Chapter 6 Structural Steel

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 except as it relates to retrofit of truss sway and portal frames and for painting of existing steel trusses.

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.

6.0.3 Retrofit of Low Vertical Clearance Truss Portal and Sway Members

A WSDOT internal study and subsequent report titled "*Low Vertical Clearance Truss Bridges: Risk Assessment and Retrofit Mitigation Study*" completed in November, 2017 identifies all the trusses on the WSDOT inventory with vertical clearances less than the current minimum of 16'-6". The report prioritizes the over 60 trusses with substandard vertical clearance for potential retrofit and provides scoping level project cost estimates. The goal is to eventually procure a funding source to gradually begin to retrofit the most vulnerable structures that have the highest risk of being struck with an over-height vehicle causing damage to the structure. Previous over-height vehicle impacts have caused damage ranging from minor distortion of members to as severe as a total collapse similar to the SR 5/712 Skagit River Bridge in 2013.

When through-truss type structures are programed for maintenance, painting, structural retrofit, or barrier/railing rehabilitation projects, the designer should reference the previously mentioned report to determine if the structure is a high priority for consideration of a portal and/or sway raising retrofit. If the structure is in the Priority Group 1 or 2 categories, inquiries should be made with the Region Project Office, the Bridge Asset Management Engineer, and the Steel Specialist to determine if a portal and/ or sway retrofit should be added to the project.

Most existing through-truss structures have portal and sway members that form a parabolic shape, where the vertical clearance at the centerline of the bridge is higher than at the left and right truss lines where the portals and sways connect to the main truss members. When a portal and/or sway retrofit is to be performed, a typical retrofit is to remove the entire parabolic portal or sway member, modify the secondary members attached to the portal or sway, and install a new beam with a straight horizontal profile. Alternatively, and because the portal or sway members often have sufficient clearance for a portion of the length, the members can be cut at the 16’-6” height and a short member added to each side in a straight horizontal profile. Examples of each one of these retrofit schemes is shown in Figures 6.0.3-1 and 6.0.3-2, respectively.

Figure 6.0.3-1 Complete Portal Replacement

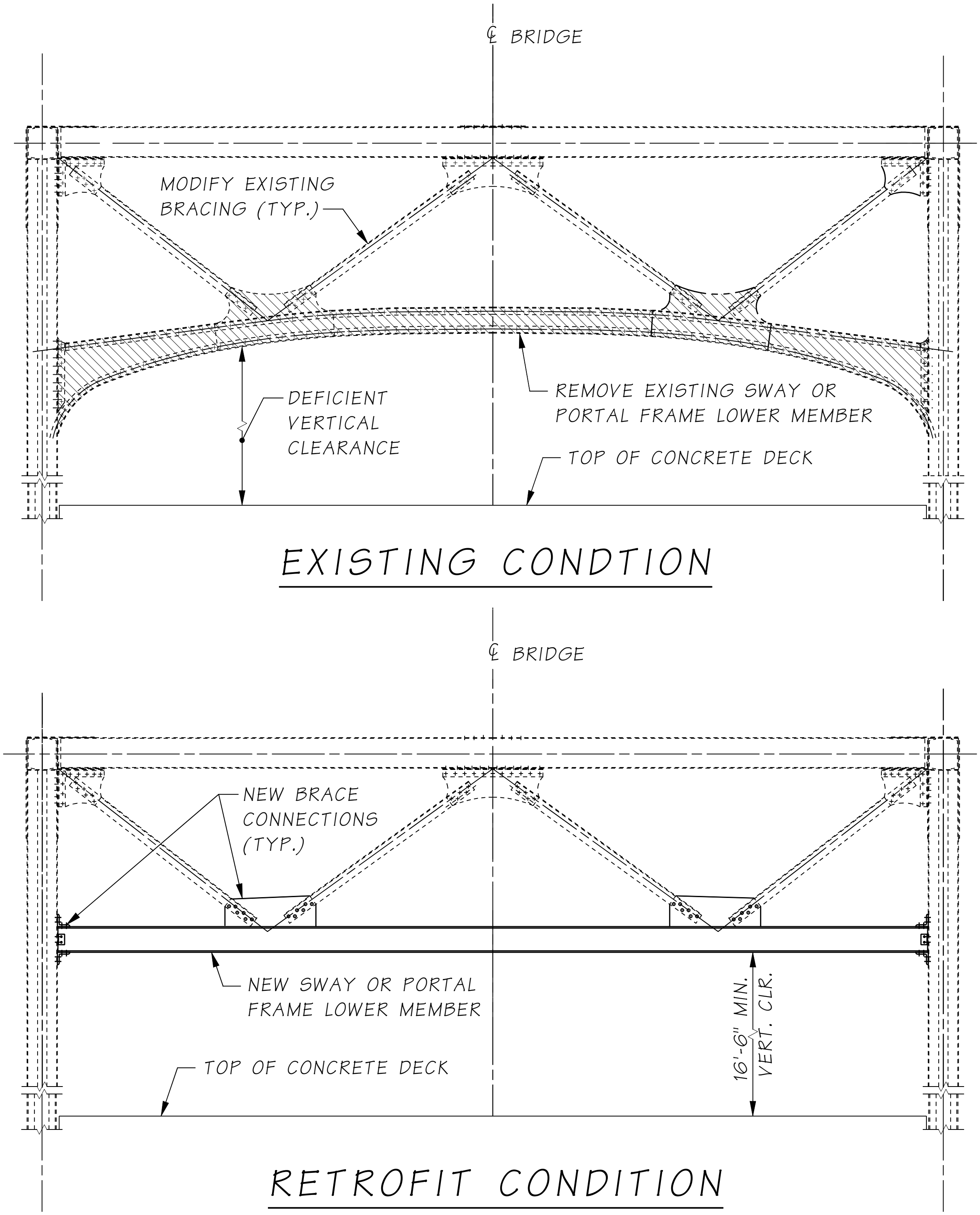
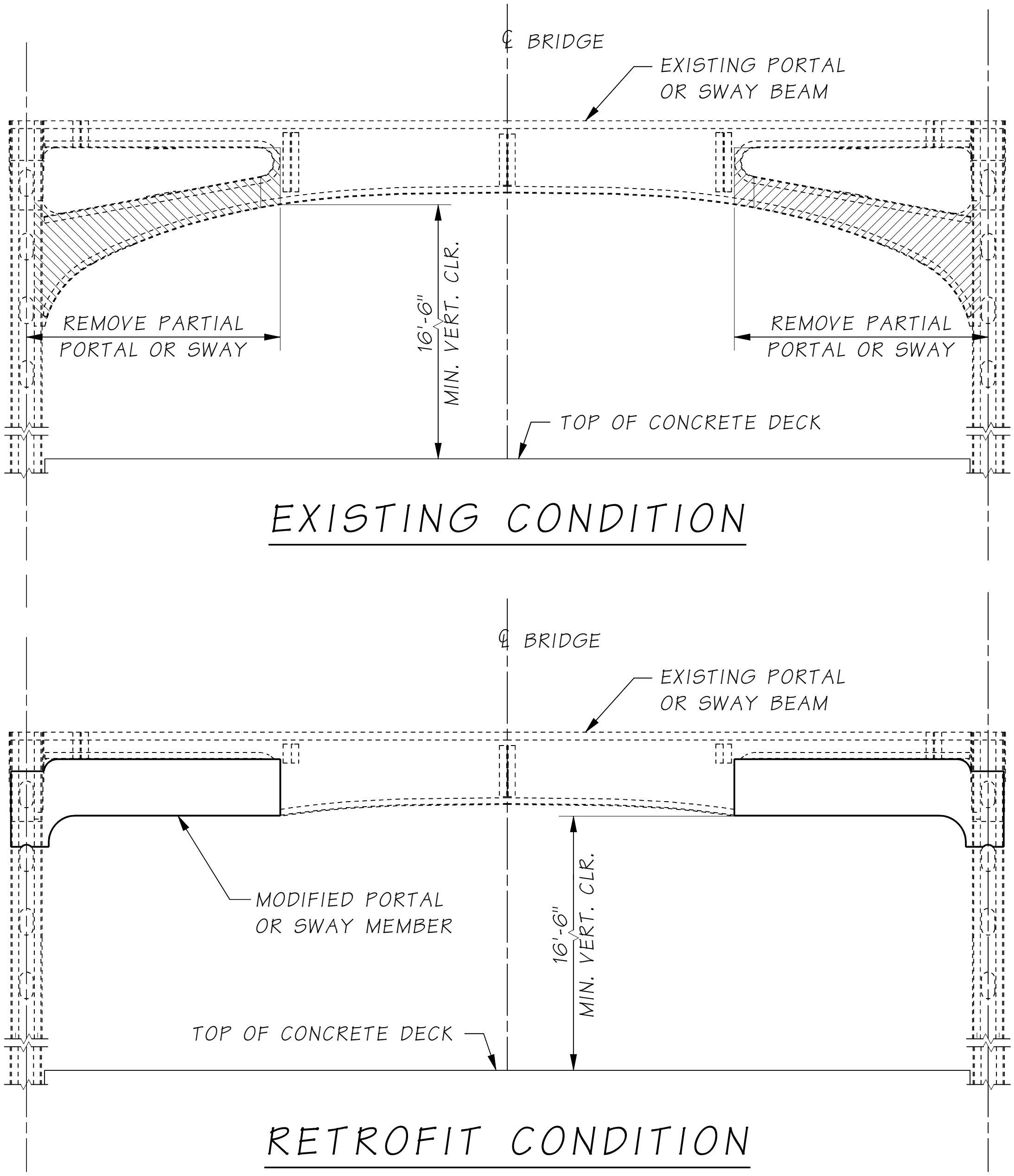


Figure 6.0.3-2 Partial Portal Replacement



When a portal raising retrofit is being performed, the designer will need to check the end posts on the truss to determine if strengthening of these highly loaded compression members is required. The horizontal portal member acts as a brace point for the end post. The capacity of the member is reduced due to an increase in the unbraced length. When the portal is raised, the distance from the bottom chord connection point to the portal horizontal connection to the end post is increased, thus increasing the unbraced length. In most instances, the end posts are strengthened by adding side plates to the member. These are typically bolted on, but have also been field welded in the past.

A relatively straight forward analysis method for determining the end post retrofit needs is to analyze the existing portal frame with a 2-D model in its existing condition. A unit load can be applied to the upper panel point of the portal frame and a displacement determined. The same analysis is then performed with the portal horizontal member in its raised condition. The section of the end post can be increased by adding steel plate until the deflections for the applied load are the same. The applied load can be the actual calculated lateral wind or seismic load or just a nominal load. This is a relative stiffness comparison analysis so the exact load is not critical. In most cases the boundary conditions for the bottom of the end post can be considered pinned. A more rigorous 3-D analysis with actual wind and seismic loads can always be performed, especially if 3-D models are available due to other retrofit needs. In addition to the lateral analysis of the end posts, the compressive capacity shall also be analyzed and verified to be sufficient in the final condition and during any construction conditions. The construction condition is particularly important if live load will remain on the structure while the existing portal is removed. In this case the unbraced length will be at its longest length (panel point to panel point) and needs to be analyzed to be structurally sufficient during the interim construction conditions. Temporary horizontal bracing has been utilized in the past to provide a temporary brace point on the end post after the existing portal is removed and prior to installation of the new.

The designer should also consider how many portal and/or sway members can be worked on at the same time during construction. In most cases, traffic will still be on the structure during the retrofit project. The stability of the truss must be maintained at all times. Previous retrofit projects determined that requiring every other sway member to remain in place during a retrofit project was sufficient. A 3-D model will likely be required to determine the number of portal and sway members that can be removed at one time.

The Bridge Architect and the Region Project Office shall also be consulted during any portal and sway raising projects. Many of the existing trusses are old enough to be on the historic register and changing any visual appearance of the structure may require additional permits or approvals.

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:

- WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (*Standard Specifications*) M 41-10, latest edition
- AASHTO/AWS D1.5M/D1.5: *Bridge Welding Code*, latest edition

The following AASHTO/National Steel Bridge Alliance (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*
- G12.1-2020, Guidelines *to* Design for Constructability *and Fabrication*
- G1.3-2002, *Shop Detail Drawing Presentation Guidelines*
- S2.1-2018, *Steel Bridge Fabrication Guide Specification*
- S4.1-2019, *Steel Bridge Fabrication QC QA Guidelines*
- G4.2-2021, *Guidelines for the Qualification of Structural Bolting Inspectors*
- G4.4-2006, *Sample Owners Quality Assurance Manual*
- S8.1-2014, *Guide Specification for Application of Coating Systems with Zinc-Rich Primers to Steel Bridges*
- S8.2-2017, *Specification for Application of Thermal Spray Coating Systems to Steel Bridges*
- G13.1-2019, *Guidelines for Steel Girder Bridge Analysis*
- G9.1-2004, *Steel Bridge Bearing Design and Detailing Guidelines*
- S10.1-2019, *Steel Bridge Erection Guide Specification*
- G1.4-2006, *Guidelines for Design Details*
- G1.1-2020, *Shop Detail Drawing Review/Approval Guidelines*
- G2.2-2016, *Guidelines for Resolution of Steel Bridge Fabrication Errors*

The *Steel Bridge Design Handbook*, which includes 19 volumes of steel bridge design aids and 6 design examples, was originally produced by US Steel in the 1970s and had previously been maintained by the FHWA. The document has now been taken over by the NSBA and was recently updated in 2021. The update was completed by the FHWA, NSBA, and HDR Engineering. The newly updated documents are also available as design aids and can be downloaded from the NSBA website at the following link: Steel Bridge Design Handbook | American Institute of Steel Construction (aisc.org). These documents are current with the AASTHO Bridge Design Specifications, 9th Edition.

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 and Design of Steel–Volume 4*
- *Selecting the Right Bridge Type–Volume 5*
- *Stringer Bridges and Making the Right Choices–Volume 6*
- *Loads and Load Combinations–Volume 7*
- *Structural Analysis–Volume 8*
- *Redundancy–Volume 9*
- *Limit States–Volume 10*
- *Design for Constructability–Volume 11*
- *Design for Fatigue–Volume 12*
- *Bracing System Design–Volume 13*
- *Splice Design–Volume 14*
- *Bearing Design–Volume 15*
- *Substructure Design–Volume 16*
- *Bridge Deck Design–Volume 17*
- *Load Rating of Steel Bridges–Volume 18*
- *Corrosion Protection of Steel Bridges–Volume 19*
- *Design Example: Three-span Continuous Straight Composite Steel I-Girder Bridge*
- *Design Example: Two-span Continuous Straight Composite Steel I-Girder Bridge*
- *Design Example: Two-span Continuous Straight Composite Steel Wide-Flange Beam Bridge*
- *Design Example: Three-span Continuous Curved Composite Steel I-Girder Beam Bridge*
- *Design Example: Three-span Continuous Straight Composite Steel Tub-Girder Bridge*
- *Design Example: Three-span Continuous Curved Composite Steel Tub-Girder Bridge*

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. Additionally, due to cracking observed in some positive moment regions on some recent steel girder structures, the one percent longitudinal deck steel is required to be placed the full length of the bridge. 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. If solid diaphragms (I-sections) are used at piers and designed for jacking, sufficient access shall be provided for form removal behind the diaphragm. For plate girders, the diaphragm shall be at least 0.75 of the girder depth.

Coordinate jack placement in substructure and girder details. Verify bearing stiffener locations are placed such that jacks can be installed, girders raised, and bearings removed. Proper access should be provided to permit the bearings to be removed and replaced in the future.

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

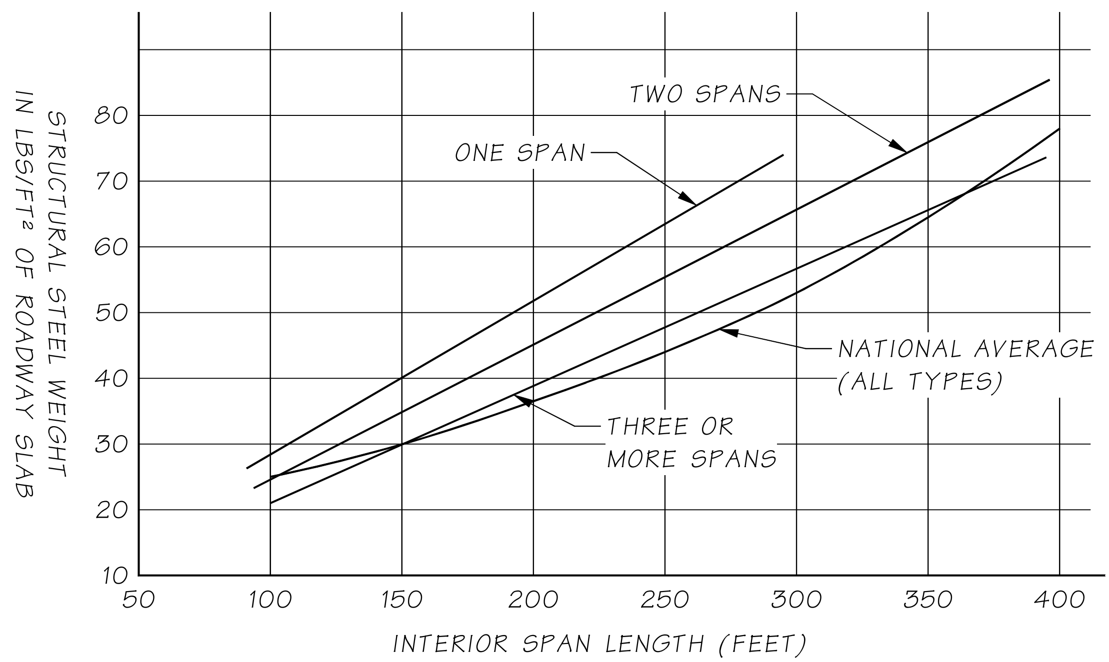
For the preliminary quantities or preliminary girder design, an estimate of steel weights for built-up plate composite I-girders can be obtained from Figure 6.1.4-1. This figure is based upon previous designs with AASHTO HS-20 live loads with no distinction between service load designs and load factor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

Figure 6.1.4-1 Composite Welded Steel Plate "I" Girder



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 HPS70W and HPS100W may only be used if allowed by the Bridge Design Engineer. HPS70W can be economical if used selectively in a hybrid design. For moderate spans HPS70W could be considered for the bottom flanges throughout and top flanges near interior piers. HPS 50W is no longer readily available and should be avoided.

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.

ASTM	ASTM A 709/AASHTO M 270
A 36	Grade 36
A 572 gr 50	Grade 50
A 992 (W and rolled sections)	Grade 50S
A 588	Grade 50W
---	Grade HPS 70W
---	Grade HPS l00W*

*Minimum yield strength is 90 ksi for plate thickness greater than 2½″.

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, and tees cut from W-shapes. Channels and angles are not readily available in ASTM A992. Angles are often more available in thicknesses that are not an increment of ¹⁄₁₆”. For example, if a ⁵⁄₁₆” thickness is required for design, it would be appropriate to increase to ⅜” for availability considerations. 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 6.6.2.1-2, 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 typically 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 may be lower than values obtained in the flats, however recent informational testing has shown the CVN values in the corners meet or exceeds the requirements for FCM steels in temperature zone 2. Heat treating of the sections can improve the values, but no data is currently available. ASTM A 1085 should not be specified for dynamic loading applications until further data is available. The designer should check with suppliers to ensure the size and quantities are readily available. In some cases minimum tonnage or bundle quantities 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 ⅞″ in ¹⁄₁₆″ increments
- ⅞″ to 1¼″ in ⅛″ increments
- 1¼″ to 4″ in ¼″ 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. Local heavy haul trucking companies can also be consulted to help determine girder shipping segment lengths and depths. Contact the Steel Specialist for more information.

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

Material	Bolt Diameter	Tensile Strength ksi	Yield Strength ksi
ASTM F 3125 GR A325 & GR F1852	½–1½ inch	120	92
	Over 1½		Not Available
ASTM A 449	¼–1 inch	120	92
	1⅛–1½ inch	105	81
	1¾ -3 inch	90	58
	Over 3		Not Available
ASTM F 1554			
Grade 105	¼–3 inch	125-150	105
Grade 55	¼–4 inch	75-95	55
Grade 36	¼–4 inch	58-80	36
ASTM F 3125 GR A490 & GR F2280	½–1½ inch	150-173 (max)	130
	Over 1½		Not Available
ASTM A 354 GR BD	¼–2½ inch	150	130
	Over 2½–4 inch	140	115
	Over 4		Not Available

General Guidelines for Steel Bolts

6.1.9.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 in accordance with ASTM F2329 or mechanically galvanized in accordance with ASTM B695 – Class 55. For painted structures, WSDOT's current policy is to require galvanized bolts for structures that are painted to avoid the need for blasting and priming of black bolts after erection of the steel in the field. The *Standard Specifications* provides requirements for surface preparation and painting of galvanized bolts. Do not specify for anchor bolts. Galvanized GR F1852 "Twist-Off" style bolts are not permitted on WSDOT structures. A recent change to the ASTM F3125 specification allows for alternate dimensions such as modified head geometry or special thread lengths. The modified head geometry would include countersunk heads, which previously needed to be specified as ASTM A449. The engineer shall specify Supplement S2 when requiring alternate dimensions.

6.1.9.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.

6.1.9.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.

6.1.9.D ASTM F 3125 GR A490 & GR F2280

High strength alloy steel, headed bolts for use in structural joints. These bolts should not be hot dip galvanized, because of the high susceptibility to hydrogen embrittlement. In lieu of galvanizing, the application of an approved zinc rich paint may be specified. Other coating applications are available and are specified in ASTM F3125. The Steel Specialist shall be consulted prior to specifying one of the alternative coatings. Do not specify for anchor bolts. Only uncoated GR F2280 "Twist-Off" bolts are permitted on WSDOT structures.

6.1.9.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 hot dip galvanized. If used in seismic applications, specify minimum CVN toughness of 25 ft-lb at 40°F.

6.1.10 Bending Steel

It is often necessary to bend steel plate or members for use in the final configuration. This is common for sign brackets or bridge and architectural railings. In researching bending and rolling facilities, the minimum size bending radius for 2-inch to 6-inch diameter, schedule 40 or 80 pipe is 3 times the diameter, 3D. The measurement is taken to the centerline of the pipe. Some facilities may not be able to bend to this tight of a radius so specifying 4D or 5D is preferable.

When bending square tubes in the 2 to 4-inch range, a 12-inch inside radius should be used as a minimum, with larger radius being preferred.

When bends to larger diameter members, square or rectangular tubes, or rolled shapes (I-beams, angles and channels) is required, the designer should contact bend and roll facilities to verify capabilities. Consult with the Steel Specialist for additional information.

Bending of AASTHO M270 (ASTM A709) bridge steel may be necessary for haunched girders or the dapped ends of girders. Typically these plates can be cold bent (room temperature) provided a minimum radius of 5 times the thickness of the plate is specified, 5t. This requirement is for bending plate perpendicular to direction of final rolling. If it is necessary to bend parallel to the rolling direction, the minimum radius shall be increased to 7.5t.

When bending other grades of steel and for ancillary components (connection plates etc.) up to 3/4" thick the minimum bend radius may be taken as 1.5t, or as recommended by the plate producer. Again, the larger the bend radius the finished part will permit the better. Cracking can originate from the outside edges of bent plate, therefore it is recommended to require the edges of bent plates to be ground to a chamfer or radius prior to bending. Require the grinding to occur one-foot beyond the end of the bend.

Hot bending of bridge steels may be used with the approval of the WSDOT State Bridge Design Engineer and Steel Specialist. The Contractor shall submit a heating and bending plan for review. The plates shall be bent at a temperature above the "blue brittle" temperature of 700F but not greater than 1200F, except for HPS 70W and HPS 100W shall be limited to 1100F. Minimum radii shall be as previously specified.

6.2 Girder Bridges

6.2.1 General

Once the material of choice, structural steel has been eclipsed by 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 (SAW). 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 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:

$$0.5 \leq N_L \,/\, N_b \leq 1.5 \qquad (6.2.3\text{-}1)$$

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 WSDOT State 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 grade 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.

The limit state load modifier relating to redundancy, η_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.

Fracture-critical members and system redundant members shall be designed for infinite fatigue life, which requires design using the Fatigue I load combination specified In Table 3.4.1-1 and the nominal fatigue resistance specified In Article 6.6.1.2.5.

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 Articles1.3.2 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 1.75. 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. 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 2” or less 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.

Plates for flange material greater than 2” thick shall 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 6.13.6.1.4 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 reinforced with fillet 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 and are required to be a minimum of 0.75 of the web depth for plate girders and 0.5 of the web depth for rolled sections.

On curved bridges, the cross frames also resist torsional twisting of the superstructure and are considered primary members. 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 with 3 or less girders may not be sufficiently stiff laterally before the deck is cured. Both wind and deck placement can cause noticeable deflections. In the case of deck placement, permanent sideway and rotation of the steel framing may result in a system buckling mode and should be checked per AASHTO section 6.10.3.4.2. Some form of temporary or permanent lateral bracing to an adjacent structure is recommended when possible. For example, when widening a steel girder bridge, the top and bottom struts of the cross frames should be installed and connected to the existing bridge with one bolt at each connection. This provides lateral stability but allows vertical deflections. The remaining connection bolts can be installed after the deck is cast.

Cross frames and connections should be detailed for repetitive fabrication 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°. Some rules of thumb for selecting either X or K-frames are as follows:

- If S/D is less than or equal to 1.0, use X-frames
- If S/D is greater than or equal to 1.5, use K-frames
- If 1 < S/D < 1.5, OK to use either

Where,

S = girder spacing
D = total depth of web

WSDOT standard practice when using K-frame configured cross frames is to orient the frame where the diagonals meet at the center of the bottom horizontal strut. For pier cross frames, it may be advantageous to flip the K-frame so the diagonals help to transfer lateral loads from the deck directly into the girder bearings.

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 cross frames units to girder stiffeners. Welding of individual pieces to make up a cross frame unit is acceptable. Oversize holes are not allowed in cross frame connections if girders are curved and should be avoided for straight and skewed bridges unless approved by the Steel Specialist.

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. Cross frames should also be sized to meet the stability bracing requirements discussed in Chapter 13 of the recently updated (Feb. 2022) Steel Bridge Design Handbook. The stability bracing forces should be added to the loads obtained from the first-order analysis loads such as dead load, wind etc.

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. Recent research has provided guidance, much of which is included in AASTHO, for cross frame arrangements for skews greater than 20°, especially near intermediate piers and abutments. The key is to avoid a cross frame spanning from one girder that is supported by a pier or abutment to an adjacent girder point away from the support. This creates large cross frame forces due to differential deflections of the girders. Consult with the Steel Specialist for recommendations on cross frame arrangements.

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. Recent ballot items approved for the 10th Edition of AASHTO provides reductions in the fatigue demands in cross frames. Currently the load factors for Fatigue I (infinite) and Fatigue II (finite) are 1.75 and 0.80, respectively. These have been found to be overly conservative for cross frame design. The approved ballot item allows the current Fatigue I and II load factors to be multiplied by a cross frame design factor of 0.65.

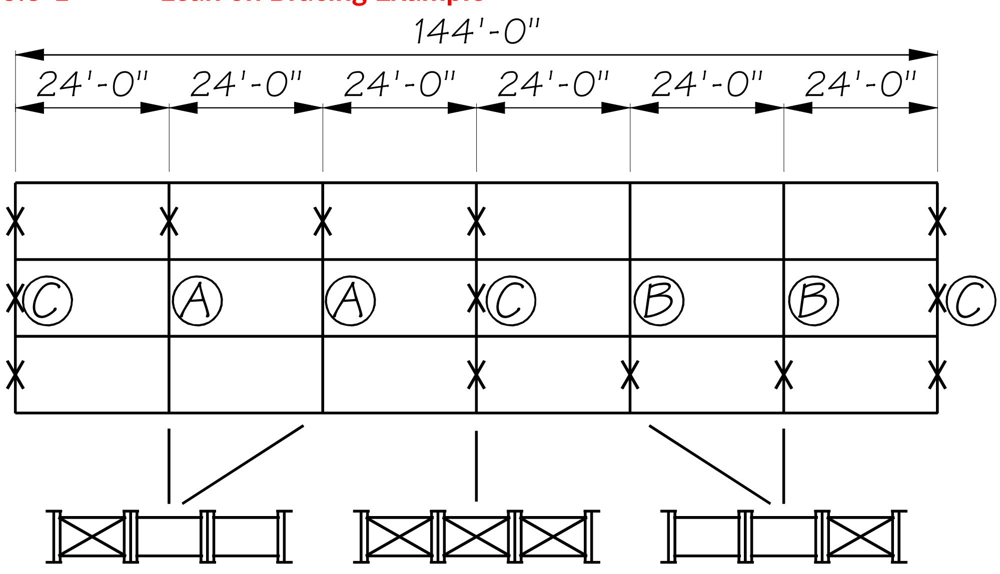
For design and detailing of cross frames, the following are additional items to consider:

1. When using a refined analysis, such as a 3D model, member stiffness for equal leg single angles, unequal leg single angles connected by the long leg, and flange connected WT members may use an effective member stiffness equal to 0.65AE. This approach will reduce the force demands by accounting for connection stiffness and second-order stiffness softening. It is recommended to create a new steel material property within the analysis model with a reduced modulus, E, versus reducing the area, A, of the section.
2. When modeling cross frames in 3D models, the ends of the members should be considered pinned and the point of the connection to the girder web modeled as close to the true location as possible. Connecting the cross frame member directly into the intersection of the web and flanges can often produce higher demands than the members will actually experience.
3. When obtaining fatigue demands on cross frames, the maximum and minimum forces shall be obtained with the fatigue truck confined to one critical transverse position per each longitudinal position throughout the length of the bridge. In other words, the max/min forces on a cross frame shall be obtained from the truck loading from one lane. For a specific cross frame, it is too conservative to use the max force from a truck positioned in lane 1 while taking the minimum force obtained from and adjacent lane. For typical FEA model software, such as CSI Bridge, consisting of a model with multiple lanes, the normal output is to provide an envelope of max and min forces considering all the lanes. Fatigue truck loading will need to be carefully obtained from FEA models and should be limited to demands produced from one lane of loading at a time.
4. Cross frames can be one of the most expensive items to fabricate for a steel bridge. Economy of repetition should be considered in the design and detailing. If the loads on the cross frames vary along the length of the bridge, the cross frame designs can be grouped into 2-3 types based on demands. For example, one design for highly loaded frames, one for intermediate, and one for lightly loaded. It can be expensive to design one cross frame for the maximum demands and use them at all locations.
5. Detailing of cross frames have historically identified work points (WP) on the plans at the intersections of cross frame centerlines and the centerline of the girder web. Consider using the center of a bolt hole at cross frame gusset connections as a defined WP. This enables the fabricators in the shop to physically layout the WP in their fabrication jigs versus having it be “off in space” beyond the limits of the actual cross frame. Additionally, distances or clearances from girder flanges to bottom of cross frame gussets or WP can be called out with +/- tolerances, especially when there are variations in girder framing geometry. If there are tolerances allowed in the actual location of the cross frame, it provides opportunity for the fabricator to detail and fabricate cross frames that are exactly the same.

6. When obtaining cross frame loads, consider the actual stiffness of the supports and allowable movements in the bearings. A true pin or fixed bearing in a model can produce high demands on the pier cross frames. Including flexibility of the support or slight movements in bearings, say ⅛” to ¼”, can greatly reduce demands. This will depend on the bearing type.
7. For straight or slightly skewed I-girder bridges, consider lean on bracing for the cross frames. Lean on bracing has not been codified in AASHTO but several states have used them successfully in steel bridges. The general concept is to minimize the number of complete cross frames between girder lines. For example, in a 4-girder system labeled A through D, a complete K or X cross frame would be used between girders A and B while only top and bottom struts are installed between girders B-C and between C-D. The positions of the complete cross frame would vary along the length of the bridge and in some bays, complete cross frames would be used between all girders. See Figure 6.3.8-1 for a general illustration. The construction sequence will need to be carefully considered so stability is provided during girder erection. See the Steel Specialist for more guidance on design and detailing of lean on bracing.
8. Single angles members for cross frames are generally preferred by fabricators. These are the easiest to fabricate. WT sections require a W-beam to be cut in half, adding an additional step to fabrication. The AASHTO code provides equations for design of single angles that account for the eccentricity of the angle and the connection point, allowing the angle to be designed for just axial demands. When checking slenderness minimum requirements L/r or KL/r, use the radius of gyration, r, from the Z-Z axis.

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

Figure 6.3.8-1 Lean on Bracing Example



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. A major revision to the design of bolted splices was implemented in the 8th Edition of the AASHTO LRFD. The design methodology is simplified and requires top and bottom flange splice plates and bolts to be sized to develop the capacity of the smaller flange. Designing web splices is outlined in AASHTO LRFD Article 6.13.6.1.3c. Web splices are sized to develop the shear capacity of the web, with a check on moments. Only 2 rows of web bolts will typically be required on each side of the splice plate. Bolted web splices should not involve thin fill material. Thickness transitions for webs, if needed, should be done with welded shop splices. The NSBA has developd a bolted splice EXCEL spreadsheet, which can be downloaded from their website to assist in splice design.

Flange splice design is outlined in AASHTO LRFD Article 6.13.6.1.3b. 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 6.13.6.1.4. 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.

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:

1. Steel weight, applied to steel section. Include 5 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.
2. Bridge deck weight, applied to steel section. This should be the majority of dead load deflection.
3. Traffic barriers, sidewalks, and overlays, applied to long-term composite section using 3n. Do not include weight of future overlays in the camber calculations.
4. Bridge deck shrinkage (if ≥ ¾″).

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, may not require a rigorous analysis. The bridge deck could 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. However, with the modern analysis tools available, it is recommended a rigorous analysis be performed for all steel bridge designs. An analysis coupled with a bridge deck placement sequence should be used. This requires 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 most finite element analysis software.

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″. 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 5.5. 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 5 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 $\frac{1}{10}^{th}$ 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, however the designer should consider economics and constructability when selecting bearings. In some cases where an elastomeric bearing could handle vertical loads and rotations, additional girder stops and/or hold downs may be needed, which could make a disc bearing a smarter and more economical choice.

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

6.3.14 Surface Roughness and Hardness

The standard measure of surface roughness is the microinch value. This can be measured in a number of scales but typically in the USA, the Ra scale is assumed. 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 the *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:

1000A surface produced by thermal cutting

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

250A 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.

125A 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.

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

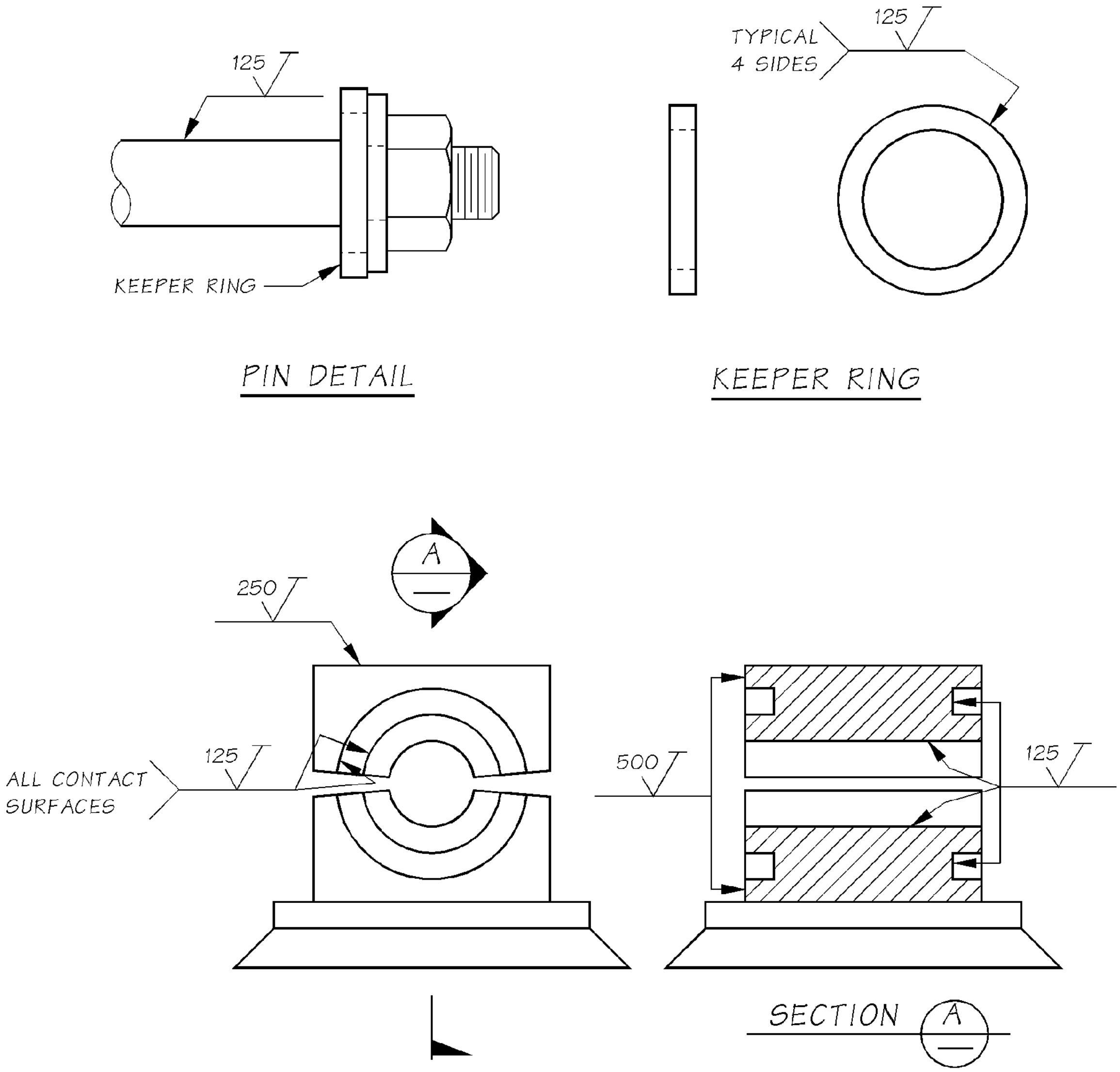
32An extremely fine machine finish suitable for steel sliding parts. This surface is generally produced by grinding.

16A 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 USA industry standards for finish on stainless steel sheet, strip, and plate. A #1 finish is basically "as rolled" and the scale goes up to a #8 finish, which is the mirror finish. No units are implied. See the Steel Specialist for examples of these finishes.

Figure 6.3.14-1 Surface Finish Examples



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*.

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.

Base Metal Thickness of Thicker Part Joined	Minimum Size of Fillet Weld
To ¾″ inclusive	¼″
Over ¾″	5⁄16″

In general, the maximum size fillet weld which may be made with a single pass is 5⁄16 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.

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 is prequalified.

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 or steel dead load only 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 National Steel Bridge Association (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. Consult with the Steel Specialist on what fit-up to specify in the plans for curved and skewed bridges. Straight bridges, fit of girders is not an issue because the webs will remain vertical in all conditions, no-load, steel dead load only, and total dead load.

6.4 Plan Details

6.4.1 General

Detailing practice shall follow industry standards. 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

Structural steel notes are dynamic in nature and often change due to changes in design, materials and industry practice. A starting point for Structural Steel Notes can be found in the Standard Design Drawings on the WSDOT Bridge and Structures website and 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 bridges to be painted and for weathering steel bridges. Amendments to each are available for box girders. Contact the Steel Specialist for specific questions related to steel notes.

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 shown 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 as FCM. If a member is identified as fracture critical with an "FCM" symbol, it is not necessary to also call the plate or member with a "V" for Charpy-V-Notch as this is covered with the "FCM" designation. 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 Bridge Welding Code Section 5.5. Allow a minimum of ¼″ for out of position web plus ⅜″ for fillet weld, or a total of ⅝″ 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 6.10.11.1.1.

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 where single angles or WT sections will not suffice, consider symmetric sections (HSS tubes) 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. The current WSDOT policy requires one percent minimum steel be provided for the entire length of the bridge so typically only one section view is required for single or continuous spans.

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 (BPO). Often, handrails are not needed if access to all details is possible from under bridge inspection trucks (UBIT’s) although recently the BPO inspection staff has requested handrails on both sides of interior girders and the inside of exterior girders. 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 SAE AMS 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), when applicable, should be submitted with shop plans. If not, they should be requested so they can be reviewed during the shop plan review process.

All the geometry in most shop plans is typically not reviewed in its entirety, however the reviewer should verify that lengths, radii, material types, 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.

Typically shop plans are submitted as Type 2 Working Drawings. See *Standard Specifications* Section 1-05.3 for definitions of Working Drawings. Upon completion of the review, the submittal shall be stamped in accordance with the WSDOT *Construction Manual* Section SS 1-05.3 guidance. The status of the submittal is typically stamped as:

1. No exceptions taken
2. Make corrections noted
3. Revise and resubmit.

When stamping as “Make corrections noted” it is often worthwhile to note whether the shop plan needs to be resubmitted or not.

6.6 Painting of Existing Steel Bridges

6.6.1 General

With the aging of our existing steel bridge inventory, painting of these existing steel bridges has become a common preservation project requiring PS&E development. The majority of the painting projects are existing steel truss bridges and this section will focus mainly on trusses, however most aspects of this section are also applicable to plate girder and box girder type structures.

The existing truss bridges range in complexity from simple span through-truss bridges to complex multi-span, arched deck trusses. As part of the PS&E development, the structures need to be analyzed to ensure the bridge can support the construction loads that will be imposed on the structure. In many instances, the structures will need to remain open to all or partial live load traffic lanes.

The existing trusses may require both a vertical and horizontal analysis. The vertical analysis is necessary to ensure the structure can withstand the additional dead and live load from painting construction activities. The horizontal analysis is necessary to ensure the structure can withstand the lateral wind loading imposed on the structure during painting operations. Containment is required to collect all debris and creates a large "sail" area with respect to the truss condition without containment. Wind load limitations must be imposed as part of the Contract documents to ensure the structure is not overloaded while the containment is in place. This will be covered in more detail in Section 6.6.3. In most cases when painting a plate or box girder structure, only a vertical analysis will be required. The containment necessary for plate or box girders will not be substantially larger than the normal exposed wind area of the bridge without containment.

Most existing steel structures were originally constructed when lead paint was used as a primer coat for protection of the steel elements. It began to phase out in the 1960's and was formally banned by the Federal government in 1978. However, lead paint was not removed from WSDOT *Standard Specifications* until the early 1990s. Structures constructed prior to these dates should be assumed to contain lead based paint. For many years, WSDOT's practice was to overcoat existing steel structures, including trusses. This involved removal of loose paint and debris, spot sandblasting and priming of areas with visible rust, and then over coating with an intermediate and top coat of paint. WSDOT's current policy is to require full paint removal on our existing steel structures that have only received overcoat paint applications in the past. This entails full removal of all existing paint, rust, mill scale etc. down to bare metal. Refer to Section 6-07 of the *Standard Specifications* for more details on full paint removal procedures. Contract Special Provisions should indicate the general paint history of a structure and note that blasting to bare metal has not occurred in the past, several layers of paint may exist on the structure, and no guarantee of total paint thickness can be made. Paint history forms can be obtained from the Bridge Asset Engineer and can be include as "For Information Only" documents.

WSDOT's current policy requires full containment of all blasting and paint debris and shall be in accordance with SSPC Technology Guide No. 6, *Guide for Containing Surface Preparation Debris Generated During Paint Removal Operations* Class 1. Emissions from the containment are limited to the Level A Acceptance Criteria – Option Level 0 Emissions standard per SSPC Technology Update No. 7. This means no emissions (debris, old paint, sand blast media, etc.) are permitted to escape the containment.

6.6.2 Vertical Analysis

A vertical analysis is typically required when developing plans and specifications for a steel bridge painting project. Cases where a vertical analysis may not be required are when the structure will be closed to live load during construction or when enough lanes will be closed to easily ensure the extra construction load will be less than the live load from the closed lanes.

The vertical analysis consists of a load rating of the bridge with added loads for construction dead and live load. The additional dead load is primarily due to containment, access platforms, and equipment. The additional live load is for workers, and debris from abrasive blasting. These are considered live loads because they vary and are not constant. For the vertical analysis, a 25 psf load is typically assumed to account for both dead and live loads. The width of the applied construction load is typically assumed to be the bridge width plus 5 feet on each side. This is only a rule of thumb and can be adjusted, but the assumptions used for the area of construction load must be clearly stated in the plans. If the Contractor chooses to use a wider or narrower width of platform, then they can adjust the allowable construction load proportionally. Construction load can also be increased if the structure allows or decreased if 25 psf creates rating concerns. The minimum reasonable construction load should be at least 20 psf.

The load rating analysis is only performed for legal vehicles, which include the AASHTO 1, 2 and 3 vehicles and the NRL. In some cases, additional vehicles may need to be included in the rating on a case-by-case basis including emergency vehicles. The construction load is subtracted from the member capacity in the numerator of the typical load rating equation. In most cases the LRFR rating method should be used.

The designer should assume the construction load is applied to the entire length of the structure. This allows the contractor the most flexibility in constructing the work access and containment. In some cases, this may not be possible due to load rating limitations and only specific zones will be permitted to be loaded with the additional construction load. Again, these limitations need to be clearly stated on the contract plans.

As with a typical load rating, the rating should include the primary truss members, the floor system and the primary member gusset plates. The designer should check with the WSDOT Bridge Preservation office’s Load Rating Engineer to obtain the latest load rating information. Often, an existing rating can be easily modified to add in construction loads or existing structural models can be utilized as part of the analysis.

In addition to the global analysis of the structure for the added construction load, analysis of localized loads on individual members may be required. The number of support points and maximum applied loads for individual stringers, floor beams or truss elements will need to be outlined in the Contract plans. The most common approach is to provide maximum loads and spacing of support points. The Contractor will need to analyze the local attachment to the member based on his construction methods and particular attachment detail. Alternatively, maximum additional shear or moment demands on specific members can be provided, however maximum loads and support point spacing is typically preferred by the Contractor.

6.6.3 Horizontal Analysis

As discussed previously, and primarily with truss type structures, a horizontal wind analysis will need to be performed to ensure the structure can resist wind loads while the containment is in place, which creates a large wind or "sail" area on the bridge. As part of the analysis, the amount of bridge to be contained needs to be determined and at what maximum wind speed the containment side walls need to be removed to provide an acceptable level of safety. This is particularly important with long span trusses and trusses with varying depths such as arch deck trusses or camel back trusses. Currently the *Standard Specifications* limit the amount of containment to one span unless otherwise specified. In some cases more or less than one span can be contained. It is possible that a Contractor may want to contain portions of multiple spans or they may want to maintain a constant containment length, but progressively move the containment down the length of the bridge. This can create many different loading conditions on the structure.

A full rigorous analysis can always be performed for the horizontal analysis taking into account member capacities and demands; however a simplified method has been successfully employed in the past and will be described herein. The following is a basic outline of the process followed by a more detailed description for each step.

Basic Outline:

1. Compute the AASHTO LRFD design wind pressure for the bridge elements based on the existing site conditions.
2. Apply the wind pressure to the bridge elements and calculate total horizontal wind shear at each support location.
3. Utilize the total design wind shear at each support as the allowable upper bound horizontal wind loading.

Assume locations for containment. Back calculate a reduced wind speed that results in the same or lower horizontal wind shear at each pier. Compute for the various containment conditions. Determine allowable containment areas and associated maximum wind speeds during construction. These maximum wind speeds will be the forecast wind speed value that triggers the Contractor to remove or lower the containment side walls.

Detailed Discussion:

The following discussion provides more detail to the outlined steps above. In the past Excel spreadsheets have been used for the analysis and have proven to be an efficient tool for this simplified analysis.

Steps 1, 2 & 3:

Determining the design horizontal wind shear at each pier or support location requires computing the exposed surface area of the existing truss members, floor system and any barrier or rail. The design wind pressure, P_z, to apply to the surface area should be computed based on the latest AASHTO LRFD Bridge Design Specifications for the Strength III Limit State. The wind speed to use in the calculation of wind pressure should be obtained from AASHTO LRFD Figure 3.8.1.1.2-1. The Wind Exposure Category and Ground Surface Roughness Category will need to be selected based on judgment of the existing site. The Drag Coefficients will vary depending on the member and can be found in AASHTO LRFD Table 3.8.1.2.1-2. Typically for truss members the sharp-edged member coefficient is used, whereas the I-girder Superstructure coefficient is used for the floor system and barrier.

The computed design wind pressure is applied to each member or bridge element to compute a horizontal shear per member. For simplicity, the individual member horizontal shear can be distributed to each pier or support by assuming simple span boundary conditions for each span, regardless if the truss is continuous over the pier(s). The load is assumed to be applied at the mid-length of the member when computing horizontal distances to the pier. The summation of all member shear forces is calculated for each pier and can be considered the maximum allowable shear force per pier or support. The calculated maximum shears will be based on the latest AASHTO wind speeds and could be higher than the original design. As a good check, the designer should review the existing stress sheets as they often will have design wind shear for the bearings. These forces will be Service level demands and should be compared with unfactored calculated demands. If no data is available, old versions of the AASHTO code in the WSDOT Bridge Office archives can be reviewed to determine design wind speeds and or pressures used when the structure was originally constructed. Historically a wind pressure of 50psf was used on trusses. The designer can then make a judgment as to what maximum shear value should be used for comparison in the next steps. Regardless of which maximum wind shear is to be used, a safety (or resistance) factor of 0.80 should be applied to the maximum permissible shear forces.

Overturning stability of a bridge should be calculated at each pier, especially when the truss has significant depth, has a variable depth, or the ratio of truss line horizontal spacing to truss vertical depth is less than ⅕. Overturning forces have been computed on previous projects and have been found to not be of concern and no uplift conditions on the bearings were encountered.

Step 4:

Once the upper bound horizontal shear force per pier has been determined, analysis of the containment areas can be computed.

Simple Spans Structures: For a simple span truss, it is recommended the designer start by assuming the entire span is covered with containment. For the total containment height, assume a vertical distance from 5 feet below the truss bottom chords to 5 feet above the top of bridge superstructure. Using the total containment area, calculate the associated wind pressure that results in the previously determined maximum shear at the pier or support. From this wind pressure, back calculate the corresponding wind speed. For containment areas use a drag coefficient for I-Girder Superstructures when back calculating wind speed. This drag coefficient most closely represents a flat surface created by the containment. This value is the maximum wind speed that can be allowed on the structure before the Contractor will be required to remove or lower the containment side walls. The Contract Specifications and Special Provisions require the Contractor to monitor forecasted wind speeds and gusts and are required to act accordingly if speeds over the maximum are expected. As a rule of thumb, 30 to 35 mph has been successfully used as maximum wind speeds on several projects. However, removal of containment can be time consuming and expensive for the Contractor. A more desirable maximum wind speed is in the range of 45 to 50 mph. The Contractor may not necessarily be working in these conditions, but will allow them to safely leave the containment in place.

Multi-span, Long-span, Variable Depth or Complex Geometry Structures: These types of truss structures typically require more analysis due to the many containment scenarios that could be employed on the structure. In many cases only portions of the span will or can be contained. When computing shear forces at the piers or supports, the portion contained and the members not contained will need to be included in the analysis. This is where an Excel spreadsheet can be a handy tool. In past analyses, each bay of the truss has been set up with essentially an “on or off” switch. If on, the total area of the truss bay or panel is assumed contained and the associated area and distances from the center of the bay are used to calculate horizontal wind shear at the piers. If off, only the truss members within the bay are assumed to be loaded with wind pressure. Shear forces are combined and total shear at the pier is computed. This tool allows many containment scenarios to be investigated at various wind pressures and speeds. As discussed earlier, the Contractor will often want to contain a constant length of the bridge, but will move the containment progressively along the length.

As discussed with the simple spans, a wind pressure and corresponding wind speed can be back-calculated and used as the maximum allowable construction wind speed.

For further questions on the analysis methods, and what method may be appropriate for a given structure, discuss with the Steel Specialist.

6.6.4 Special Considerations

6.6.4.A Coast Guard

Coast Guard permits will be required for any painting project over a navigable waterway. The containment will typically extend below the structure thereby reducing the vertical and/or horizontal clearance of the navigation channel. Temporary navigation lights may also be required as part of the project as existing lights may be covered by containment.

The WSDOT Coast Guard Liaison should be contacted early in the design process so there is sufficient time to procure the necessary permits. When a moveable span is included in the project, such as a bascule, lift, or swing bridge, the operation of the moveable span may need to be modified during the contract. This will require either a deviation or rule change to the operation of the bridge. The deviation can be handled at the local level and allows for changes up to 6 months. If changes to the operation of the bridge are required for longer than 6 months, a temporary rule change will be required and will need approval at the national level.

6.6.4.B Moveable Spans

When working with moveable spans, consideration for machinery and span balancing must be considered. For most moveable bridges, counterweights are necessary to balance the span while it is being lifted or swung open. This is to keep the span operating smoothly and to reduce demand on the machinery and associated components. The Bridge Preservation Office should be contacted to discuss the specifics of the project. In some cases measurements of the bridge machinery must be taken prior to any work starting and then again after painting is complete and prior to operating. Removing all the existing paint can change the weight and balance of the span and adjustments to the counterweights may be required. The machinery will also need to be protected during painting operations, particularly sandblasting. Sandblasting can severely damage bearings, gears, motors etc. if not protected. These elements are typically excluded from the project painting limits.

Operation of the moveable spans with containment in place is typically not permitted. This will need to be coordinated with the Coast Guard and durations where the span is inoperable should be minimized to avoid impacts to the marine traffic.

6.6.4.C Traffic Control

Traffic Control options will need to be discussed with the Region early in the project, particularly with through-truss type bridges. Ideally the bridge can be closed during the painting operation; however that is not the norm. In most cases all or partial lanes will need to remain open during the painting project. This will control the amount and location of containment the Contractor will be permitted to install. The trade-off is the overall duration of the project. If several containment set-ups are required, the longer the project duration and number of working days required.

6.6.4.D Review of Construction Submittals

The *Standard Specifications* requires a comprehensive painting plan be submitted by the Contractor for all painting projects. Included in the submittal will be the engineered containment plans. These will need to be reviewed carefully to ensure the limitations outlined in the Contract have been followed. In many cases, attachments to existing piers, bearings or walls are required as part of the containment support system to resist horizontal forces. These attachment locations and associated calculations also need careful review to ensure no damage will occur to existing elements.

6.6.4.E Structural Steel Repairs

The designer should review the existing inspection reports for the bridge(s) included in the painting project to determine if structural repairs are required and would be appropriate to add to the scope of work. In addition, the design engineer should consider a site visit to review and inspect the existing structure during the design effort. The designer should consult with the Bridge Preservation Office to coordinate the site visit (traffic control, UBIT or other equipment needs). During the site visit the designer should verify any repair areas identified in the inspection reports and look for additional areas that may need repairs. An assessment of the condition of the existing rivets and amount of existing pack rust should be made to help in estimating quantities for the project. In most cases it will be difficult to inspect the entire bridge, but representative areas can be inspected and evaluated and then extrapolated to the entire structure.

The Bridge Asset Management Engineer should be consulted as to available funding for structural repairs. If known structural repairs are not required, all paint projects should include a bid item for Misc. Steel Repairs to account for items that are discovered during the painting operations. Typically this bid item is included as a Force Account item so that WSDOT can direct the work when needed repairs are identified. In addition, existing rivets that are corroded or loose should be replaced during the contract. A detail is provided in the plans that outlines in what condition a rivet should be removed and replaced with a high strength bolt. All rivets being replaced with high strength bolts should be replaced with galvanized bolts. Tension control "twist–off" bolts shall not be permitted. Use of galvanized bolts avoids having to go back and sandblast the black bolts in preparation for prime paint in areas that have already been blasted and primed. Refer to Standard Drawing 6.4-A16 for details on evaluation and replacement of existing rivets. Rivets shall be evaluated after blasting and priming operations. In many cases rivet deterioration can't be evaluated until after blasting operations are completed. However, to avoid flash rust on the newly blasted steel, the Contractor will need to prime the existing surfaces prior to any evaluation inspection can occur. The rivet evaluation and repair work shall be included in a separate per each bid item.

The engineer should review the inspection reports to determine a reasonable estimate for number of rivets in addition to any assessments made during a site visit. Consult with the Steel Specialist if the inspection reports do not have an estimated quantity of deteriorated rivets. The Misc. Steel Repair Force Account bid item should be included to cover any unforeseen structural repairs. Known structural repairs are typically included as a Lump Sum bid item based on estimated steel weight.

6.6.5 Quantities and Estimates

As part of the PS&E development, quantities of surface area will need to be calculated for the entire bridge. This can be a time consuming effort but is necessary to estimate project costs and durations. On truss bridges, it is typically sufficient to compute the surface areas of the members from panel point to panel point and ignore the gusset plates. Detailed comparisons have been performed and when a member length between panel points is used, the area of the gusset plates is typically accounted for. A factor for miscellaneous area in the range of 5% to 15% can be added depending on the complexity of the bridge to cover connections, bolt or rivet surfaces, and any miscellaneous items not accounted for in the surface quantity take-off. Once the quantity of surface area is computed, the Specification and Estimate Engineers should be consulted and can provide estimates on cost.

When estimating schedules, experience from previous painting projects can be used as a guide. A good rule of thumb is to assume one painting crew consisting of 6-8 people can complete 250SF of surface area per 8 hour shift. This would include set-up and take-down of containment, blasting and painting. From these initial estimates, durations can be estimated for the entire project by adjusting number of crews on the project and number of shifts worked per day. The engineer/estimator must take into account the practical limitations of a given project. For example, a simple span truss will have limited amount of access and/or crews that can be working at one time. Alternatively a large multi-span truss may have several crews working or several shifts per day. Constraints at a particular site will also need to be considered in the working day estimate. Limitations on night work may be required due to noise concerns. The sandblasting operation is loud and often exceeds the noise limits, particularly in populated areas.

6.7 Corrosion of Steel Foundations and Buried Structures

6.7.1 Corrosion of Steel Foundations

The following section provides corrosion rates for the design of steel H-piling, pipe piling, concrete filled steel tubes (CFSTs), and sheet piling. The design wall, flange, and web thickness, as applicable, for structural steel sections shall be reduced for corrosion over a 75-year minimum design life. The remaining steel thickness at 75 years shall be sufficient to resist the anticipated design loads. Minimum corrosion rates for section loss are specified in Table 6.7-1 below.

Table 6.7-1 Section Loss (inch per year)

Location	Marine or Non-Marine: Corrosive	Non-Marine: Non-Corrosive
Soil embedded zone (undisturbed soil)	0.001	0.0005
Soil embedded zone (fill or disturbed soils)	0.0015	0.00075
Immersed zone	0.003	0.0015
Tidal zone	0.004	---
Splash zone	0.006	---
Atmospheric	0.002	0.001

Definitions of the terms used in Table 6.7-1 are as follows:

- **Marine** – a site is considered a marine environment if the structure is less than 1000 feet measured from the surface or edge of salt or brackish water. Water shall be considered brackish if the chloride concentration is measured at 500 ppm or greater measured at mean tide level or higher.
- **Non-Marine: Corrosive** – a non-marine site is greater than 1000 feet from salt or brackish water and is considered corrosive if one or more of the following conditions exist based on representative soil and/or water samples:
  1. The chloride concentration is 500 ppm or greater,
  2. The sulfate concentration is 1500 ppm or greater,
  3. The pH is 5.5 or less.

  If none of the following conditions exist, the site is considered non-marine: non-corrosive. Refer to Section 6.7.2.1 for information on appropriate testing procedures.
- **Immersed zone –** portion of structural steel element which is continuously immersed or submerged in water. Immersed non-marine: non-corrosive are environments with fresh water or are tested and found not to meet the marine or non-marine: corrosive values.
- **Tidal zone –** portion of structural steel element in a marine environment between the Mean Lower Low Water (MLLW) and the Mean High Water (MHW) based on the MLLW Datum.
- **Splash zone –** portion of structural steel element in a marine environment located above the MHW plus five additional feet or as otherwise determined for a specific site.
- **Atmospheric** - portion of structural steel element above the splash zone or above ground line as applicable.

The section loss for H-piling and sheet piling shall be doubled to account for both surfaces being exposed. For piling exposed on one face and against the soil on the opposite, the appropriate exposure section loss values can be used per side and then added for a total loss. The interior surface of pipe piling and CFSTs are assumed to be sealed by concrete fill or a soil plug, which prevents sufficient oxygen to support significant corrosion. A micro-pile is considered a subset of pipe piles.

The corrosion rates are based on information published in the CALTRANS March 2018, Corrosion Guidelines, version 3.0, the FHWA NHI-16-009 Design and Construction of Driven Pile Foundations - Volume I, and the Washington State Ferries *Terminal Design Manual*, 2016.

A site-specific assessment should be performed to determine if a site is considered marine, non-marine: corrosive, or non-corrosive. If a site investigation is not performed, the values for a marine/non-marine: corrosive site shall be used unless otherwise approved by the State Bridge Design Engineer. Sampling of a site for corrosion assessment requires samples of the soil and/or water to be obtained from both the surface and subsurface materials to ensure representation of the strata at the site. Water samples in flowing streams and rivers shall not be taken when the water level is elevated due to storm conditions as they may dilute the chemical concentrations.

The potential for scour need not be considered when choosing a design corrosion rate as it relates to zones of exposure. It is assumed any significant scour would be repaired and the applicable zone of a structural element would not be changed. However, if abrasion on the section is a concern, additional wall thickness should be considered. Refer to the discussion on abrasion in Section 6.7.2.5 for more guidance.

A protective coating may be applied to the structural steel sections to increase the corrosion protection. If a coating is utilized, then the section loss corrosion is assumed to begin at the end of the effective life of the coating. Coating effective life is generally assumed to be 15 years. An appropriate coating system shall be specified to withstand damage from handling and driving. Contact the WSDOT Steel Specialist for recommendations on paint systems and coating thicknesses for use on steel piling.

Coating requirements for steel piles in soldier pile walls (cantilever and tie-back) is covered by the *Standard Specifications* Section 6-16.3(4). Any portion of the pile embedded in concrete or CDF need not consider corrosion or section loss, but portions adjacent to the soil or exposed to the atmosphere shall consider section loss for the design life of the structure.

6.7.2 Corrosion and Abrasion of Metal Buried Structures

Metal buried structures consist of steel or aluminum structural plate pipes, arches, and boxes and shall be designed in conformance with Section 12 of the AASHTO LRFD Bridge Design Specifications. Once a structural shape, size, material, and gage are selected, the designer shall perform a service life analysis to ensure a minimum structure service life of 75 years is provided. Issues affecting the service life include corrosive action of the exterior and interior environments and abrasive action of the hydraulic flow adjacent to the structure. Metal buried structures consisting of steel structural plate shall not be used in Marine and Non-Marine: Corrosive environments as defined in Section 6.7.1.

6.7.2.A Corrosion

The design service life analysis shall include a check on both the outside or backfill and the inside or water side environments to determine which governs.

The characteristics of the soil and water in contact with the structure that can contribute to corrosion include, but are not limited to, soluble salts, sulfates, soil and water resistivity, soil and water pH, and the presence of oxygen.

During design, corrosion investigations of in-situ soils, fill material (native or imported) to be used as backfill, soils/materials to be used inside the buried structure and any water flowing into or around the structure shall be performed. Soils shall meet the requirements of *Standard Specification* 6-20.3(6) and 6-20.3(9)A.

Soil and water sampling and testing procedures shall follow the requirements of the following or other approved test protocol:

1. WSDOT T 417 (resistivity and pH of soil and water)
2. CALTRANS Test 643 (resistivity of soil and water)
3. AASHTO T 288 (resistivity of soil)
4. AASHTO T 289 (soil pH)
5. AASHTO T 290 (sulfate in soil)
6. AASHTO T 291 (chloride in soil)
7. ASTM D 1293 (water pH)
8. CALTRANS Test 417 (soil and water sulfate content)
9. CALTRANS Test 422 (soil and water chloride content)

Water samples shall not be taken when the water level is elevated due to storm conditions as this may dilute chemical concentrations.

6.7.2.B Steel Structures

For galvanized steel buried structures, once the controlling pH and resistivity values of the environment are determined, the designer shall utilize the chart in Figure 6.7-1 to determine the estimated service life of the structure. The chart is based on 18 gage galvanized sheet steel. If a gage thicker than 18 is used, the thickness factor table within the chart can be used as a multiplier on the estimated service life.

Note: CALTRANS developed thickness modification factors up to 8 gage, which were then interpolated out to 1 gage by CONTECH Engineered Solutions. Thicknesses greater than 1 gage shall use the factors for 1 gage.

If the estimated service life is less than 75 years, additional thickness shall be added beyond what is required for structural demands to achieve the 75 year service life.

For aluminized steel buried structures, using aluminum-coated (Type 2) sheet steel in accordance with AASHTO M274, the service life and resistance to corrosion is generally more than galvanized steel structures. A similar chart and methodology can be used for aluminum coated structures and is found in Figure 6.7-2. The chart is based on 16 gage aluminized sheet steel. If a gage thicker than 16 is used, the thickness factor table within the chart can be used as a multiplier on the estimated service life. The service life is based on years to first perforation.

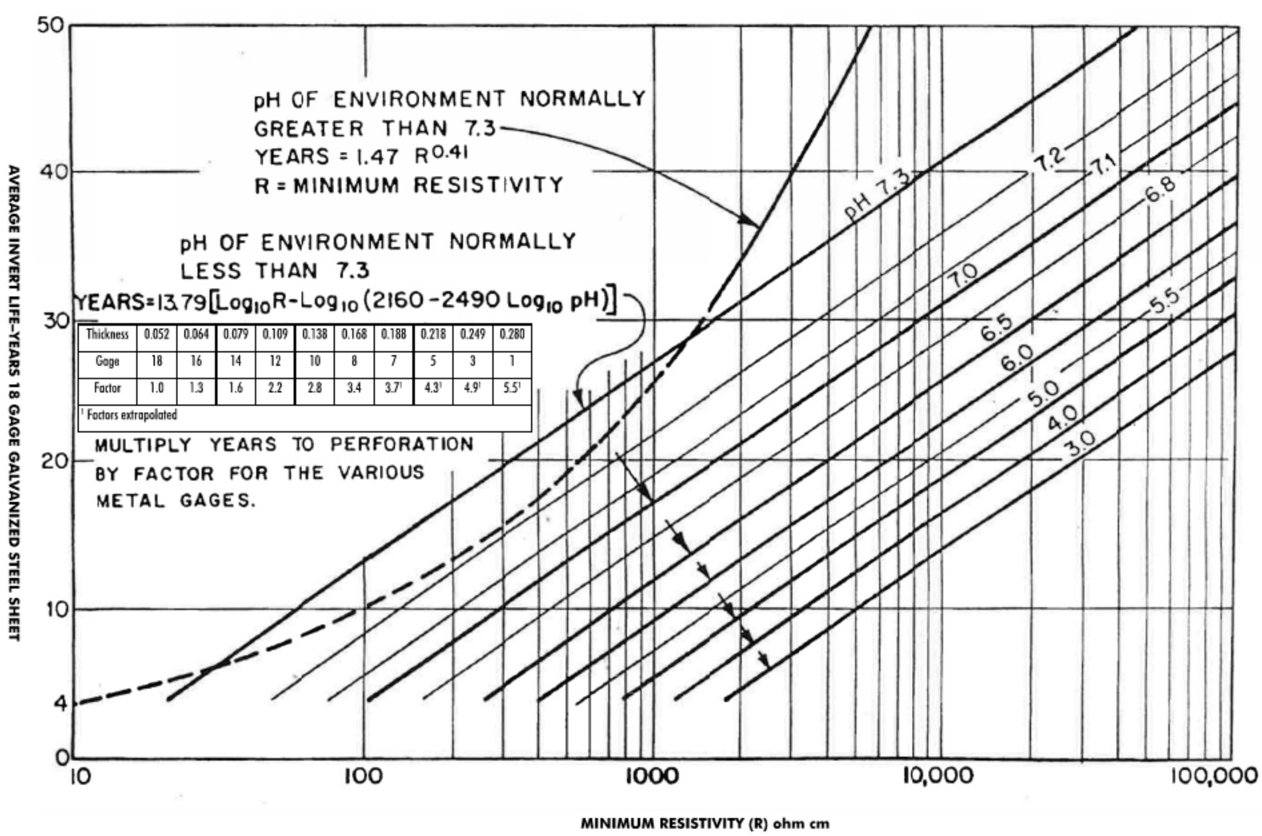
Aluminum-coated steel structures should generally only be used when environmental conditions have a pH between 5 and 9 and a resistivity greater than 1500 ohm-cm. Recent studies documented in the National Corrugated Steel Pipe Association (NCSPA) Pipe Selection Guide indicated that 14 gage aluminum-coated structures can achieve over 75 year service life in these environmental conditions. The WSDOT *Hydraulics Manual* also recommends aluminum-coated pipe only be used with pH between 5 and 8.5 and soil resistivity greater than 1000 ohm-cm. These recommendations are based on assumed 50 year service life.

6.7.2.C Aluminum Alloy Structures

Aluminum alloy plate has been shown to be more resistant to corrosion than either galvanized or aluminized steel. A chart for determining service life, similar to that for galvanized and aluminized steel, is located in Figure 6.7-3. The chart is based on 16 gage aluminum alloy plate and as with the aluminum-coated chart, the service life is based on years to first perforation. The chart was developed for aluminum pipe (Al-Clad 7072/3004) but is applicable to aluminum structural plate (5050 alloy).

Aluminum alloy plate structures should generally only be used when environmental conditions have a pH between 4.5 and 9 and a resistivity greater than 500 ohm-cm. There is no upper limit on resistivity as soft water is not a concern.

Figure 6.7-1 Chart for Estimating Average Invert Life for 18 Gage Plain Galvanized Buried Structures (CALTRANS *Highway Design Manual*/CONTECH Structural Plate Design Guide)



Thickness	0.052	0.064	0.079	0.109	0.138	0.168	0.188	0.218	0.249	0.280
Gage	18	16	14	12	10	8	7	5	3	1
Factor	1.0	1.3	1.6	2.2	2.8	3.4	3.7[1]	4.3[1]	4.9[1]	5.5[1]

[1] Factors extrapolated

Figure 6.7-2 **Chart for Estimating Average Invert Life for 16 Gage Aluminized Steel Buried Structures (Florida DOT Drainage Design Guide, 2019)**

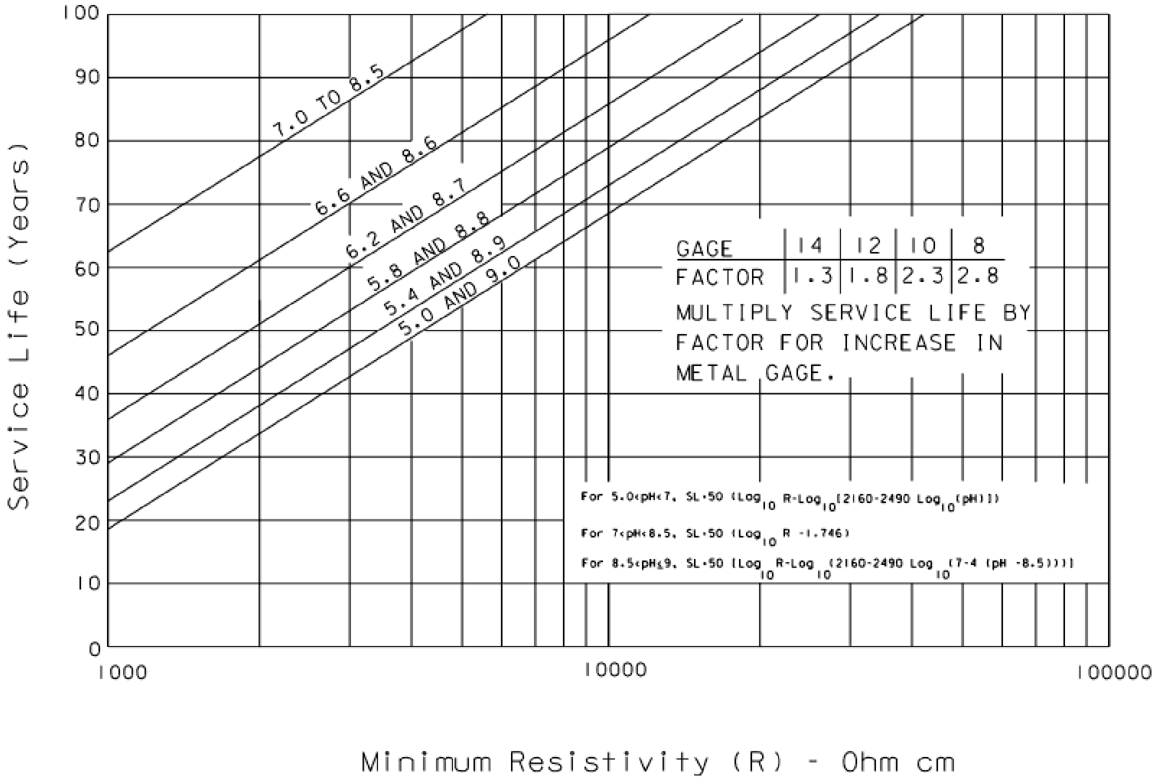
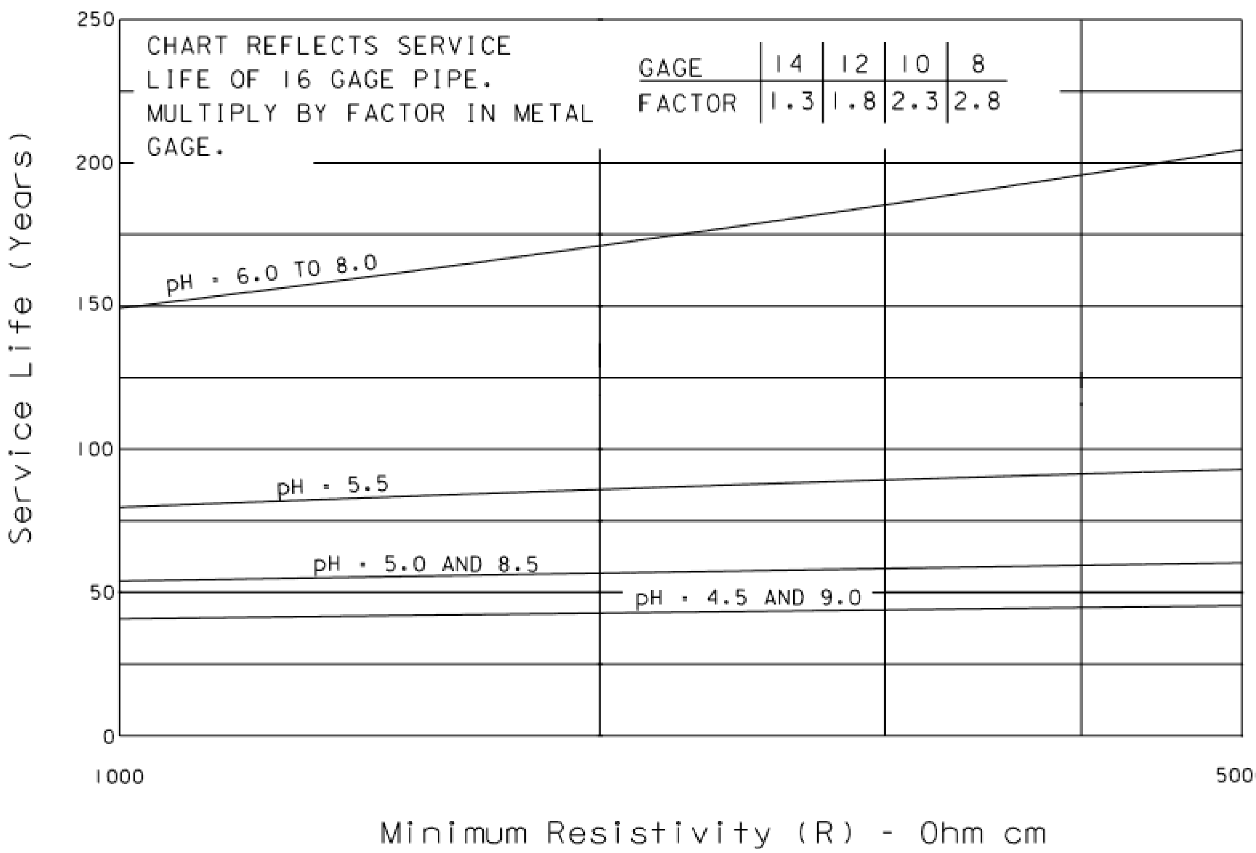


Figure 6.7-3 **Chart for Estimating Average Invert Life for 16 Gage Aluminum Alloy Buried Structures (Florida DOT Drainage Design Guide, 2019)**



6.7.2.D Alternative Coatings and Mitigation for De-icing Salts

Alternate coatings for buried metal structures, such as polymer, asphalt, concrete linings, and others are also available but are not discussed in detail in this section. The designer will need to evaluate the corrosion and abrasive potential and select the appropriate coating to achieve a 75 year service design life. If a coating is utilized, then the section loss corrosion and/or abrasion is assumed to begin at the end of the effective life of the coating. Any alternative coating selected shall be approved by the WSDOT Bridge Design Office.

When a metal buried structure supports a roadway, and the minimum Fill Depth, as defined in Section 8.3, is less than 8 feet, a protection against roadway de-icing salts and chlorides by one of the following methods shall be provided:

- Providing an impermeable geomembrane with welded seams in the backfill over the Buried Structure that is sloped to drain water away from the Structure. The membrane shall be a minimum 30 mil thick polyvinyl chloride, ethylene interpolymer alloy, or polyurethane polymer, or a combination of these polymers.
- Preventing Roadway drainage from entering into the fill above the Buried Structure.
- Providing additional metal plate thickness.

Regional WSDOT Maintenance staff shall be consulted to determine the type and frequency of de-icing salts used for the site specific location.

6.7.2.E Abrasion

In addition to service life analysis for corrosion, abrasion of the invert and side walls shall be investigated. The designer shall consider the potential for lateral stream migration when considering abrasion on side walls. Abrasion risk and mitigation shall be considered the same for galvanized or aluminized steel and for aluminum plate structures. Three factors that must combine to cause abrasion include:

1. Abrasive bed load
2. Sufficient stream velocity to carry the bed load
3. Flow duration and frequency

The WSDOT *Hydraulics Manual* M 23-03.06 (2019) provides guidelines for abrasion levels and the general site characteristics, which are described below:

1. Level 1 - Nonabrasive: areas of little or no bed load and very low velocities of 3 feet per second (fps) or less. Slopes are generally less than 1 percent. This condition can be assumed for the soil side of the structure.
2. Level 2 - Low abrasive: areas of minor bed loads of sand, silts and clays with velocities less than 6 fps and slopes generally range from 1 percent to 2 percent.
3. Level 3 - Moderate abrasive: areas of moderate bed loads of sand and gravel with stone sizes up to approximately 3 inches and velocities between 6 and 15 fps. Slopes generally range from 2 percent to 4 percent.
4. Level 4 - Severe abrasive: areas of heavy bed loads of sand, gravel, and rock with stone sizes up to 12 inches and larger and velocities exceeding 15 fps. Slopes are generally greater than 4 percent.

Stream velocities shall be based upon typical flows and water surface elevations for a 2-year event, minimum.

For Abrasion Levels 1 and 2, no additional protection is required.

For Abrasion Level 3, the thickness of the material shall be increased by one or two gages (approximately 1/64 inch) and/or a concrete lining shall be used. Alternatively, other protection such as streambed material or stilling basins are acceptable with the approval of the WSDOT Bridge Design Office.

For Abrasion Level 4, abrasion protection shall be provided such as increased gage thickness, alternative materials, coatings, concrete linings, etc. and shall require approval of the WSDOT Bridge Design Office.

6.8 Bridge Standard Drawings

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. *Steel Bridge Design Handbook* (February, 2022)

   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.

4. *Design of Welded Structures* by Omer H. Blodgett.

   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.

5. *Guide to Stability Design Criteria for Metal Structures, Sixth Edition* by Ronald D. Ziemian

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

8. *Steel Construction Manual*, Latest Edition, by American Institute of Steel Construction

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

9. *Machinery's Handbook*, Latest Edition by Industrial Press

   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.

12. Report No. WA-RD 876.1, Low Vertical Clearance Truss Bridges: Risk Assessment and Retrofit Mitigation Study, November 2017, WSDOT Bridge and Structures Office.