Oregon Department of Transportation

SECTION 1.6
Steel Structure Design and Detailing

Revisions for the current edition are highlighted yellow. Deleted text is not marked.

Past editions of the Bridge Design Manual are available through our website.

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1.6 STEEL STRUCTURE DESIGN AND DETAILING

1.6.1 Structural Steel, General

This chapter covers primarily the design and construction of steel plate and box girder bridge superstructures. It provides guidance for bridge designers working on ODOT projects to achieve optimal quality and value in steel bridges.

1.6.1.1 Structural Steel, Design Considerations

Take into consideration, fabrication requirements, construction techniques and future maintenance needs in the design, including the following items:

- Use the most recent version of the design specifications.
- Become familiar with construction and fabrication specifications and standard drawings applicable to steel structures.
- Evaluate how construction and fabrication specifications influence the design and what modifications or special provisions may be required.
- During the initial evaluation of design options, consider consulting with the Steel Bridge Standards Engineer, fabricators, steel erectors or contractors for ideas on achieving an economical, easy to build and robust design.
- Ensure that all individual bridge components fit well together by accounting for how rotation, deflection (especially differential deflection), twist, stiffness (vertical bending, lateral bending, and torsion), and skew affect interaction between different elements.

C1.6.1.1

Familiarity with design and construction specifications are key to steel bridge design. Designs that merely satisfy the design specifications are often problematic.

- Provide clear and distinct load paths that mitigate or preferably, eliminate out-of-plane bending.
- Provide adequate access for bolting, welding, and painting.
- Keep design simple by maximizing the use of common details and minimizing the number of plate sizes and rolled shapes fabricators are required to purchase. Complicated details are always difficult to fabricate and build.
- Do not use details that permit water and debris to collect on girders.

1.6.1.2 Codes and Standards

Design according to *AASHTO LRFD Bridge Design Specifications* unless specified otherwise in this document. See *NSBA Guide to Navigating Routine Steel Bridge Design* for a guide to designing routine steel I-girder bridges.

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 AISC website:

- *G13.1-2019, Guidelines for Steel Girder Bridge Analysis.*
- *G12.1-2020, Guidelines to Design for Constructability and Fabrication.*
- *G1.4-2006, Guidelines for Design Details.*
- *S10.1-2019, Steel Bridge Erection Guide Specification.*
- *G9.1-2022, Steel Bridge Bearing Guidelines.*
- *G1.2-2003, Design Drawing Presentation Guidelines.*
- *S2.1-2018, Steel Bridge Fabrication Guide Specification.*
- *G1.3-2002, Shop Detail Drawing Presentation Guidelines.*
- *G1.1-2020, Shop Drawing Approval Review/Approval Guidelines.*
- *G4.1-2019, Steel Bridge Fabrication QC QA Guidelines.*
- *G2.2-2016, Guidelines for Resolution of Steel Bridge Fabrication Errors.*
- *G4.2-2021, Guidelines for the Qualification of Structural Bolting Inspectors.*
- *G4.4-2006, Sample Owners Quality Assurance Manual.*
- *S8.1-2014, Guide Spec. for Application of Coating Systems w/ Zinc-Rich Primers to Steel Bridges.*
- *S8.2-2017, Specification for Application of Thermal Spray Coatings for Steel Bridges.*

The Steel Bridge Design Handbook publication includes 19 chapters of steel bridge design topics with six design examples to be used as design aids when deciding the design, fabrication and construction of steel bridges.

1.6.1.3 ODOT Steel Bridge Practice

ODOT does not require Certified Erector Qualification for erection of steel bridges. For complex projects in which a contractor with such qualification is deemed necessary, obtain Bridge Engineering Section approval prior to including such requirement in the contract documents.

Specify caulking according to *Standard Specifications 00594.43(f)* to fill and seal crevices and gaps between structural shapes and plates, around bolt heads and nuts, weld terminations and similar areas that would retain moisture.

Refer to *AASHTO/NSBA G1.4-2006, Guidelines for Design Details* for design details that allow for the economical fabrication and erection.

1.6.1.3.1 Fatigue Design Requirements

Design all welded and bolted connections for infinite fatigue design life using ADTT from *LRFD Table 6.6.1.2.3-2*. Do not use details category E or E′ in any connections for primary load-carrying members.

C1.6.1.3

C1.6.1.3.1

The fatigue live load specified in *AASHTO LRFD Article 3.6.1.4* should 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. As specified in *Article 3.6.1.1.2*, multiple presence factors are also not to be applied to the fatigue limit state check for which one design truck is used. The load factor is 1.75. 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.

Use G13.1-2019, Guidelines for Steel Girder Bridge Analysis *Section 4 and Appendix B* on selecting the appropriate level of analysis.

1.6.1.4 Estimating Structural Steel Weights

For preliminary quantities or preliminary girder design, obtain an estimate of steel weights for built-up plate composite I-girders from the AISC/NSBA Standard Plans for Steel Bridges from the AISC website.

Forshort span bridges (40 to 140 feet), refer to Short Span Steel Bridge Alliance website.

For span and girder configurations beyond the above cases, obtain an estimate of steel weights for built-up plate composite I-girders from the Span to Weight Curves from the AISC website.

Curved and skewed deck girder bridges have the potential for three-dimensional deflection and rotation. Longer spans magnify the rotation of the girders and cause unaccounted stresses on the diaphragm connections.

C1.6.1.4

The *AISC/NSBA Standard Plans for Steel Bridges* provide comprehensive designs for straight steel I-girder bridges, encompassing one- to four-span configurations with span lengths from 80 to 300 feet and girder spacings of 8, 10, 12, and 14 feet. These plans offer optimized solutions that detail girder plate sizes, diaphragm spacing, intermediate stiffener sizes and locations, shear connector spacing, camber, and girder weights.

The *Steel Span to Weight Curves* are the quickest way to determine the weight of steel per square foot of bridge deck for preliminary girder design of straight, low skew, plate girder bridges. The curves are organized by span arrangement (1, 2 or 3 or more span bridges) and girder spacing. The curves also provide a double check for final quantities.

1.6.1.4.1 Bid items

Use the following bid items for structural steel. Use horizontally curved steel (plate or box) girder bid item when the radius of horizontal curve on the structure is less than 1,000 feet.

- Steel Rolled Beam.
- Steel Plate Girder.
- Steel Box Girder.
- Steel Plate Girder with haunch.
- Trapezoidal Steel Box Girder with haunch.
- Horizontally Curved Steel Plate Girder.
- Horizontally Curved Steel Box Girder.
- Specialty Bridges (tied arches, Cable Stayed).
- Structural Steel Maintenance.

For seismic retrofit project, break down the retrofit item under Structural Steel Maintenance:

- Structural Steel Maintenance Cross Frame Strengthening.
- Structural Steel Maintenance Bumper Blocks.
- Structural Steel Maintenance Restrainers.
- Structural Steel Maintenance Shear Key.

1.6.2 Structural Steel, Design

1.6.2.1 Steel Grade Selection

C1.6.2.1

Identify all steel by grade on the contract plans.

Provide structural steel for bridges conforming to *ASTM A709* (*AASHTO M270*).

ASTM A36, A572 or *A588* may be used for structural steel for bridges provided the supplementary Charpy V-Notch Impact test requirements are included in the *Special Provisions*. When Charpy V-Notch Impact tests are required for *ASTM A36, ASTM A572* or *ASTM A588* structural steel, use the supplementary requirements of *ASTM A709*.

These specifications include grades 36, 50, 50W, HPS 50W and HPS 70W. *ASTM A709* steel specifications are written exclusively for bridges wherein supplementary requirements for Charpy V-Notch Impact tests are mandatory.

Specify *ASTM A709* Grade 50 steel for all structures that require yield strengths between 36 ksi and 50 ksi and are to be painted or galvanized.

Do not use *A709* (Grades 36, 50, 50W) steels for plates thicker than 3-inches, or butt welds in tension members over 3 inches.

Specify Grade HPS (high-performance steel) 50W and HPS 70W to be **Quenched and Tempered** in the contract document. For thermo-mechanical control processed steel, require the contractor to provide test samples at both ends of each rolled plate. Plates that pass the required test are acceptable for fabrication.

Grade HPS 70W steel has recently been developed and provides high strength, enhanced durability and improved weldability. Depending on the availability, grade HPS 70W may be economical only in hybrid girders. With grade 50W webs, use a hybrid configuration with HPS 70W tension and compression flanges in high moment regions. See Steel Bridge Design Handbook Chapter 1 Article 2.1.6 Grades HPS 50W, HPS 70W and HPS 100W for more background.

Limit plate thickness for HPS 50W and HPS 70W to 2 inches. Consult with the Steel Bridge Standards Engineer for specific project needs.

Provide structural steel for steel piling, metal sign structures and other incidental structures conforming to *ASTM A36, ASTM A572 or ASTM A588*. Incidental structures include luminaire and traffic signal supports, bridge metal rails and metal rail posts, guardrail connections, earthquake restraints, bridge deck expansion joints and fencing post connections, etc.

Use merchant quality steel (non-spec) in items such as catch basin frame, catch basin, deck drain grate, manhole rungs and steps, access hole cover, guardrail spacer blocks, shims, anchor bolt plate embedded in concrete, etc. and where a high degree of internal soundness, chemical uniformity or freedom of surface defects are not required. Acceptance of such items is based on visual inspection.

1.6.2.2 Weathering Steel

Avoid requiring weathering steel in painted applications.

To achieve the performance expected of uncoated weathering steel (UWS), prevent prolonged exposure to wetness or high levels of chlorides, without the opportunity to dry, by avoiding use in the environments stated below.

C1.6.2.2

Weathering steel performs as well but not better than non-weathering steel in painted applications.

Review the following references for appropriate application of weathering steel:

- NSBA Uncoated Weathering Steel Reference Guide.
- Steel Bridge Design Handbook: Chapter 19 – Corrosion Protection of Steel Bridges.
- Computer Modeling and Analysis of Truck Generated Salt Spray Transport Near Bridges.

Similar to regular construction steel, weathering steels also rust under a wide range of exposure conditions. But during this process it forms oxides that remain tightly adherent to the steel substrate and

develops a much more stable oxide layer than non-weathering steel. Shortly after blast cleaning to remove mill scale, weathering steel turns rusty in appearance. Through several cycles of wetting and drying (usually between 6 and 24 months, depending on the environment), the surface of the steel develops a tight oxide coating (patina) that provides its own corrosion resistant surface finish, eliminating the need for painting and resulting in minimal future maintenance and lower life cycle costs.

Do not use weathering steel in the environments and locations listed below:

Design deviations will be approved for environments and locations of concern when there is a reasonable case for weathering steel at the specific site. The intent is not to discourage reasonable use of weathering steel, but to provide an orderly, documented process for the judgment necessary to properly apply weathering steel in environments and locations of concern. See NSBA Uncoated Weathering Steel Reference Guide Article 2.2 and 2.3 for detailed consideration related to the macro- and micro- environment.

1.6.2.2.1 Environment

- When the bridge site is within two aerial miles of the coastline.
- At coastal areas (*BDM 1.2.4.2*) with eight or more months of the year experiencing average relative humidity between 76 to 80 percent or higher.
- Industrial areas where concentrated chemical fumes may drift onto the structure.

C1.6.2.2.1

Average relative humidity by month can be obtained through *NOAA*:

- Archived data on from stations in Oregon in NOAA's Annual Local Climatological Data publications.
- Morning and afternoon averages by a station's period of record from NOAA's Comparative Climatic Data.

1.6.2.2.2 Location

- Grade separations in tunnel like conditions.
- Low level water crossings.
    - Ten feet or less over stagnant, sheltered water.
    - Eight feet or less over moving water.
- Conditions that do not allow for the drying of the steel necessary to develop a good patina.

With the proper details, good performance from weathering steel can be achieved by reduce or eliminate unsightly concrete staining. Comply with the following design detailing these requirements:

- Provide adequate drainage beneath overpass structures to prevent ponding and continual traffic spray from below. Communicate the importance of adequate drainage to roadway designers.
- Do not detail deck drains that can discharge water onto the steel, especially in regions that use de-icing chemicals.
- Avoid any type of open joint that allows runoff to reach the steel.
- Provide details that take advantage of natural drainage.
- Provide drip plates (also called drip tabs) to divert runoff water and protect abutments and columns from staining.

C1.6.2.2.2

Provide flange thickness transitions instead of flange width transitions for bottom flanges (with the thickness added to the underside of the flange).

- Eliminate details that retain water, dirt, and other debris. Provide stiffener clips for proper ventilation and drainage

Use haunched girders with generous allowances for drainage through stiffener copes, snipes, and drain holes since the use of haunched girders complicates the efficient removal of run-off and can result in the collection of debris at the haunch.

1.6.2.3 Simple for Dead Load Continuous for Live Load

Consider a simple steel bridge system (SDCL) for dead load and continuous for live load in the design of multi-span structures when simple spans or full continuous spans do not provide efficient span configuration and design.

Critical to the functionality of SDCL structures is the continuity connection at the interior bents. Consider use of concrete diaphragms over the bridge piers. See *DET3620*.

For designs intending to eliminate the deck joint at interior bents while allowing simple-span bridge behavior to be retained, consider using ultra-high performance concrete (UHPC) link slabs as a continuity connection. Do not use link slab for bridges with SDC C and D due to unknown performance during a large earthquake.

C1.6.2.3

Span configuration plays an important role in steel efficiency. Two span continuous girders for dead and live loads are not always efficient because of high negative moments. Three span units with interior spans about 20 to 30 percent longer than end spans are preferable when possible.

Relative to full continuous spans, SDCL bridges reduce uplift in unbalanced spans, reduce negative moments at the bents, simplify fabrication, and eliminate the need for bolted field splices. See ABC-UTC Guide for Superstructure to Pier Connection in SDCL Steel Bridge Systems for additional guidance.

Concrete diaphragms are a feasible connection detail for providing live load continuity over the pier in a SDCL steel girder system when conventional construction methods are employed.

Link slabs are proven to be an economical detail for eliminating deck joints on bridges. Elimination of the deck joint increases a bridge's service life, reduces the need for frequent maintenance, and provides a smooth riding surface. See *BDM 1.9.2.3.1* for more information on UHPC.

Research in steel industry practices is continuously improving. Discuss appropriate connection details with the Steel Bridge Standards Engineer.

1.6.2.4 Uplift

Avoid span layouts with uplift at end bents for DL case.

Always consider the presence of uplift at the ends of continuous girders, particularly with light, rolled beam units or short end spans. Uplift restraint, when needed, must satisfy the Strength limit state and the Fatigue limit state.

C1.6.2.4

When the end spans are much shorter than the interior spans, uplift at the girder ends can occur and create design and construction problems. When end spans are too long in relation to interior spans, a disproportionate amount of steel will be required for the end spans.

Commentary to *AASHTO LRFD Bridge Design Specifications, Article C3.4.1*, indicates that uplift needs be checked under a strength load combination. It also provides guidance in the appropriate use of minimum and maximum load factors.

1.6.2.5 Girder Spacing

Use wider girder spacing to reduce the number of lines of girders, which will reduce shop and field labor. Consider the following to optimize girder spacing:

- ODOT prefers a minimum of four I-shaped beams/girder span for vehicular bridges.
- Stability and redundancy of the structure during future re-decking.
- Thicker concrete deck results in more concrete and reinforcing steel, and possibly in more superstructure weight.
- Wide girder spacing can create challenges for deck formwork, and slabs (or floor systems) cannot adequately support certain overloads.
- On straight bridges, interior and exterior girders must be detailed identically. Spacing must be such that the distribution of wheel loads to the exterior girder is close to that of the interior girder.

C1.6.2.5

Many studies show that the weight of structural steel per square foot of deck area decreases as girder spacing increases. Refer to *G12.1-2020, Guidelines to Design for Constructability Article 1.2* for more information. Generally, for a bridge with an average span length less than 175 feet, there is not an appreciable difference in the structural steel unit weight for the various girder spacing summarized in the graph in *G12.1 Article 1.2*. For a bridge with an average span length more than 175 feet, the designer may want to consider a wider girder spacing, perhaps between 11 and 13 feet, as this wider girder spacing trends to a lighter steel superstructure.

With wider girder spacing and *AASHTO* simplified live load distribution factor, the fatigue limit-state check at the cross-frame connection-plate weld to the bottom flange could control the design of the bottom flange. It is prudent for the designer to consider moving the cross-frame away from high flexural location or use refined analysis

to determine the stress at the cross-frame connection-plate weld to the bottom flange to achieve economical girder design, rather than increase the bottom flange thickness for the entire girder segment.

Bolted tab plate detail is not recommended (Refer to *G12.1-2020, Guidelines to Design for Constructability Article 2.1.2.2*).

1.6.2.6 Shop Lengths of Welded Girders

Optimize the girder segments by reducing the number of field splices.

All field splices must be bolted. Locate field splices in welded steel girders by not exceeding the following shop lengths and mass:

- 150 feet when bridge site is readily accessible.
- 130 feet when bridge site is not readily accessible.
- Fabricators are limited to their shop crane sizes. Contact fabricators in the state of Oregon for project specific needs and requirements.

For curved girders, limit the girder sweep plus the flange width to 6 feet for ease of shipping. The current legal vehicle width is 8 feet 6 inches without a permit. Limiting the overall shipping width of curved girders to 6 feet permits fabricators to offset the girder on the trailer, as is frequently done, while not exceeding an overall width of 8 feet 6 inches. Add optional field splices if required.

C1.6.2.6

Maximum girder or girder field segment lengths without a field splice ranges from 130 feet to 150 feet depending upon cross section. There may be locations where girder lengths will be controlled by weight or access to the bridge site. Long and deep girders may also require auxiliary lateral support during transportation.

It is typical to show an optional bolted field splice to allow the fabricator and contractor some flexibility in fabrication and transportation.

Longer girders have been fabricated and hauled to project sites, however contact fabricators and the Steel Bridge Standards Engineer when project requires girder segments longer than 150 feet.

1.6.2.7 Rolled Beam Sections

C1.6.2.7

Rolled beams can be more economical than plate girders for their applicable span lengths (up to 80 feet) because of decreased fabrication costs. Do not use rolled beam sections smaller than W18.

Select beams that have a top flange that is sufficiently wide to provide adequate spacing for three stud connectors per row.

Beams must be large enough that the elastic neutral axis of the composite section is within the steel beam (not within the slab or haunch).

Rolled beams are not exempt from camber requirements of *BDM 1.6.2.13*.

Verify the need for bearing stiffeners for rolled beams by using the provisions in *LRFD Appendix D6.5*.

Rolled beams usually do not need bearing stiffeners.

The diaphragms between beams usually consist of rolled shapes with channels being the most common choice.

1.6.2.8 Plate Girders

1.6.2.8.1 Flange Width and Thickness

C1.6.2.8.1

The minimum size flange is ¾ inch thick by 12 inch wide.

The minimum ¾ inch flange thickness is to minimize the distortion of the flange due to welding of the flange to the web.

It may not be prudent to minimize the top flange. Flange width affects girder stability during handling, erection, and deck placement. Keep the girder length (length of the unspliced individual girder field section) to flange width ratio below 85.

The girder needs significant lateral load capacity to resist lateral transportation loads and lateral loads from deck overhang brackets and deck placements. Another side benefit of providing generous top flange is that the non-composite deflections are reduced. See *LRFD Appendix C6.10.2.2* for commentary.

Minimize the number of changes in flange size, as the cost of a butt weld will offset a considerable length of excessive flange area. When locating flange thickness transitions (shop flange splices), include no more than two butt splices or three different flange thicknesses for an individual flange between field splices, except for unusual cases such as very long or heavy girders or mill length availability limits.

Constant width flanges enable the fabricator to order the flanges in multiple width plates which are more economical than universal ill plates. The shop flange splices can be made while the plates are in wide slabs and cut to widths simultaneously with multiple cutting torches. Also, the recommended practice for general constructability is to favor flange thickness transitions instead of flange width transitions.

Limit the maximum change between adjacent plates to 6 inches in width, at both welded and bolted connection section changes.

Efficiently locating thickness transitions in plate girder flanges is a matter of plate length availability and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. Refer to NSBA G12.1 Guidelines to Design for Constructability Article 1.5.1 for optimizing flange thickness transitions.

The rule of thumb is to limit flange transitions such that the smaller flange at a welded transition is no less than 50 percent of the area of the larger flange.

The rule of thumb accomplishes two things. First, the bending stress gradient in the girder web due to the change in section properties does not become overly steep when this criterion is met. Second, it has been demonstrated in past designs that, when the flange transition results in greater than a 50 percent reduction in flange area, either the transition is not in the optimum location or an additional transition may prove to be economical.

Minimizing the number of changes in the top flange will also facilitate easier deck forming.

Make top and bottom flanges a constant width where possible. When a change in bottom flange width is needed, make it at a bolted splice location.

Limit the maximum flange thickness to 3 inches. At welded flange splices, the thinner plate must not be less than one-half the thickness of the thicker plate.

Generally, use a minimum flange width that is equal to the width of the flange resisting the maximum positive moment. Widen the flange as necessary in negative moment areas so the flange thickness will not exceed 3 inches at the bent.

1.6.2.8.2 Web Depth and Thickness

Girder depths, particularly for haunched girders, may be limited because of transportation constraints.

Use web depths in whole-inch increments. Minimum web thickness is ½ inch. Thinner plates are subject to excessive distortion from welding.

Use web plate of sufficient thickness to eliminate the need for transverse stiffeners either entirely or partially. In high shear regions, when transverse stiffeners are spaced at 8 to 10 feet prevent the need for a thicker web, the use of a stiffened web can be justified.

C1.6.2.8.2

Commonly used web plates range from 48 to 96 inches deep. The recommendations in *LRFD 2.5.2.6.3* provide a good estimate of a minimum web depth for straight girders. If vertical clearance is not a problem, adding depth can result in lighter girders. For curved girder web depth, use either the *AASHTO* recommended minimum depth for straight girders, increased by 10 to 20 percent, or use *LRFD Equation 2.5.2.6.31* as a starting point. A rule of thumb for a well-proportioned superstructure is to have total section depth (slab plus girder) in the range of 0.033L to 0.04L (L = c–c bearing length). For web depths up to 96 inches, provide sufficient thickness to preclude the need for longitudinal stiffeners.

The labor to place and weld one foot of stiffener is equal to about 25 pounds of steel. Unstiffened webs reduce fabrication, painting costs (for non-weathering steel) and flange sizes. Thicker webs are also helpful in reducing web distortion due to welding and in supporting deck overhang brackets for the deck placement.

Design web plates in 1/16 inch increments with a note that the contractor may increase the web thickness shown by 1/16 inch at no additional cost to the State. Minimize web transitions as the cost of a butt weld web splice often exceeds the cost of the added material between sections.

The cost of a square butt joint web splice is equal to about 800 pounds of steel per foot of splice. When web plates are over 80 feet long and constant thickness, provide the fabricator an optional shop splice on the design plans. The most economical bid can then be prepared according to the mill length extras, market areas available, and transportation and handling costs.

1.6.2.8.3 Girder Splices

C1.6.2.8.3

Locate splices to avoid conflicts with wind bracing, diaphragms and intermediate stiffeners. Layout locations of all intermediate stiffeners, diaphragms and wind bracing to avoid conflicts with the flange cutoff points (and possible splice locations).

Splices are a natural location to make changes in the flange size to eliminate flange welds. Maintain the same web thickness on each side of the splice.

For flexural members, it is recommended that the smaller section at the point of splice be taken as the side of the splice that has the smaller calculated moment of inertia for the non-composite steel section.

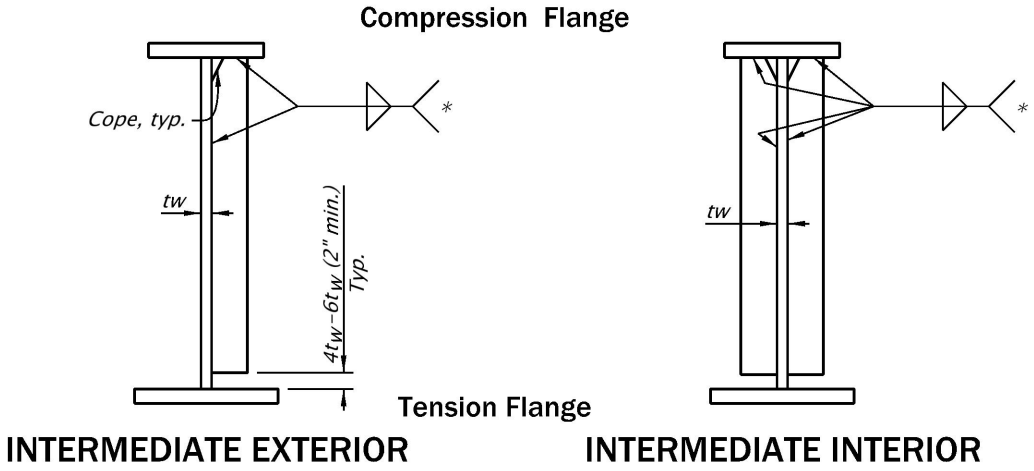
1.6.2.8.4 Intermediate Web Stiffeners

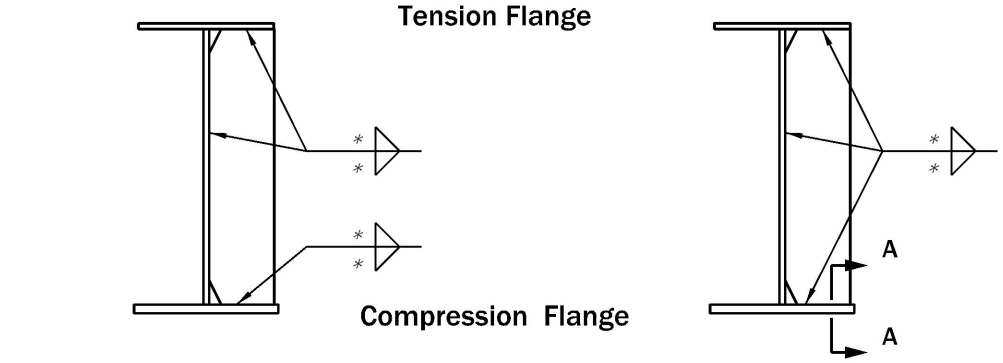
C1.6.2.8.4

When used, provide transverse intermediate stiffeners on both faces of the webs of interior girders and on the interior faces, only, of exterior girders. Specify stiffener widths in ½ inch increments. Specify thickness in 1/8 inch increments using 3/8 inch as an absolute minimum.

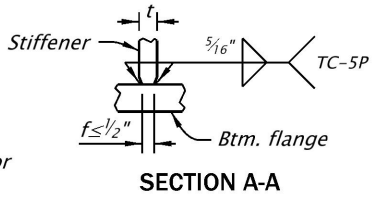
Rigidly connect the stiffeners to the compression portions of the flanges. Stiffeners may be welded to compression flanges. Ends welds ¼ inch away from the edge (snipe, clip, etc.) to avoid a poor quality weld termination. See *Figure 1.6.2.8.4-1* for details of intermediate web stiffeners.

Figure 1.6.2.8.4-1 Intermediate Web Stiffeners





For stiffener thickness connecting to tension flange ≤ ½".
∗ Size fillet welds in accordance with AASHTO LRFD. Minimum weld sizes not less than ¼" for t ≤ ¾" or 5/16" for t > ¾".
∗ Compression and tension flanges reverse near interior bents for continuous girder.



1.6.2.8.5 Bearing Stiffeners

Select bearing stiffener widths in increments of ½ inch. Specify thickness in 1/8 inch increments. Bearing stiffeners should extend to ½ to ¾ inch from the flange edge. Minimum size of fillet weld is specified in *BDM 1.6.3.2*. See *Figure 1.6.2.8.5-1* for details of bearing stiffeners.

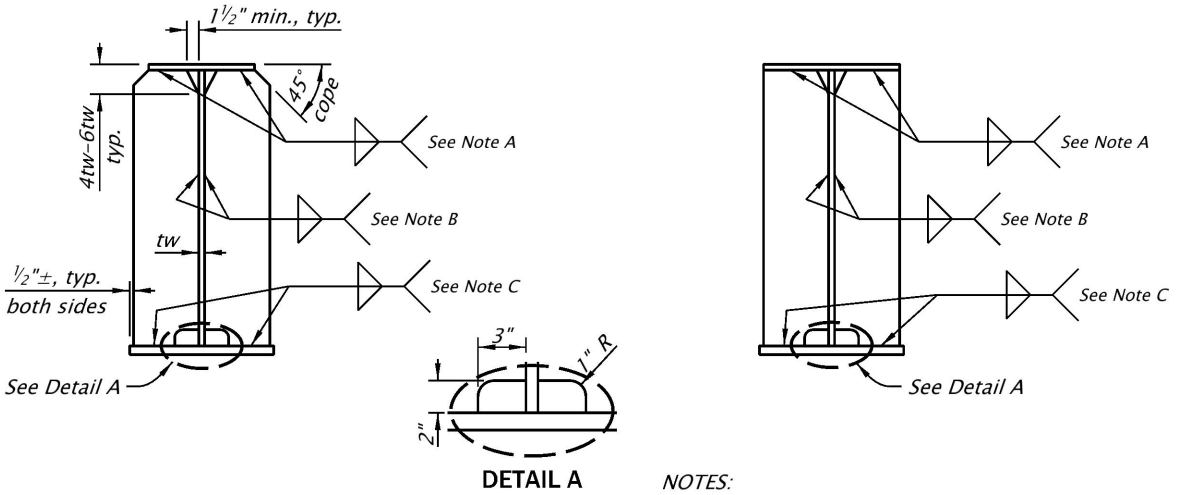
Fabricators strongly discourage full-penetration welding of bearing stiffeners to flanges.

C1.6.2.8.5

Bearing stiffeners and the web act as a column section, transferring loads from the superstructure to the substructure. In combination with the end frames, they also transfer lateral loads from the superstructure to the substructure.

Full-penetration welds distort the bearing area of the bottom flange.

Figure 1.6.2.8.5-1 Bearing Stiffener Detail



NOTES:

A. Use fillet weld in lieu of tight fit when a diaphragm or crossframe attaches to stiffener. Tight fit only without a diaphragm or crossframe attached.

B. Fillet weld sizes shall be shown on the design drawings or a note stating that minimum AASHTO/AWS welds shall be used. Where ever possible avoid fillet welds over 5/16".

C. Finish to bear (typ.) and fillet weld if a diaphragm or crossframe attaches to stiffener. Insure clearance to bearing attachment bolts. Investigate load for requirement of CJP welds.

Limit bearing stiffeners skew angle at end bents or interior bents to the values shown in *Figure 2.3* of the *AWS D1.5* for bearing stiffeners to web connection. Discard the footnotes of the figure which permits angles

less than 60 degrees. When the skew angle exceeds limit shown on *Figure 2.3*, use bent plates or half-round bearing stiffeners.

1.6.2.9 Check Samples and Nonredundant Steel Tension Members

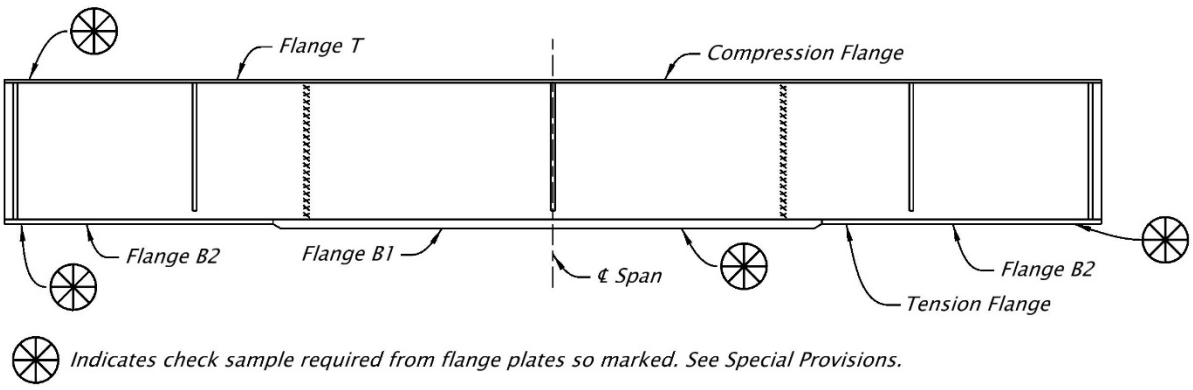
1.6.2.9.1 Check Samples

C1.6.2.9.1

Identify tension members and elements that require notch toughness check samples on the plans.

Check samples are required for cross-frame members on curved steel girders.

Figure 1.6.2.9.1-1 Check Sample Identification Detail



1.6.2.9.2 Nonredundant Steel Tension Members

C1.6.2.9.2

Clearly identify nonredundant steel tension members (NRST) on the plans. See FHWA-HIF-19-088 Bridge Welding Manual *Article 7.2.4 Design Detailing Practice* for identifying nonredundant steel tension members.

1.6.2.10 Fit-up and Intermediate Cross-Frames

Fit-up of steel bridge members is a critical component in the overall success of a project. Clearly convey to the fabricator and erector the intent of the design as it relates to bolting and pinning, camber, bolted field splices, and differential deflections. State in the contract documents the fit condition for which the cross-frames or diaphragms are to be detailed according to *LRFD 6.7.2*.

Include a note in the contract drawings stating that the girder webs are plumb in the final condition.

This requires the erector to force fit the diaphragms with the girders out-of-plumb prior to deck placement. Rotation of girders resulting from the deck placement plumbs the girders web and releases stresses caused from force fitting the diaphragms.

C1.6.2.10

Steel bridges, including straight and skewed bridges, should be detailed so they are plumb in the final condition. For steel girder bridges this means that the girder webs should be plumb after deck and barrier placement. This is accomplished by detailing the cross-frames to the final position. The girders are then installed to fit the cross-frames, requiring that for skewed bridges they be **rolled** during fit-up so that they are out of plumb under steel dead loads. Tolerances for web plumbness/girder layover are specified in the *AASHTO/NSBA Guide Specification S10.1-2014, Steel Bridge Erection Guide Specification Section 9*.

Steel Dead Load Fit (SDLF, also known as Erected Fit): For bridges which are detailed for SDLF the girder webs should be plumb (within reasonable construction tolerance) at the end of steel erection, prior to deck placement. When they are not plumb at the end of steel erection (prior to deck placement), the engineer should be consulted and remedial action should be considered. Later, when the deck is placed, the webs will lay over and be out of plumb. This sequence of webs being plumb prior to deck placement and out of plumb after deck placement is normal and generally does not represent a problem.

Total Dead Load Fit (TDLF, also known as Final Fit): For bridges which are detailed for TDLF the girder webs should be plumb (within reasonable construction tolerance) at the end of deck placement. The webs will be out of plumb at the end of steel erection, prior to deck placement. When the webs are plumb at the end of steel erection (prior to

deck placement), or are out of plumb in the wrong direction or beyond reasonable construction tolerances, remedial action should be considered. When the webs are in their correct, anticipated out of plumb position prior to deck placement, then when the deck is placed the webs will rotate (twist) to a plumb position (within reasonable construction tolerance), at least at the supports. This sequence of webs being out of plumb prior to deck placement and plumb after deck placement is normal and generally does not represent a problem.

SKEWED AND CURVED I-GIRDER BRIDGE FIT AND FRAMING ARRANGEMENTS

See Skewed and Curved Steel I-Girder Bridge Fit (Standalone Summary) and Skewed and Curved Steel I-Girder Bridge Fit (Full Document) for more information.

Recommended Fit Conditions for the following I-girder bridges:

- Straight bridges where one or more support lines are skewed more than 20 degrees from normal;
- Horizontally curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03;
- Horizontally curved bridges with or without skewed supports and with a maximum L/R greater than 0.03;

where:

L = actual span length bearing to bearing along the centerline of the bridge (feet)

R = girder radius at the centerline of the bridge (feet)

Fit Condition – deflected girder geometry associated with a targeted dead load condition for which the cross-frames are detailed to connect to the girders

Table 1.6.2.10-1 Recommended Fit Conditions for I-Girder Bridges

Loading Condition Fit	Construction Stage Fit	Description
No-Load Fit (NLF)	Fully-Cambered Fit	The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.
Steel Dead Load Fit (SDLF)	Erected Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under bridge steel dead load at the completion of the erection.
Total Dead Load Fit (TDLF)	Final Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.

Table 1.6.2.10-2 Recommended Fit Conditions for Straight I-Girder Bridges (including Curved I-Girder Bridges with L/R in all spans less than or equal to 0.03)

Square Bridges and Skewed Bridges up to 20 deg Skew

Span Length	Recommended	Acceptable	Avoid
Any span length	Any	Any	None

Skewed Bridges with Skew greater than 20 degree and $\boldsymbol{Is} \leq \mathbf{0.30}$ $+/-$

Span Length	Recommended	Acceptable	Avoid
Any span length	TDLF or SDLF	--	NLF

Skewed Bridges with Skew greater than 20 degree and $\boldsymbol{Is} > \mathbf{0.30}$ $+/-$

Span Length	Recommended	Acceptable	Avoid
Span lengths up to 200 feet +/-	SDLF	TDLF	NLF
Span lengths greater than 200 feet +/-	SDLF	--	TDLF & NLF

Table 1.6.2.10-3 Recommended Fit Conditions for Horizontally Curved I-Girder Bridges ($(L/R)MAX > 0.03$) on Radial or Skewed Supports

Span Length	Recommended	Acceptable	Avoid
$(L/R)MAX \geq 0.2$	NLF	SDLF	TDLF
All other cases	SDLF	NLF	TDLF

Tables notes:
Detail for a Steel Dead Load Fit, unless the maximum L/R is greater than or equal to 0.2.

When $(L/R)MAX \geq 0.2$, detail for No-Load Fit, unless the additive locked-in force effects from Steel Dead Load Fit detailing are considered.

CROSS-FRAME DESIGN

Design and detail cross-frame based on rational analysis for all stages of construction and service life according to *LRFD 6.7.4*. See G13.1-2019, Guidelines for Steel Girder Bridge Analysis Section 4 and Appendix B on selecting the appropriate level of analysis. Rational analysis of cross-frames depends on accurately analyzing the structure and determining the appropriate force effects.

- When a bridge is designed using line girder analysis methods, evaluate the cross-frames for minimum stability bracing strength and stiffness requirements as well as wind loads and construction loads. Per *LRFD 6.6.1.3.1*, in the absence of better information, design the welded or bolted connection to resist a 20.0-kip lateral load for straight, nonskewed bridges. Per *LRFD C6.10.10.1.2*, in lieu of a refined analysis, cross-frame or diaphragm force for an exterior girder may be taken as 25.0 kips.
- Where cross frame fatigue forces control the design, live load force effects in cross-frames and diaphragms should be calculated by 2D grid or 3D analysis model, careful consideration for modeling of cross-frames and the distribution of loads among the girders is crucial.
    - *LRFD C6.6.1.2.1* – Calculate fatigue stress using only a single transverse lane position. Determine maximum fatigue stresses in cross frames with the truck confined to one critical transverse position per each longitudinal position throughout the length of the bridge. Standard grid design software is generally not considering this. This requires isolating loading from individual lanes in the analysis. To satisfy this requirement, an influence-surface grid of 5 feet (longitudinal) by 4 feet (transverse) has been shown to adequately capture the maximum force effects.
    - For 2D models using equivalent beam, use Timoshenko beam approach, see G13.1-2019, Guidelines for Steel Girder Bridge Analysis *Section 3.11.3*. The Timoshenko beam approach provides the most realistic estimate of the cross-frame stiffness because it considers both flexural and shear deformations. Include the transverse stiffness of the deck.
    - Using a single adjustment factor of 0.65 applied to the Fatigue I load factor of 1.75 and the Fatigue II load factor of 0.8.
    - Apply the stiffness reduction factor in the model (apply to AE). Independent stiffness modification factors are proposed for the construction stages (R_{con} = 0.65) and for in-service conditions (R_{ser} = 0.75).

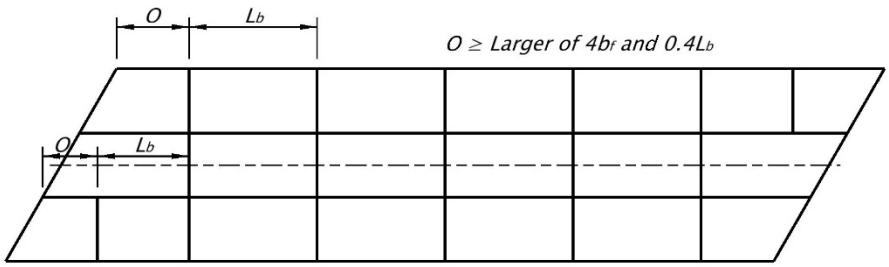
See the following references for additional guidance:

- Proposed Modification to AASHTO Cross-Frame Analysis and Design
    - Appendix B: Cross-Frame Design Example (Straight Bridge)
    - Appendix C: Cross-Frame Design Example (Curved Bridge)
- *G12.1-2020, Guidelines to Design for Constructability Section 2.2.6*
- *FHWA Steel Bridge Design Handbook: Volume 13 – Bracing System Design*

For skewed (greater than 20 degrees) and curved I-girder bridges:

- See *LRFD C6.7.4.2* for discussion about beneficial framing arrangements in skewed and curved I-girder bridges to alleviate detrimental transverse stiffness effects.
- It is recommended to offset the first intermediate cross-frame placed normal to the girders adjacent to a skewed support as shown in *Figure 1.6.2.10-1*.

Figure 1.6.2.10-1 Cross-Frame Layout for Bridge with Skewed Support



- Framing of a normal intermediate cross-frame into or near a bearing location along a skewed support line is strongly discouraged unless the cross-frame diagonals are omitted.
- At skewed interior bents and end bents, place cross-frames along the skewed bearing line, and locate intermediate cross-frames greater than or equal to the recommended minimum offset from the bearing lines.
- For curved I-girder bridges, provide contiguous intermediate cross-frame lines within the span in combination with the recommended offset at skewed bearing lines.

MEMBER SELECTION AND DETAILING

Select the most economical cross-frame members which meet the strength and detailing requirements of the design. Minimize the number of different types or sizes of members used in a given design. Consider the following for cross-frame member selection:

- Equal leg angles are often more cost-effective than unequal leg angles. Do not use back-to-back angles as cross-frame members. Common angle sizes for diaphragms range from L3 x 3 x 5/16 to L6 x 6 x 5/8.
- Only use WT sections when design forces exceed the capacity of angle sections. Do not use WT shapes in small quantities.
- When channel sections are used, C shapes are preferable to MC shapes. A bent-plate diaphragm, in the shape of a channel, is a possible option to provide to the fabricator for diaphragms on shallow plate girders (4-foot-deep web or less) or rolled beams.
- Design and detail cross-frames such that they can be erected as a single unit. Fabricators and erectors discourage diaphragms that require erection in separate pieces.
- Design cross frames as deep as practical so that the diagonals of the cross frame have large enough angles to prevent the gusset-type plates at the ends of the cross frame from becoming too large.

In choosing between intermediate cross-frames of **K** or **X** form, use the **X** form when the ratio of the beam spacing to the frame depth is less than two. Use the **K** form when the ratio is greater than two. Consider a solid plate diaphragm when the depth of the frame approaches 3 feet or less.

Consider maintenance and inspection requirements in the cross-frame design. Provide adequate clearance for sandblasting and painting. For cross-frame weld detailing, avoid weld callouts that leave the top edge open, as this can trap moisture and lead to long-term corrosion issues. Avoid inaccessible areas. It may be necessary to provide for maintenance walkways or utilities through the cross-frames. See *BDM 1.15.5* for accessibility requirements and *BDM 1.12.6* for utility requirements.

Rigidly connect cross-frames to the top and bottom flanges to prevent web distortions and cracking. Weld stiffeners to compression and tension flanges as shown on *Figures 1.6.2.10-2 and 1.6.2.10-3*. Stop ends of welds about 1/4 inch away from the edge (snipe, clip, etc.) to avoid a poor quality weld termination.

Where two adjacent plate girders have significant differential deflection, such as the first row of cross-frame from the end bents, do not use the **K** or **X** type of cross-frames. Use details shown on *Figure 1.6.2.18-2*. Check fatigue requirements of all welded connections.

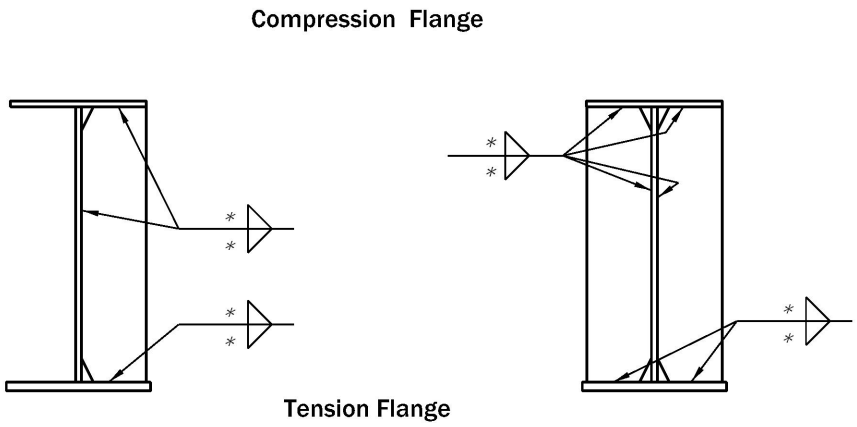
Provide intermediate cross-frames between the box girders and steel tub girders.

CONNECTION PLATES FOR BRACING MEMBERS

Cope diaphragm connection plates, which are welded to both the web and flange of a plate girder, a minimum of 1 ½ inches to prevent intersection of the two welds. Avoid lateral connection plates for lateral bracing which will be connected to the web of the plate girder or box girders. Bolt lateral connection plates to the flange of the steel girder. Cope lateral connection plates to be clear of any transverse web stiffener or diaphragm connection plate.

It is desirable to have all cross-frame member center of gravities intersecting at a common point (identified as work points, WP on design drawings). Locate work points either at the centerline of the girder web or at the corner bolt within the connection. It is difficult to develop details such that the member force vectors intersect the bolt group center of gravity. Therefore, design for the eccentric loads in the connection.

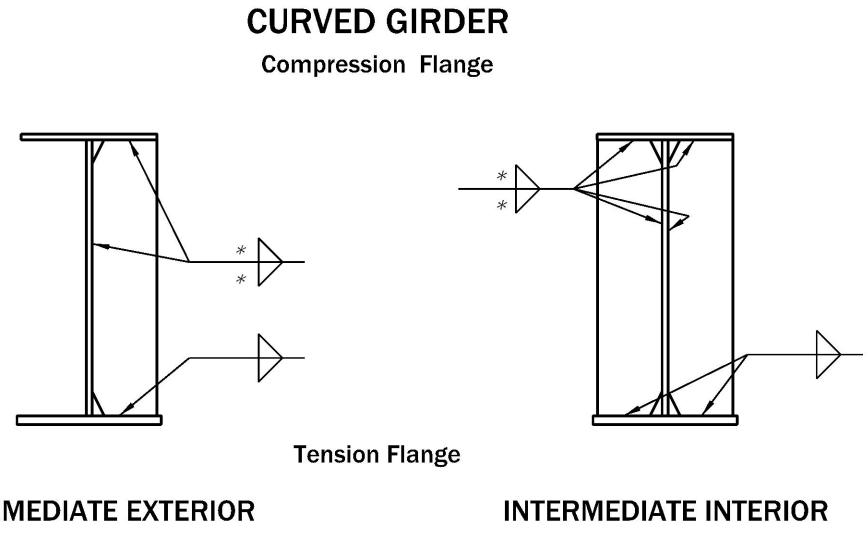
Figure 1.6.2.10-2 Transverse Connection Plates



∗ Size fillet welds in accordance with AASHTO LRFD minimum welds sizes shall

Seal all weld terminations and unwelded connections of crossframes and stiffeners with structural steel caulking from QPL, typical.

Figure 1.6.2.10-3 Transverse Connection Plates on Curved Girders



Compression and Tension flanges reverse near interior bent of continuous girder.

1.6.2.11 Cross Frames at Bents

Design cross-frames at bent for seismic loading obtained from a linear elastic seismic analysis or the force V_{po}, whichever is less for bridges with SDC B and C. For bridges with SDC D, use V_{po}, associated with the overstrength moment, M_{po}.

Diaphragms or cross-frames are required along skewed interior bents and end bents. See *BDM 1.6.2.10* for more information.

C1.6.2.11

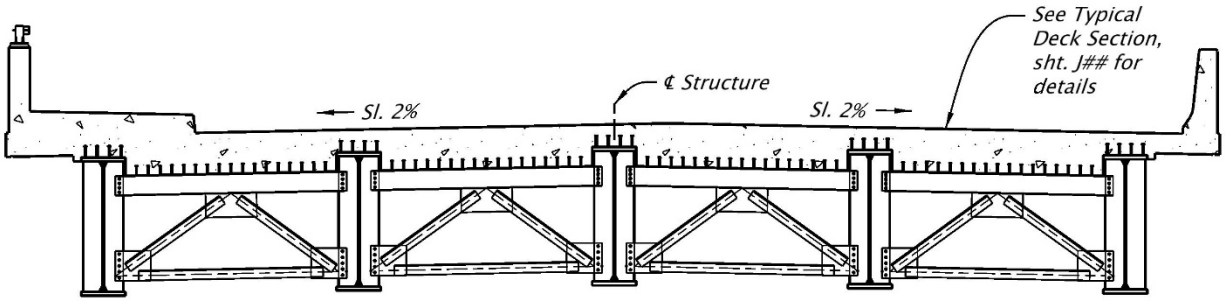
The assumed load path for the flow of forces is accommodated through deck slab to-girder interfaces to main beams or girders and cross-frames, and to all components of the bearing assembly to substructure (see *LRFD 4.6.2.8*).

The shear studs at the ends are critical in transferring the transverse seismic forces from the deck slab into the steel superstructure and down to substructure. A top chord connecting the deck slab to the end cross frames was found to be effective in transferring these forces and resist rotation at the top of the girders in the absence of sufficient studs on top of the girders.

Cross-frames at bents are more critical to transfer seismic forces from the superstructure to the substructure. One solution is to use detail *Figure 1.6.2.11-1* with welded studs added to the top flange of top chord members to provide the lateral resistance and transfer seismic loads from the concrete deck to substructure.

When a joint system is required for a cross-frame at end bents, it may be necessary to use details similar to cross-frames at continuous beam interior bents.

Figure 1.6.2.11-1 Cross-Frame Details at Bent



1.6.2.12 Composite Action and Flange Shear Connectors

Provide shear connectors in all portions of continuous spans, positive or negative moment.

For deflection and moment calculations include longitudinal reinforcing steel in the composite section properties of the girder in the negative moment areas.

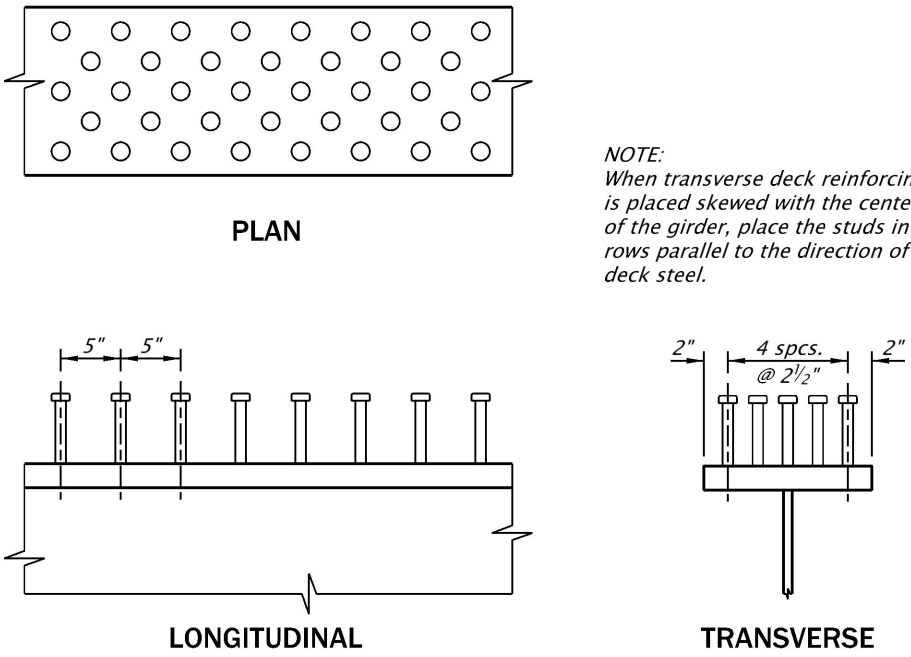
Extend shear connectors at least one inch above the mid depth of the deck for composite action. No less than 2.0 in clear cover over the tops of the shear connector.

C1.6.2.12

Previous practice was to not use concrete reinforcement to increase the moment capacity of composite girders in the negative moment areas.

Generally, the deck build up on steel girders is constant except for bridges with variable cross-slopes (super elevation) along the bridge. However the top flange plate thicknesses may vary. Consider the effect of top flange thickness variation and bridge deck super elevations when checking the shop drawings or specifying the shear connector's length. The advantages of longer shear connectors are in distributing load to larger area of the bottom mat reinforcing steel when a girder fails in fatigue. The concrete deck will distribute a portion of the unsupported load of the failed girder to adjacent girder(s).

Figure 1.6.2.12-1 Shear Connector Layout



1.6.2.13 Beam Camber

C1.6.2.13

Steel beams are cambered to compensate for dead load, shrinkage deflections and gradelines. The final position of the bottom flange is either flat or follows the grade, except in a sag vertical curve. Do not place a final negative camber in a beam. Profile grades can be incorporated into the camber by either added camber in the beam or by varying the deck flange build-ups along the beam. Sag vertical curves always require flange build-ups.

Consider the superelevation of the deck in the design of minimum flange build-ups.

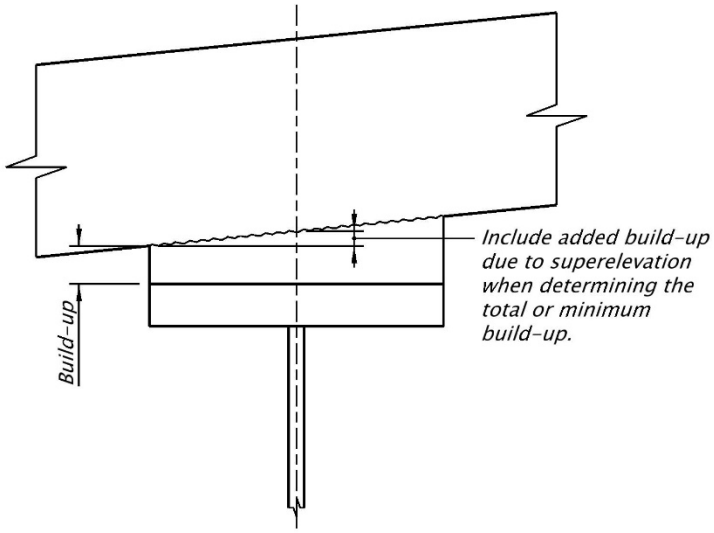
Slope adjustment or build-up for straight girders on curved roadways must also be

considered. Deck grades are based on the roadway centerline and straight girders are offset at midspan from the centerline. As a result, the adjustment is the superelevation times the midspan offset. Additional beam camber at midspan or additional build-up at the ends will be required. See *Figure 1.6.2.13-1*.

In addition to girder deflections, show girder rotations at bearing stiffeners.

Showing girder rotations will allow shop plan detailers to compensate for the rotations so that bearing stiffeners will be vertical in their final position.

Figure 1.6.2.13-1 Superelevation Deck Build-Up



Sketches of camber options for simple spans are shown in *Figures 1.6.2.13-2* through *1.6.2.13-4*.

Figure 1.6.2.13-2 Case 1: Crest Vertical Curve with Beam Grade Camber

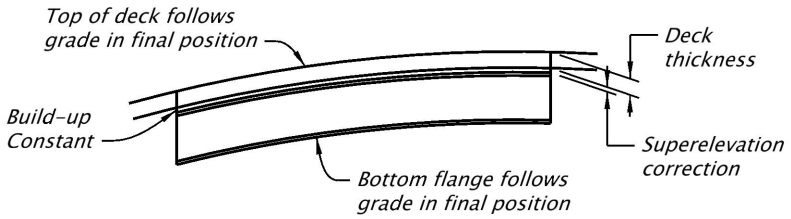


Figure 1.6.2.13-3 Case 2: Crest Vertical Curve with Build-Up for Grade Camber

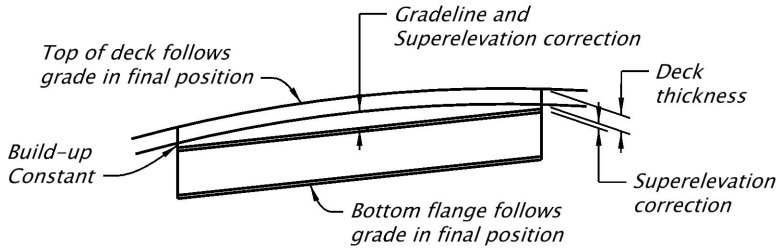
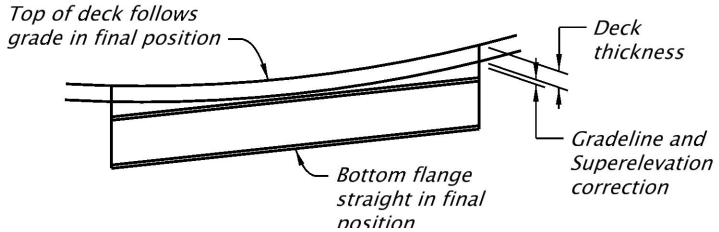


Figure 1.6.2.13-4 Case 3: Sag Vertical Curve with Build-Up for Grade Camber



1.6.2.13.1 Shrinkage Camber

Bridge deck shrinkage has a varying degree of effect on superstructure deflections. Use some judgment in evaluating this effect on camber.

A shrinkage strain of 0.0002 applied to the long-term composite section using 3n may be used to estimate the long-term effects of shrinkage using an approximate method.

For structures requiring close tolerances on girder cambers, perform refined analysis of shrinkage effects using structural analysis software such as MIDAS Civil.

C1.6.2.13.1

1.6.2.13.2 Camber Diagrams

Show the data in *Table 1.6.2.13.2-1* for steel beam camber on the contract drawings.

C1.6.2.13.2

Bridge deck shrinkage should be the smallest portion of the total camber. It has greater influence on shallower girder sections, e.g. rolled beams. Simple spans will see more effect than continuous spans.

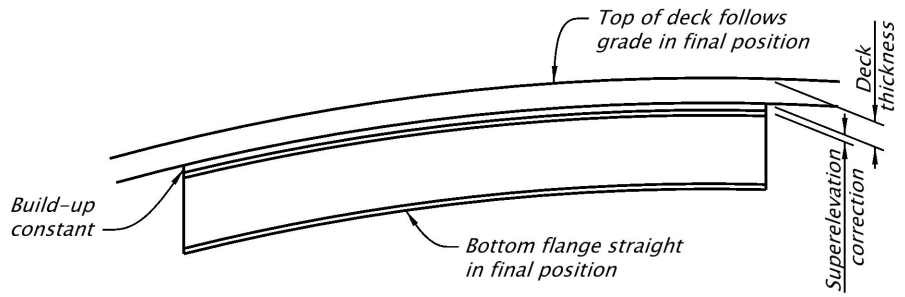
Tests have indicated that the unit shrinkage of the slab in composite beams (i.e., the shrinkage strain adjusted for long-term relaxation effects) may be taken equal to 0.0002.

Table 1.6.2.13.2-1 Steel Beam Camber Template

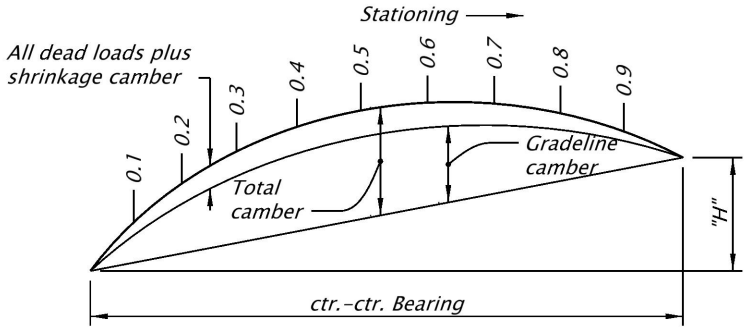
Camber Item	Value
Grade line camber	
Dead load camber	
Superimposed Dead load camber	
Shrinkage camber	
Total Camber	
Camber due to weight of steel beam and diaphragm	

Sketches of camber diagram examples are shown in *Figures 1.6.2.13.2-1 and 1.6.2.13.2-2.*

Figure 1.6.2.13.2-1 Camber Diagram Example 1



CREST VERTICAL CURVE WITH BUILD-UP FOR GRADE CAMBER



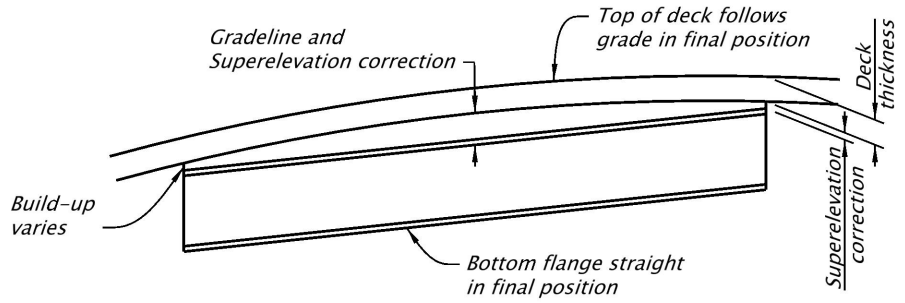
Positive "H" indicates ahead end higher than back end.

Approximately 50% of shrinkage camber will occur prior to curb and handrail pour.

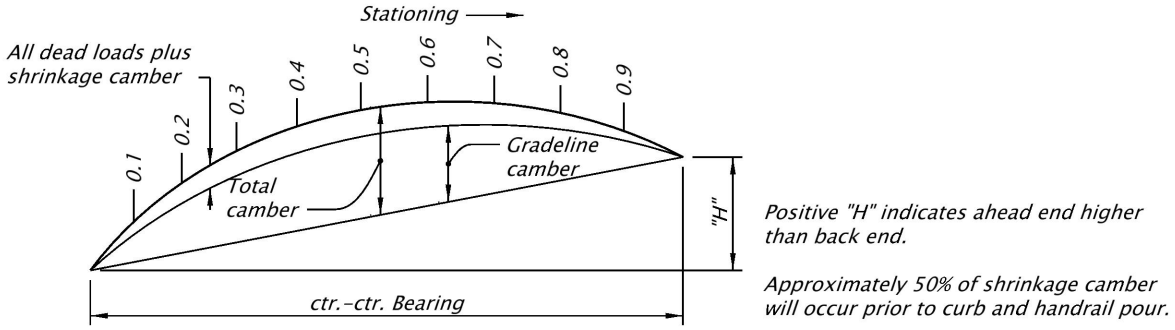
GIRDER CAMBER

SPAN	ITEM	CAMBER									H Inches
		0.1 Pt.	0.2 Pt.	0.3 Pt.	0.4 Pt.	0.5 Pt.	0.6 Pt.	0.7 Pt.	0.8 Pt.	0.9 Pt.	
1	Beam Dead Load + Diaphragms	5/8"	1"	1 3/8"	1 1/2"	1 3/8"	1 1/8"	3/4"	3/4"	1/16"	+16 3/8"
	Deck Dead Load + Form	1 3/8"	2 1/2"	3 3/16"	3 1/4"	3 3/16"	2 3/16"	1 3/8"	1 1/16"	1/4"	
	Sidewalk, Rail & WS Dead Load	3/8"	5/8"	1 3/16"	7/8"	1 3/16"	1 5/16"	5/8"	5/16"	1/16"	
	Shrinkage	5/8"	1"	1 1/4"	1 1/4"	1 1/8"	1"	3/4"	1/2"	1/4"	
	Sub Total	3"	5 1/8"	6 5/8"	6 7/8"	6 3/8"	5 1/8"	3 1/2"	1 7/8"	5/8"	
	Gradeline	3/8"	3/4"	1"	1 1/8"	1 1/4"	1 1/8"	1"	3/4"	3/8"	
	Total	3 3/8"	5 7/8"	7 5/8"	8"	7 5/8"	6 1/4"	4 1/2"	2 5/8"	1"	
2	Beam Dead Load + Diaphragms	1/16"	1/4"	5/8"	7/8"	1 1/16"	1/16"	5/8"	5/16"	1/16"	+7 5/8"
	Deck Dead Load + Form	1/8"	3/4"	1 5/8"	2 7/16"	3 1 1/16"	3 1/4"	1 5/8"	1 3/16"	1/8"	
	Sidewalk, Rail & WS Dead Load	1/16"	1/4"	1/2"	1 3/16"	7/8"	1 3/16"	1/2"	1/4"	1/16"	
	Shrinkage	-1/4"	-3/8"	-3/8"	-3/8"	-3/8"	-1/4"	-1/8"	-1/8"	0	
	Sub Total	0	7/8"	2 3/8"	3 3/4"	4 1/4"	3 7/8"	2 3/4"	1 1/4"	1/4"	
	Gradeline	5/8"	1 1/4"	1 5/8"	1 3/4"	1 7/8"	1 3/4"	1 5/8"	1 1/4"	5/8"	
	Total	3/8"	2 1/8"	4"	5 1/2"	5 1/8"	5 5/8"	4 3/8"	2 1/2"	7/8"	

Figure 1.6.2.13.2-2 Camber Diagram Example 2



CREST VERTICAL CURVE WITH BUILD-UP FOR GRADE CAMBER



GIRDER CAMBER

SPAN	ITEM	CAMBER								H Inches
		0.1 Pt.	0.2 Pt.	0.3 Pt.	0.4 Pt.	0.5 Pt.	0.6 Pt.	0.7 Pt.	0.8 Pt.	0.9 Pt.	
1	Beam Dead Load + Diaphragms	5/8"	1"	1 3/8"	1 1/2"	1 3/8"	1 1/8"	3/4"	3/4"	1/16"	+16 3/8"
	Deck Dead Load + Form	1 3/8"	2 1/2"	3 3/16"	3 1/4"	3 3/16"	2 3/16"	1 3/8"	1 1/16"	1/4"	
	Sidewalk, Rail & WS	3/8"	5/8"	13/16"	7/8"	1 3/16"	15/16"	5/8"	5/16"	1/16"	
	Shrinkage	5/8"	1"	1 1/4"	1 1/4"	1 1/8"	1"	3/4"	1/2"	1/4"	
	Sub Total	3"	5 1/8"	6 5/8"	6 7/8"	6 3/8"	5 1/8"	3 1/2"	1 7/8"	5/8"	
	Gradeline	None	None	None	None	None	None	None	None	None	
	Total	3"	5 1/8"	6 5/8"	6 7/8"	6 3/8"	5 1/8"	3 1/2"	1 7/8"	5/8"	
2	Beam Dead Load + Diaphragms	1/16"	1/4"	5/8"	7/8"	1 1/16"	1/16"	5/8"	5/16"	1/16"	+7 3/8"
	Deck Dead Load + Form	1/8"	3/4"	1 5/8"	2 7/16"	3 1 1/16"	3 1/4"	1 5/8"	1 3/16"	1/8"	
	Sidewalk, Rail & WS	1/16"	1/4"	1/2"	13/16"	7/8"	13/16"	1/2"	1/4"	1/16"	
	Shrinkage	-1/4"	-3/8"	-3/8"	-3/8"	-3/8"	-1/4"	-1/8"	-1/8"	0	
	Sub Total	0	7/8"	2 3/8"	3 3/4"	4 1/4"	3 7/8"	2 3/4"	1 1/4"	1/4"	
	Gradeline	None	None	None	None	None	None	None	None	None	
	Total	0	7/8"	2 3/8"	3 3/4"	4 1/4"	3 7/8"	2 3/4"	1 1/4"	1/4"	

1.6.2.14 Deck Pour Sequence

See *BDM 1.9.4.3*.

1.6.2.15 End Bents Detailing

It is desirable to eliminate end bent joints or make construction jointless to protect the girder steel from leaking joints.

Use the extended deck detail or semi integral abutments similar to *Figures 1.6.2.15-1* or *1.6.2.15-2*.

Use the integral abutments when geometry and span length allow.
Show a coated or painted section at the ends of plate girders. On jointless bridges coat or paint the end of the girder with three coat paint system for a length of 1 foot outside the concrete interface and 4 inches inside the concrete interface. See *Figure 1.6.2.15-3*.

Where joints cannot be avoided, show a coating or painting detail at the end of plate girders. Coat or paint the end of the girder for a length at least 1.5 times the depth of the girder and all attachments within this limit. See *Figure 1.6.2.15-4*. Match the paint color developed by the weathering steel patina 2.5 years after completion of the bridge construction. See *Figure 1.6.2.15-4*.

C1.6.2.15

See *BDM 1.11.2.5* and *1.11.2.6* for additional guidance.

Figure 1.6.2.15-1 Extended Deck Details at End Bents

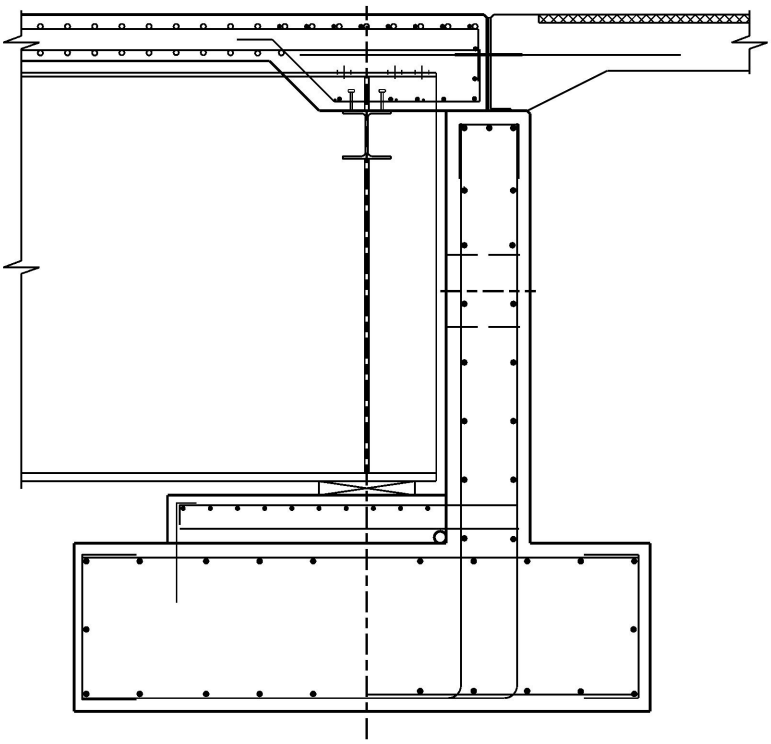


Figure 1.6.2.15-2 Semi Integral Bent Detail

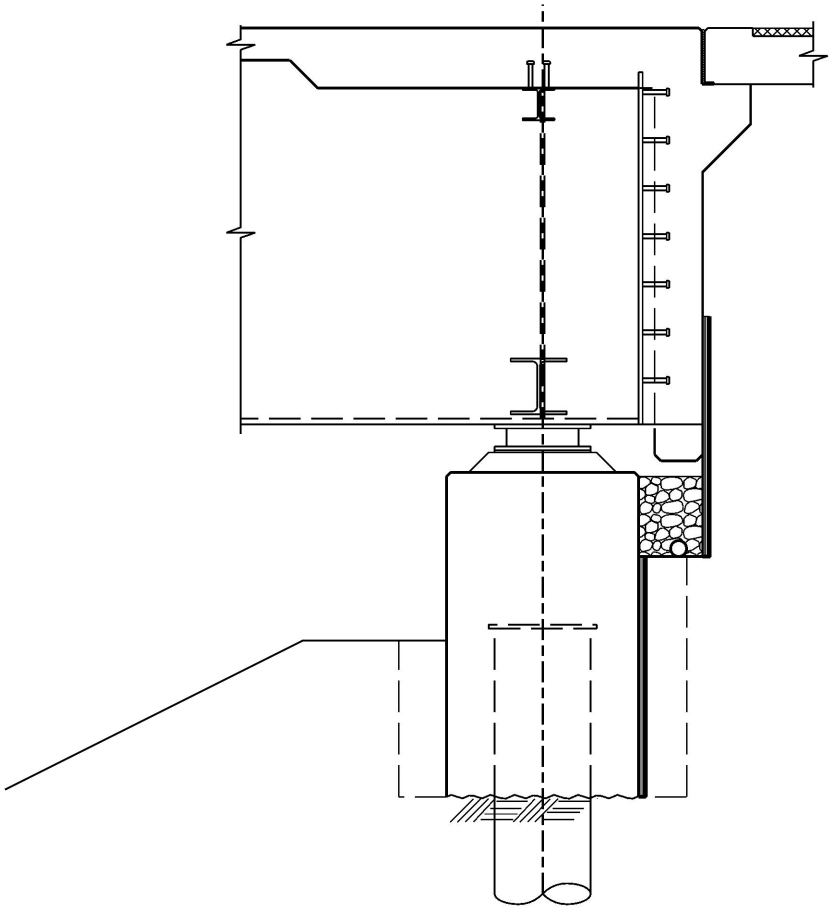


Figure 1.6.5.15-3 Integral Bent Detail

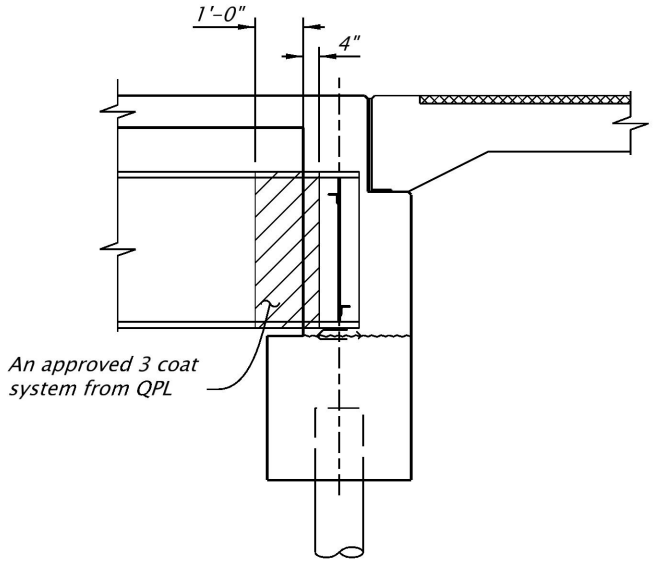
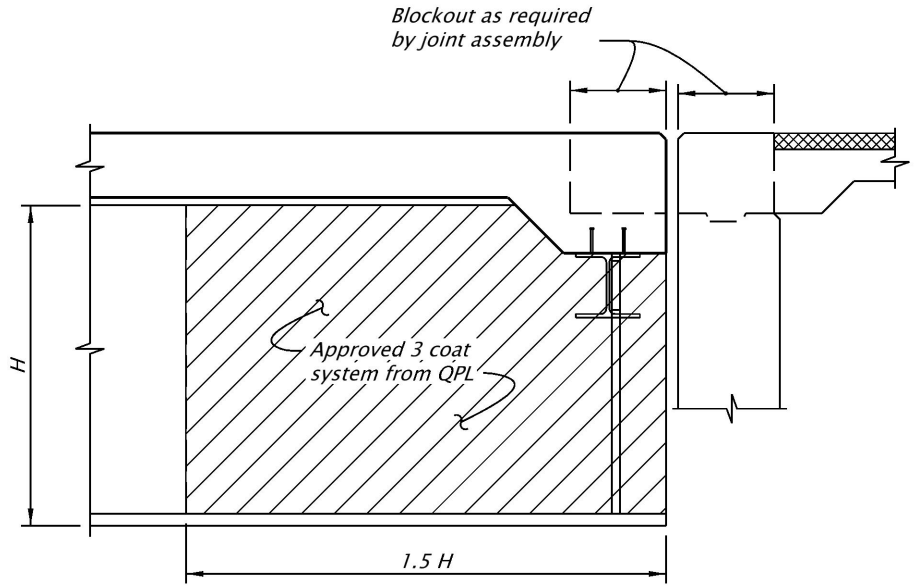


Figure 1.6.2.15-4 Girder End Coating Limit



1.6.2.16 Expansion Joint Blockouts

Show a blockout detail (see *Figure 1.14.2.4-1*) on the plans to allow the expansion joint assembly to be placed after the final deck pour.

C1.6.2.16

Providing a blockout makes the adjacent deck pour easier, provides smoother deck transition to joint, and allows the majority of the superstructure shrinkage to occur prior to joint assembly placement.

1.6.2.17 Bearings and Anchor Rods

Due to high cost, avoid using built up steel bearings, pot bearings, and spherical bearings.

Use elastomeric bearings wherever possible.

Use circular elastomeric bearings on curved steel girders.

See also *BDM 1.14.1* and G9.1-2022, Steel Bridge Bearing Guidelines for additional guidance. See *Drawing E2.3* for preferred bearing anchor rods connection detail.

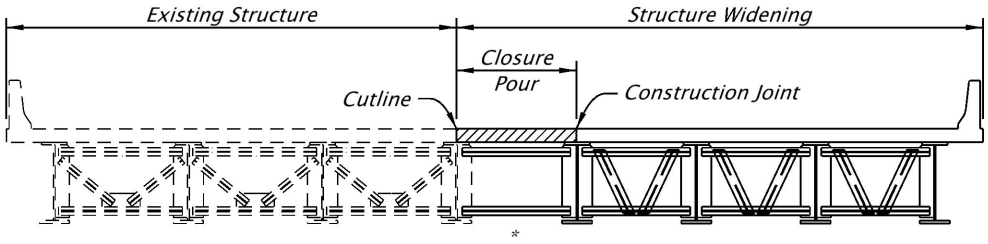
C1.6.2.17

1.6.2.18 Structure Widening

Avoid transferring dead loads from the widening to existing beams. Specify diaphragms between the widening and existing girders with a closure pour at the deck level as shown in *Figure 1.6.2.18-1 and Figure 1.6.2.18-2*. The diaphragms are for lateral load resistance.

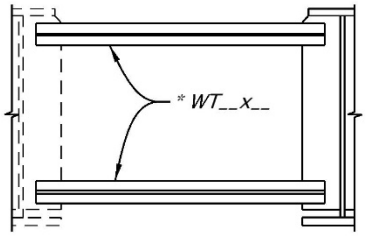
C1.6.2.18

Figure 1.6.2.18-1 Widening Closure Pour Detail



NOTE:
Place closure pour a minimum of 3 days after adjacent deck widening concrete placement.

Figure 1.6.2.18-2 Diaphragm Connection Detail



** Install temporary 4- __ dia. H.S. bolts (A 325) (snug tight) in each end of each horizontal WT__x__ and omit the diagonal WT__x__'s until the entire deck and closure pours have been completed.*

1.6.3 Welding

1.6.3.1 Welding, Design Considerations

C1.6.3.1

The following four categories loosely describe the most common types of welding needed for design work in roadway and bridge sections.

1. **INCIDENTAL STRUCTURES** (*AWS D1.1*): Welding under this category consists of light structural joining such as handrails, fencing, and sheet metal products.

In general the weld is not required to fully develop the strength of the joining parts. Visual inspection of the final product is all that is expected.

2. **GENERAL STRUCTURAL WELDING** (*AWS D1.1*): Welding under this category consists of partially or fully developing the strength of the joining parts such as pile splices and attachments, guard rails, signing and lighting support, expansion joints (unless prefabricated by an approved supplier), seismic restraint fixtures and bearings (unless directly welding to main structural elements of a bridge).

In general the weld will develop the ultimate strength of the joining parts but is not expected to provide maximum fatigue life unless nondestructive testing is specified for acceptance.

3. **STRUCTURAL WELDING OF REINFORCING STEEL** (*AWS D1.4*): Welding under this category consists of splicing and anchoring either new construction or existing reinforcing steel in concrete columns and girders. Note that *LRFD 9.7.2.5* does not allow welded splices of bridge deck reinforcement due to fatigue considerations. The particular weld joint design usually consists of either flare-bevel welds or butt joints with back up bars see *Figure 1.11.3.6-2* for examples. Ensure the contractor provides a CWI during field welding.

In general it is desired to develop the full strength of the reinforcing steel to be joined. Almost any type of reinforcing steel can be successfully welded provided the chemistry of the steel is known (from either mill certifications or field testing) and an appropriate welding procedure is developed and followed. Unknown steels need to have a sample extracted (approximately 2 to 4 grams) and testing for chemistry.

The welding procedure is developed from *AWS D1.4* using the carbon equivalent method. This type of welding is almost always performed in the field and thus needs

4. **BRIDGE WELDING** (*AWS D1.5*): Welding under this category consists of fabricating or modifying any main load path carrying members of a bridge that have some or all portions that experience tensile stresses under normal loads. This includes girders, floor beams, stringers, trusses, and hanger assemblies. The member does not necessarily have to be fracture critical.

Certification of Steel Fabricators: *SP 00560.30* requires the American Institute of Steel Construction (AISC) Certified Bridge Fabricator - Intermediate (IBR) certification. For fracture critical Structures, the fabricator is also required to have an AISC Fracture Critical Endorsement (FC). All fabricators of earthquake restraints are required to have either a current AISC IBR certification or a Certified Bridge Fabricator - Simple (SBR) certification.

to be monitored by a certified welding inspector (CWI). Acceptance is usually based on visual examination, but other methods can be used when concerned about fatigue.

In general the welding is expected to develop both full ultimate strength of the joining parts and maximum fatigue performance. Joint toughness and nondestructive testing are typically required for acceptance.

1.6.3.1.1 Typical Pathways for Successful Welding in Your Design

INCIDENTAL WELDING:

- Specify the welds needed on the drawings (type, size, and length).
- In general, welding procedure specifications and welder certification are not required to be submitted.
- Quality assurance will be based on general appearance (visual testing) only. When you want a trained

C 1.6.3.1.1

person to inspect the workmanship send a copy of the plans to the ODOT Portland Materials Inspection Crew. The same inspectors will also check for quality of painting and galvanizing. When the workmanship is poor then the parts can be rejected.

GENERAL STRUCTURAL WELDING:

- Specify the welds needed on the drawings (type, size, and length).

Even though the *Standard Specifications* invoke *AWS D1.1* welding code for all incidentals structures, it is recommended that the following statement be included on the drawings (usually the plan and elevations):

  - All welding shall conform to the *AWS D1.1 Structural Welding Code*.

- Generally welding procedure specifications (WPS) and welder certification are required to be submitted and approved. Any shop drawings that have welding shown are not legally approved until the WPS are approved under *AWS D1.1*.

  Quality assurance is typically based on visual inspection by a certified welding inspector (CWI) and may also incorporate nondestructive testing such as ultrasonic (UT), radiographic (RT), and magnetic particle (MT) testing when specified on the design drawings. Various stages of the fabrication process may also be monitored if necessary. It is recommended that a copy of all plans and specifications that require this category of welding be sent to the ODOT Portland Materials Inspection Crew.

REINFORCING STEEL WELDING:

- Specify the welds needed on the drawings (type, size, and length).
- In the general notes, put the following:
    - All reinforcing steel welding shall conform to *AWS D1.4* Structural Reinforcing Steel.
- When the steel is not *ASTM A706* extract a field chemistry sample and analyze for the carbon equivalent. Base the welding procedure on this information. When the steel is *A706* the *D1.4* welding code has recommended heat inputs.
- Inform the ODOT Portland Materials Office of the work and have a CWI review the welding procedure, welder certification and observe the welding.

BRIDGE WELDING:

- Specify the welds needed on the drawings (type, size, and length).

Calling out the specific weld ID number (i.e. TC-U4a is an example) is preferable but not required. Typically this category of welding requires a significant Quality Assurance (QA) effort so please include this in your construction cost estimate.

- Even though the *Standard Specifications* invoke *AWS D1.5* welding code for all bridge welding it is recommended that the following statement be included on the drawings (usually the plan and elevations):

    All welding shall conform to the *AWS D1.5 Bridge Welding Code*.

- Welding procedure specifications (WPS) and welder certification are required to be submitted and approved by the Engineer of Record. Any shop drawings that have welding shown are not legally approved until the WPS are approved under *AWS D1.5*.

Quality assurance is based on a more complicated Owner/Fabricator relationship that involves frequent inspections during the entire fabrication and erection process. Most individuals involved have stringent requirements for their duties including certified welders, inspectors, fabricators, and testing personnel. Most welding in this category requires some form of nondestructive testing for acceptance. Theoretically all materials and processes are traceable with archived documentation. Send a copy of all plans and specifications that require this category of welding to the ODOT Portland Materials Inspection Crew.

1.6.3.2 Fillet Welds

When adequate structural performance from fillet welds in **T** and corner joints can be obtained, use fillet weld in preference to groove welds.

The minimum fillet weld size for prequalified joints is shown in *Table 1.6.3.2-1*.

C1.6.3.2

Fillet welds can be non-destructively inspected with greater certainty of result and at lower cost.

Table 1.6.3.2-1 Minimum Fillet Weld Size for Joints

Material Thickness of Thicker Part Joined (T) (inch)	Minimum Size* of Fillet Weld (inch)
To ¾ inclusive	¼**
Over ¾	5/16**

Table notes:
* Except that the weld size need not exceed the thickness of the thinner part joined. For this exception, take particular care to provide sufficient preheat to ensure weld soundness.

** Welds of this size must be made in a single pass.

Size fillet welds in accordance with *AASHTO LRFD Design Specifications*.

1.6.3.2.1 Web to flange connection

Use the minimum fillet weld necessary to join the flange to the web.

Calculate shear stress capacity of fillet welds (equal legs) using equation *LRFD 6.13.3.2.2b*.

C1.6.3.2.1

The weld size will vary along the length of the girder depending on the size of the plates being joined.

1.6.3.3 Flange Welds

C1.6.3.3

The design tensile stress in butt welded joints may equal the allowable stress in the base metal.

Show flange butt weld splices as in *Figures 1.6.3.3-1 or 1.6.3.3-2*. Include this detail on all steel structure plans. Indicate the type of butt weld splice for each splice on the plans. This may be accomplished by:

Adding the word **tension** or **compression**, whichever is the case, to the tail of the weld symbol.
Indicating which flanges or which portions of the flanges are in compression (C) and which are in tension (T).

Figure 1.6.3.3-1 Unequal Thickness Flange Splice

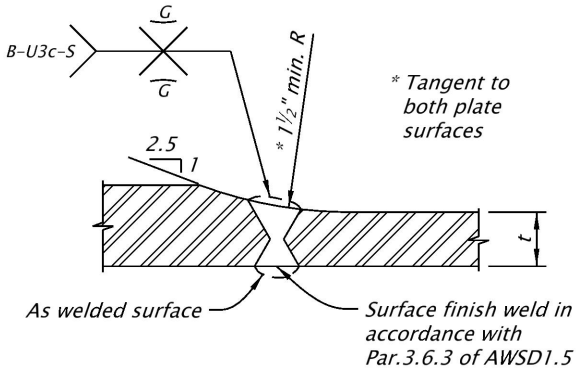
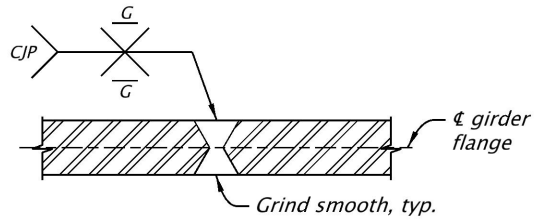


Figure 1.6.3.3-2 Equal Thickness Flange Splice



1.6.3.4 Welded Web Splices in Steel Bridge Girders

C1.6.3.4

Use complete joint penetration butt weld in web splices. Grind off 100 percent of the weldments reinforcing of all web splices. To facilitate NDE during fabrication, specify on the design drawings which portion of the girder webs are tension and compression. See *Figures 1.6.3.4-1* and *1.6.3.4-2*.

Figure 1.6.3.4-1 Unequal Thickness Web Splice

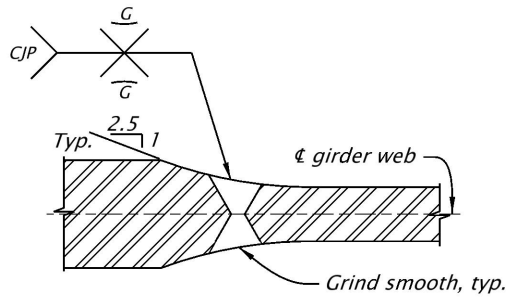
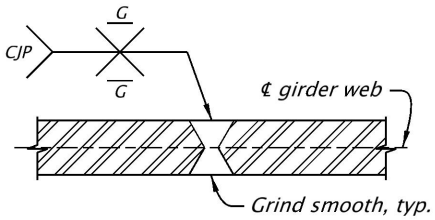


Figure 1.6.3.4-2 Equal Thickness Web Splice



1.6.4 Galvanizing and Painting

1.6.4.1 Processes

Hot-dip galvanizing has proven to provide better long term corrosion protection and should be required for all galvanized items.

C1.6.4.1

Galvanizing is a process of applying a sacrificial metal (zinc) to a base metal. The zinc will corrode, or sacrifice itself, to protect the base metal. Hot-dip galvanizing involves cleaning the items with a combination of caustic and acidic solutions and the dipping them into a tank of molten zinc for a specified period of time. After removal, small items are spun to remove excess zinc.

Mechanical galvanizing involves cleaning as mentioned above and then loading the items in a multi-sided rotating barrel. The barrel contains a mixture of various sized beads and water. As the barrel turns, chemicals and powdered zinc are added. The collision between the items, the glass beads and zinc causes the zinc to cold weld to the part. Powdered zinc is added until the required thickness is obtained.

1.6.4.2 Detailing

To ensure proper hot-dip galvanizing, provide venting and drain holes in details.

Provide a minimum vent opening of 25 to 30 percent of the cross-sectional area of a tubular section when full open venting is not possible.

Provide drains holes at closed corners or clip all corners at gusset plates to allow complete drainage.

C1.6.4.2

These insure proper circulation and removal of cleaning solutions and the molten zinc. They may also prevent potential explosions during dipping caused by trapped air.

1.6.4.3 Silicon Control

The silicon content of the steel must be held within either of the range of 0 to 0.06 percent, or 0.153 to 0.25 percent to obtain and maintain a pleasing appearance.

Call out all members that will have visual impact on the drawings with Galvanize - Control Silicon.

For economic reasons, silicon need not be controlled in galvanized structural members that are hidden from motorist view or are too small to have significant visual impact. Generally, these members that are too small to have significant visual impact are steel shapes whose least dimension does not exceed 3 inches.
The general notes on each contract drawing that includes members are to be called out as Galvanize-Control Silicon.

C1.6.4.3

The silicon content of the steel influences the corrosion resistance and strength of the galvanized coating and the thickness of the zinc layer.

Examples of these members are the chords, posts and diagonals of sign bridges; arms and shafts of luminaire, sign and signal support structures; steel traffic rail posts and railing members and pedestrian railings.

An example of an exception is pedestrian rail members that should have silicon control. Examples of hidden members and others which for practical reasons do not require silicon control are base plates and guard rail connection plates, flex-beam rails and their posts and single-post, breakaway sign posts.

The specification for control of silicon in steels to be galvanized is included in the *Standard Specifications for Construction*.

1.6.4.4 Painting or Coating of New or Existing Metal

Coating of metal structures is discouraged in most circumstances due to maintenance costs of recoating. Weathering steel (see *BDM 1.6.2.2*) and galvanizing are preferred options.

Sacrificial thickness is another option that may be appropriate in some circumstances.

C1.6.4.4

Coating work consists of preparing and coating new metal structures and features in the shop and in the field, and preparing and coating existing metal structures. This includes all:

- Interior and exterior steel surfaces.
- Steel railings, bridge bearings, and bridge expansion joint assemblies.
- Other miscellaneous steel.

Perform a life cycle cost comparison when considering sacrificial thickness vs coated steel. Include the cost comparison in the TS&L narrative.

Coating steel may be appropriate in the following situations:

- Marine areas (as defined in *BDM 1.2.4.1*).
- When use of weathering steel is improper per *BDM 1.6.2.2.*
- Structure is easily accessible and has minimally restricted lane closures (i.e. does not cross a roadway).

Specify coating of metal structures in conformance with *SP 00594* and the special provisions. Powder coating is discussed in *SP 00593*.

In the construction drawings, require inside surfaces of boxes or tubs (bottom flange, top flange, web and diaphragm) to be painted with a silver gray prime coat.

One of the main concerns in steel tubs or box girders in the State of Oregon is corrosion inside the girders.

Painting inside the tub (box) girders will prevent corrosion resulting from leakage thru the deck and condensation. Light color paint also increases illumination inside the tub (box) and eases detection of corrosion or cracks in steel members.

1.6.4.4.1 Design Features of Coated Steel

C1.6.4.4.1

Provide the following design features for bridges fabricated from coated structural steel:

1. Where structure access and lane closures are expected to be improbable, consider providing additional vertical clearance beyond the required minimum (per *BDM 2.14.4.2*) according to the following criteria. Allow for future ACP overlay when applicable.
   - For box girders: half of the box girder bottom flange width plus 1 foot, but not exceeding 3 feet.
   - For plate girders: 1.5 foot minimum, but not exceeding 3 feet.
   - All other situations: 1.5 feet.

This is to allow a minimal amount of access for work platforms and performing the work on the lowest members over traffic, without affecting freight movement.

Submit a design deviation when providing additional clearance beyond the required minimum.

Additional vertical clearance only needs to be provided at girders over traffic lanes or where low water clearance is expected.

When only the minimum required clearance is provided (no additional), it will be more difficult and costly to recoat the bridge in the future. Reduced clearances typically require lane closures, detours, or night work to gain access to the work. In high traffic areas where lane closure is prohibited the efficiency of the work can be further degraded when allowable work shifts are too short.

Evaluate providing additional clearance with the project team based on the following criteria:

- Projected AADT – projected AADT high enough to impact future lane closures.
- Railroad project involvement – railroad involvement will restrict structure access.
- Freeway projects - either on or above.
- Urban locations – surroundings (buildings, structures, utilities, etc.) may confine and limit access.

Evaluate the lifecycle cost of increasing vertical clearances against future recoating maintenance.

- When evaluating increased project construction cost due to a raised profile, consider the following items that may be affected:
    - Roadway construction due to a grade profile change.
    - Environmental impact and mitigation.
    - Additional right-of-way needs.
    - Retaining wall(s) needs.

  - Seismic requirements (i.e. increased column heights, foundation stabilization).
  - Over-height warning system - for protecting workers on platforms over live traffic.

Evaluate the following items, with assistance from the Senior Cost Engineer and the Senior Structure Coatings Engineer, when considering lifecycle costs of future painting when only the minimum required clearance (no additional) is provided:

- Over-height warning system.
- Additional traffic control.
- Detours.
- Effect of night work.
- Mobilizing and demobilizing equipment each work shift; effect of work shifts that are not long enough for reasonable efficiency.
  - Increase in access and containment cost due to complexity and additional setups.
  - Increase in surface preparation cost due to reduced efficiency.
  - Increase in coating application cost due to reduced efficiency.
  - Increase in traffic control or detour costs due to reduced efficiency.
- Increased overhead costs due to reduced efficiency (longer calendar duration of work).
  - Delay costs borne by the public.

Document the various clearance alternatives, including cost comparisons (based on the items outlined above) and justifications in the TS&L narrative.

2. Provide a minimum of 3 feet horizontal separation between the front face of traffic rail and the nearest steel surface (e.g. tied arch and stacked deck structures).

This is to allow a minimal amount of access for scaffolding, worker and traffic protection shielding, and performing the work on the members closest to traffic, without affecting traffic or freight movement.

3. Provide stainless steel padeyes, stainless steel eyebolts or deck inserts located at appropriate intervals (approximately 15 by 15 foot to 20 by 20 foot grid typical) for support of future work platforms from upper structure members above roadway, main structure members below roadway, and the deck.

This is to allow surface preparation and coating of members without having to move hangers, beam clamps, chains, cables or chokers. These temporary attachment points require extra work and often receive poor surface preparation and coating.

List the allowable loading for the attachment points in the structural notes on the bridge plans. This requirement may be coordinated with the inspection requirements of *BDM 1.15.5*.

4. Include a temporary load representing the dead loads, live loads and wind loads acting on work platforms, scaffolding and containment needed to recoat the bridge. Add this Temporary Load to each applicable Load Combination as shown in *LRFD Table 3.4.1*, using a load factor of:

- 1.50 for Strength and Extreme Event limit states.
- 1.00 for Service I and Fatigue limit states.
- 1.25 for Service II limit state.

Design work platforms that access the structure with little need for scaffolding for a

When the member is too small for a worker to turn around in, but larger than 2 by 3 feet, provide access openings 18 by 30 inches minimum with semicircular ends, spaced from 42 to 60 inches on centers.

For members up to 2 by 3 feet, provide hand hole access 6 by 12 inches with semicircular ends, spaced 30 inches on center. For members large enough for workers to enter see *BDM 1.2.10* for additional accessibility guidance.

$DL + LL$ of 25 pounds per square foot of platform area. Design work platforms that require significant scaffolding to reach the structure for a $DL + LL$ of at least 50 pounds per square foot. See *BDM 1.3.4* for temporary loading.

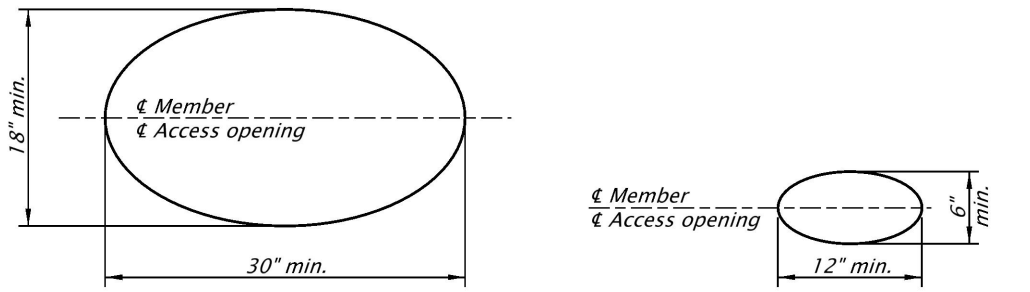
Assume work platforms and containment extend 5 feet beyond the sides of the structure and full length to 5 feet beyond the ends of the entire structure or each span.

List the permissible work platform sizes and loading, and the permissible containment sizes and wind speeds in the structural notes on the bridge plans. It may be necessary to separate the work platforms and containment into zones for structural reasons, in which case carefully size the zones to allow efficient work by the recoating contractor.

5. Do not create spaces where blind sides of members cannot be reached for surface preparation and coating work.

6. Closed members or areas which are too small for workers to enter to perform surface preparation and coating work are highly discouraged. See *Figure 1.6.4.4.1-1*.

Figure 1.6.4.4.1-1 Access Openings for Closed Members



MEMBERS LARGER THAN 2'x3'

MEMBERS UP TO 2'x3'

Be aware of an existing structure's condition prior to completing TS&L of a recoating project. Recoating of an existing bridge is very costly and requires a careful examination of the structure's condition.

Collect all necessary information for such projects. The Steel Bridge Recoating Checklist provides a list of required information and guidance on specifications. Complete the checklist and include in the TS&L Report.

Older structures are typically painted and have potential deficiencies that may need to be addressed during a recoating project.

1.6.5 Bolts and Connections

Design all high-strength bolted connections as slip-critical connections. Assume Class B faying surfaces where inorganic zinc primer is used. When steel will be given a full paint system in the shop, mask the primed faying surfaces to maintain the Class B surface.

Include additional costs for access, paint removal and recoating rivet or bolt replacements, when rivets or bolts are outside normal paint area limits.

1.6.5.1 High Strength Bolts

HIGH-STRENGTH BOLT USE GUIDELINES:

- *ASTM F3125* GR A325 – Heavy hex headed structural bolt for use in structural connections. Do not specify for anchor bolts.

C1.6.5.1

These may be hot-dip galvanized. Limit the maximum bolt diameter for girder bolted connection to 7/8 inch. Bolts with diameters greater than 7/8 inch are difficult to tighten. Checking the inspection torque in the field becomes problematic from an equipment standpoint when bolts with diameters greater than 7/8 inch are used.

• Use Type 3 bolts conforming to *ASTM F3125* when specifying weathering steel.	
• *ASTM A449* - Steel bolts and studs for general applications including anchor bolts. When using bolts of larger diameter, accounted for a reduction in strength as indicated in the *Table 1.9.5.2-1*. Do not use these as anchor bolts for seismic applications due to low CVN impact toughness.	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 A449* bolts are equivalent to GR A325 up to 1 inch diameter. These bolts may be hot-dip galvanized.
• *ASTM F3125* GR A490 & GR F2280 - Do not use *ASTM F3125* GR A490 bolts in bridge applications. When a design deviation is approved for use of these bolts, do not galvanize these bolts because of high susceptibility to hydrogen embrittlement. Instead of galvanizing, require two or three coats of approved zinc rich paint. Do not specify for anchor bolts.	Alloy steel headed structural bolt for use in structural connections.
• *ASTM F1554* grade 105 - Higher strength anchor bolts used for larger sizes (1½ to 4 inch). When used in seismic applications, such as bridge bearings that resist lateral loads, specify supplemental CVN requirement S4 with a test temperature of -20 degree Fahrenheit.	Lower grades may also be suitable for sign structure foundations. This specification should also be considered for seismic restrainer rods, and may be galvanized.

1.6.5.2 Properties of High-Strength Bolts

Table 1.6.5.2-1 Properties of High Strength Bolts

Material	Bolt Diameter (inch)	Tensile Strength (ksi)	Yield Strength (ksi)
ASTM F3125 GR A325 & GR F1852	½ – 1½	120	92
--	Over 1½	--	Not Available
ASTM A449	¼ – 1	120	92
--	1⅛ – 1½	105	81
--	1¾ – 3	90	58
--	Over 3	--	Not Available
ASTM F1554 GR 105	¼ – 3	125 – 150	105
ASTM F1554 GR 55	¼ – 4	75 – 95	55
ASTM F1554 GR 36	¼ – 4	58 – 80	36
ASTM F3125 GR A490 & GR F2280	½ – 1½	150 – 173 (max)	130
--	Over 1½	--	Not Available