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INTRODUCTION

It is the intent of this chapter to establish the practices and specific requirements of the Structure and Bridge Division for the design and detailing of steel members. It will also provide design aids and other sources of information along with cross references to other Parts of this manual to assist in the design and preparation of plans.


The practices and specific requirements contained in this chapter have been established based on the Structure and Bridge Division's experience, industry standards and recommendations, and technological advancements made over the years.

The practices and requirements set forth herein are intended to supplement or clarify the requirements of the AASHTO Standard and LRFD specifications and to provide additional information to assist the designer. In the event of conflict(s) between the practices and the requirements set forth herein and those contained in the AASHTO Standard or LRFD specifications, the more stringent requirement shall govern.

This chapter in the manual contains specific requirements and/or guidelines for the detailing of various steel components and related areas. It is not the intent of these requirements and guidelines to supersede the requirements contained in Chapter 1 of this manual but to convey necessary information to the designer for the detailing of steel components.

Standard sheets and cell libraries dealing with details in this chapter can be found in the following Parts of this manual:

   Part 7: Steel Plate Girder Standards
   Part 8: Steel Beam with Timber Deck Superstructure Standards (SS-8)

Major change(s) and/or addition(s) to past practices are as follows:

   1. Revised camber diagrams.

NOTE:

Due to various restrictions on placing files in this manual onto the Internet, portions of the drawings shown do not necessarily reflect the correct line weights, line types, fonts, arrowheads, etc. Wherever discrepancies occur, the written text shall take precedence over any of the drawn views.
FRACTURE CRITICAL STRUCTURES:

Fracture critical structures/elements are undesirable due to a lack of redundancy and also because they require more frequent inspections under VDOT’s FHWA approved inspection program. Designers shall be familiar with and adhere to FHWA Memorandum HIBT-10, Clarification of Requirements for Fracture Critical Members, dated June 20, 2012.

Examples of fracture critical structures (vehicular or pedestrian) include, but are not limited to:

- Steel straddle bents
- Steel integral pier caps
- Two beam/girder systems
- Trusses (tubular and/or open shapes)

New fracture critical structures are to be used only when appropriate. Such requisite factors influencing the use of a fracture critical structure are:

- Geometrics
- Hydraulics
- Historical and/or Context Sensitive Solutions
- Average daily traffic
- Project cost

When the design year ADT is greater than 400, two beam/girder systems and trusses are not allowed. Two beam/girder configurations during phased construction are not considered fracture critical.

MATERIALS:

ASTM A709 Grade 50W shall be used for structural steel except as noted below.

ASTM A709 Grade HPS 50W shall be used for fracture critical members, except for trusses.

ASTM A709 Grade HPS 50W (except for fracture critical members) and HPS 70W shall not be used without a design waiver. Thus, use of a hybrid girder requires an approved design waiver.

Painted or galvanized ASTM A709 Grade 50 shall be used for trusses.

Structural pipe and tubing shall be galvanized inside and outside.

Structural steel in pedestrian bridges shall be painted.

A design approval is required to use steel in locations where uncoated weathering steel is not recommended according to FHWA Technical Advisory of Uncoated Weathering Steel in Structures (T 5140.22, dated October 3, 1989).

ASTM A709 Grade 50CR may be used in structures in accordance with the criteria and restrictions indicated on File No. 11.11-1.

ASTM A709 Grade 100/100W shall not be used.
HIGH STRENGTH BOLTS:

High strength ASTM F3125 A325 bolts shall be used. High strength ASTM F3125 A490 bolts shall not be used.

Bolts shall be ASTM F3125 A325 Type 3 when used with weathering steel. For all other steel types, use hot dip galvanized bolts ASTM F3125 A325 Type 1.

Where round head bolts are required, ASTM A449 bolts shall be used. Example: High strength bolts for rails on traffic side to avoid snagging.

Use 7/8" diameter bolts when bolt diameter is not specified.

ROLLED BEAMS:

Rolled beam use may be limited due to the availability of some sizes and are primarily used for widening projects or replacement of damaged members, but can also be used on new bridges.

COVER PLATES:

Welded cover plates on steel rolled beams shall be limited to one on any flange. Changes in areas of cover plates at welded splices shall be made by changing the thickness while maintaining the same width. Due to fatigue considerations, partial length cover plates shall be extended beyond the theoretical end of the cover plate by the terminal distance and no further than 3 feet from the centerline of bearing. Plates and welds shall be detailed with square ends (weld across the ends). The width of cover plates shall be 1 ½ inch less than the flange to which attached. Cover plates wider than the flange to which attached is not permitted. Minimum thickness of cover plates shall be 5/8”. Cover plates shall not be used in the negative moment regions.

Cover plates on plate girder flanges shall not be permitted.

FIELD CONNECTIONS:

All field connections, including splices, shall be bolted connections unless otherwise specified by the District Bridge Engineer.
COMPOSITE SECTIONS:

All simple spans and the positive moment regions of continuous spans shall be designed as composite sections and shall be proportioned so that the neutral axis lies below the top surface of the steel beam/girder at service load. Concrete on the tension side of the neutral axis shall not be considered in calculating stresses or resisting moments. The gross section of beam/girder and slab throughout the entire length of the beam/girder shall be used for the distribution of moments and computation of deflections.

In the negative moment region(s) of continuous spans, the longitudinal reinforcement shall be provided as required for a composite section; however, additional contribution to capacity of the longitudinal reinforcing steel shall be ignored for evaluating service or strength resistance. Minimum negative flexure concrete deck reinforcement shall be designed in accordance with AASHTO LRFD 6.10.1.7. The reinforcement used to satisfy the requirement shall extend into the regions specified by AASHTO LRFD 6.10.1.7 specified, or at least a tension development length past the point of the dead load contraflexure, whichever is greater. Dead load contraflexure points shall be established based on the entire deck dead load being in place.

No bolster (haunch) height shall be used in computing moments of inertia for the section(s) with the maximum top flange thickness. For section(s) where the top flange thickness has been reduced, the bolster height used in computing moments of inertia shall correspond to the difference in the maximum and actual plate thickness; however, the bolster itself shall not be considered for strength (width = 0) in computing moments of inertia.

CONSTRUCTABILITY:

The following Specification references provide some guidance and requirements to be considered by the designer when evaluating constructability:

1. AASHTO LRFD 2.5.3
2. AASHTO LRFD 6.10.3 – Constructability

Additionally the designer may consult the National Steel Bridge Alliance, Steel Bridge Design Handbook, for additional information on constructability.

For complex bridges and/or situations, the designer may need to show an erection/construction sequence for the bridge on the contract plans. As an example, to limit lane closures on an interstate due to construction of a new overpass, it may be desirable to field bolt and erect girder sections together leaving a cantilever extending past the pier and beyond the shoulder. Falsework should be indicated on the plans if needed for support.
PLAN ASSEMBLY:

A plan assembly with steel members (beams, girders, bents, columns, etc.) shall include the following, as a minimum:

**BEAM/GIRDER ELEVATION** sheet(s) detailing, as applicable, span length(s), plate sizes and lengths, weld sizes, range of tension members for Charpy V-Notch (CVN) requirements, spacing of stud shear connectors, location of bolted field splices and radius of beam/girder (for curved members). Details of diaphragms, cross frames, stiffeners (bearing and intermediate), connector plates, CVN requirements and fracture critical members shall be included either on this sheet or additional sheets.

**FRAMING PLAN** sheet(s) detailing, as applicable, plan layout of beams/girders, span length(s), skew angle(s), spacing of members, labeling of bearings, stiffeners (transverse, bearing), spacing and type(s) of cross frames (with connection plates shown) and location of bolted field splices.

**BOLTED SPLICE** sheet(s) detailing flange and web splices (splice plates, filler plates and spacing and arrangement of bolts).

**DEAD LOAD DEFLECTION** sheet(s), **CAMBER** diagram(s) and **BEAM/GIRDER ELEVATIONS** detailing the deflected shape of the steel member(s), dead load deflections and top of slab elevations along the top of the beams/girders, as applicable.

Detailed information and check lists for the above plan sheets are noted in the remainder of this chapter.
SPAN CONFIGURATION:

Simple Spans:

Multi-span bridges composed of steel simple spans are not allowed.

Continuous Spans:

Ratios of end span to interior span lengths between 1 to 1.2 and 1 to 1.4 generally result in the most economical use of structural steel.

Span arrangements that result in uplift shall not be used.

Curved Spans:

Horizontally curved (or curved) girder bridges are commonly used at locations that require complex geometries and have limited right-of-way, such as urban interchanges. Compared with straight steel girders, curved steel girders have greater effects of torsion, flange lateral bending, stability issues, and constructibility concerns. Curved girder bridges require special attention during design and construction.

AASHTO LRFD Bridge Design Specifications have provided a unified design approach for both straight and horizontally curved girders. Designers shall be adhere to the AASHTO Specs and the guidelines provided herein for design of curved girder bridges.

Geometric Design:

Geometric design standards often dictate the orientation of a bridge. In many cases, bridges must adapt to the highway alignment. However, bridge designers should take part in the roadway design in the scoping / preliminary design phase to mitigate the impact that curved alignments have on the design and construction of bridges, as well as safety of the bridge in service. The bridge designer should discuss, at least, the following items with the roadway designer:

- Move a curve off the bridge by adjusting roadway design criteria (such as design speed)
- Consider a different alignment
- Increase the radius of curvature at the bridge location if a curve cannot be avoided
- Use multiple supports/piers rather than a single span
- Eliminate spirals on bridges
- Avoid a reversed curve on the bridge
- Avoid a curved bridge with high skews (> 30°)

Curved bridges have higher cost, longer design and construction time, and higher risks of bridge failure during construction. Curved alignments may also cause more safety issues. For example, on horizontally curved structures, potential sight distance problems may occur due to bridge barriers / parapets or pedestrian fencing.

If an alignment requires a curved girder bridge, then the external longitudinal lines, traffic barriers, and fascia lines of the structure should follow the curved centerline to provide a smooth visual flow. Parallel lines should be maintained by matching barrier, sidewalk, curb and fascia depth across the structure. Curved beams/girders shall be parallel.
Curved girder bridges with skew greater than 30° shall not be used without an approval from the District Structure and Bridge Engineer.

When the roadway horizontal geometrics provide the opportunity to design and build a straight beam/girder structure, the designer shall investigate the cost effectiveness of doing so. The benefits in the design and construction should be considered. The roadway striping on the structure shall follow the curvature of the roadway.

Structural Design:

Analysis:

The AASHTO LRFD 4.6.1.2 provides guidelines for structures that are curved in plan. The moments, shears, and other force effects required to proportion the superstructure components are to be based on a rational analysis of the entire superstructure. Equilibrium of horizontally curved I-girders is developed by the transfer of load between the girders, thus the analysis must recognize the integrated behavior of structural components. Therefore cross frames are primary load-carrying members. Curved girder bridges shall be analyzed with a grid or 3D model.

Plans:

The AASHTO LRFD 6.7.2 specifies that the plans should state the fit condition for which the cross-frames or diaphragms are to be detailed for the following I-girder bridges:

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

where L is the span length bearing to bearing along the centerline of the bridge and R is the radius of the centerline of the bridge cross-section.

One feasible erection sequence shall be defined in the plans when the above conditions are met. Although it is not the responsibility of the designer to consider all potential conditions during the construction of the bridge, sufficient conditions should be considered during a study of the erection scheme to ensure that it is feasible.

Construction:

The geometry of horizontally curved girders generally leads to a de-stabilizing or overturning moment that needs to be considered in the plans. The overturning moment is the result of the offset of the center of gravity with respect to the axis of rotation. The axis of rotation is defined by a line connecting the end points at supports of a curved girder segment.
Due to torsional behavior during lifting of the girders and during erection, additional lifting points and temporary supports may be required to provide stability and deflection control.

In cases where overturning is a concern, intermediate points of support, such as shoring towers or holding cranes can be added to reduce the likelihood for such problems.

However, in any case, erecting a single curved girder will require significant bracing to ensure that the segment in question does not overturn.

The potential for overturning is dramatically reduced, and in most cases, eliminated, by erecting curved girder segments in pairs. As depicted in the following figure, erection of girders is to be performed by assembling and lifting pairs of girders with the cross frames between the girders bolted into place. In this case, the axis of rotation moves out to the line connecting the end points of support of the outermost curved girder segment.

![Diagram of girder erection](image)

The following FHWA publications may be used to aid in the design and construction of curved bridges.

- Guidance for Erection and Construction of Curved I-Girder Bridges, FHWA/TX-10/0-5574-1
- Engineering for Structural Stability in Bridge Construction, Publication No. FHWA-NHI-15-044

**Single Point Interchanges:**

Single Point Interchanges involve complex geometries, complicate rehabilitation, cannot be widened and require an approved design waiver before proceeding with preliminary design.
BEAM/GIRDER SPACING:

Studies show that the weight of structural steel per square foot of deck area decreases as beam/girder spacing increases. However, the designer should consider deck slab design, deck cracking, re-decking, stage construction, weight/size of individual beam/girder pieces, limit of the depth-to-span ratios and other factors for determining the beam/girder spacing.

The maximum beam/girder spacing shall be 12'-0". For splayed (variable spaced straight beams/girders), the maximum spacing is applied at mid-span, but the spacing at the widest splayed end shall not exceed 14'-0".

The deck slab overhang for the exterior beam/girder is dependent on these factors:

1. For overhangs exceeding 0.3 x beam spacing, a yield-line analysis is required.
2. If the exterior beam/girder controls the design, reduce the deck slab overhang to the point that the interior and exterior beam/girder design is nearly equal.
3. Check the space required for deck drains for conflicts with the location of the exterior beam/girder lines.
4. Aesthetic considerations.

The minimum deck overhang shall be 10" beyond the edge of the flange and the maximum overhang shall be 0.35 x the beam/girder spacing or 4'-0", whichever is less including where straight beams/girders are used on a curved alignment. Deck overhang and bridge railing/parapet geometry shall meet the layout requirements on File No. 10.01-6.

NUMBER OF BEAMS/GIRDERS:

Because of concerns for redundancy, bridges shall have a minimum of four beams/girders per span with the following exceptions:

- One lane bridges on low-volume (ADT < 400) roads where a minimum of three beams/girders may be used;
- Typically, pedestrian bridges use two-beam/girder systems and are fracture critical structures.

DEFLECTION LIMITS:

Deflection limits as noted in AASHTO LRFD 2.5.2.6.2 shall be adhered to.
SPAN TO DEPTH RATIO:

The first step in sizing the steel girder elements is to establish the web depth. The proper web depth is an extremely important consideration affecting the economy of steel girder design. The minimum depth shall meet the span-to-depth ratios in AASHTO LRFD 2.5.2.6.3 and Table 2.5.2.6.3-1 (see below). The minimum depth is not necessarily optimal. The optimum web depth can be established by preparing a series of designs with different web depths to arrive at an optimum cost-effective depth. Cost estimates should include substructure and roadway costs.

The thickness of the future wearing surface shall not be included.
MINIMUM WEB THICKNESS:

The minimum web thickness, $t_w$, shall be:

- $\frac{1}{2}"$ for girder depths $\geq 42"$
- $\frac{7}{16}"$ for girder depths $< 42"$ and when no transverse stiffeners are required. Otherwise, $\frac{1}{2}"$ is required.

Changes in the web thickness along the girder shall be made at field bolted splices.

The trade-off between adding more transverse stiffeners versus increasing the thickness of web material shall be investigated. As a rule of thumb, stiffeners can be considered to cost four times the material cost of web steel. The decrease in cost from eliminating stiffeners should be compared to the increase in cost for the thicker web. If transverse stiffeners are required, the following note shall be included on the beam/girder elevation:

The Contractor has the option of eliminating the transverse stiffeners by increasing the web thickness to ___ inch.

MINIMUM AND MAXIMUM FLANGE THICKNESS AND WIDTH:

The minimum thickness of any flange shall be $\frac{3}{4}"$. Thicknesses of flange plates shall be specified in multiples of $\frac{1}{8}"$ for plates 2" and under and multiples of $\frac{1}{4}"$ for plates over 2".

The minimum flange width shall be 12".

To minimize potential out-of-plane distortions, the flange width of the compression flange for each shipping piece shall satisfy $b_{fc} \geq L/85$ recommendation set forth in AASHTO LRFD C6.10.3.4.

Where:

- $b_{fc}$ = the width of the compression flange (inches)
- $L$ = the length of girder shipping piece (inches); which is the distance between adjacent bolted field splices; and also the distance from girder end to adjacent bolted field splice

When flange thickness and width transitions are necessary, keep flange widths preferably constant in a field section. For very long spans (> 200 feet) with thick flanges, a width transition may be appropriate in the negative moment region. Flange width changes shall preferably be made at a bolted field splice. Reduction of flange area shall not be more than one half the area of the heavier section at shop splices.
WEB HAUNCH INFORMATION:

The decision to use a haunched girder is usually driven by consideration of clearance requirements, economics, unbalanced span arrangement, and/or aesthetics. In cases where there is an underclearance or deflection problem, it may be beneficial to haunch the girders at interior piers.

The total angle at the point of haunch shall be between 135 and 160 degrees to prevent the appearance of too sharp a haunch at the bearing point.

The distance from the edge of the sole plate to the transition shall be a minimum of 12 inches in order to clear any distortion that may result from bending or welding of the flange plate and to accommodate future jacking needs.

Fish belly haunches shall not be used.

The depth of haunches shall be limited to twice the midspan depth. The length of haunches is preferred about 1/3 of the span. The haunch shall terminate prior to the bottom flange bolted splice plate.

Bolted field splices shall not be detailed in the variable depth regions. Haunches shall not be used for hybrid steel girders. Haunches shall not be used with horizontally curved girders.
SPLICES:

Splices for steel beams/girders can be field splices or shop splices. Field splices shall be bolted splices. Shop splices shall be welded splices. Bolted field splices are generally required for members that are too long to be transported to the bridge site in one piece or too heavy to be erected.

Changes in flange width should be at bolted field splices, which are also good locations to change flange thickness.

To avoid a penalty on the Moment Gradient Modifier, \( C_b \), locate the change in section within 20% of the unbraced length near the brace point with smaller moment. See AASHTO LRFD A6.3.3 and Appendix C6.4.10.

**Bolted Field Splices:**

For details and design, see File No. 11.05.

The designer shall investigate the feasibility of having the beam/girder segment(s) hauled to the project site and erected.

**Shop Welded Splices:**

The designer should maintain constant flange widths within a field section for economy of fabrication. No more than three plate sizes should be included in the top or bottom flange within a single field section. In determining the points where changes in plate thickness occur within a field section, the designer should weigh the cost of groove-welded splices against extra plate area.

To facilitate testing of the weld, web thickness transition or flange thickness transition shall be at least 2 feet away from web splices and at least 6 inch from transverse stiffeners.

Minimum flange plate length is 10 feet between transitions. The ratio of thickness shall be not more than 2:1.

A flange area ratio of 2 to 1 is preferred. Flange area ratios less than 1.5 to 1 are typically not economical. Use of the AASHTO/NSBA Steel Bridge collaboration document chart shown on the next page is recommended for evaluating the economy of splices.
SPLICES: (cont’d)

Weight Savings Factor Per Inch of Plate Width For Non-Fracture Critical Flanges Requiring Temperature Zone 1 Charpy V Notch (CVN) Testing

Multiply weight savings factor x flange width (length of butt weld) and compare to the actual pounds saved

<table>
<thead>
<tr>
<th>Thinner Plate at Splice (inches)</th>
<th>Thicker Plate at Splice (inches)</th>
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<tbody>
<tr>
<td>1.0</td>
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</tbody>
</table>

Notes:

1. Values in table shown for ASTM A709 Grade 50. Values for ASTM A709 36 and 50W are similar. Approximate cost increase for ASTM A709 Grade 70W over ASTM A709 Grades 36, 50 and 50W is 40% for plate thicknesses ≤ 2" and 60% for plate thicknesses > 2".

2. Weight factors for non-fracture critical Zone 2 materials are the same as for Zone 1, as shown, except that in the bold areas the factors should be reduced by 20%.

3. For compression flanges where CVN testing is not required, the factors should be increased by about 10%, except the bottom two rows should be increased by about 30%.

4. For fracture critical material, the factors should be reduced by values between 10% and 25% depending on thickness.

5. For intermediate thicknesses, interpolate between the closest values.

6. Where equal plate thicknesses are joined, table values indicate welded splice cost in terms of steel weight. Steel cost per pound is based on unfabricated steel plate, not the bid price of fabricated, delivered steel.
WELDED SPlice INFormation:

Field welded splices may be used for maintenance projects with approval of the District Bridge Engineer.

Notes:

1. Minimum dimension between flange splice and web splice recommended by AISC is 6".

2. Minimum dimension between flange splice and web splice to transverse stiffener is 6"

3. Minimum distance between web thickness transitions and flange thickness transitions shall be 2'-0".
STUD SHEAR CONNECTORS:

Shear connectors shall be used to achieve composite sections for steel beam/girder bridges with concrete deck.

Continuous composite beams/girders shall be designed with shear connectors throughout the entire length. As such, additional shear connectors at the contraflexure points are not required in accordance with AASHTO LRFD Article 6.10.10.

Use 7/8” diameter shear connectors.

Studs shall be at least 4 inches long. Heads shall project at least 2 inches above the plane of the bottom of the deck slab and shall be 3 inches below the plane of the top of the deck slab.

Current OSHA regulations require that shear connectors shall be installed (welded) in the field. Construction Specifications require fabricators to show location, spacing and height of stud shear connectors on working drawings.

Pitch of Shear Connectors:

The center to center pitch of shear connectors shall not exceed 24” including negative moment regions of continuous spans.

At simply supported ends (simple and continuous spans), connectors shall be as close as practical to end of the beam/girder (approximately 3”) and shall have a maximum pitch of 6” within a distance of 36” from the first row of shear connectors.

Transverse Spacing of Shear Connectors:

Shear connectors shall be placed parallel to main deck reinforcement.

The center of shear connectors shall not be less than 1½ inches from the edge of flange.
STIFFENERS:

See AASHTO LRFD 6.10.11 for stiffener design and detailing requirements.

The minimum thickness of stiffeners shall be ½".

Three different types of stiffeners are generally used for steel girder bridges: transverse, bearing, and longitudinal. Longitudinal stiffeners are not permitted.

Transverse stiffeners increase the shear resistance of a girder. They are aligned perpendicular to the top flange. Transverse stiffeners shall consist of plates or angles welded or bolted to either one or both sides of the web. If transverse stiffeners are used on one side, they shall be alternated about the web for the interior girders. Stiffeners in straight girders not used as connection plates shall be attached (welded) to the compression flange and tight fit to the tension flange, but need not be in bearing with the tension flange.

Use of transverse stiffeners should be limited to situations when they are economically practical. Designers should analyze several web thicknesses and stiffener arrangements to determine the most cost effective design.

For panel spacing, see AASHTO LRFD 6.10.9.

Bolted splices shall be considered equivalent to a transverse stiffener placed at the centerline of the splice.

Except as allowed on File No. 11.07-3 (rolled beams or girders 36" or less in depth), bearing stiffeners shall be perpendicular to the webs of plate girders.

Bearing stiffeners shall consist of one or more plates or angles welded to both sides of the web. Multiple bearing stiffeners are usually considered for cases where large horizontal thermal movements may occur (9 × t_w) or large bearings are needed. Additional stiffeners shall be spaced at 18 × t_w. The connections to the web shall be designed to transmit the full bearing force due to the factored loads. The stiffeners shall extend the full depth of the web and as closely as practical to the outer edges of the flanges.
STIFFENERS: (cont’d)

When multiple bearing stiffeners are required, provide 8” minimum spacing or 1½ times the bearing stiffener width, whichever is greater, for welding access. In the case of skewed bearing stiffeners, the spacing should be measured perpendicular to the bearing stiffeners.

Where multiple bearing stiffeners are required, the bearing sole plate size and required connection bolts may affect where stiffeners can be located. There must be sufficient room to install bolts and replace the bearing in the future, as well as to inspect the connection during routine maintenance activities.

When multiple bearing stiffeners are used along with cross-frames/diaphragms, the designer should evaluate the ability of the erector to install the cross frame. There may not be space for the cross frame to swing into place if multiple bearing stiffeners are present.
CROSS FRAME CONNECTOR AND STIFFENER DETAILS:

Notes to designer:

1. See Part 7 of this manual for standard sheets containing cross frame connector plate, transverse stiffener and bearing stiffener details.
2. Bearing and transverse stiffeners to be perpendicular to web (except as allowed on 11.07-3 for rolled beams or girders 36" or less in depth).
3. For details of cross frames and diaphragms, see File No. 11.07.
4. Cross frame / diaphragm connector plates shall be 7 ½” minimum width to allow for two rows of bolts in the connection. Minimum thickness shall be ½”.
5. Where both top and bottom flanges can experience tension, tight fit stiffener to both flanges.

WEB THICKNESS:

- ⅜, ⅝ and ⅝ 2⅛
- ¾ and ¾ 3
- ¾ and ¾ 4

Use single stiffeners alternating from side to side.
GENERAL INFORMATION:

This section of the chapter establishes the practices/requirements necessary for the completion of the framing plan sheet for a plan assembly. Included are sample framing plan sheets with a checklist for completing these sheets.

It is not the intent of the sample framing plan sheets and checklist contained in this section to show how to lay out the superstructure of a bridge.

A typical project will normally have a single framing plan sheet which will include notes. Space permitting, other details pertaining to the framing plan or steel beam/girder may be shown on this sheet. In all cases, the framing plan sheet shall contain all of the items shown in this section. Information placed in blocks on the sample framing plan sheets is for designer’s information only and are not to be placed on the framing plan sheet.

For major projects or long structures, the framing plan sheet may have to be extended to additional sheets to adequately show the complete structure. Designation of typical units shall be indicated on the title sheet (plan view).

The practices for the completion of interior sheets contained in Chapter 4 shall be adhered to.

CROSS FRAME / DIAPHRAGM LAYOUT:

If the skew is 20 degrees or less, the cross frames/diaphragms and connector plates shall be parallel with the supports. This arrangement permits the cross frames/diaphragms to be attached to the girders at points of equal stiffness, thus reducing the relative deflection between cross frame/diaphragm ends and thus, restoring forces in these members. See File No. 11.03-2 for sample sheet.

For skews greater than 20 degrees, the cross frames/diaphragms shall be placed perpendicular to the girders. Typically, they are placed in a contiguous line with the cross frames/diaphragms matched up on both sides of the interior girders. This arrangement provides the greatest transverse stiffness. See File No. 11.03-3 for sample sheet.

For curved steel bridges, the intermediate cross frames/diaphragms shall be radial and support cross frames/diaphragms shall be parallel to the supports. See File No. 11.03-4 for sample sheet.

An option to consider where supports are skewed more than 20 degrees is to remove highly stressed cross frames/diaphragms, which typically results in discontinuous cross frame/diaphragm lines, or lines that do not form a continuous line between multiple girders. Removal of highly stressed cross frames/diaphragms, particularly at obtuse corners near supports (see AASHTO LRFD C6.7.4.2), releases the girders torsion and is often beneficial in reducing the overall transverse stiffness of the bridge superstructure, along with the restoring forces in the remaining transverse bracing members, as long as the girder twist is not excessive. See File No. 11.03-5 for sample sheet.

End cross frames/diaphragms are required with all abutment types except full integral. For full integral abutments, end cross frames/diaphragms may be used at the designer’s discretion if needed for stability during construction. Cross frames/diaphragms are required at piers in all cases.
SAMPLE FRAMING PLAN FOR STEEL PLATE GIRDERS WITH TRANSVERSE STIFFENERS AND SKEW ≤ 20°
For bolted splice details, see sheet 13.

For girders details, see sheet 13.

Sample Framing Plan for Steel Plate Girder
Without Transverse Stiffeners and Skew > 20°
Dimensions shown in the tables are measured along the girder L.
SAMPLE FRAMING PLAN FOR STEEL PLATE GIRDER
WITH VIRGINIA PIER CAP AND SKEW ≤ 20°
CHECK LIST FOR SAMPLE FRAMING PLAN SHEET:

1. Framing plans shall be drawn to a scale of sufficient size to fit the full size sheet and be legible when reduced to half-size. Drawings drawn to a scale other than those listed in File No. 01.04 shall be indicated as not to scale.

2. Show skew angle(s) if applicable. For a 0° skew, show as 90° to L / R.

3. Label L / R of roadway. This designation should match that shown on the title sheet.

4. At abutments, label line thru center of bearings. Provide station.

5. Label L pier and line thru center of bearings if applicable. Provide station.

6. Dimension span length(s) and label span(s).

7. Label girders.

8. Dimension girder spacing.

9. Dimension the spacing of cross frames.

10. Dimension the spaces of stiffeners.

11. Label the bearing stiffeners or transverse stiffeners.

12. Label the cross frames.

13. Label the cross frame connector plates.

14. Dimension location(s) of the bolted field splices.

15. Label bolted field splices.

16. Label radial if applicable.

17. Show the radius of the horizontal curve if applicable.

18. Show and label PCC, POC, POT, etc. if applicable (e.g. curved girder).

19. Show and label utility line if applicable.

20. Show dimensions in a table if applicable (e.g. curved girder).

21. Label FRAMING PLAN.


23. For instructions on completing the title block, see File No. 03.03.

24. For instructions on completing the notes, see File Nos. 04.03-1 and -2.
CHECK LIST FOR SAMPLE FRAMING PLAN SHEET: (cont’d)

25. For instructions on completing the project block, see File No. 04.01.

26. For instructions on developing the CADD sheet number, see File Nos. 01.01-6 and 01.14-4.

27. For instructions on completing the block for sealing, signing and dating this sheet, see File Nos. 01.16-1 thru -5.

28. Show dimensions along the line thru center of bearings.

29. For interior girders, connection plates shall be place in pairs (even when there is no corresponding cross frame in the adjacent bay).
GENERAL INFORMATION:

This section of the chapter establishes the practices/requirements necessary for the completion of the girder elevation sheet for a plan assembly. Included are sample girder elevation sheets with a checklist for completing these sheets.

A typical project will normally have a single girder elevation sheet which will include notes. Space permitting, other details pertaining to the girder elevation or steel beam/girder may be shown on this sheet. In all cases, the girder elevation sheet shall contain all of the items shown in this section. Information placed in blocks on the girder elevation plan sheets is for designer’s information only and are not to be placed on the framing plan sheet.

For major projects or long structures, additional girder elevation sheet(s) may have to be added to adequately show the complete structure.

The practices for the completion of interior sheets contained in Chapter 4 shall be adhered to.

Charpy V-Notch Requirements/Fracture Critical Members:

AASHTO LRFD Bridge Design Specifications require all primary longitudinal superstructure components and connections sustaining tensile stress due to Strength Load Combination I and transverse floorbeams subject to such stresses shall require mandatory Charpy V-Notch testing. Tension members in trusses, cross frames in curved steel bridges and other primary components in steel bridges shall require mandatory Charpy V-Notch testing.

The FHWA Memorandum of Clarification of Requirements for Fracture Critical Members, HIBT-10, dated June 20, 2012, shall be complied with for determining the requirements for FCMs. For internal staff, the memo is available at the following link in the memorandum by others folder:


AASHTO LRFD 6.6.2 requires that components subject to tensile stress be shown on the plans. The designer shall determine which, if any, component is a fracture critical member (FCM). FCMs shall be clearly delineated on the plans.

See examples of notes required on beam/girder elevation sheet(s) for Charpy V-Notch on next page.
Charpy V-Notch Requirements/Fracture Critical Members: (cont’d)

**EXAMPLE 1**: Simple span rolled beam without cover plates

Note: The bottom flange and web are areas of tensile stress for Charpy V-Notch impact requirements.

**EXAMPLE 2**: Simple span rolled beam with cover plates

Note: The bottom flange including cover plates and web are areas of tensile stress for Charpy V-Notch impact requirements.

**EXAMPLE 3**: Simple span plate girder

Note: The bottom flange and web are areas of tensile stress for Charpy V-Notch impact requirements.

**EXAMPLE 4**: Simple span plate girder/rolled beam with bolted field splice

Note: The bottom flange, web and all splice plates are areas of tensile stress for Charpy V-Notch impact requirements.

**EXAMPLE 5**: Continuous plate girder with splice at dead load point of contraflexure. For splices at other locations note shown below will be applicable (rolled beam similar).

Note: The top and bottom flanges as shown, the web, and all splice plates are areas of tensile stress for Charpy V-Notch impact requirements.
CHECK LIST FOR SAMPLE GIRDER ELEVATION SHEET:

1. GIRDER ELEVATION shall be drawn to a scale horizontally and proportional vertically.
2. Label Line thru center of bearings at Abutment A or Abutment B.
3. Label Line thru center of bearings and Pier 1 or Pier 2.
4. Dimension span length(s). Dimension girder lengths past lines thru center of bearings.
5. Dimension and label tension flange bottom.
6. Dimension and label tension flange top.
7. Dimension length of plates.
8. Label top and bottom flange plates.
9. Dimension location(s) of bolted field splice.
10. Label bolted field splice.
11. Dimension spacing of stud shear connectors.
12. Show and label number of rows and size of stud shear connectors.
13. Show and label type and size of welds.
15. Show table values where applicable. Remove unused columns.
16. Label GIRDER ELEVATION. Add horizontal scale.
17. Show details of bearing stiffeners.
18. Show detail of transverse intermediate stiffener.
19. Show cross frame connector plate.
20. Show typical beam end detail.
21. For instructions on completing the title block, see File No. 03.03.
22. For instructions on completing the notes, see File Nos. 04.03-1 and -2.
23. For instructions on completing the project block, see File No. 04.01.
CHECK LIST FOR SAMPLE GIRDER ELEVATION SHEET (cont’d.):

24. For instructions on developing the CADD sheet number, see File Nos. 01.01-7 and 01.14-4.

25. For instructions on completing the block for sealing, signing and dating this sheet, see File Nos. 01.16-1 thru -6.

26. For curved and/or skewed bridges, add the following to Notes:
   Fabricator to detail the girders so that the webs are plumb under steel dead load at supports.
BOLTED SPLICE CONNECTIONS:

1. Applicable section of AASHTO LRFD is 6.13.

2. For curved girders, the girder sweep plus the flange width shall not exceed 6 feet for ease of shipping. The current legal vehicle width is 8’-6” without a permit. Limiting the overall shipping width of curved girders to 6 feet permits fabricators to offset the girder on the trailer.

3. Bolted splices and connections shall be detailed for standard holes but shall be designed for oversize and short slotted holes ($K_h = 0.85$) and for Class B surface condition ($K_s = 0.50$).

4. Bolted web splices shall be considered equivalent to a transverse stiffener placed at the center of the splice.

5. In continuous spans, splices should be made at or near points of dead load contraflexure. Web and flange splices in areas of stress reversal shall be investigated for both positive and negative flexure. For simple spans, the splices shall be made to maximize the flange thickness transition.

6. Minimum distance between the end of splice plates and transverse stiffeners and connection plates shall be 6 inches.

7. Where an option is noted on the plans allowing a thicker web (steel plate girders) to eliminate transverse stiffeners, no change in the bolted web splice will be made for the thicker web.

8. All bolted splices shall be designed as slip critical.

9. Bolted splices shall be symmetrical about the splice.

10. All flange splices shall include inside and outside splice plates.

11. When the width of flanges being spliced differs by more than 2 inches, the larger flange shall be beveled. If a flange width transition occurs at the bolted splice, size the flange splice plate to the smaller width.

12. Filler plates shall not extend beyond the splice plate. Minimum filler plate thickness shall be $\frac{1}{4}$”.

13. Any reduction factor, $R$, used based upon thickness of filler plates shall be applied to both inside and outside flange splice plates.

14. A minimum of two rows of bolts shall be used on each side of the splice for both flange and web splices.

15. Design bolts for shear assuming that threads are not included in the shear plane.

16. Staggered bolt patterns are preferred in the flange splice to maximize the net section.
BOLTED SPLICE CONNECTIONS: (cont’d)

17. For flange splices the first set of bolts on each side of the splice shall be a minimum of 2 inches from the centerline of the splice. For web splices the first set of bolts on each side of the splice shall be a minimum of 1 ¾” from the centerline of the splice.

18. Outside splice plates of bolted flange splices shall match the width of the narrower flange plate at the splice. The center of gravity of the gross area of the inside and outside splice plates shall be as close to the center of gravity of the thicker flange as possible.

19. The minimum thickness of web splice plates shall be $\frac{5}{16}"$. Minimum thickness of flange splice plates shall be $\frac{3}{8}"$.

20. When minimum criteria control the splice design, the gross area of the splice plates shall equal or exceed 75% of the gross area of the controlling flange for redundant members and 100% of the gross area of the controlling flange for non-redundant members. Bolted splices of all steel members shall be determined in accordance with AASHTO LRFD Section 6, except that for non-redundant members splices shall be designed for 100% of the member capacity at the spliced location.

21. For curved girders, place the following note on the plans: “Oversized or slotted holes shall not be permitted.”

22. Designer shall check bolt interferences between web and flanges splice bolts using the information on the next page.
BOLTED SPLICE CONNECTIONS: (cont’d)

Bolts are normally installed with nut down in the top flange (splice) and nut up in the bottom flange (splice). Assume \( \frac{1}{2} \)" extension of bolt threads beyond face of nut.

High Strength, ASTM A325 Bolts, \( \frac{7}{8} \)" dia.

Bolt Head: \( x = 1 \frac{7}{16} \), thickness = \( \frac{35}{64} \)
Nut: \( x = 1 \frac{7}{16} \), thickness = \( \frac{55}{64} \)
Washer: \( \Phi = 1 \frac{3}{4} \), thickness = 0.177
Designer should check clearance between impact wrench for tightening splice bolts and top head of shear studs. For a $\frac{7}{8}$" Ø stud shear connector, the top head is $1 \frac{3}{8}$". For impact wrench, assume a 3" dia. for a heavy-duty socket.
Typical splice detail below is for an example only. The top flange is shown with a uniform pattern; the bottom flange is shown with a staggered pattern.

Notes to designer:
1. Show top and bottom views only if flanges and/or bolt patterns are different.
2. Do not show edge clearances of bolts unless other than standard.
3. Do not show length of plates.
4. Dimensions indicated by * are minimum to be used.
5. Size of flange plates and stresses will determine whether uniform or staggered bolt pattern is required.

BOLTED SPLICE DETAILS
NOTES:

Steel rolled beams and plate girders shall be cambered in accordance with VDOT Road and Bridge Specifications, Section 407.04.

Bolster thickness is normally set so that the top of the beam/girder web is parallel with the grade. If bolster over 4" are used, the bolster needs to be reinforced; see File No. 10.02-1.

DEFINITIONS:

C = total camber required for beam/girder = \( \Delta_s + \Delta'_s + \Delta_c + \text{V.C.C.} \)

\( \Delta_s \) = deflection of girder from its own weight after erection including diaphragms or cross frames, connectors, etc.

\( \Delta'_s \) = deflection of girder from dead load of concrete deck slab, bolster and construction tolerance

\( \Delta_c \) = deflection of girder from permanent dead load added after deck slab is cast (e.g., parapet), future wearing surface not included

V.C.C. = vertical curve camber = distance between the reference line and top of web after full dead load deflection (i.e., V.C.C. is the vertical curve ordinate at any point being considered, therefore finished grade elevations along C/L of each girder may be used to calculate V.C.C.). V.C.C. is positive if the top of web is above the reference line, and negative if the top of web is below the reference line. V.C.C. is zero if the entire bridge is in a gradient.

Reference line = line used on camber diagram standards in Part 7 = line between top of web at C/L of bearing at the beginning of the continuous girder unit and top of web at C/L of bearing at the end of the unit (usually this is abutment-to-abutment). The reference line is a straight line, but not necessarily a horizontal line (unless the elevations of top of web at C/L bearing are the same at the beginning of the unit and end of the unit).

CT = camber tolerance (positive numerical value)

\( \sum D \) = girder depth tolerance + (cross slope x \( \frac{1}{2} \) max. flange width) + (cross slope x horizontal curve ordinate) + \( \frac{1}{4} " \)

B = thickness of bolster at C/L girder over the thickest top flange

\( B \geq CT + \sum D \), but not greater than (\( CT + \sum D + 1" \))
DEFINITIONS (Cont’d):

Camber Tolerance (CT) for computation of bolsters only (simple and continuous spans):

\[
\text{Rolled beam} = + \frac{1}{8}" \times \frac{\text{span length(feet)}}{10}
\]

\[
\text{Plate girder} = + \frac{1}{16}" \times \frac{\text{span length(feet)}}{10} \quad \text{or} \quad \frac{3}{4}" \quad \text{whichever is greater}
\]

In setting bolster at interior piers, use larger of the two adjacent span lengths for computing CT.

Depth tolerance:

\[
\text{Rolled beam} = \frac{1}{4}"
\]

\[
\text{Plate girder} = \frac{1}{8}" \quad \text{for depth} \leq 36"; \quad \frac{3}{16}" \quad \text{for depth} > 36" \quad \text{but} \leq 72"; \quad \frac{5}{16}" \quad \text{for depth} > 72"
\]

TOP OF DECK ELEVATIONS and DEAD LOAD DEFLECTIONS

Elevations and deflections shall be given at tenth points of each span along C/L girders using the SGDLD standards and Notes to Designer in Part 7.

See File No. 11.06-6 for sample sheet for Dead Load Deflections and Top of Deck Elevations.

CAMBER

Camber values shall be given at tenth points of each span using the SGCAM standards and Notes to Designer in Part 7.

Deflected shape on the SGCAM standards is shown for a hump vertical curve. These standards are intended to be generic, therefore it is not necessary to modify the shape for bridges in a sag vertical curve or gradient. Since the generic diagram is intended for all situations (hump, sag, gradient, combinations of these), the sign convention given on the standard is important. Fabricators have been consulted in the development of the SGCAM standards, and have confirmed the practice of using a generic diagram with this sign convention.

See File Nos. 11.06-7 thru -9 for a sample Camber Diagram sheets. The examples are for hump vertical curve, sag vertical curve and a gradient. The purpose is to illustrate the difference in V.C.C. for these scenarios. For the hump vertical curve example (11.06-7), V.C.C. is based on the top of slab elevations from File No. 11.06-6.

File No. 11.06-10 shows a sample Camber Diagram for stage construction. The bridge has 10 steel girders, 6 in Stage 1 and 4 in Stage 2. The closure pour between the two stages has construction joints on Girder 6 and 7. In such cases, the deflection due to the closure pour should consider the stiffness contribution of the deck slabs that have been placed in Stage 1 and 2.
BOLSTER:

SIMPLE SPAN
(Diagram shown for simple span. Continuous spans similar)

Refer to diagram above and the Elevation diagram on the following page.

\[ h = B \text{ (as defined on 11.06-1) } + \text{ thickest top flange in span(s)} \]

\[ h = \text{ bolster at any point along girder } + \text{ corresponding top flange thickness} \]

h is used for setting seat elevation.

h is constant in order to keep top of web parallel to the grade. Bolster thickness along the girder changes as the top flange thickness changes.

See File Nos. 11.06-4 and -5 for sample calculations for bolster and h.
DESIGN EXAMPLE FOR COMPUTING MINIMUM BOLSTERS:

**Given:** Curved alignment - straight girders (laid out on long chord)
- Cross slope: \( \frac{1}{4} \)" per foot
- Horizontal curve ordinates, Pier 1 = 1' - 2 \( \frac{1}{4} \)", Pier 2 = 1' - 1 \( \frac{1}{8} \)"
- Slab = 8 \( \frac{1}{2} \)" min
- Flange sizes shown at piers only

Span layout as follows:

**ELEVATION**

**PLAN**
DESIGN EXAMPLE FOR COMPUTING MINIMUM BOLSTERS: (cont’d)

Required: Compute minimum bolster B and “h” which is needed for setting seat elevations.

Spans a and b at Pier 1:

\[ CT = \frac{1}{16} \times (150' \text{ span} / 10) = 0.9375'' \]

\[ \Sigma D: \text{girder depth tolerance (maximum depth < 72'')} = \frac{3}{16}'' = 0.1875'' \]

\[ \text{cross slope x } \frac{1}{2} \text{ flange width} = (0.25'' / 12) \times (18'' / 2) = 0.1875'' \]

\[ \text{cross slope x horizontal curve ordinate} = (0.25'' / 12) \times (1' - 2 \frac{1}{4}'') = 0.2969'' \]

\[ \Sigma D = 0.9219'' \]

\[ B_1 \geq CT + \Sigma D = 0.9375'' + 0.9219'' = 1.8594'' \]

use \( B_1 = 1 \frac{7}{8}'' \)

Bolster at Pier 2 was also checked, but it did not control.

So \( h = 1.875'' (B_1) + 2.375'' \text{ (thickest top flange)} = 4.625'' \)

This value of h is used for setting seat elevations at all support locations.

Note that bolster thickness along the girder changes when the top flange thickness changes. For example:

Bolster at Pier 2, \( B_2 = h - \text{(top flange thickness at Pier 2)} = 4.625'' - 2.5'' = 2.125'' \)
DEAD LOAD DEFLECTIONS

**Notes:**
- For center diagram, see Sheet 13.
- For girder details, see Sheet 12.

### Structural Engineer
Richmond, VA
VDOT S&B Division

### DESIGN AND SLAB ELEVATIONS

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**TOP OF DECK ELEVATIONS ALONG GIRDER**

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**DEAD LOAD DEFLECTIONS**

- \( \Delta_w \): Deflection of girder from dead load of concrete deck slab, bolster and construction tolerance.
- \( \Delta_d \): Deflection of girder from permanent dead load added after deck slab is cast (e.g., parapet).

---

**FILE NO. 11.06-6**
**SHEET 6 of 10**
**DATE: 30 Apr 2020**
**PART 2**
CAMBER DIAGRAM

Reference line - line between top of web at abutment A and top of web at abutment B (C bearing to C bearing).

Δs - Deflection of girder from its own weight after erection including diaphragms, connectors, etc.
ΔL - Deflection of girder from dead load of concrete deck slab, bolster and construction tolerance.
V.C.C. - Vertical curve camber - Distance between the reference line and top of web after full dead load deflection.

Total camber = ∑ Δs + ΔL + V.C.C.

Sign convention: Deflections are positive if downward, negative if upward.

V.C.C. is positive if the top of web after full dead load deflection is above the reference line, and negative if below the reference line.
Total camber is positive if the top of web as fabricated is above the reference line, and negative if below the reference line.

Diagram depicts all values being positive, and is not meant to reflect actual conditions.
### Camber Diagram

Reference line: The line between the top of web at abutment A and top of web at abutment B (C bearing to C bearing).

- \( \Delta_s \) = Deflection of girder from its own weight after erection including diaphragms, connectors, etc.
- \( \Delta_g \) = Deflection of girder from dead load of concrete deck slab, bolster and construction tolerance.
- \( \Delta_p \) = Deflection of girder from permanent dead load added after deck slab is cast (e.g. parapet).
- \( \Delta_{V.C.C.} \) = Deflection of girder from its own weight after erection including diaphragms, connectors, etc.

V.C.C. = Vertical curve camber = Distance between the reference line and top of web after full dead load deflection.

**Total camber**: Positive if the top of web as fabricated is above the reference line; and negative if below the reference line.

**V.C.C.**: Positive if the top of web after full dead load deflection is above the reference line; and negative if below the reference line.

**Sign convention**: Deflections are positive if downward, negative if upward.

Diagram depicts all values being positive, and is not meant to reflect actual conditions.

---

**Sag Vertical Curve**

For top of slab elevations, see Sheet 14.

For girder details, see Sheet 12.

---

**Sample Camber Diagram Sheet**

Not to scale

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File No. 11.06-8

Sheet 8 of 10

DATE: 30Apr2020

PART 2
For top of slab elevations, see Sheet 14.

Notes:

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STRUCTURAL ENGINEER
VDOT S&B DIVISION
RICHMOND, VA

Reference line = line between top of web at abutment A and top of web at abutment B (C bearing to C bearing).
Total camber = V.C.C = Vertical curve camber = Distance between the reference line and top of web after full dead load deflection.

Sign convention:
1 and 4 Girders
2 and 3 Girders

'c' at abutment A of bearings
Line thru center

'c' s at abutment B
of bearings

's' = Deflection of girder from permanent dead load added after deck slab is cast (e.g. parapet).
'c' = Deflection of girder from dead load of concrete deck slab, bolster and construction tolerance.
'c' = Deflection of girder from its own weight after erection including diaphragms, connectors, etc.

V.C.C. is positive if the top of web after full dead load deflection is above the reference line; and negative if below the reference line.
Deflections are positive if downward; negative if upward.

Diagram depicts all values being positive, and is not meant to reflect actual conditions.

SAMPLE CAMBER DIAGRAM SHEET

GRADIENT

CAMBER DIAGRAM

CAMBER DIAGRAM
CAMBER DIAGRAM

Reference line = line between top of web at abutment A and top of web at abutment B (C bearing to C bearing).

$\Delta_{x}$ = Deflection of girder from its own weight after erection including dead load, construction tolerances, etc.

$\Delta_{d}$ = Deflection of girder from dead load of concrete slab, bolster and construction tolerances.

V.C.C. = Vertical curve center = Distance between the reference line and top of web after full dead load deflection.

Total center = $V.C.C. - \Delta_x - \Delta_d + \Delta_{x0}$

Sign convention: Deflections are positive if deflection is away from the reference line.

V.C.C. is positive if the top of web after full dead load deflection is above the reference line.

Total center is positive if the top of web is raised above the reference line.

Diagram depicts all values being positive, and is not meant to reflect actual conditions.

CAMBER DIAGRAM SHEET
STAGED CONSTRUCTION WITH CLOSURE POUR HUMP VERTICAL CURVE
Not to scale
©2004, Commonwealth of Virginia

CAMBER DIAGRAM
GENERAL INFORMATION:

Cross frames and diaphragms shall be used to provide discrete bracing of rolled beams and plate girders. It is the designer’s responsibility to design the cross frames, including location, member sizes, weld lengths, location and geometry of bolt groups, size of bolts, gusset plate sizes, connection plate sizes, etc. The cross frame standards in Part 3 provide the configuration and minimum size sections for straight girders with skew angle ≤ 30°, girder spacing up to 12'-0" and cross frame spacing ≤ 25'-0". It is the designer’s responsibility to size the members appropriately. Grid analysis or 3D analysis is required for curved girders or highly skewed bridges. In these situations, the cross frames shall be designed as primary members and carry the loads based on the analysis.

Intermediate cross frames/diaphragms shall be placed level. Cross frames/diaphragms placed at support locations (abutments and piers) shall be placed parallel to the deck.

Diaphragms:

Diaphragms may be used on plate girders with a depth of 36" or less and rolled beams. Diaphragms may be coped and bent or may be used with bent gusset plates when diaphragms and bearing stiffeners are not aligned.

Cross Frames:

1. Angles shall be equal leg angles. Angle legs shall be oriented with the vertical leg below the other leg.
2. Welds of individual cross frame members to gusset plates or each other shall be made on 3 sides as shown in Detail C. Weld length on each side shall be at least four times weld size and in no case less than 1.5 inches.
3. The minimum weld shall also be based on capacity or fatigue considerations, whichever controls.
4. In checking slenderness, the effective length factor for single angles shall be taken as k = 1.0 and radius of gyration for single angles shall be checked using \( r_z \).
5. For interior beams/girders, connection plates shall be placed in pairs (even when there is no corresponding cross frame in the adjacent bay). See check list item 29 on 11.03-5.
6. All connection plates shall be welded to the web and both flanges.
Designed by Line Girder Analysis:

Line girder analysis may be used for straight girders with 30° skew or less. In these situations, cross frame elements (chords and diagonals) are considered to be secondary members. Gravity axes of diagonals and chords do not need to intersect. Minimum angle size shall be L6 x 6 x 3/8 for chords and L5 x 5 x 3/8 for diagonals.

Designed by Grid or 3D Analysis:

Grid analysis or 3D analysis is required for curved girders or straight girders with greater than 30° skew. Cross frame or diaphragm members (equal leg angles, channels, WT’s, etc.) in curved girder or highly skewed bridges are primary members. Most analysis programs treat the cross frame members as truss members (includes DESCUS.) If frame action is considered, forces may not be applied except at the centerline of the web. With this issue in mind, the following restrictions are placed on the geometry of cross frames for design and detailing in plans:

1. The centers of gravity axes (C.G.) of intersecting cross frame members shall intersect at the centerline of the web. The C.G. of the chord on either side of one girder shall intersect at a point on the centerline of web or the girder shall be designed to carry the moment at that location.

2. Cross frames shall be connected to the connection plates using gusset plates. No members of the cross frames should be directly connected to the girder.

3. The ends of angles shall be welded or bolted to gusset plates.
Notes to designer:

1. These details may be used for plate girders with a maximum depth of 36" and rolled beams.
2. Skew bearing stiffeners to 20°. For skews to 20°, use 5/16" fillet weld both sides. Over 20°, use single bevel groove weld. The groove weld is to run the full length of the plate.
3. Diaphragm connections are to be detailed as bolted connections.
Notes to designer:

1. These details may be used for plate girders with a maximum depth of 36" and rolled beams.
2. Two spaces at 3" for 24" and smaller beams. Three spaces at 3" for beams/girder greater than 24".
3. Bolt spacing is based on 7/8" dia. bolts and applies regardless of skew.
4. Diaphragms shall be normal to the beams when the supports are skewed more than 20°.
Cross Frame Detail for Staged Construction:

For staged construction bridges, because of differential deflections, the cross frames between the construction stages need special design. The following detail may be used with appropriate modifications.

The following detail was based on the deflection in File No. 11.06-10. The designer needs to modify all the dimensions including the bolt size.

The slot size may be computed as:

\[
\text{Width} = \text{diameter of bolt} + 1/16''
\]
\[
\text{Length} = \text{diameter of bolt} + \text{differential deflection} + 3/4''
\]

Note A:
Slot to be positioned so that 3/8'' of slot is below the 6 of bolt and 2/32'' is above 6 of bolt at time of stage II erection.
Notes to designer:

1. See CF-1 on standard sheet BCF-4 in Part 3 of this manual.
2. Cross frame as detailed is limited to a girder depth of 8'-0".
3. Maximum angle is limited to 60°. Use V-type cross frames when angle is less than or equal to 60°.
4. For bridge requiring utility attachments, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as long as the angle limit in note 3 is not exceeded.
5. Details shown are also used at interior support lines (piers) of continuous spans.
6. The 3" dimension shown on the bottom angle shall be increased to 6" at piers.
7. For Detail C, see File No. 11.07-1.
Notes to designer:

1. See CF-2 on standard sheet BCF-4 in Part 3 of this manual.

2. Cross frame as detailed is limited to a girder depth of 8'-0".

3. Maximum angle is limited to 60°. Use V-type cross frames when angle is less than or equal to 60°.

4. For bridge requiring utility attachments, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as long as the angle limit in note 3 is not exceeded.

5. Details are shown for normal end cross frames of simple spans and exterior support lines of continuous spans.

6. For Detail C, see File No. 11.07-1.
Notes to designer:

1. See CF-3 on standard sheet BCF-4 in Part 3 of this manual.
2. Cross frame as detailed is limited to a girder depth of 8'-0".
3. Maximum angle is limited to 60°. Use V-type cross frames when angle is less than or equal to 60°.
4. For bridge requiring utility attachments, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as long as the angle limit in note 3 is not exceeded.
5. Details are shown for skewed end cross frames of simple spans and exterior skewed support lines of continuous spans.
6. For Detail C, see File No. 11.07-1.

* See Note 4.
Notes to designer:

1. See CF-4 on standard sheet BCF-4 in Part 3 of this manual.
2. Cross frame as detailed is limited to a depth of 8'-0".
3. Maximum angle is limited to 60°. Use V-type cross frames when angle is less than or equal to 60°.
4. For bridge requiring utility attachments, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as long as the angle limit in note 3 is not exceeded.
5. Details shown are also used at skewed interior support lines (piers) of continuous spans.
6. The 3" dimension shown on the bottom angle shall be increased to 6" at piers.
7. For Detail C, see File No. 11.07-1.
Notes to designer:

1. See CF-5 on standard sheet BCF-5 in Part 3 of this manual.
2. Use V-type cross frames if the requirements for their use are met.
3. Details shown are also used at interior support lines (piers) of continuous spans.
4. The 3” dimension shown on the bottom angle shall be increased to 6” at piers.
5. For Detail C, see File No. 11.07-1.
Notes to designer:

1. See CF-6 on standard sheet BCF-5 in Part 3 of this manual.

2. Use V-type cross frames if the requirements for their use are met.

3. Details are shown for normal end cross frames of simple spans and exterior support lines of continuous spans.

4. For Detail C, see File No. 11.07-1.
Notes to designer:

1. See CF-7 on standard sheet BCF-5 in Part 3 of this manual.
2. Use V-type cross frames if the requirements for their use are met.
3. Details are shown for skewed end cross frames of simple spans and exterior skewed support lines of continuous spans.
4. For Detail C, see File No. 11.07-1.
Notes to designer:

1. See CF-8 on standard sheet BCF-5 in Part 3 of this manual.
2. Use V-type cross frames if the requirements for their use are met.
3. Details shown are used also at skewed interior support lines (piers) of continuous spans.
4. The 3" dimension shown on the bottom angle shall be increased to 6" at piers.
5. For Detail C, see File No. 11.07-1.
Notes to designer:

1. A standard sheet is not provided for curved girders or girders skewed greater than 30° because the details required will be project specific. It is the designer's responsibility to design the cross frames, including location, member sizes, weld lengths, location and geometry of bolt groups, size of bolts, gusset plate sizes, connection plate sizes, etc.

2. For bridge requiring utility attachments, avoid mixing V-type and X-type bracing throughout the length of bridge.

3. For Detail C, see File No. 11.07-1.
1. A standard sheet is not provided for curved girders or girders skewed greater than 30° because the details required will be project specific. It is the designer’s responsibility to design the cross frames, including location, member sizes, weld lengths, location and geometry of bolt groups, size of bolts, gusset plate sizes, connection plate sizes, etc.

2. For bridge requiring utility attachments, avoid mixing V-type and X-type bracing throughout the length of bridge.

3. For Detail C, see File No. 11.07-1.
STEEL PIER CAPS* AND STEEL BOX (TUB) GIRDERS:

FRACTURE CRITICAL:

See File No. 11.01-1.

DESIGN:

Flanges (tension and compression) and web shall be fabricated using ASTM A709 Grade HPS 50W, unless a design waiver for ASTM A709, Grade HPS 70W has been approved.

When using any system that has twin girders/elements and which is intended to function after the failure of one of the girders/elements, the individual girder/element(s) must be designed at no less than operating level capacity after the initial girder failure. No yielding will be allowed after the initial fracture.

The minimum clear height for maintenance and inspection access is 84” between flanges. Steel pier cap/steel box webs shall be designed at no less than operating level capacity after the initial girder failure. Steel pier cap/steel box webs shall require a design approval from the District Structure and Bridge Engineer. For locations where 84” webs are impractical, designer should consider alternatives. Some alternatives might include paired girders, alternative span arrangements and/or profiles.

Design the steel pier cap without the use of AASHTO’s lane reduction factors.

In addition to the moments and shears introduced by girder reactions, steel pier cap sections shall be designed for the torsion induced by the longitudinal moments and vertical shears of the girders. Designer is cautioned that steel pier caps are not rigid and the elastic deflections and rotations of the box must be accounted for during design. Designer shall follow detailing practices for curved girders if torsion is significant. When connection of plates (stiffeners or plate diaphragms) to the tension flange is required by analysis, a welded connection shall not be used. Alternatives to welding are bolting, increasing plate sizes and altering box geometry.

Steel pier caps/steel box girders are often highly congested. The Designer is encouraged to minimize congestion by decreasing stiffening requirements through the use of thicker plate sizes.

Permissible splices shall be designed such that the splice develops 100% of the smaller section.

A camber diagram and dead load deflection diagram are both required.

*Steel pier caps refer to straddle bents, integral caps and integral straddle bents. See File No. 15.01-3 for description.
DEFLECTION:

Steel pier cap deflections shall be limited to L/800 and span deflections, including effects of cap deflections, shall also be limited to L/800.

FATIGUE:

Steel pier caps/box girders shall be detailed using infinite life fatigue requirements.

DETAILS: SPLICES, CONNECTIONS, DIAPHRAGMS, CROSS FRAMES, ETC.

The width of the pier cap/box girder shall be a minimum of 4'-0" clear interior between the webs.

Layouts which include steel pier caps at consecutive substructure locations (e.g. Pier 2 followed by Pier 3) are discouraged as construction tolerances are stringent and special details are required.

Interior plate diaphragms spaced at a minimum of 4'-0" improves inspection and accessibility. This includes the distance between the end plate with the exterior access door and interior plate diaphragm serving as the bearing stiffener.

Diaphragms/bulkheads shall not have stress hoops on the inside of pass through openings. Bulkheads or diaphragms shall be spaced at least 24" apart to ensure that a person does not have to straddle two at a time.

Steel pier caps shall include jacking/erection diaphragms centered between 2'-0" (preferable) and 4'-0" from the face of each supporting column, but not closer than 4'-0" from the center line bearing.

Interior plate diaphragms shall not be welded to the tension flange.

Transverse stiffeners between plate diaphragms should be avoided. Transverse stiffeners shall be welded to the compression flange and the web. When used, transverse stiffeners shall not be welded to the tension flange. Torsional rigidity may require welding to bottom flange for flexible boxes. See design guidance below. Bearing stiffeners and jacking stiffeners shall be welded to the tension flange. Designers are reminded that clips for transverse, bearing and jacking stiffeners must follow AASHTO LRFD Specifications and must be of adequate size to allow for backing bars if complete penetration welds are required for the flange to web connection of the box.

Bolted field splices should be considered to limit the weight of shipping pieces. When detailing bolted field splices, the edge distance of the exterior splice plates shall be no more than the edge distance of the interior splice plates. The web splice bolt locations should clear the flanges 6". The center line of web splice shall be perpendicular to the flanges.

Longitudinal stiffeners shall not be permitted.

Grade between ends of pier caps/box girders shall be 2% minimum.
The top of the box on the high side shall have a drip diverter to keep water from puddling next to the slab.

Ventilation/drainage holes of 2" diameter shall be placed at low points and at a maximum spacing of 25'-0". All exterior holes not covered by a door shall be covered on the inside by a screen of not greater than 1/4" mesh opening. If a threaded insert is provided on the outside of the tube, it shall also be covered by screen.

Girder Connection at Straddle:

The girder connection at a straddle shall use the concepts as shown in the three following figures.

Section though Straddle
Access Door and Internal Access Opening:

An access door shall be provided at each end of a steel box or straddle. Internal access opening shall be provided from one end of the straddle to the other end.

The opening of an access door or interior access opening shall be 24" x 36" minimum. Round corners with 6" radius. For an access door, locate a vertical grab bar on the end plate next to access door hasp and place a horizontal grab bar over the access doors to assist in passing through access doors. Location restrictions shall be the same as for internal access openings. The length of grab bars should be the same dimension as the parallel edge of the opening or greater.
DETAILS: SPLICES, CONNECTIONS, DIAPHRAGMS, CROSS FRAMES, ETC. (cont’d)

An access door shall be provided with hinges and hasp for locking with a positive method to keep hatch closed and sealed in each end plate. Access doors should swing out and be easy to handle. Designer shall provide ladders or rungs on the interior of the box to improve access.

Access doors shall have a hasp and means of positively closing doors. Access doors shall have a neoprene gasket which will be compressed when the door is closed and be waterproof. In addition to the gasket, a drip edge shall be provided over the top of the end opening.

Mechanical keepers shall be provided on all exterior access doors to maintain doors in the open position and prevent closure by wind or other unintentional causes.

Material for hinges and pins shall be forged stainless steel. Hinges and pins shall be easily lubricated and replaceable in the field.

A means to positively attach a ladder to the outside of the pier cap/box girder at the access door during inspection shall be provided. Placing a ladder to gain access to the pier cap/box girder can be difficult after heavy rains on a shoulder slope. Designer should consider identifying an area to set a ladder at the base of each column. A concrete slab sloped to drain to accommodate both the ladder and generator shall be provided as directed by the District Structure and Bridge Engineer.

When detailing internal access and openings, the maximum distance between foot placement locations shall be 60" measured as shown (from footprint to footprint along the inspector's inseam, follow the path of the arrow) taking into account the "straddle" effect of plates and stiffeners. Steps may be provided to limit "straddle" problems.

The bottom of openings in the plate diaphragms shall be between 2'-0" and 2'-6" from either the bottom flange of the pier cap/box girder or the horizontal stiffener if it is used.

Placement of stiffeners effectively reduces the size of the openings by creating a tunnel effect. Details shall place any vertical stiffeners back from the edge of the openings by 6" min. Horizontal stiffeners below openings should be of a width that can be straddled. If they cannot be straddled, place where they can be used as steps or be able to be bypassed through the use of steps.

Finishes:

Paint inside of box white. Paint exterior to match bridge girders. If girders are unpainted then the straddle bent shall be painted brown, Federal Color No. 595-20059.

Use epoxy grit on the floor and on any horizontal surfaces on which inspectors will be required to step. Grit should be kept 3” to 6” from welds.
DETAILS: SPLICES, CONNECTIONS, DIAPHRAGMS, CROSS FRAMES, ETC. (cont’d)

Electrical Provisions:

Permanent electrical service shall be provided and installed in accordance with the latest edition of the NFPA70/National Electrical Code (NEC) adopted by VDOT. Providing a generator does not meet this requirement. The grounding system for electrical service shall be installed and tested in accordance with the Road & Bridge Specifications (Section 700).

Provide a duplex receptacle inside the box between each plate diaphragm. The wiring (electrical conductors) shall be in metal conduit. Receptacles shall be located in the compression region of the girder and should be within reach of inspectors standing on the bottom flange of the box. Drilling and tapping for securing conduit shall be done by the structural steel fabricator in the shop, not after erection in the field. A note indicating that drilling and tapping for the electrical conduit shall be performed by the structural steel fabricator shall be placed on the plans and elevation sheet of the steel pier caps.

If the contract documents specify that electrical service is to be provided by a generator, then provide a quadplex receptacle inside the box between each plate diaphragm. In addition, extend the conduit down one pier to a safe location as near as practical to the ground generally between 3'-0" and 6'-0" but where it will not be subject to splash or spray from traffic and/or bridge drains. At the termination, provide a male connection such that the installed wiring can be powered by a generator. Termination shall also be protected by a protective lockable enclosure.

Welding of Box Girders:

For closed box sections with a width of 4'-0" or greater, use double fillet welds for attaching all webs to flanges.

For closed box sections with width less than 4'-0" (if approved at Preliminary Stage 1 phase), use double fillet welds for attaching first flange to each web and use full penetration welds from outside for attaching second flange to each web.
CONSTRUCTION CONSIDERATIONS:

Fabrication:

To improve fabrication of boxes, flange to web welds shall be fillet welds on both inside and outside of the box webs. Stiffeners shall be attached using fillet welds. At diaphragms which transmit superstructure beam bending and shear, connections should generally not involve milling to bear or full penetration welds to the box webs or flanges.

A note shall be provided on the plans for any fracture critical pier cap requiring that the pier cap be fully bolted up in the shop including fully tensioning all bolted splices and connections.

Erection:

Field sections (segments) shall be blocked in the field to the correct orientation before connecting the two segments. Contractor may attach splice plates to one box segment only after the blocking is complete. Contractor may use drift pins to facilitate alignment. Once the bolting process begins, if a segment must be moved all the bolts must be removed and replaced before the splice is complete. Bolts shall not be reused. Inspector must check each bolted splice inside and outside the box while on the blocking and after erection to ensure that the splice has been correctly installed, no beams shall be attached to the box until after the construction inspection has been completed.
STEEL TRUSSES:

All new trusses on projects with a design year ADT greater than 400 require a design waiver.

Most often projects requiring a truss will either be: a desired aesthetic requirement from the district/locality, replacement of a similar existing structure to retain historical appearances, hydraulic issues or as a practical solution for long spans over an obstacle while at the same time minimizing elevation changes of approach embankments. The latter option also provides an opportunity to minimize the depth between the profile grade and low chord, which helps in circumstances where vertical clearance needs to be provided.

Trusses are considered to be a specialty superstructure type. Therefore, when a project requires a truss, either it shall be designed by a consultant with previous experience in designing truss bridges or a prefabricated truss shall be proposed for the project.

When a prefabricated truss is planned, the designer shall coordinate with prefabricated truss manufacturer(s). The manufacturer should be AISC Certified for Major Bridges and should have the AISC Fracture Critical Endorsement. The coordination will include obtaining the following information;

1. Design loadings and reactions to all substructures.
2. Estimated fabrication costs.
3. Estimated delivery costs.
4. Estimated erection costs.
5. Estimated duration of erection.
6. Sequencing of on-site assembly and erection that may affect traffic detour or erection duration.
7. Other requirements which may affect cost, fabrication (including finishes) and duration.

See also File No. 11.01-1 for fracture critical requirements.

Barriers:

Barriers shall be crash-tested barrier and mounted to the deck except for pedestrian only structures. Barrier shall function independently of the truss members.

Sample plans for prefabricated truss are provided on the following pages.
STEEL SUPERSTRUCTURE:

Steel trusses are for a 67'-9" to 13'-0" wide bridge deck. The superstructure design load is to be determined as per VDOT S&B Division Instructional Memorandum IIM-S&B-86 and be approved by the Engineer. Design/Shop drawings shall clearly define those members or components requiring CVN testing. Full length edge dams, acceptable to the Engineer, shall be field welded to both sides of bridge flooring in order to retain the concrete in the deck. Edge dams and splash guards may be welded without removing the zinc coating.

TRUSS SUPERSTRUCTURE REQUIREMENTS:

One simple span 118'-0" (measured from centerline of bearing to centerline of bearing), 120'-8" (measured from end of slab to end of slab), with a clear roadway width of 13'-0" face-to-face of rails.

Bridge deck shall be 8 in. concrete deck cast with the connections using commercial bridge flooring meeting the requirements of AASHTO LRFD Specifications. All members or components requiring CVN testing shall be power brushed and painted with two coats of paint meeting the requirements of ASTM A123.

STEEL TRUSSES:

Vehicular Truss Bridge

The shop drawings and the material specification and grade for all steel structural steel and hardware.

STRUCTURAL STEEL:

Welding shall be accomplished using electrodes and procedures acceptable to the Engineer. Welds shall be made to A570 Grade 70 with AWS Grade E7016. All welds shall be made in accordance with the VDOT Structure and Bridge Division Instructional Memorandum IIM-S&B-80.3. Fasteners shall be installed and tightened in accordance with the VDOT Specifications. All rivets, nuts and bolts shall be of the proper grade and size and shall be of the proper size and grade as specified in the VDOT Structure and Bridge Division Instructional Memorandum IIM-S&B-80.3. Fasteners shall be installed and tightened in accordance with the VDOT Specifications.

Bridge deck shall be 8 in. concrete deck cast with the connections using commercial bridge flooring meeting the requirements of AASHTO LRFD Specifications. Full length edge dams, acceptable to the Engineer, shall be field welded to both sides of bridge flooring in order to retain the concrete in the deck. Edge dams and splash guards may be welded without removing the zinc coating.

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**TYPICAL TRANSVERSE SECTION ON TRUSS BRIDGE**

**STEEL SUPERSTRUCTURE:**

The details shown are for a fully engineered single span steel superstructure with a concrete deck and shall be regarded as the minimum standards for design and construction. The elevation shown is of the centerline of the roadway and the transverse section shall not be interpreted. The engineer shall be responsible for all additional work not shown.

The bridge shall be an all welded, fully galvanized steel truss bridge and shall be designed and manufactured by one of the following: Steel Truss Bridge Co., 14685 Avion Parkway, Chantilly, VA 20151-1104 or any other fabricator to be approved by the Engineer prior to fabrication and erection. The contract drawings and field erection procedures, shall be approved by the Engineer prior to any erection activities.

**PROJECT L Stairwell 22**

**ROUTE 999**

**VDOT S&B DIVISION**

**RICHMOND, VA**

**DATE: 31 Oct 2018**

**FILE NO. 11.09-3 SHEET 3 of 3**

**COMMUNITY OF VIRGINIA DEPARTMENT OF TRANSPORTATION**

**STRUCTURE AND DESIGN DIVISION**

**DATE: 31 Oct 2018**

**SHEET NO. 272**

**PLAN DRAWN: S&BD 31 Oct 2018**

**DRAWN: S&BD 31 Oct 2018**

**TECHNICAL SPECIFICATIONS:**

**ROW TRUSS BRIDGE STRUCTURAL STEEL:**

The shop drawings shall include all structural components complete to include, but not limited to: members, fabrication details, truss stringers, truss truss members identified as main load carrying members subject to calculated tension or stress reversal, with the exception of the truss fabricator and the requirements of the fence fabricator. Structural members fabricated to the requirement of the fence fabricator shall be coordinated to ensure there are no conflicts between the requirements of the fence fabricator and the requirements of the bridge fabricator. The Contractor shall be responsible for the design and details necessary to construct and install the fence enclosure.

**GENERAL NOTES:**

- The truss shall be of the Bowstring type. Floor beam and stringer members, floor and spandrel, truss members, and any other components designed to resist loading shall be designed as slip-critical connections per the requirements of Section 232 of the Specifications. Design shop drawings shall clearly define those members or components requiring CVN testing.

- All shop drawings shall clearly define those members or components required for CVN testing.

**WELDING AND NON-DESTRUCTIVE TESTING:**

- All fabrication, welding, non-destructive testing and visual testing shall be accomplished using acceptable methods.

**GENERAL COMMENTS:**

- The Contractor shall be responsible for the necessary tolerances and joints to be fabricated on the bridge. All fabrication, welding, non-destructive testing and visual testing shall be accomplished using acceptable methods.
Supplementary Symbols Used with Welding Symbols

Location of Elements of a Welding Symbol

Supplementary Symbols

<table>
<thead>
<tr>
<th>Weld-All-Around</th>
<th>Field Weld</th>
<th>Melt-Thru</th>
<th>Backing, Spacer</th>
<th>Contour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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</tbody>
</table>

STEEL
WELDING
STANDARD WELDING SYMBOLS
Typical Welding Symbols

<table>
<thead>
<tr>
<th>Square-Groove Welding Symbol</th>
<th>Back or Backing Welding Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Omission of size indicators complete joint penetration Root opening</td>
<td>Any applicable single groove weld symbol</td>
</tr>
</tbody>
</table>

Edge- and Corner- Flange Welding Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{3}{8} )</td>
<td>( \frac{3}{8} + \frac{1}{16} )</td>
</tr>
<tr>
<td>Radius</td>
<td>( \frac{3}{16} + \frac{1}{16} )</td>
</tr>
<tr>
<td>Size of weld</td>
<td>( \frac{1}{16} )</td>
</tr>
<tr>
<td>Height above point of tangency</td>
<td></td>
</tr>
</tbody>
</table>

Plug Welding Symbol

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Included angle of counterflank</td>
<td>Pitch (distance between centers) of welds</td>
</tr>
<tr>
<td>Size (diameter of hole at root)</td>
<td>Depth of filling in inches (omission indicates filling is complete)</td>
</tr>
<tr>
<td>30</td>
<td>4</td>
</tr>
</tbody>
</table>

Welding Symbols for Combined Welds

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{1}{4} )</td>
<td>( \frac{1}{8} ) 60°</td>
</tr>
<tr>
<td>( \frac{5}{8} )</td>
<td></td>
</tr>
<tr>
<td>( \frac{1}{4} )</td>
<td></td>
</tr>
</tbody>
</table>
Typical Welding Symbols

Single-V Groove Welding Symbol Indicating Root Penetration

Depth of preparation
Effective throat
Groove angle

Slot Welding Symbol

Depth of filling in inches (omission indicates filling is complete)
Orientation, location and all dimensions other than depth of filling are shown on the drawing

Backgouging Welding Symbol

Second reference line used for back gouging and welding as a second operation
Note: Total effective throat not to exceed thickness of member

Double-Bevel-Groove Welding Symbol

Omission of size dimension indicates a total depth of preparation equal to thickness of members

STEEL WELDING STANDARD WELDING SYMBOLS
Typical Welding Symbols

Spot Welding Symbol

- Size (diameter of weld)
- Strength (in, lb, per weld) may be used instead
- Number of welds
- Pitch (distance between centers) of weld

Process reference must be used to indicate process desired

Seam Welding Symbol

- Size (width of weld)
- Strength (in lb per linear inch) may be used instead
- Pitch (distance between centers) of increments
- Process reference must be used to indicate process desired

Length of welds or increments
Omission indicates that weld extends between abrupt changes in direction or as dimensioned

Projection Welding Symbol

- Projection welding reference must be used
- Size (strength in lb per weld)
- Diameter of weld may be used instead for circular projection welds
- Pitch (distance between centers) of weld

Double-Fillet Welding Symbol

- Size (length of leg)
- Specification, process, or other reference

Length of weld extends between abrupt changes in direction or as dimensioned
Typical Welding Symbols

Flash or Upset Welding Symbol

No Arrow Side or Other Side Significance

Plug Weld Symbol

Chain Intermittent Fillet Welding Symbol

Size (length of leg)

Pitch (distance between centers of increments)

Length of Increments

Flare-V and Flare-Bevel-Groove Welding Symbols

Root opening

Size is considered as extending only to tangent points

Surfacing Welding Symbol Indicating Built-up Surface

Size (height of deposit)

Omission indicates no specific height desired

Orientation, location and all dimensions other than size are shown on the drawing

Staggered Intermittent Fillet Welding Symbol

Pitch (distance between centers of increments)

Size (length of leg)

Length of Increments
### Supplementary Symbols Used with Welding Symbols

<table>
<thead>
<tr>
<th>Weld - All - Around Symbol</th>
<th>Joint with Backing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weld-all-around symbol</td>
<td>With groove weld symbol</td>
</tr>
<tr>
<td>Indicates that weld</td>
<td>Note: Material and</td>
</tr>
<tr>
<td>extends completely</td>
<td>dimensions of backing</td>
</tr>
<tr>
<td>around the joint</td>
<td>as specified</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Joint with Spacer</th>
<th>Melt - Thru Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>With modified groove weld</td>
<td>Any applicable weld</td>
</tr>
<tr>
<td>symbol</td>
<td>symbol</td>
</tr>
<tr>
<td>Note: Material and</td>
<td>Melt-thru symbol is</td>
</tr>
<tr>
<td>dimensions of backing</td>
<td>not</td>
</tr>
<tr>
<td>as specified</td>
<td>dimensioned (except</td>
</tr>
<tr>
<td></td>
<td>height)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Field Weld Symbol</th>
<th>Complete Joint Penetration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field Weld symbol</td>
<td>Indicates complete</td>
</tr>
<tr>
<td>indicates that weld is</td>
<td>penetration regardless</td>
</tr>
<tr>
<td>to be made at a place</td>
<td>of type of weld or</td>
</tr>
<tr>
<td>other than that of</td>
<td>joint preparation</td>
</tr>
<tr>
<td>initial construction</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Convex Contour Symbol</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Convex contour symbol</td>
<td>Finish symbol (user's</td>
</tr>
<tr>
<td>indicates face of weld</td>
<td>standard) indicates</td>
</tr>
<tr>
<td>to be finished to convex</td>
<td>method of obtaining</td>
</tr>
<tr>
<td>contour but not degree</td>
<td>specified contour but not</td>
</tr>
<tr>
<td>of finish</td>
<td>degree of finish</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Flush Contour Symbol</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flush contour symbol</td>
<td>Finish symbol (user's</td>
</tr>
<tr>
<td>indicates face of weld</td>
<td>standard) indicates</td>
</tr>
<tr>
<td>to be made flush. When</td>
<td>method of obtaining</td>
</tr>
<tr>
<td>used without a finish</td>
<td>specified contour but not</td>
</tr>
<tr>
<td>symbol, indicates weld</td>
<td>degree of finish</td>
</tr>
<tr>
<td>without subsequent</td>
<td></td>
</tr>
<tr>
<td>finishing</td>
<td></td>
</tr>
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<table>
<thead>
<tr>
<th>Multiple Reference Lines</th>
<th></th>
</tr>
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<tbody>
<tr>
<td>First operation shown on</td>
<td></td>
</tr>
<tr>
<td>reference line nearest</td>
<td></td>
</tr>
<tr>
<td>arrow</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1st</td>
</tr>
<tr>
<td>Second operation or</td>
<td></td>
</tr>
<tr>
<td>supplementary data</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2nd</td>
</tr>
<tr>
<td>Third operation or</td>
<td></td>
</tr>
<tr>
<td>test Information</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3rd</td>
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</tbody>
</table>
### Basic Welding Symbols and Their Location Significance

<table>
<thead>
<tr>
<th>Location Significance</th>
<th>Surfacing</th>
<th>Scoof for Brazed Joint</th>
<th>Flange</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Edge</td>
<td>Corner</td>
<td></td>
</tr>
<tr>
<td>Arrow Side</td>
<td>SUA</td>
<td>SCA</td>
<td>FEDA</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>FCOA</td>
</tr>
<tr>
<td>Other Side</td>
<td></td>
<td></td>
<td>FEDO</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>FC00</td>
</tr>
<tr>
<td>Both Sides</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SCB</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Arrow Side or Other Side Significance</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location Significance</th>
<th>Plug or Slot</th>
<th>Spot or Projection</th>
<th>Seam</th>
<th>Back or Backing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arrow Side</td>
<td>PLA</td>
<td>SPA</td>
<td>SEA</td>
<td>Groove weld symbol</td>
</tr>
<tr>
<td></td>
<td>BKA</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other Side</td>
<td>PLO</td>
<td>SP0</td>
<td>SEO</td>
<td>Groove weld symbol</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Both Sides</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Arrow Side or Other Side Significance</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Basic Welding Symbols and Their Location Significance

<table>
<thead>
<tr>
<th>Location Significance</th>
<th>Fillet</th>
<th>Groove</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Square</td>
</tr>
<tr>
<td>Arrow Side</td>
<td>FIA</td>
<td>GSOA</td>
</tr>
<tr>
<td>Other Side</td>
<td>FIO</td>
<td>GSOQ</td>
</tr>
<tr>
<td>Both Sides</td>
<td>FIB</td>
<td>GSOB</td>
</tr>
<tr>
<td>No Arrow Side or Other Side Significance</td>
<td>GSQN</td>
<td>GSQN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Location Significance</th>
<th>Groove</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>U</td>
</tr>
<tr>
<td>Arrow Side</td>
<td>GUA</td>
</tr>
<tr>
<td>Other Side</td>
<td>GOU</td>
</tr>
<tr>
<td>Both Sides</td>
<td>GUB</td>
</tr>
<tr>
<td>No Arrow Side or Other Side Significance</td>
<td>GSQN</td>
</tr>
</tbody>
</table>
Basic Joints - Identification of Arrow Side and Other Side of Joint

<table>
<thead>
<tr>
<th>Butt Joint</th>
<th>Lap Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Butt Joint Diagram]( Diagram 1 )</td>
<td>![Lap Joint Diagram]( Diagram 2 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Corner Joint</th>
<th>T-Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Corner Joint Diagram]( Diagram 3 )</td>
<td>![T-Joint Diagram]( Diagram 4 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Edge Joint</th>
<th>Process Abbreviations</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Edge Joint Diagram]( Diagram 5 )</td>
<td>Where process abbreviations are to be included in the tail of the welding symbol, reference is made to Table A, Designation of Welding and Allied Processes by Letter, of AWS 2.4-75, 71.</td>
</tr>
</tbody>
</table>

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STEEL WELDING
STANDARD WELDING SYMBOLS
INTRODUCTION:

ASTM A709 Grade 50CR (formerly designated as A1010) steel is highly corrosion resistant structural steel. It is relatively new to the bridge industry. The major bridge fabrication issues involving ASTM A709 Grade 50CR steel have been addressed by industry. At least eight bridges have been built with ASTM A709 Grade 50CR steel in the United States and Canada. The initial cost of ASTM A709 Grade 50CR steel is higher than ASTM A709 Grade 50W. A Life Cycle Cost estimate may justify the selection of ASTM A709 Grade 50CR steel for some projects.

MATERIAL AND SPECIFICATIONS:

ASTM A709 Grade 50CR steel shall be in accordance with ASTM A709/A709M – 17, Standard Specification for Structural Steel for Bridges.

ASTM A709 Grade 50CR steel structures shall conform to the requirements specified in the Special Provision for Corrosion Resistant Steel Plate Girders.

SELECTION CRITERIA:

Steel girders in highly corrosive environments may use ASTM A709 Grade 50CR steel. Corrosive environments include exposure to deicing salt, airborne sea salt and airborne chemicals in heavy industrial areas. For instance, a bridge site where a heavy industry is located may be treated as a corrosive environment; and steel girders within 15 feet of mean high tide in areas east of the red highlighted routes (including the bridges on these routes) in Figure 1 on File No. 12.07-2 may use ASTM A709 Grade 50CR steel.

Corrosive environments should be identified or defined in the scoping stage.

ASTM A709 Grade 50CR steel can be used for projects where uncoated weathering steel would not be recommended according to FHWA Technical Advisory of Uncoated Weathering Steel in Structures, T 5140.22, October 3, 1989.

ASTM A709 Grade 50CR steel can also be used in areas where repainting steel elements will be difficult.

Cross frames can be ASTM 709 galvanized steel or weathering steel except for curved and highly skewed bridges where cross frames are primary structural elements. ASTM A709 Grade 50CR steel shall be used for the cross frames for curved and highly skewed bridges.

When galvanized steel is used for cross frames, galvanized ASTM F3125 Grade A325 steel bolts shall be used. When weathering steel is used for cross frames, Type 3 ASTM F3125 Grade A325 steel bolts shall be used.

ASTM A709 Grade 50CR steel may be desirable for some rehabilitation projects. Contact the Structure and Bridge Division Central Office Maintenance Section for guidance on using A709 Grade 50CR for maintenance projects.

Currently, ASTM A709 Grade 50CR steel is only available as hot-rolled plate product up to 2” in thickness.

The use of ASTM A709 Grade 50CR steel requires an approved Design Waiver from the State Structure and Bridge Engineer.