<td></td>
</tr>
<tr>
<td>ASTM A 449</td>
<td>¼–1 inch</td>
<td>120</td>
<td>92</td>
</tr>
<tr>
<td></td>
<td>1½–1½ inch</td>
<td>105</td>
<td>81</td>
</tr>
<tr>
<td></td>
<td>1¾ -3 inch</td>
<td>90</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>Over 3</td>
<td>Not Available</td>
<td></td>
</tr>
<tr>
<td>ASTM F 1554</td>
<td>Grade 105</td>
<td>¼–3 inch</td>
<td>125-150</td>
</tr>
<tr>
<td></td>
<td>Grade 55</td>
<td>¼–4 inch</td>
<td>75-95</td>
</tr>
<tr>
<td></td>
<td>Grade 36</td>
<td>¼–4 inch</td>
<td>58-80</td>
</tr>
<tr>
<td>ASTM F 3125 GR A490 &amp; GR F2280</td>
<td>½–1½ inch</td>
<td>150-173 (max)</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>Over 1½</td>
<td>Not Available</td>
<td></td>
</tr>
<tr>
<td>ASTM A 354 GR BD</td>
<td>¼–2½ inch</td>
<td>150</td>
<td>130</td>
</tr>
<tr>
<td></td>
<td>Over 2½–4 inch</td>
<td>140</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td>Over 4</td>
<td>Not Available</td>
<td></td>
</tr>
</tbody>
</table>
General Guidelines for Steel Bolts

A. **ASTM F 3125 GR A325 & GR F1852**

   High strength steel, headed bolts for use in structural joints. These bolts may be hot-dip galvanized but shall not be used in structures that are painted. Do not specify for anchor bolts.

B. **ASTM A449**

   High strength steel bolts and studs for general applications including anchor bolts. Recommended for use where strengths equivalent to **ASTM F 3125 GR A325** bolts are desired but custom geometry or lengths are required. **Strengths for ASTM A 449** bolts are equivalent to GR A325 up to 1” diameter. If using bolts of larger diameter, a reduction in strength as indicated in the previous table shall be accounted for. These bolts may be hot-dip galvanized. Do not use these as anchor bolts for seismic applications due to low CVN impact toughness.

C. **F1554 – Grade 105**

   Higher strength anchor bolts to be used for larger sizes (1½” to 4”). When used in seismic applications, such as bridge bearings that resist lateral loads, specify supplemental CVN requirement S4 with a test temperature of -20°F. Lower grades may also be suitable for sign structure foundations. This specification should also be considered for seismic restrainer rods, and may be galvanized. The equivalent AASHTO M 314 shall not be specified as it doesn’t include the CVN supplemental requirements.

D. **ASTM F 3125 GR A490 & GR F2280**

   High strength alloy steel, headed bolts for use in structural joints. These bolts should not be galvanized, because of the high susceptibility to hydrogen embrittlement. In lieu of galvanizing, the application of an approved zinc rich paint may be specified. Do not specify for anchor bolts.

E. **A354–Grade BD**

   High strength alloy steel bolts and studs. These are suitable for anchor bolts where strengths equal to **ASTM F 3125 GR A490** bolts are desired. These bolts should not be galvanized. If used in seismic applications, specify minimum CVN toughness of 25 ft-lb at 40°F.
6.2 Girder Bridges

6.2.1 General

Once the material of choice, structural steel has been eclipsed by reinforced and prestressed concrete. Fabrication, material and life cycle costs have contributed to steel’s relative cost disadvantage. Costs may be minimized by simplifying fabrication details, optimizing the number of girder lines, allowing for repetitive fabrication of components such as cross frames and stiffeners, and ensuring ease of shipping and erecting.

The specifications allow a combination of plastic design in positive moment regions and elastic design in negative moment regions. Plate girders, of the depths typically built in this state, have traditionally been designed to elastic limits or lower. High performance and weathering steel can be used to save weight and life cycle painting costs, thereby minimizing the cost gap between steel and concrete bridges.

I-girders may require accommodation for bridge security. Security fences may be installed within the confines of the superstructure to deter inappropriate access. Coordinate with the State Bridge and Structures Architect during final design where required.

6.2.2 I-Girders

Welded plate I-girders constitute the majority of steel girders designed by WSDOT. The I-girder represents an efficient use of material for maximizing stiffness. Its shortcoming is inefficiency in resisting shear. Office practice is to maintain constant web thickness and depth for short to medium span girders. Weight savings is achieved by minimizing the number of webs used for a given bridge. This also helps minimize fabrication, handling, and painting costs. Current office practice is to use a minimum of three girders to provide redundant load path structures. Two girder superstructures are considered non-redundant and hence, fracture critical.

Buckling behavior of relatively slender elements complicates steel plate girder design. Most strength calculations involve checks on buckling in some form. Local buckling can be a problem in flanges, webs, and stiffeners if compression is present. Also, overall stability shall be ensured throughout all stages of construction, with or without a bridge deck. The art of designing steel girders is to minimize material and fabrication expense while ensuring adequate strength, stiffness, and stability.

I-girders are an excellent shape for welding. All welds for the main components are easily accessible and visible for welding and inspection. The plates are oriented in the rolling direction to make good use of strength, ductility, and toughness of the structural steel. The web is attached to the top and bottom flanges with continuous fillet welds. Usually, they are made with automatic submerged arc welders. These welds are loaded parallel to the longitudinal axis and resist horizontal shear between the flanges and web. Minimum size welds based on plate thickness will satisfy strength and fatigue requirements in most cases. The flanges and webs are fabricated to full segment length with full penetration groove welds. These welds are inspected by ultrasound (UT) 100 percent. Tension welds, as designated in the plans, are also radiographed (RT) 100 percent. Office practice is to have flanges and webs fabricated full length before they are welded into the “I” shape. Weld splicing built-up sections results in poor fatigue strength and zones that are difficult to inspect. Quality welding and inspection requires good access for both.
6.2.3 Tub or Box Girders

Typical steel box girders for WSDOT are trapezoidal tub sections. Using single top flange plates to create true box sections is very uncommon when reinforced concrete decks are used. Tub girders will be referred to herein as box girders, as in AASHTO LRFD Article 6.11.

The top lateral system placed inside the girder is treated as an equivalent plate, closing the open section, to increase torsional stiffness before bridge deck curing. Although not required by the code, it helps ensure stability that may be overlooked during construction. Partial or temporary bracing may be used provided it is properly designed and installed. Dramatic construction failures have occurred due to insufficient bracing of box girders. Stability of the shape shall be ensured for all stages of construction in accordance with AASHTO LRFD Article 6.11.3. The cured bridge deck serves to close the section for torsional stiffness. Internal cross frames or diaphragms are used to maintain the shape and minimize distortion loading on individual plates and welds making up the box. Box segments will have considerable torsional stiffness when top lateral bracing is provided. This may make fit-up in the field difficult.

The ability to make box girders with high torsional stiffness makes them a popular choice for short radius curved structures. Curved box girders, because of inherent torsional stiffness, behave differently than curved I-girders. Curved box girder behavior can be approximated by the M/R method, rather than the V-load method, although true behavior can be easily modeled with modern finite element software. See curved girder references listed at the end of this chapter for complete description.

Straight box girders, when proportioned in accordance with AASHTO LRFD Article 6.11.2 may be designed without consideration of distortional stresses. The range of applicability for live load distribution is based on:

\[ 0.5 \leq N_L / N_b \leq 1.5 \]  

(6.2.3-1)

which limits the number of lanes loading each box. Wide box girder spacing, outside this range, will require additional live load analysis. Consideration must be given to differential deflection between boxes when designing the bridge deck. Generally, use of cross frames between boxes is limited to long spans with curvature.

Box girders shall have a single bearing per box for bridges with multiple box girders, not bearings under each web. If bearings are located under each web, distribution of loads is uncertain. For single box girder bridges, two bearings per box shall be used. Generally, plate diaphragms with access holes are used in place of pier cross frames.

With the exception of effects from inclined webs, top flanges and webs are designed as if they were part of individual I-girders.

The combined bottom flange is unique to box girders. In order to maximize web spacing while minimizing bottom flange width, place webs out of plumb on a slope of 1 in 4. Wide plates present two difficulties: excessive material between shop splices and buckling behavior in compression zones (interior piers). To keep weight and plate thickness within reason, it is often necessary to stiffen the bottom flange in compression with longitudinal stiffeners. Office practice is to use tee sections for longitudinal stiffeners and channel bracing at cross frame locations (transverse stiffeners). If possible, bottom flange stiffeners are terminated at field splices.
Otherwise, carefully ground weld terminations are needed in tension regions with high stress range. Due to the transverse flexibility of thin wide plates, stiffener plates are welded across the bottom flange at cross frame locations, combined with web vertical stiffeners. For the design of the bottom flange in compression, see AASHTO LRFD Articles 6.11.8.2 and 6.11.11.2.

6.2.4 Fracture Critical Superstructures

Non-redundant, fracture critical single tub superstructures, and twin I-girder systems, may sometimes be justified. In which case, approval for this bridge type must be obtained from the Bridge Design Engineer. Conditions that favor this option are narrow one lane ramps, especially with tightly curved alignments, at locations within existing mainline interchanges. Flyover ramps often fall into these constraints. The box section allows in-depth inspection access without significant disruption to mainline traffic. UBIT access over urban interstate lanes is becoming increasingly difficult to obtain.

Where curvature is significant, the box section is a stiffer, more efficient load carrying system than a twin I-girder system. If a twin I-girder system is to be used, approval must also be secured. Some form of permanent false decking or other inspection access needs to be included over mainline lanes that will be difficult to close for UBIT access. This access needs to be appropriate for fracture critical inspections. If curvature is not severe, the twin I-girder system may prove to be more economical than a single box.

The maximum roadway width for either a single box or twin I-girder superstructure is about 27 feet. Where roadway width exceeds this, additional girders shall be used. Mainline structures, usually exceeding 38 feet in width, will require a minimum of three webs, with four webs being the preferred minimum.

Increased vertical clearance from mainline traffic should be obtained for either of these bridge types. The desired minimum is 20 feet. Box sections tend to offer greater stiffness than equal depth I-girders, especially on curved alignment. The web depth may be reduced below AASHTO LRFD Table 2.5.2.6.3-1 minimums provided live load deflection criteria are met. However, avoid web depth less than 5′-0″ so that inspection access is within reason. The desirable minimum web depth for boxes is 6′-6″. Box sections with web depth of 6′-6″ should be capable of interior spans up to 250 feet. Main spans of 150 feet should be considered the low end of this girder type’s economical range. Because of the proximity of flyover ramps to high numbers of observers, attempt to streamline their superstructure depths where economical and deflection criteria can be achieved.

Consider use of high performance steels, AASHTO M270 grades HPS50W or HPS70W for these girder types. Grades of steel with equal CVN toughness may be considered, however the improved through-thickness properties of the HPS grades should also be considered. If practical, maintain a maximum flange thickness of 2″ when using HPS for better properties and plate availability. The improved toughness of HPS will lower the chance of sudden crack propagation if a crack does become visible to casual observation. HPS50W is not commonly used and the Steel Specialist should be consulted before specifying this grade of steel in a structure.
The limit state load modifier relating to redundancy, \( \eta_r = 1.05 \), as specified in AASHTO LRFD Article 1.3.4 shall be used in the design of non-redundant steel structures. However, for load rating non-redundant structures, a system factor of 0.85 is currently required in the AASHTO Manual for Bridge Evaluation (MBE) on the capacity of the girders, which is not consistent with the redundancy load modifier used in design. The designer shall design some reserve capacity in the girders so the load rating value for HL93 Inventory is greater than or equal to 1.0.

The AASHTO LRFD approximate live load distribution factors are not applicable to these girder types. The level rule or the preferred refined analysis shall be used. Where highly curved, only a refined analysis shall be used.
6.3 Design of I-Girders

6.3.1 Limit States for AASHTO LRFD

Structural components shall be proportioned to satisfy the requirements of strength, extreme event, service, and fatigue limit states as outlined in AASHTO LRFD Articles 1.3.2 and 6.5.

Service limit states are included in Service I and Service II load combinations. Service I load combination is used to check the live load deflection limitations of AASHTO LRFD Article 2.5.2.6. Service II places limits on permanent deflection, no yielding, slenderness of the web in compression, and slip of bolted connections.

The fatigue live load specified in AASHTO LRFD Article 3.6.1.4 shall be used for checking girder details in accordance with Article 6.6. A single fatigue truck, without lane loading or variable axle spacing, is placed for maximum and minimum effect to a detail under investigation. The impact is 15 percent, regardless of span length. The load factor is 1.5. It is generally possible to meet the constant amplitude fatigue limit (CAFL) requirement for details with good fatigue performance. Limiting the calculated fatigue range to the CAFL ensures infinite fatigue life. Webs shall be checked for fatigue loading in accordance with AASHTO LRFD Article 6.10.5.3, using the calculated fatigue stress range for flexure or shear. Shear connector spacing shall be according to AASHTO LRFD Article 6.10.10. Generally, the fatigue resistance (Zr) for ⅞" diameter shear connectors can be taken as 4.2 kips for an infinite number of cycles (CAFL = 4.2 kips).

Flanges and webs shall meet strength limit state requirements for both construction and final phases. Constructability requirements for flanges and webs are covered in AASHTO LRFD Article 6.10.3. Flexure resistance is specified in AASHTO LRFD Articles 6.10.7 and 6.10.8; shear resistance is specified in AASHTO LRFD Article 6.10.9

Pier cross frames shall be designed for seismic loading, extreme event load combination. Bolts are treated as bearing type connections with AASHTO LRFD Article 6.5.4.2 resistance factors. The resistance factor for all other members is 1.0 at extreme limit state.

6.3.2 Composite Section

Live load plus impact shall be applied to the transformed composite section using Es/Ec, commonly denoted n. Long-term loading (dead load of barriers, signs, luminaries, overlays, etc.) is applied to the transformed composite section using 3n. Positive moments are applied to these composite sections accordingly; both for service and strength limit states. The bridge deck may be considered effective in negative moment regions provided tensile stresses in the deck are below the modulus of rupture. This is generally possible for Service I load combination and fatigue analysis. For strength limit state loadings, the composite section includes longitudinal reinforcing while the bridge deck is ignored.
6.3.3 Flanges

Flange thickness is limited to 4” maximum in typical bridge plate, but the desirable maximum is 3”. Structural Steel Notes on contract plans shall require all plates for flange material to be purchased such that the ratio of reduction of thickness from a slab to plate shall be at least 3.0:1. This requirement helps ensure the plate material has limited inclusions and micro-porosity that can create problems during cutting and welding.

As an alternative, plates for flange material greater than 2” thick not meeting the 3.0:1 ratio of reduction may be supplied based on acceptable ultrasonic testing (UT) inspection in accordance with ASTM A 578. UT scanning and acceptance shall be as follows:

- The entire plate shall be scanned in accordance with ASTM A 578 and shall meet Acceptance Standard C, and
- Plate material within 12-inches of flange complete joint penetration splice welds shall be scanned in accordance with ASTM A 578 Supplementary S1 and shall meet Acceptance Standard C

The number of plate thicknesses used for a given project should be kept to a minimum. Generally, the bottom flange should be wider than the top flange. Flange width changes should be made at bolted field splices. Thickness transitions are best done at welded splices. AASHTO LRFD Article 6.13.6.1.5 requires fill plates at bolted splices to be developed, if thicker than ¼”. Since this requires a significant increase in the number of bolts for thick fill plates, keeping the thickness transition ¼” or less by widening pier segment flanges can be a better solution. Between field splices, flange width should be kept constant.

6.3.4 Webs

Maintain constant web thickness throughout the structure. If different web thickness is needed, the transition shall be at a welded splice. Horizontal web splices are not needed unless web height exceeds 12'-6". Vertical web splices for girders should be shown on the plans in an elevation view with additional splices made optional to the fabricator. All welded web splices on exterior faces of exterior girders and in tension zones elsewhere shall be ground smooth. Web splices of interior girders need not be ground in compression zones.

6.3.5 Transverse Stiffeners

Transverse stiffeners shall be used in pairs at cross frame locations on interior girders and on the inside of webs of exterior girders. They shall be welded to the top flange, bottom flange and web at these locations. This detail is considered fatigue category C’ for longitudinal flange stress. Stiffeners used between cross frames shall be located on one side of the web, welded to the compression flange, and cut short of the tension flange. Stiffeners located between cross frames in regions of stress reversal shall be welded to one side of the web and cut short of both flanges. Alternatively, they may be welded to both flanges if fatigue Category C’ is checked. Transverse stiffeners may be dropped when not needed for strength. If cross frame spacing is less than 3 times the web depth, additional stiffeners may only be necessary near piers.
Stiffened webs require end panels to anchor the first tension field. The jacking stiffener to bearing stiffeners space shall not be used as the anchor panel. The first transverse stiffener is to be placed at no greater spacing than 1.5 times the web depth from the bearing or jacking stiffener.

Transverse stiffeners shall be designed and detailed to meet AASHTO LRFD Article 6.10.11.1. Where they are used to connect cross frames, they should be a minimum width of 8” to accommodate two bolt rows.

6.3.6 Longitudinal Stiffeners

On long spans where web depths exceed 10 feet, comparative cost evaluations shall be made to determine whether the use of longitudinal stiffeners will be economical. The use of longitudinal stiffeners may be economical on webs 10 feet deep or greater. Weld terminations for longitudinal stiffeners are fatigue prone details. Longitudinal stiffener plates shall be continuous, splices being made with full penetration welds before being attached to webs. Transverse stiffeners should be pieced to allow passage of longitudinal stiffeners.

Design of longitudinal stiffeners is covered by AASHTO LRFD Article 6.10.11.3.

6.3.7 Bearing Stiffeners

Stiffeners are required at all bearings to enable the reaction to be transmitted from the web to the bearing. These stiffeners are designated as columns, therefore, must be vertical under total dead load. The connection of the bearing stiffener to flanges consists of partial penetration groove welds, of sufficient size to transmit design loads.

Pier cross frames may transfer large seismic lateral loads through top and bottom connections. Weld size must be designed to ensure adequate load path from deck and cross frames into bearings.

Design of bearing stiffeners is covered by AASHTO LRFD Article 6.10.11.2.

6.3.8 Cross Frames

The primary function of intermediate cross frames is to provide stability to individual girders or flanges. Cross frames or diaphragms are required at each support to brace girders; they should be as near to full-depth as practical. Cross frames share live load distribution between girders with the concrete deck. The approximate AASHTO LRFD live load distribution factors were based on the absence of intermediate cross frames. Where cross frames are present, the exterior girder distribution factors are also determined according to AASHTO LRFD Article 4.6.2.2.2d (conventional approximation for loads on piles). On curved bridges, the cross frames also resist twisting of the superstructure. Pier cross frames are subjected to lateral loads from wind, earthquake, and curvature. These forces are transmitted from the roadway slab to the bearings by way of the pier cross frames. Intermediate cross frames also resist wind load to the lower half of the girders. The primary load path for wind is the concrete deck and pier diaphragms. Wind load on the bottom flange is shed incrementally to the deck through intermediate cross frames. The essential function, however, is to brace the compression flanges for all stages of construction and service life. As such, continuous span girders require bottom flange bracing near interior supports.
Office practice requires intermediate cross frames, at spacing consistent with design assumptions. The 25 foot maximum spacing of older specifications is not maintained in the AASHTO LRFD code. A rectangular grid of girders and cross frames is not significantly stiff laterally before the deck is cured. Both wind and deck placement can cause noticeable deflections. In the case of deck placement, permanent sideways and rotation of the steel framing may result. Some form of temporary or permanent lateral bracing is therefore required.

Cross frames and connections should be detailed for repetitive fabrication, adjustment in the field, and openness for inspection and painting. Cross frames consisting of back-to-back angles separated by gusset plates are not permitted. These are difficult to repaint. Cross frames are generally patterned as K-frames or as X-frames. Typically the configuration selected is based on the most efficient geometry. The diagonals should closely approach a slope of 1:1 or 45°. Avoid conflicts with utilities passing between the girders. Detailing of cross frames should follow guidelines of economical steel bridge details promoted by the National Steel Bridge Alliance. Office practice is to bolt rather than weld individual pieces. Oversize holes are not allowed in cross frame connections if girders are curved.

Intermediate cross frames for straight girders with little or no skew should be designed as secondary members. Choose members that meet minimum slenderness requirements and design connections only for anticipated loads, not for 75 percent strength of member.

In general, cross frames should be installed parallel to piers for skew angles of 0° to 20°. For greater skew angles, other arrangements may be used. Consult with the Design Unit Manager or the Steel Specialist for special requirements.

Intermediate cross frames for curved I-girders require special consideration. Cross frames for curved girder bridges are main load carrying members and tension components should be so designated in the plans. For highly curved systems, it is more efficient to keep members and connections concentric, as live loads can be significant. Welded connections should be carefully evaluated for fatigue.

Web stiffeners at cross frames shall be welded to top and bottom flanges. This practice minimizes out-of-plane bending of the girder web.

Bridge widening requires special attention to girder stability during bridge deck placement. Lateral movement and rotation has been common with widening projects around the country. Narrow framing, such as a two girder widening, requires bracing to an existing structure. A common method for bracing is to install cross frames (in the bay between existing and new girders) with only one bolt per connection to allow for girder differential deflection but no rotation. Remaining bolts can be installed through field-drilled holes after the slab has cured.
6.3.9 **Bottom Laterals**

Bottom lateral systems are expensive to install permanently. If possible, they should be avoided in favor of alternative bracing methods. They are seldom required in the completed structure, but may contribute to nuisance fatigue cracking or fracture in the main girders.

The primary function of a bottom lateral system is to stabilize the girders against lateral loads and translation before the bridge deck hardens. The layout pattern is based on number of girder lines, girder spacing, and cross frame spacing. Cost considerations should include geometry, repetition, number, and size of connections. If used, limit bottom laterals to one or two bays.

For both straight and curved structures, bottom laterals carry dead and live loads, in proportion to distance from the neutral axis. They should be modeled in the structure to determine the actual forces the member's experience. Since they carry slab dead load, they should be accounted for when calculating camber.

Where lateral gusset plates are fillet welded to girder webs, the fatigue stress range in the girder is limited to Category E without transition radius, or Category D with carefully made transition radius. The gusset plates should be bolted to the girder web in regions of high tension stress range.

For widening projects, bottom laterals are not needed since the new structure can be braced against the existing structure during construction. Framing which is adequately braced should not require bottom laterals.

6.3.10 **Bolted Field Splice for Girders**

Field splices shall be bolted. Splices are usually located at the dead load inflection point to minimize the design bending moment. See AASHTO LRFD Articles 6.13.2 and 6.13.6.1 for bolted splice design requirements. Designing web splices is outlined in AASHTO LRFD Article 6.13.6.1.4b. Bolted web splices should not involve thin fill material. Thickness transitions for webs, if needed, should be done with welded shop splices.

Flange splice design is outlined in AASHTO LRFD Article 6.13.6.1.4c. For splice plates at least ⅜″ thick and ⅞″ diameter bolts, threads may be excluded from all shear planes for a 25 percent increase in strength, per AASHTO LRFD Article 6.13.2.7. Bolts designed with threads excluded from shear planes shall be designated as such in the plans. Generally, bolts in girder field splices may be designed for double shear.

A requirement has been added for developing fillers used in bolted splices, AASHTO LRFD Article 6.13.6.1.5. When fill plates are greater than ¼″, the splice or filler needs to be extended for additional bolts. As filler thickness increases, the shear resistance of bolts decreases. A way of minimizing filler thickness is to transition flange width for pier segments. Using equal plate thickness by this method has the added benefit of reducing the number of plate sizes in a project.

**Note:** A major revision to the design of bolted splices was approved by AASTHO in 2016, but has yet to be published. The revised methodology and or the current provisions in the AASTHO 7th Edition may be used for design. The new provisions were developed to simplify the design process. Consult the Steel Specialist for additional information and guidance.
Splice bolts shall be checked for Strength load combinations and slip at Service II load combination. When faying surfaces are blasted and primed with inorganic zinc paint, a Class B surface condition is assumed.

Fabrication of girder splices is covered by Standard Specifications Sections 6-03.3(27) and 6-03.3(28). Method of field assembly is covered by section 6-03.3(32) and bolting inspection and installation by Section 6-03.3(33). Since bolted joints have some play due to differences in bolt diameter and hole size, field splices are drilled while segments are set in proper alignment in the shop. The joint is pinned (pin diameter equals hole size to prohibit movement) for shop assembly and also during initial field fit-up. Normally, this ensures repeatability of joint alignment from shop to field.

### 6.3.11 Camber

Camber shall include effects of profile grade, superelevation, anticipated dead load deflections, and bridge deck shrinkage (if measurable). Permanent girder deflections shall be shown in the plans in the form of camber diagrams and tables. Dead load deflections are due to steel self-weight, bridge deck dead load, and superimposed dead loads such as overlay, sidewalks, and barriers. Since fabricated camber and girder erection have inherent variability, bridge deck form height is adjusted after steel has been set. Although a constant distance from top of web to top of deck is assumed, this will vary along the girders. Bridge deck forms without adjustment for height are not allowed. Girders shall be profiled once fully erected, and before bridge deck forms are installed. See Standard Specifications Section 6-03.3(39).

Girder camber is established at three stages of construction. First, girder webs are cut from plates so that the completed girder segment will assume the shape of reverse dead load deflections superimposed on profile grade. Only minor heat corrections may be made in the shop to meet the camber tolerance of the Bridge Welding Code AWS D1.5 Chapter 3.5. Camber for plate girders is not induced by mechanical force. The fabricated girder segment will incorporate the as-cut web shape and minor amounts of welding distortion. Next, the girder segments are brought together for shop assembly. Field splices are drilled as the segments are placed in position to fit profile grade plus total dead load deflection (no load condition). Finally, the segments are erected, sometimes with supports at field splices. There may be slight angle changes at field splices, resulting in altered girder profiles. Errors at mid-span can be between one to two inches at this stage.

The following is a general outline for calculating camber and is based on girders having shear studs the full length of the bridge.

Two camber curves are required, one for total dead load plus bridge deck formwork and one for steel framing self-weight. The difference between these curves is used to set bridge deck forms.

Girder dead load deflection is determined by using various computer programs. Many steel girder design programs incorporate camber calculation. Girder self-weight shall include the basic section plus stiffeners, cross frames, welds, shear studs, etc. These items may be accounted for by adding an appropriate percentage of basic section weight (15 percent is a good rule of thumb). Total dead load camber shall consist of deflection due to:
A. Steel weight, applied to steel section. Include 10 psf bridge deck formwork allowance in the total dead load camber, but not in the steel weight camber. The effect of removing formwork is small in relation to first placement, due to composite action between girders and bridge deck. It isn’t necessary to account for the removal.

B. Bridge deck weight, applied to steel section. This should be the majority of dead load deflection.

C. Traffic barriers, sidewalks, and overlays, applied to long-term composite section using 3n. Do not include weight of future overlays in the camber calculations.

D. Bridge deck shrinkage (if $\geq \frac{3}{4}^\prime$).

Bridge deck dead load deflection will require the designer to exercise some judgment concerning degree of analysis. A two or three span bridge of regular proportions, for example, should not require a rigorous analysis. The bridge deck may be assumed to be placed instantaneously on the steel section only. Generally, due to creep, deflections and stresses slowly assume a state consistent with instantaneous bridge deck placement. For unusual girder arrangements (multiple spans, unbalanced spans, curved structures, etc.), and especially structures with in-span hinges, an analysis coupled with a bridge deck placement sequence shall be used. This would require an incremental analysis where previous bridge deck placement are treated as composite sections (using a modulus of elasticity for concrete based on age at time of second pour) and successive bridge deck placements are added on non-composite sections. Each bridge deck placement requires a separate deflection analysis. The total effect of bridge deck construction is the superposition of each bridge deck placement. This analysis can also be accomplished using staged construction features in finite element analysis software. With the availability of these staged construction features, performing an incremental analysis should be considered for all structures with 3 or more spans.

Traffic barriers, sidewalks, overlays, and other items constructed after the bridge deck placement should be analyzed as if applied to the long-term composite section full length of the bridge. The modulus of elasticity of the slab concrete shall be reduced to one third of its short term value. For example, if $f'_c = 4000$ psi, then use a value of $n = 24$.

Bridge deck shrinkage has a varying degree of effect on superstructure deflections. The designer shall use some judgment in evaluating this effect on camber. Bridge deck shrinkage should be the smallest portion of the total camber. It has greater influence on shallower girder sections, say rolled beams. Simple spans will see more effect than continuous spans. For medium to long span continuous girders (spans over 200 feet without any in-span hinges), bridge deck shrinkage deflection can be ignored. For simple span girders between 150 and 250 feet, the deflection should not exceed $1^\prime$. For calculation, apply a shrinkage strain of 0.0002 to the long-term composite section using 3n.

In addition to girder deflections, show girder rotations at bearing stiffeners. This will allow shop plan detailers to compensate for rotations so that bearing stiffeners will be vertical in their final position.
Camber tolerance is governed by the *Bridge Welding Code AWS D1.5*, chapter 3.5. A note of clarification is added to the plan camber diagram: “For the purpose of measuring camber tolerance during shop assembly, assume top flanges are embedded in concrete without a designed haunch.” This allows a high or low deviation from the theoretical curve, otherwise no negative camber tolerance is allowed.

A screed adjustment diagram shall be included with the camber diagram. This diagram, with dimension table, shall be the remaining calculated deflection just prior to bridge deck placement, taking into account the estimated weight of deck formwork and deck reinforcing. The weight of bridge deck formwork may be taken equal to 10 psf, or the assumed formwork weight used to calculate total camber. The weight of reinforcing may be taken as the span average distributed uniformly. The screed adjustment should equal: (Total Camber – Steel Camber) – (deflection due to forms + rebar). The screed adjustment shall be shown at each girder line. This will indicate how much twisting is anticipated during bridge deck placement, primarily due to span curvature and/or skew. These adjustments shall be applied to theoretical profile grades, regardless of actual steel framing elevations. The adjustments shall be designated “C”. The diagram shall be designated as “Screed Setting Adjustment Diagram.” The table of dimensions shall be kept separate from the girder camber, but at consistent locations along girders. That is, at 1/10th points or panel points. A cross section view shall be included with curved span bridges, showing effects of twisting. See Appendix 6.4-A6.

For the purpose of setting bridge deck soffit elevations, a correction shall be made to the plan haunch dimension based on the difference between theoretical flange locations and actual profiled elevations. The presence of bridge deck formwork shall be noted at the time of the survey. The presence of false decking need not be accounted for in design or the survey.

### 6.3.12 Bridge Deck Placement Sequence

The bridge deck shall be placed in a prescribed sequence allowing the concrete in each segment to shrink with minor influence on other segments. Negative moment regions (segments over interior piers) must be placed after positive moment regions have had time to cure. This helps minimize shrinkage cracking and provides manageable volumes of concrete for a work shift.

Positive moment regions should be placed first, while negative moment regions are placed last. Successive segments should not be placed until previous segments attain sufficient strength, typically about 2000 psi or cure of 3 to 7 days. This general guideline is sufficient for typical, well balanced span, however the designer should check slab tensile stresses imposed on adjoining span segments. Required concrete strength can be increased, but needless delays waiting for higher strengths should be avoided. Also, the contractor should be given the option of placing positive moment segments with little influence on each other at a convenient rate, regardless of curing time. That is, segments separated by a span could be placed the same or next day without any harm. These can be lumped in the same pour sequence.
6.3.13 Bridge Bearings for Steel Girders

Make bearing selection consistent with required motions and capacities. The following order is the general preference, high to low:

- No bearings (integral abutments or piers)
- Elastomeric bearings
- Fabric pad bearings
- Disk bearings
- Spherical bearings

6.3.14 Surface Roughness and Hardness

The standard measure of surface roughness is the microinch value. Surface roughness shall be shown on the plans for all surfaces for which machining is required unless covered by the Standard Specifications or Special Provisions. Consult Machinery’s Handbook for common machining practice. Edge finishing for steel girders is covered in Standard Specifications Section 6-03.3(14). Surface hardness of thermal cut girder flanges is also controlled.

Following is a brief description of some finishes:

1000 A surface produced by thermal cutting

500 A rough surface finish typical of “as rolled” sections. Suitable for surfaces that do not contact other parts and for bearing plates on grout.

250 A fairly smooth surface. Suitable for connections and surfaces not in moving contact with other surfaces. This finish is typical of ground edges in tension zones of flanges.

125 A fine machine finish resulting from careful machine work using high speeds and taking light cuts. It may be produced by all methods of direct machining under proper conditions. Suitable for steel to steel bearing or rotational surfaces including rockers and pins.

63 A smooth machine finish suitable for high stress steel to steel bearing surfaces including roller bearings on bed plates.

32 An extremely fine machine finish suitable for steel sliding parts. This surface is generally produced by grinding.

16 A very smooth, very fine surface only used on high stress sliding bearings. This surface is generally produced by polishing.

For examples, see Figure 6.3.14-1.

For stainless steel sliding surfaces, specify a #8 mirror finish. This is a different method of measurement and reflects industry standards for polishing. No units are implied. See the Steel Specialist for examples of these finishes.
Surface Finish Examples

Figure 6.3.14-1
6.3.15 Welding

All structural steel and rebar welding shall be in accordance with the Standard Specifications, amendments thereto and the special provisions. The Standard Specifications currently calls for welding structural steel according to the AASHTO/AWS D1.5 Bridge Welding Code (BWC), latest edition and the latest edition of the AWS D1.1 Structural Weld Code. The designers should be especially aware of current amendments to the following sections of the Standard Specifications Sections 6-03.3(25) and 6-03.3(25)A.

Exceptions to both codes and additional requirements are shown in the Standard Specifications and the special provisions.

Standard symbols for welding, brazing, and nondestructive examination can be found in the ANSI/AWS A 2.4 by that name. This publication is a very good reference for definitions of abbreviations and acronyms related to welding.

The designer shall consider the limits of allowable fatigue stress, specified for the various welds used to connect the main load carrying members of a steel structure. See AASHTO LRFD Article 6.6. Most plate girder framing can be detailed in a way that provides fatigue category C or better.

The minimum fillet weld size shall be as shown in the following table. Weld size is determined by the thicker of the two parts joined unless a larger size is required by calculated stress. The weld size need not exceed the thickness of the thinner part joined.

<table>
<thead>
<tr>
<th>Base Metal Thickness of Thicker Part Joined</th>
<th>Minimum Size of Fillet Weld</th>
</tr>
</thead>
<tbody>
<tr>
<td>To ¾” inclusive</td>
<td>¼”</td>
</tr>
<tr>
<td>Over ¾”</td>
<td>⅛”</td>
</tr>
</tbody>
</table>

In general, the maximum size fillet weld which may be made with a single pass is ⅛ inch for submerged arc (SAW), gas metal arc (GMAW), and flux-cored arc welding (FCAW) processes. The maximum size fillet weld made in a single pass is ¼ inch for the shielded metal arc welding (SMAW) process.

The major difference between AWS D1.1 and D1.5 is the welding process qualification. The only process deemed prequalified in D1.5 is shielded metal arc (SMAW). All others must be qualified by test. Qualification of AASHTO M 270 grade 50W (ASTM A709 grade 50W) in Section 5 of D1.5 qualifies the welding of all AASHTO approved steels with a minimum specified yield of 50 ksi or less. Bridge fabricators generally qualify to M 270 grade 50W (A709 grade 50W).

All bridge welding procedure specifications (WPS) submitted for approval shall be accompanied by a procedure qualification record (PQR), a record of test specimens examination and approval except for SMAW prequalified. Some handy reference aids in checking WPS in addition to PQR are:

- Matching filler metal requirements are found in BWC Section 4.
- Prequalified joints are found in BWC Section 2.
- AWS electrode specifications and classifications can be obtained from the structural steel specialist. Many electrode specification sheets may be found online.
Lincoln Electric Arc Welding Handbook.

Many of Lincoln Electric’s published materials and literature are available through those designers and supervisors who have attended Lincoln Electric’s weld design seminars.

WSDOT *Standard Specifications* for minimum preheat temperatures for main members.

**Notes:** Electrogas and electroslag welding processes are not allowed in WSDOT work. Narrow gap improved electroslag welding is allowed on a case-by-case basis.

Often in the rehabilitation of existing steel structures, it is desirable to weld, in some form, to the in-place structural steel. Often it is not possible to determine from the original contract documents whether or not the existing steel contains high or low carbon content and carbon equivalence. Small coupons from the steel can be taken for a chemical analysis. Labs are available in the Seattle and Portland areas that will do this service quickly. Suitable weld procedures can be prepared once the chemical content is measured.

### 6.3.16 Shop Assembly

In most cases, a simple progressive longitudinal shop assembly is sufficient to ensure proper fit of subsections, field splices, and cross frame connections, etc., in the field. Due to geometric complexity of some structures, progressive transverse assembly, in combination with progressive longitudinal assembly may be desirable. The designer shall consult with the Design Unit Manager and the Steel Specialist to determine the extent of shop assembly and clarification of *Standard Specifications* Section 6-03.3(28) A. If other than line girder progressive assembly is required, the method must be included by special provision. High skews or curved girders should be done with some form of transverse and longitudinal assembly. Complex curved and skewed box girder framing should be done with full transverse progressive assembly. For transverse assembly, specify cross frame and pier diaphragm connections to be completed while assembled.

During shop assembly, girder segments are blocked or supported in the no-load condition (no gravity effects). Simple line girder assembly is often done in the horizontal position. The primary reason for shop assembly is to ensure correct alignment for girder field splices. For straight bridges, cross frame connections are normally done by numerically controlled (NC) drilling (no trial shop assembly). This is generally of sufficient accuracy to allow cross frame installation in the field without corrective action such as reaming.

For curved I-girders, cross frames are to be fabricated to fit the no-load condition. During field erection, girder segments will need to be adjusted or supported to make fit-up possible. This is not unreasonable since curved girders are not self-supporting before cross frames are in place. However, the method results in out-of-plumb girders. For most cases, making theoretical compensation to arrive at plumb in final condition is not justified.
Highly skewed girders present difficult fit-up conditions. Setting screeds is also complicated because of differential deflections between neighboring girders. Design of cross frames and pier diaphragms must take into account twist and rotations of webs during construction. This situation should be carefully studied by finite element analysis to determine amount and type of movement anticipated during construction. Details should be consistent. Unlike curved girders rotating away from plumb at midspan, girder webs for skewed construction should be kept plumb at piers. The NSBA has published *Skewed and Curved Steel I-girder Bridge Fit*, which is a good reference on how to deal with fit-up of skewed and curved girders.
6.4 Plan Details

6.4.1 General

Detailing practice shall follow industry standards. Designations for structural steel can be found in AISC Detailing for Steel Construction. Previous plans are a good reference for detailing practices. Detailing should also conform to national unified guidelines published by AASHTO/NSBA Steel Bridge Collaboration listed in Section 6.1.1.

Details for plate girders are continually being revised or improved to keep up with changing fabrication practice, labor and material costs, and understanding of fatigue behavior. Uses and demands for steel girder bridges are also changing. Cost benefits for individual details vary from shop to shop and even from time to time. For these reasons, previous plan details can be guides but should not be considered standards. Options should be made available to accommodate all prospective fabricators. For example, small shops prefer shorter, lighter girder segments. Some shops are able to purchase and handle plates over 90 feet long. Large shop assembly may be prohibitive for fabricators without adequate space.

WSDOT practice shall be to use field bolted connections. Cross frame members may be shop bolted or welded assemblies and shall be shipped to the field in one unit. Connections of bolted cross frame assemblies shall be fully tensioned prior to shipping. Cross frame assemblies shall be field bolted to girders during erection.

6.4.2 Structural Steel Notes

Due to their dynamic nature, the structural steel notes are not shown in this manual. They are available as standards in the drafting system. Since each project has unique requirements, these notes should be edited accordingly. Material specifications are constantly changing. Separate sets of notes are available for "I" and box girders.

6.4.3 Framing Plan

The Framing Plan shall show plan locations of girders, cross frames, and attachments and show ties between the survey line, girder lines, backs of pavement seats, and centerlines of piers. Locate panel points (cross frame locations). Show general arrangement of bottom laterals. Provide geometry, bearing lines, and transverse intermediate stiffener locations. Show field splice locations. Map out different lateral connection details. See Bridge Standard Drawing 6.4-A1.

6.4.4 Girder Elevation

The Girder Elevation is used to define flanges, webs, and their splice locations. Show shear connector spacing, location, and number across the flange. Show shear connector locations on flange splice plates or specifically call out when no connectors are required on splice plates. Locate transverse stiffeners and show where they are cut short of tension flanges. Show the tension regions of the girders for the purpose of ordering plate material, inspection methods (NDE), and Bridge Welding Code acceptance criteria. See Charpy V-notch testing requirements of the Standard Specifications. Identify tension welded butt splices for which radiographic examination (RT) is required. See Standard Specifications Section 6-03.3(25)A. V and X are also defined in the Structural Steel Notes. Permissible welded web splices should be shown, however, the optional welded web splice shall on the Girder Details sheet permits
the fabricator to add splices subject to approval by the engineer. If there are fracture critical components, they must be clearly identified along with CVN call-outs. See Bridge Standard Drawing 6.4-A2.

6.4.5 Typical Girder Details

One or two plan sheets should be devoted to showing typical details to be used throughout the girders. Such details include the weld details, various stiffener plates and weld connections, locations of optional web splices, and drip plate details. Include field splice details here if only one type of splice will suffice for the plans. An entire sheet may be required for bridges with multiple field splice designs. See Bridge Standard Drawings 6.4-A3 and 6.4-A4. Note: Do not distinguish between field bolts and shop bolts. A solid bolt symbol will suffice.

Field splices for flanges should accommodate web location tolerance of ± ¼” per BWC 3.5.1.5. Allow a minimum of ¼” for out of position web plus ¾” for fillet weld, or a total of ¾” minimum clear between theoretical face of web and edge of splice plate. The bottom flange splice plate should be split to allow moisture to drain (use 4 equal bottom flange splice plates). The fill plate does not need to be split.

Vertical stiffeners used to connect cross frames are generally 8” wide to accommodate two bolt rows. They shall be welded to top and bottom flanges to reduce out-of-plane bending of the web. All stiffeners shall be coped, clipped (or cut short in the case of transverse stiffeners without cross frames) a distance between 4t_w and 6t_w to provide web flexibility, per AASHTO LRFD Article 6.10.11.1.

6.4.6 Cross Frame Details

Show member sizes, geometrics (work lines and work points), and connection details. Actual lengths of members and dimensions of connections will be determined by the shop plan detailer. Details shall incorporate actual conditions such as skew and neighboring members so that geometric conflicts can be avoided. Double angles shall not be used for cross frames. Cross frames shall be complete subassemblies for field installation. For highly loaded cross frames, such as at piers or between curved girders, consider symmetric sections with little or no eccentricity in the connections. Where possible, allow for repetitive use of cross frame geometrics, especially hole patterns in stiffener connections, regardless of superelevation transitions. See Bridge Standard Drawing 6.4-A5.

Internal cross frames and top lateral systems for box girders are shop welded, primarily. All connection types should be closely examined for detail conflict and weld access. Clearance between bridge deck forming and top lateral members must be considered.
6.4.7 **Camber Diagram and Bearing Stiffener Rotation**

Camber curves shall be detailed using conventional practices. Dimensions shall be given at tenth points. Dimensions may also be given at cross frame locations, which may be more useful in the field. In order to place bearing stiffeners in the vertical position after bridge deck placement, it is necessary to show expected girder rotations at piers. See Bridge Standard Drawing 6.4-A6.

Office practice is to show deflection camber only. Geometric camber for profile grade and superelevation will be calculated by the shop detailer from highway alignment shown on the Layout sheets.

A separate diagram and table, with bridge cross section, should be included to show how elevations at edges of deck can be determined just before concrete placement. This will give adjustments to add to profile grades, based on remaining dead load deflections, with deck formwork and reinforcing being present.

The camber diagram is intended to be used by the bridge fabricator. The screed setting adjustment diagram is intended to be used by the contractor and inspectors.

6.4.8 **Bridge Deck**

New bridge decks for steel I-girders or box girders shall use Deck Protection System 1. The bridge deck slab is detailed in section and plan views. For continuous spans, add a section showing negative moment longitudinal reinforcing. If possible, continue the positive moment region reinforcing pattern from end-to-end of the bridge deck with the negative moment region reinforcing superimposed on it. The plan views should detail typical reinforcing and cutoff locations for negative moment steel. Avoid termination of all negative moment steel at one location. See Bridge Standard Drawings 6.4-A7 and 6.4-A8.

The “pad” dimension for steel girders is treated somewhat differently than for prestressed girders. The pad dimension is assumed to be constant throughout the span length. Ideally, the girder is cambered to compensate for dead loads and vertical curves. However, fabrication and erection tolerances result in considerable deviation from theoretical elevations. The pad dimension is therefore considered only a nominal value and is adjusted as needed along the span once the steel has been erected and profiled. The screed for the slab is to be set to produce correct roadway profile. The plans should reference this procedure contained in Standard Specifications Section 6-03.3(39). The pad dimension is to be noted as nominal. As a general rule of thumb, use 11” for short span bridges (spans less than 150’), 12” for short to medium span bridges (150’ to 180’), 13” for medium spans (180’ to 220’) and 14” to 15” for long spans (over 220’). These figures are only approximate. Use good engineering judgment when detailing this dimension.
6.4.9 **Handrail Details, Inspection Lighting, and Access**

If required, include handrails with typical girder details. Locations may be adjusted to avoid conflicts with other details such as large gusset plates. Handrail use shall be coordinated with the Bridge Preservation Office. Often, handrails are not needed if access to all details is possible from under bridge inspection trucks (UBIT’s). Also, easy public access to girder ends and handrails may represent a nuisance. Examine the bridge and site to determine the need for handrails. Fences may be required to deny public access.

Box girders require special consideration for inspection access. Access holes or hatches shall be detailed to exclude birds and the public. They shall be positioned where ladders, as a minimum, are required to gain access. If possible locate hatches in girder webs at abutments. Hatches through webs may reduce shear capacity but are easier to use. Webs can be thickened to compensate for section loss. Provide for round trip access and penetrations at all intermediate diaphragms. Openings through girder ends are preferred if space behind end walls permits. Bottom flange hatches are difficult to operate. Pier diaphragms will require openings for easy passage. Access for removing bridge deck formwork shall be planned for. Typically, block-outs in the deck large enough to remove full size plywood are detailed. Block-outs require careful rebar splicing or coupling for good long term performance. Box girders shall have electrical, inspection lighting, and ventilation details for the aid of inspection and maintenance. Refer to the Design Manual Chapter 1040 for bridge inspection lighting requirements. Coordinate with the Region Design Office to include lighting with the electrical plans.

To facilitate inspection, interior paint shall be Federal Standard 595 color number 17925 (white). One-way inspection of all interior spaces should be made possible by round trip in adjoining girders. This requires some form of walkway between boxes and hatch operation from both sides. If locks are needed, they must be keyed to one master. Air vents shall be placed along girder webs to allow fresh air to circulate. Refer to previous projects for details.

6.4.10 **Box Girder Details**

A few details unique to box girders will be presented here. Office practice has been to include a top lateral system in each box, full length of a girder. There is a possibility of reducing some bays of the top laterals in straight girders without sacrificing safety during construction. However, most WSDOT box girders are built to some level of curvature, and the practice of using a full length top lateral system should be adhered to unless a careful stability analysis is undertaken. In the past, the top lateral system was detailed with 6” to 8” clearance between lateral work line and bottom of top flange. The intent was to provide adequate clearance for removable deck forming. This requires the introduction of gusset connecting plates with potentially poor fatigue behavior if welded to the web.

A cleaner method of attaching the top laterals is by bolting directly to the top flange or intermediate bolted gusset plate (in which case, the lateral members may be welded to the gusset plate). The flange bolting pattern shall be detailed to minimize loss of critical material, especially at interior supports. In order to maximize the clearance for bridge deck forms, all lateral connections should progress down from the bottom surface of the top flange. The haunch distance between top of web and deck soffit shall
be 6” or greater to allow deck forming to clear top lateral members. Supplemental blocking will be required to support deck forms on the typical waler system. See example top lateral details Bridge Standard Drawings 6.4-A11.

Ideal girder construction allows full length web and flange plates to be continuously welded without interruption of the welder. This process is routinely accomplished with I-girder shapes, where web stiffeners are attached after top and bottom flanges are welded to the web. With box girders, however, due to handling constraints, most fabrication shops need to progress from top flange-to-web welding, welding stiffeners to webs, and then welding the top flange plus web assemblies to the bottom flange. This introduces a start and stop position at each web stiffener, unless enough clearance is provided for the welder. To achieve this, the stiffener should be held back and attached to the bottom flange by a member brought in after the bottom longitudinal welds are complete. See detail Bridge Standard Drawings 6.4-A11.

Small tractor mounted welders are able to run a continuous pass on the bottom external weld, provided there is adequate shelf width. The standard offset between center of web and edge of bottom flange is now 2”. In the past, this weld was primarily performed by hand.

The most significant design difference between I-girders and box girders occur in bottom flange compression regions. Using thicker material to provide stability is not usually economical, given the typically wide unsupported flange widths. The standard practice has been to stiffen relatively thin compression plates with a system of longitudinal and transverse stiffeners. WSDOT practice is to use tee shapes, either singly or in pairs for the wider plates. Ideally, the stiffeners are terminated at bolted field splices. If the stiffener is terminated in a region of live load tension cycles, careful attention needs to be paid to design fatigue stresses and the termination detail. See details Bridge Standard Drawings 6.4-A13.

Box girder inside clear height shall be 5 feet or more to provide reasonable inspection access. Less than 5 feet inside clear height is not be permitted. Other girder types and materials shall be investigated.

Drain holes shall be installed at all low points.

Geometrics for boxes are referenced to a single workline, unless box width tapers. The box cross section remains tied to a centerline intersecting this workline and normal to the bridge deck. The section rotates with superelevation transition rather than warping. See box girder geometrics and proportions Bridge Standard Drawings 6.4-A10.

Box girders shall be supported by single centralized bearings when two or more boxes make up the bridge section. This requires diaphragms between boxes for bracing. See pier diaphragm details Bridge Standard Drawings 6.4-A12.
6.5  Shop Plan Review

Shop plans shall be checked for agreement with the Contract Plans, *Standard Specifications*, and the Special Provisions. The review procedure is described in Section 1.3.5. Material specifications shall be checked along with plate sizes.

Welding procedure specifications (WPS) and procedure qualification records (PQR) should be submitted with shop plans. If not, they should be requested so they can be reviewed during the shop plan review process.

Most shop plans may be stamped:

“GEOMETRY NOT REVIEWED BY THE BRIDGE AND STRUCTURES OFFICE”

However, the reviewer should verify that lengths, radii, and sizes shown on shop plans are in general agreement with the contract. The effects of profile grade and camber would make exact verification difficult. Some differences in lengths, between top and bottom flange plates for example, are to be expected.

The procedures to follow in the event changes are required or requested by the fabricator can be found in Section 1.3.6. In the past, shop plans with acceptable changes have been so noted and stamped:

“STRUCTURALLY ACCEPTABLE, BUT DOES NOT CONFORM TO THE CONTRACT REQUIREMENTS”
6.6 Bridge Standard Drawings

Structural Steel

6.4-A1 Example Framing Plan
6.4-A2 Example Girder Elevation
6.4-A3 Example Girder Details
6.4-A4 Steel Plate Girder Example–Field Splice
6.4-A5 Example–Crossframe Details
6.4-A6 Example–Camber Diagram
6.4-A7 Steel Plate Girder Example–Roadway Section
6.4-A8 Steel Plate Girder Example–Slab Plan
6.4-A9 Example – HANDRAIL
6.4-A10 Example–Box Girder Geometrics and Proportions
6.4-A11 Example–Box Girder Details
6.4-A12 Example–Box Girder Pier Diaphragm Details
6.4-A13 Example–Box Girder Miscellaneous Details
6.4-A14 Example–Access Hatch Details
6.4-A15 NGI-ESW CVN Impact Test for Heat Affected Zone
6.99 References

The following publications can provide general guidance for the design of steel structures. Some of this material may be dated and its application should be used with caution.

1. FHWA Steel Bridge Design Handbook (November, 2012)
   This includes 19 volumes of detailed design references for I-girders and box girders, both straight and curved, utilizing LRFD design. This reference also has 6 detailed design examples for I-girder and box girder bridges, straight and curved.

2. Composite Steel Plate Girder Superstructures, by US Steel
   Example tables and charts for complete plate girders, standardized for 34 and 44 ft roadways and HS-20 loading. Many span arrangements and lengths are presented.

3. Steel Structures, Design and Behavior by Salmon and Johnson
   A textbook for steel design, formatted to AISC LRFD method. This is a good reference for structural behavior of steel members or components, in detail that is not practical for codes or other manuals.

   This publication is quite helpful in the calculation of section properties and the design of individual members. There are sections on bridge girders and many other welded structures. The basics of torsion analysis are included.


6. AASHTO/NSBA Steel Bridge Collaboration Publications
   These publications include several guidelines for design, detailing, fabrication, inspection and erection of steel structures.

7. A Fatigue Primer for Structural Engineers, by John Fisher, Geoffrey L Kulak, and Ian F. C. Smith

   The essential reference for rolled shape properties, design tables, and specifications governing steel design and construction.

   A reference book for the machine shop practice; handy for thread types, machine tolerances and fits, spring design, etc.

10. Painting of Steel Bridges and Other Structures, by Clive H. Hare
    This is a good reference for paint systems, surface preparation, and relative costs, for both bare and previously painted steel. Explanations of how each paint system works, and comparisons of each on the basis of performance and cost are provided.

11. NCHRP Report 314, Guidelines for the Use of Weathering Steel in Bridges
    This reference contains detailing information if weathering steel will be used. Protection of concrete surfaces from staining and techniques for providing uniform appearance is provided.