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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)

It is expected that the users of this chapter will adhere to the practices and requirements stated herein.

Several major changes and/or additions to the past office practice (Part 2) are as follows:

1. Revised bridge layout points to line thru center of bearings/piles.

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.
GENERAL INFORMATION:

Most of the items listed below are also noted in the VDOT Modifications to the AASHTO Standard and LRFD specifications. Additional notes pertinent to these areas will be found in other sections throughout this chapter.

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

Two beam/girder systems and trusses are not allowed for new structures or replacement of existing structures on projects with a design year ADT greater than 400 without a design waiver. Two beam/girder configurations during phased construction are not considered fracture critical.

MATERIALS:

ASTM A709 Grade 50W shall be used in the design for main (primary) members 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.

ASTM A709 Grade 50, galvanized, or painted, shall be used in the design for trusses.

Structural pipe and tubing shall be galvanized inside and outside.

If main (primary) members are weathering steel, secondary members shall also be weathering steel.
MATERIALS: (cont.)

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, coated ASTM A709 Grade 50W shall be used except where a design waiver is approved to use ASTM A709 Grade 50CR. The coating may be painted, galvanized or metalized. Use of coated ASTM A709 Grade 50 is not recommended.

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.

The use of light weight concrete (LWC) for spans over 150 feet shall be investigated for cost effectiveness.

HAULING LENGTHS/WEIGHTS:

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

Lengths of beams/girders over 65 feet may be shipped with a hauling permit on Virginia’s roadway system. There are benefits to fabricating larger (longer) members in the shop. Members shall be designed for one or more bolted field splices where the investigation indicates conditions limiting length and/or weight.

HIGH STRENGTH BOLTS:

High strength ASTM A325 bolts shall be used. High strength ASTM A490 bolts shall not be used. ASTM A490 bolts cannot be galvanized.

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

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.

For low volume roads (ADT ≤ 750 and where the design speed does not exceed 45 mph), new and replacement bridges may be detailed using Part 8 of this manual with rolled beams and timber decks.
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.

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.

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 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 DEFORMATION** 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 without a design waiver except for low volume roads (ADT ≤ 750 and where the design speed does not exceed 45 mph), where new and replacement bridges may be detailed using Part 8 of this manual with rolled beams and timber decks.

Continuous Spans:

The ratio of shorter end span to longer interior span should range between 1 to 1.2 and 1 to 1.4 for the most economical span arrangements.

No span arrangement which results in uplift will be allowed without a design approval. Mitigation methods include counterweights or hold downs.

Curved Spans:

Working with the road designer to eliminate or minimize spirals on bridges is highly encouraged.

Impact of spiral or compound curves can be minimized by placing transitions at field splices as much as is practical. Spirals can be approximated in most instances by compound curves.

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.

Curved beams/girders shall be parallel. Uplift shall be checked.

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" unless a design waiver is approved. For splayed (variable spaced beam/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. 0.3 x beam/girder spacing is preferred for the deck slab overhang.
2. For overhangs exceeding 0.3 x beam spacing, a yield-line analysis is required.
3. 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.
4. Check the space required for deck drains for conflicts with the location of the exterior beam/girder lines.
5. 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, new 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. In the absence of any depth restrictions, the minimum depths in AASHTO LRFD 2.5.2.6.3 and Table 2.5.2.6.3-1 shall be used as a starting point. The minimum depths are 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 overall depth illustrated in the following figure shall be used for determining the minimum overall depth of straight composite I-girders. The thickness of the future wearing surface shall not be included.
MINIMUM WEB THICKNESS:

The minimum web thickness 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 are preferably made at field bolted splices. Changes in the web thickness in shop welded splices are allowable.

The web thickness shall meet the limits of the ratio of web depth to web thickness as stated in AASHTO LRFD 6.10.2.1.

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".

The maximum thickness of any flange shall be $2 \frac{1}{2}''$ if the flange is to be field welded (butt spliced). Otherwise, the maximum thickness of flange plates shall be in accordance with the limits indicated in the AASHTO specifications. Field welding requires a design waiver.

The relationship between the flange thickness and web thickness shall satisfy the requirements of AASHTO LRFD 6.10.2.2

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

Where:

- $b_{fc} = \text{the width of the compression flange (inches)}$
- $L = \text{the length of girder shipping piece (inches). Shipping piece is minimal distance between bolted field splice.}$

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 be not be detailed in the variable depth regions. Haunches shall not be used for hybrid steel 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 C6.4.10.

**Bolted Field Splices:**

For details and design, see File No. 11.05.

**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 CVN Testing

Multiply weight savings/inch x flange width (length of butt weld)

<table>
<thead>
<tr>
<th>Thinner Plate at Splice (inches)</th>
<th>Thicker Plate at Splice (inches)</th>
</tr>
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<tbody>
<tr>
<td>1.0</td>
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<tr>
<td>1.5</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 \( \leq 2" \) 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 DETAILS:

Notes to designer:

1. Items in blocks are for designer’s information only and are not to be placed on the plans.

2. Butt splices in plate girder flanges and webs or in cover plates for rolled beams should preferably be detailed as shown above. The use of additional dimensions or symbols may unnecessarily restrict the fabricator’s choice of process and could actually result in a more expensive and/or weaker joint, e.g., the addition of a convex-contour symbol would require additional reinforcement of the weld, tending to produce a notch effect and thus weakening the member.

3. AWS D1.5 covers types of prequalified welds, joint preparations, root openings, bevel angles, transitions for plates of different thicknesses, etc.

4. Minimum dimension between flange splice and web splice recommended by AISC is 6”.

5. Minimum dimension between flange splice and web splice to transverse stiffener is 6”

6. Minimum distance between web thickness transitions and flange thickness transitions shall be 2’-0”.

Note A:
Welding of structural steel shall be in accordance with VDOT Road and Bridge Specifications, Section 407.04.
STUD SHEAR CONNECTORS:

Shear connectors are used to achieve composite sections for steel beam/girder bridges. 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. 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.

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.

The maximum pitch 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 have a maximum pitch of 6” within a distance of 36” from the first row of shear connectors.

Shear connectors shall be as close as practical to the end of the beam/girder (approx. 3 inches). The minimum pitch of shear connectors shall not be less than 6 inches or six diameters of shear connectors.

Shear connectors shall be placed parallel to main deck reinforcement and may be spaced at regular or variable intervals. Stud shear connector spacing shall not be closer than the smaller of four stud diameters or 3½ inches.

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

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.

Shear connectors shall be designed in accordance with AASHTO LRFD 6.10.10.

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.
STIFFENERS:

See AASHTO LRFD 6.10.11 for stiffener design and detailing requirements.

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.

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 x t_w) or large bearings are needed. Additional stiffeners shall be spaced at 18 x 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.
CROSS FRAME CONNECTOR AND STIFFENER DETAILS:

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.
3. For details of diaphragms, see File No. 11.07.
4. Diaphragm connector plates shall be 7 ½” minimum width to allow for two rows of bolts in the connection.
5. Where both top and bottom flanges can experience tension, tight fit stiffener to both flanges.
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. For structures with repetitive spans, only one framing plan need be drawn for each typical unit. 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 and both supports have the same skew, 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, the restoring forces in these members.

For skews greater than 20 degrees, the cross frames/diaphragms shall be placed perpendicular to the girders. Typically, they are placed in a contiguous pattern with the cross frames/diaphragms matched up on both sides of the interior girders, except near the bearings. This arrangement provides the greatest transverse stiffness.

For curved steel bridges, the intermediate cross frames/diaphragms shall be radial and support cross frames/diaphragms shall be parallel to the supports.

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 near obtuse corners at supports, releases the girders torsionally 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.
SAMPLE FRAMING PLAN FOR STEEL PLATE GIRDER
WITH TRANSVERSE STIFFENERS AND SKEW = 20°
For bolted splice details, see sheet 13.
For girder details, see sheet 15.

Sample Framing Plan for Steel Plate Girder
Without Transverse Stiffeners 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 C / R.

3. Label C / 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 C 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.
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 stress 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.
STEEL
BEAM/GIRDER ELEVATION
SAMPLE GIRDER ELEVATION

Notes:
1. Tight fit to tension flange(s).
2. Tight fit to compression flange(s).
3. Note A: ¼" fillet weld (both sides)

TRANSVERSE INTERMEDIATE STIFFENER

See Note A

Note A: ¼" fillet weld (both sides)
Tight fit to tension flange(s).

CROSS FRAME CONNECTOR PLATE

DETAIL A

TYPICAL BEAM END DETAIL

BEARING STIFFENERS

Not to scale

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FILE NO. 11.04-4
DATE: 18 May 2016
SHEET 4 of 6

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
STRUCTURE AND BRIDGE DIVISION

ORDER ELEVATION

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FILE NO. 11.04-4
DATE: 18 May 2016
SHEET 4 of 6

COMMONWEALTH OF VIRGINIA
DEPARTMENT OF TRANSPORTATION
STRUCTURE AND BRIDGE DIVISION

ORDER ELEVATION
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 $\frac{5}{8}"$.

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 ¼" 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 $\frac{3}{16}"$.

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)

High Strength, ASTM A325 Bolts, $7/8$" dia.

Bolt Head: $x = 1\frac{7}{16}$", thickness = $35/64$"
Nut: $x = 1\frac{7}{16}$", thickness = $55/64$"
Washer: $\varnothing = 1\frac{3}{4}$", thickness = 0.177"

Bolts are normally installed with nut down in the top flange (splice) and nut up in the bottom flange (splice). Assume $1/2$" extension of bolt threads beyond face of nut.
BOLTED SPLICE CONNECTIONS: (cont’d)

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 clearance(s) 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.

The values of the deflections and bolsters shall be multiples of $\frac{1}{8}''$.

DEFINITIONS:

C = Required camber for beam/girder

$\Delta_s$ = Deflection of steel from its own weight

$\Delta'_s$ = Deflection of steel section from dead load of deck slab concrete including construction tolerance

$\Delta_c$ = Deflection of composite section from dead load (e.g., parapet, rail and curb) added after deck slab is cast, future wearing surface not included

CT = Camber Tolerance (positive numerical value)

$\Sigma D$ = Beam/girder depth tolerance + (cross slope x $\frac{1}{2}''$ max. flange width) + (cross slope x horizontal curve ordinate) + $\frac{1}{16}''$

B = Thickness of bolster over beam/girder at supports (thickness at intersection of beam/girder and bearing)

H = Distance between horizontal reference line and deflected top of web at any point being considered

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

- Rolled beam = $+ \frac{1}{8}'' \times \frac{\text{span length (feet)}}{10}$
- Plate girder = $+ \frac{1}{8}'' \times \frac{0.5 \times \text{span length (feet)}}{10}$ or $\frac{3}{16}''$ whichever is greater
  
  $= + \frac{1}{16}'' \times \frac{\text{span length (feet)}}{10}$ or $\frac{3}{16}''$ whichever is greater

Depth tolerance:

- Rolled beam = $\frac{1}{8}''$
- Plate girder = $\frac{1}{8}''$ for depth $\leq 36''$; $\frac{3}{16}''$ for depth $> 36''$ but $\leq 72''$; $\frac{5}{16}''$ for depth $> 72''$
BOLSTER

\( h = \text{min. bolster} + \text{thickest top flange} \)

(Diagram shown for hump vertical curve; sag vertical curve similar)

**SIMPLE SPANS:**

For Straight Gradient:

\[
C = \Delta_s + \Delta'_s + \Delta_c \\
B \geq CT + \Sigma D
\]

For Hump Vertical Curve:

\[
C = \Delta_s + \Delta'_s + \Delta_c \\
B \geq CT + \Sigma D
\]

Camber may be increased to an upper limit of \( \Delta_s + \Delta'_s + \Delta_c + \text{V.C. ordinate} \) where

\( \text{V.C. ordinate exceeds CT} \)

For such case(s):

\[
B \geq (\text{excess of CT over V.C. ordinate}) + \Sigma D
\]

For Sag Vertical Curve:

\[
C = \Delta_s + \Delta'_s + \Delta_c - \text{V.C. ordinate} \\
B \geq CT + \Sigma D
\]
CAMBER NOTE:

Simple Spans:

The following note is to be placed with the beam/girder details:

Camber note: Beam(s)/girder(s) shall be cambered (up) (down) _____" at midspan. Computed deflection of beam/girder from its own weight after erection, including struts, (diaphragms), connectors, etc., is _____" at midspan.

Continuous Spans:

File Nos. 11.06-6 thru -8 show the method of computing bolster thickness and the camber diagram for continuous spans. File No. 11.06-11 shows the dead load deflection diagram for continuous spans.
Adjustment of deck slab forms to correct for dead load deflections shall be made by varying thickness of concrete bolster between slab and beam without alteration of slab thickness. Longitudinal screed should be set above final finished grade by amounts equal to $\Delta_c$.

$\Delta_s$, $\Delta_c$, $\Delta'_s$, $\Delta_c$

<table>
<thead>
<tr>
<th>Dead Load Deflection at a</th>
<th>Dead Load Deflection at b</th>
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</thead>
<tbody>
<tr>
<td>$\Delta_s$</td>
<td>$\Delta_c$</td>
</tr>
<tr>
<td>$\Delta'_s$</td>
<td>$\Delta_c$</td>
</tr>
</tbody>
</table>

$\Delta'_s = \text{Deflection of beam from dead load of concrete deck slab and bolsters including construction tolerance}$

$\Delta_c = \text{Deflection of beam from dead load (e.g. parapet) added after deck slab is cast}$

**DEAD LOAD DEFLECTION DIAGRAM**

Camber Note: Beams shall be cambered up ____" at midspan. Computed deflection of beam from its own weight after erection, including struts (diaphragms), connectors, etc., is ____" at midspan.

Notes to designer:

1. In deflection diagram and camber note, show dimensions as required and draw finished grade and bottom of deck slab to approximate correct shape (gradient, hump or sag).
2. For steel plate girders, change wording beam/beams to girder/girders and change diaphragms to cross frames.
3. Cells for dead load deflection diagrams and camber note are located in the steel girder cell library.
Notes to designer:

1. Change "BEAM" to "GIRDER" in table heading and "Beam" to "Girder" in column heading for plate girder span(s).

2. Change $L$ beam to $L$ girder in the plan view for plate girder span(s).

3. Cells for plan and table of elevations are located in the rolled beam and steel plate girder cell libraries.

4. Deck elevations correspond to dead load deflection dimension points for simple spans.

5. For spans < 100’, show elevations at 1/5 points. For spans $\geq$ 100’, show elevations at 1/10 points.
Notes to designer:

1. Set bolster thickness so that top of web is parallel to grade. Bolster thickness may exceed but shall not be less than that in Note 2 below. In setting bolster at interior piers, use larger of the two spans for computing CT.

2. Bolster = CT + \( \Sigma D \)

3. \( h = \text{bolster} + \text{thickest top flange in span(s)} \)

4. For end spans, Midpoint is the halfway between \( \ell \) bearing and \( \ell \) splice. If there is no splice, Midpoint coincides with the midspan. For interior spans, Midpoint is the halfway between the splices.

6. Where "H" values differ for each line of girders, the form of the table may be changed to conveniently show each girder.

7. See File No. 11.06-1 for definition of symbols.

8. See File Nos. 11.06-9 and -10 for sample calculations for "h".

9. See Part 7, of this manual, Steel Plate Girders, for standard drawings for 2, 3 and 4 span continuous units.
Notes to designer:

1. Set bolster thickness so that top of web is parallel to grade. Bolster thickness may exceed but shall not be less than that in Note 2 below. In setting the bolster at interior piers, use the larger of the two spans for computing CT. In situations where this will give excessive bolster, the top of web need not be held parallel to the grade.

2. Bolster = CT + \Sigma D

3. \( h = \) bolster + thickest top flange in span(s).

4. For end spans, Midpoint is the halfway between top of web as fabricated and \( \bar{H} \) bearing and \( \bar{L} \) splice. If there is no splice, Midpoint coincides with the midspan. For interior spans, Midpoint is the halfway between the splices.

5. Where "H" values differ for each line of girders, the form of the table may be changed to conveniently show each girder.

6. See File No. 11.06-1 for definition of symbols.

7. See File Nos. 11.06-9 and -10 for sample calculations for "h".

8. See Part 7, of this manual, Steel Plate Girders, for standard drawings for 2, 3 and 4 span continuous units.
Notes to designer:

1. Set bolster thickness so that top of web is parallel to grade. Bolster thickness may exceed but shall not be less than that in Note 2 below. In setting the bolster at interior piers, use the larger of the two spans for computing CT. In situations where this will give excessive bolster, the top of web need not be held parallel to the grade.
2. 
3. 
4. For end spans, Midpoint is the halfway between £ bearing and £ splice. If there is no splice, Midpoint coincides with the midspan. For interior spans, Midpoint is the halfway between the splices.
5. Where “H” values differ for each line of girders, the form of the table may be changed to conveniently show each girder.
6. See File No. 11.06-1 for definition of symbols.
7. See File Nos. 11.06-9 and -10 for sample calculations for "h".
8. See Part 7, of this manual, Steel Plate Girders, for standard drawings for 2, 3 and 4 span continuous units.
DESIGN EXAMPLE FOR COMPUTING MINIMUM BOLSTERS:

Given: Curved alignment - straight girders (laid out on long chord)
Cross slope: ¼" per foot
Offsets at Abutment A = 0, Pier 1 = 1'- 2 1/4", Pier 2 = 1'- 1 1/8" and Pier 3 = 0
Slab = 8 1/2" min
Flange sizes shown at piers only

Span layout as follows:

[Diagram with labels: Slab, PL 2 3/4 x 18, Bolster, Pi 2/2 x 18, Top of slab, Web PL 1/2 x 66, 135'-0", 150'-0", 125'-0", 1'-2 1/4", 1'-1 1/8", Crte. 100, Bearing at Abutment A, Bearing at Pier 1, Bearing at Pier 2, Bearing at Pier 3]
DESIGN EXAMPLE FOR COMPUTING MINIMUM BOLSTERS: (cont’d)

Required: Compute minimum bolster and "h" which is needed for setting seat elevations.

Spans a and b at Pier 1:

\[
CT = \frac{1/8 \times (0.5 \times 150)}{10} = 0.9375'' \rightarrow \frac{15}{16}''
\]

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

\[
\text{cross slope x } \frac{1}{2} \text{ flange width = } \frac{1}{4}'' \times \frac{1}{12} \times (\frac{1}{2} \times 18'') = 0.1875''
\]

\[
\text{cross slope x horizontal ordinate = } \frac{1}{4}'' \times \frac{1}{12} \times (1' - 2 \frac{1}{4}'' \times 18') = 0.2969''
\]

\[
\Sigma D = 0.9219'' \rightarrow \frac{15}{16}''
\]

Minimum bolster = \[CT + \Sigma D \]

\[= \frac{15}{16}'' + \frac{15}{16}'' \]

\[= 1 \frac{7}{8}'' \]

\[h_1 = \text{minimum bolster + thickest flange in spans a and b} \]

\[h_1 = 1 \frac{7}{8}'' + 2 \frac{3}{4}'' = 4 \frac{3}{8}'' \]

Note: Spans b and c were also checked, but did not control the design.
Notes to designer:

1. Information in boxes is for designer's information only and is not to be placed on plans.

2. For spans $\geq 100'$, show deflections and elevations at 1/10 points. For spans $< 100'$, show deflections and elevations at 1/5 points.

3. See File No. 11.06-1 for definition of symbols.

4. See Part 7, of this manual, Steel Plate Girders, for standard drawings for dead load deflection diagrams and tables for top of slab elevations along 4 girder for two, three and four span continuous units.
CHECK LIST FOR SAMPLE DEAD LOAD DEFLECTION SHEET:

1. Dead load deflections diagrams should be drawn to an appropriate deflected shape.
2. Label Line thru center of bearings at Abutment A or Abutment B.
3. Label Line thru center of bearings and C/L pier(s).
4. Show and label finished grade after full dead load deflection.
5. Show and label top of web after full dead load deflection.
6. Show and label top of web after deflection from steel dead load only.
7. Show and label fifth or tenth points as required.
8. Label 5 or 10 equal spaces as required.
9. Show the dead load deflections and total deflections in table.
10. Label Dead Load Deflections.
11. Show top of slab elevations along C/L girder.
12. For instructions on completing the title block, see File No. 03.03.
13. For instructions on completing the notes, see File Nos. 04.03-1 and -2.
14. For instructions on completing the project block, see File No. 04.01.
15. For instructions on developing the CADD sheet number, see File Nos. 01.01-6 and 01.14-4.
16. For instructions on completing the block for sealing, signing and dating this sheet, see File Nos. 01.16-1 thru -6.
CHECK LIST FOR SAMPLE CAMBER SHEET:

1. Camber diagrams should be drawn to an appropriate deflected shape and scale.
2. Label Line thru center of bearings at Abutment A or Abutment B.
3. Label Line thru center of bearings and pier(s).
4. Show and label horizontal reference line.
5. Show and label deflected top of web without CT, matching the direction and shape of grade.
6. Show and label top of web as fabricated.
7. Show total = dead load deflection plus H.
8. Dimension the distance between midpoint and field splice/abutment. Dimension the distance between the field splice and pier.
9. Show and label midpoint (the middle point of an individual shipping piece).
10. Show and label bolted field splices.
11. Label H, distance between horizontal reference line and deflected top of web at any point being considered.
12. Show camber in table.
13. Show definition of symbols.
14. Label CAMBER.
15. For instructions on completing the title block, see File No. 03.03.
16. For instructions on completing the notes, see File Nos. 04.03-1 and -2.
17. For instructions on completing the project block, see File No. 04.01.
18. For instructions on developing the CADD sheet number, see File Nos. 01.01-7 and 01.14-4.
19. For instructions on completing the block for sealing, signing and dating this sheet, see File Nos. 01.16-1 thru -6.
GENERAL INFORMATION:

Cross frames and diaphragms shall be used to provide discrete bracing of rolled beams and plate girders. VDOT has standard diaphragms and cross frames which shall be used when a line beam/girder program is used to design girders/beams. When a grid analysis or a 3D analysis is used to design the girder/beam, the cross frames shall be designed as primary members and carry the loads based on the analysis.

Diaphragms:

Diaphragms may be used on plate girders with a depth of 36" or less and rolled beams. Diaphragms and connector plates shall be placed parallel to supports for skew angle up to 20 degrees. For skews over 20 degrees, diaphragms shall be normal to the beams/girders. Diaphragms shall be placed level except at supports where they shall be parallel to the deck. Diaphragms may be coped and bent or may be used with bent gusset plates when diaphragms and bearing stiffeners are not aligned.

Cross Frames:

Cross frames shall be placed parallel to supports for skew angle up to 20 degrees. For skews over 20 degrees, cross frames shall be normal to the girders. Cross frame chords shall be parallel to the deck. Diagonal and horizontal members shall be equal leg angles. Angle legs shall be oriented with the horizontal leg above the vertical leg. When welding individual cross frame members to gusset plates or to each other welds shall be on three (3) sides. 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 $r_z$.

It is the designer’s responsibility to design the cross frames including location and member sizes, weld lengths, location and geometry of bolt groups and size of bolts, gusset plate sizes, connection plate sizes.

DETAIL C

STEEL
CROSS FRAME/DIAPHRAGM DETAILS
GENERAL INFORMATION
Designed by Line Girder Analysis:

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:

Cross frame members (equal leg angles, channels, WT’s, etc.) in curved girder 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. Weld length shall be a minimum of ½ of the leg width. The minimum weld shall also be based on capacity or fatigue considerations, whichever controls.

4. For interior beams/girders, the connection plates shall be placed in pairs even when there is no corresponding cross frame.

5. All connection plates shall be welded to both flanges and web.
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 \( \frac{5}{16} \)” 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.
4. Items in double parentheses are for designer information and are not to be placed on plans.
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. Do not show spacing on plans.
4. Diaphragms shall be normal to the beams when the supports are skewed more than 20°.
5. Items in double parentheses are for designer information and are not to be placed on plans.
Notes to designer:


2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" and a girder depth of 8'-0".

3. Maximum angle is limited to 60°.

4. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as needed.

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:


2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" and a girder depth of 8'-0".

3. Maximum angle is limited to 60°.

4. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as needed.

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:

2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" (along skew) and a girder depth of 8'-0".
3. Maximum angle is limited to 60°.
4. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as needed.
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.
Notes to designer:

2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" (along skew) and a girder depth of 8'-0".
3. Maximum angle is limited to 60°.
4. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge. The 2" dimension shown between diagonals may be adjusted as needed.
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:


2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" and a girder depth of 8'-0".

3. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge.

4. Details shown are also used at interior support lines (piers) of continuous spans.

5. The 3" dimension shown on the bottom angle shall be increased to 6" at piers.

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


2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" and a girder depth of 8'-0".

3. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge.

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

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

2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" (along skew) and girder depth of 8'-0".
3. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge.
4. Details are shown for skewed end cross frames of simple spans and exterior skewed support lines of continuous spans.
5. For Detail C, see File No. 11.07-1.
Notes to designer:

2. Cross frame spacing as detailed is limited to a girder spacing of 12'-0" (along skew) and a girder depth of 8'-0".
3. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge.
4. Details shown are used also at skewed interior support lines (piers) of continuous spans.
5. The 3" dimension shown on the bottom angle shall be increased to 6" at piers.
6. For Detail C, see File No. 11.07-1.
Notes to designer:

1. No Standard sheet is provided for curved girder because the details required will be project specific.

2. The designer shall design the cross frames including member sizes, weld lengths, location and geometry of bolt groups, size of bolts, gusset plate sizes, connection plate sizes, etc.

3. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge.
Notes to designer:

1. No Standard sheet is provided for curved girder because the details required will be project specific.

2. The designer shall design the cross frames including member sizes, weld lengths, location and geometry of bolt groups, size of bolts, gusset plate sizes, connection plate sizes, etc.

3. For bridge requiring future utility attachments, if reasonable, avoid mixing V-type and X-type bracing throughout the length of bridge.
STEEL PIER CAPS* AND STEEL BOX (TUB) GIRDERS:

FRACTURE CRITICAL:

See File No. 11.01-1.

DESIGN:

Fracture critical elements 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 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, post-tensioned concrete beams, 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 should consider following 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, designer should consider alternatives to welding such as bolting, increasing plate sizes and altering box geometry before considering welding.

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, are also 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.

Longitudinal stiffeners shall not be permitted.

Grade between ends of pier caps/box girders shall be 2% minimum.
DETAILS: SPLICES, CONNECTIONS, DIAPHRAGMS, CROSS FRAMES, ETC. (cont’d)

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.

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 1/8" mesh opening. If a threaded insert is provided on the outside of the tube, it shall also be covered by screen.

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. For boxes over 100'-0" in length, the designer shall receive guidance from the District Structure and Bridge Engineer on the need of adding a third door in the top of the box with a ladder bolted to the interior.

Access from the bottom of the pier cap/box girder is discouraged. If used, doors shall be counterweighted to improve access. Designer shall provide ladders or rungs on the interior of the box to improve access.

Access door size shall be 24" x 36" minimum. Round corners with 6" radius. 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 same as for internal openings.

Top and end access doors should swing out and be easy to handle. Bottom hatches shall swing in and counterweighted for easy handling.

Access doors shall have a hasp and means of positively closing doors. Bolting doors shut should be a last resort. 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 causes.

Material for hinges shall be wrought or forged stainless steel with permanently lubricated hinge pins that are replaceable in the field with hand tools.

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.
DETAILS: SPLICES, CONNECTIONS, DIAPHRAGMS, CROSS FRAMES, ETC. (cont’d)

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.

Plate diaphragms shall provide an opening for fabrication/inspection personnel. A 24" x 36" minimum opening is required.

Openings shall be located in a position that will allow best access passing through the pier/box. Openings for plate diaphragms shall be rounded corners with 6" radius. Stress rings around the perimeters of openings are strongly discouraged as they impede access.

Placement of stiffeners effectively reduces the size of the openings by creating a tunnel effect. Details should place vertical stiffeners back from the edge of the openings by 6". 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.

Grab bars:

One grab bar oriented vertically shall be installed at openings on each side of the plate diaphragm.

If the opening is located in such a position that someone must stand on something other than the bottom of the box (a step or a flange) to pass through the opening, an additional horizontal grab bar shall be placed at the top of the opening on both sides of the plate diaphragm. The length of grab bars should be the same dimension as the parallel edge of the opening or greater.

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:

Where electrical service is available, provide a duplex receptacle inside the box between each plate diaphragm. Electrical service shall be in accordance with the latest edition of the National Electrical Code.

The wiring 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.

Where electrical service is not available, 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:

Structural Steel:

Kip 8.1

Section at Floorbeam

Description 21.0

Route 999 -99_008 .dgn

VDOT S&B Division

L Truss

Price for Prefabricated Steel Truss Superstructure.

Superstructure. Cost of the representative shall be included in the bid.

Erection to provide technical assistance in planning and erecting the prefabricated truss superstructure supplier at the bridge site during

TRUSS SUPERSTRUCTURE REQUIREMENTS:

These plans are for a fully engineered clear span steel superstructure and

must be submitted to the Engineer for review and approval.

Modifications to AASHTO (IIM-S&B-80.3).

For construction tolerances and future wearing surface are given in VDOT

according to Load and Resistance Factor Design. Design loading allowances


The superstructure shall be designed for HL-93 loading in accordance

Lower chord members shall allow for free drainage and be configured as to

Welding of lateral bracing to the flooring system shall not be permitted.

The truss shall be of the Modified Warren type (similar to U.S. Bridge

Cambridge Flat Truss). Floor beam and stringer connections shall be bolted.

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Modifications to AASHTO (IIM-S&B-80.3).

For construction tolerances and future wearing surface are given in VDOT

according to Load and Resistance Factor Design. Design loading allowances

TRUSS SUPERSTRUCTURE REQUIREMENTS:

Structural steel shall be ASTM A572 Grade 50.

All truss members, including gussets, plates, floor beams, and any other components designated as main load carrying shall be designed in accordance with AASHTO LRFD Bridge Design Specifications, AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members, and the shop drawings shall be reviewed by the Engineer prior to fabrication and erection. Design plans shall be approved by the Engineer prior to fabrication and erection. Design shall be accomplished under the direction of the Engineer and certified to the specifications of the Virginia Department of Transportation. Structural steel shall be designed and fabricated in accordance with AASHTO LRFD Bridge Design Specifications.

Truss members shall be capable of being installed without any support. Truss connections shall be bolted. Welding of truss members shall be accomplished using electrodes and procedures acceptable to the Engineer. Field welds shall be power brushed and painted with two coats of paint meeting the requirements of ASTM A570 Grade 36 and hot-dipped galvanized after fabrication in accordance with ASTM A123.

Welding and Non-Destructive Testing:

Field welding shall be accomplished using electrodes and procedures acceptable to the Engineer. Field welds shall be power brushed and painted with two coats of paint meeting the requirements of ASTM A570 Grade 36 and hot-dipped galvanized after fabrication in accordance with ASTM A123.

Welding and Non-Destructive Testing:

Welding and Non-Destructive Testing shall be accomplished using electrodes and procedures acceptable to the Engineer. Field welds shall be power brushed and painted with two coats of paint meeting the requirements of ASTM A570 Grade 36 and hot-dipped galvanized after fabrication in accordance with ASTM A123.

GENERAL COMMENTS:

The Contractor is advised that substructure details shown in these Plans shall be modified based upon the final superstructure details provided during design. Some modification/redesign of the substructure may be necessary to accommodate the final superstructure details. The Contractor is advised that substructure details shown in these Plans shall be modified based upon the final superstructure details provided during design. Some modification/redesign of the substructure may be necessary to accommodate the final superstructure details.

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Supplementary Symbols Used with Welding Symbols

Location of Elements of a Welding Symbol

- Finish symbol
- Contour symbol
- Effective throat size
- Depth of preparation; size or strength for certain welds
- Specification, process or other reference
- Tall (omitted when reference is not used)
- Basic weld symbol or detail reference
- Groove angle; included angle of countersink for plug welds
- Root opening; depth of filling for plug and slot welds
- Length of weld
- Pitch (c-c spacing) of welds
- Field weld symbol
- Weld-all-around symbol
- Reference line
- Arrow connecting reference line to arrow side member of joint or arrow side of joint
- Number of spot, seam, plug, slot, or projection welds

Elements in this area remain as shown when tall and arrow are reversed

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>
<td></td>
<td></td>
<td>Flush</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Convex</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Concave</td>
</tr>
</tbody>
</table>
### 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</td>
<td>Any applicable single groove weld symbol</td>
</tr>
<tr>
<td>Root opening</td>
<td></td>
</tr>
</tbody>
</table>

#### Edge- and Corner- Flange Welding Symbols

<table>
<thead>
<tr>
<th>$\frac{1}{16}$</th>
<th>$\frac{1}{8} + \frac{1}{16}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Radius $\rightarrow$</td>
<td>Size of weld $\rightarrow$ $\frac{1}{16}$</td>
</tr>
<tr>
<td>Height above point of tangency</td>
<td></td>
</tr>
</tbody>
</table>

#### Plug Welding Symbol

<table>
<thead>
<tr>
<th>Included angle of countersink</th>
<th>Pitch (distance between centers of welds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size (diameter of hole at root)</td>
<td></td>
</tr>
<tr>
<td>$\frac{30}{4}$</td>
<td>$\frac{4}{4}$</td>
</tr>
<tr>
<td>Depth of filling in inches (omission indicates filling is complete)</td>
<td></td>
</tr>
</tbody>
</table>

#### Welding Symbols for Combined Welds

- $\frac{1}{4}$
- $\frac{1}{8}$
- $\frac{5}{16}$
Typical Welding Symbols

Single-V Groove Welding Symbol Indicating Root Penetration

Size

Depth of preparation

Effective throat

Root opening

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

Arrow points toward member to be prepared

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

Root opening

Groove angle

STEEL
WELDING
STANDARD WELDING SYMBOLS
Typical Welding Symbols

Spot Welding Symbol

Size (diameter of weld)
Strength (in. lb per weld)
may be used instead

Process reference must
be used to indicate
process desired

RSW 0.25 (5) 4

Number of welds
Pitch (distance
between centers)
of weld

Seam Welding Symbol

Size (width of weld)
Strength (in lb per linear
inch may be used instead

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

0.30 3 9 RSW

Process reference must
be used to indicate process
desired

Projection Welding Symbol

Projection welding
reference must be used

RSW 500 (7) 6

Pitch (distance
between centers)
of weld

Size (strength in lb per weld)
Diameter of weld may be used
Instead for circular projection
welds

Number of welds

Double-Fillet Welding Symbol

Size (length of leg)

Specification, process,
or other reference

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

STEEL
WELDING
STANDARD WELDING SYMBOLS

PART 2
DATE: 27Mar2013
SHEET 4 of 10
FILE NO. 11.10-4
Typical Welding Symbols

Flash or Upset Welding Symbol

No Arrow Side or Other Side Significance

Plug Weld Symbol

Chained Intermittent Fillet Welding Symbol

Size (length of leg)

Flare-V and Flare-Bevel-Groove Welding Symbols

Size (height of deposit)

Omission indicates no specific height desired

Surfacing Welding Symbol Indicating Built-up Surface

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

Staggered Intermittent Fillet Welding Symbol

Size (length of leg)

Length of Increments

STEEL WELDING
STANDARD WELDING SYMBOLS

PART 2
DATE: 27Mar2013
SHEET 5 of 10
FILE NO. 11.10-5
## 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>Indicating 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 symbol</td>
<td>Any applicable weld 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</td>
<td>penetration regardless</td>
</tr>
<tr>
<td>is 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>
</tr>
</thead>
<tbody>
<tr>
<td>Convex contour symbol</td>
</tr>
<tr>
<td>Indicates face of weld</td>
</tr>
<tr>
<td>to be finished to</td>
</tr>
<tr>
<td>convex contour</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Flush Contour Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flush contour symbol</td>
</tr>
<tr>
<td>Indicates face of weld</td>
</tr>
<tr>
<td>to be made flush. When</td>
</tr>
<tr>
<td>used without a finish</td>
</tr>
<tr>
<td>symbol, indicates weld</td>
</tr>
<tr>
<td>without subsequent</td>
</tr>
<tr>
<td>finishing</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Multiple Reference Lines</th>
</tr>
</thead>
<tbody>
<tr>
<td>First operation shown on</td>
</tr>
<tr>
<td>reference line nearest</td>
</tr>
<tr>
<td>arrow</td>
</tr>
<tr>
<td>1st</td>
</tr>
<tr>
<td>Second operation or</td>
</tr>
<tr>
<td>supplementary data</td>
</tr>
<tr>
<td>2nd</td>
</tr>
<tr>
<td>Third operation or</td>
</tr>
<tr>
<td>test Information</td>
</tr>
<tr>
<td>3rd</td>
</tr>
</tbody>
</table>
## Basic Welding Symbols and Their Location Significance

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

### Plug or Slot  
- Arrow Side: PLA, BKA  
- Other Side: PLO, BK0  
- Both Sides: SP0, EO0

### Spot or Projection
- Arrow Side: SPA  
- Other Side: SP0  
- Both Sides: SPN

### Seam
- Arrow Side: SEA  
- Other Side: SEO  
- Both Sides: SEN

### Back or Backing
- Arrow Side: Groove weld symbol  
- Other Side: Groove weld symbol  
- Both Sides: Groove weld symbol
## Basic Welding Symbols and Their Location Significance

### Fillet Welds

<table>
<thead>
<tr>
<th>Location Significance</th>
<th>Square</th>
<th>V</th>
<th>Bevel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arrow Side</td>
<td>FIA</td>
<td>GSQA</td>
<td>GVA</td>
</tr>
<tr>
<td>Other Side</td>
<td>FIO</td>
<td>GSOQ</td>
<td>GVD</td>
</tr>
<tr>
<td>Both Sides</td>
<td>FIB</td>
<td>GSQB</td>
<td>GBV</td>
</tr>
<tr>
<td>No Arrow Side or Other Side Significance</td>
<td>GSQN</td>
<td>GSQB</td>
<td>GEBN</td>
</tr>
</tbody>
</table>

### Groove Welds

<table>
<thead>
<tr>
<th>Location Significance</th>
<th>U</th>
<th>J</th>
<th>Flare-V</th>
<th>Flare-Bevel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arrow Side</td>
<td>GUA</td>
<td>GJA</td>
<td>GFVA</td>
<td>GFBA</td>
</tr>
<tr>
<td>Other Side</td>
<td>GQO</td>
<td>GJO</td>
<td>GFVO</td>
<td>GFB0</td>
</tr>
<tr>
<td>Both Sides</td>
<td>GUB</td>
<td>GJB</td>
<td>GFVB</td>
<td>GFBB</td>
</tr>
<tr>
<td>No Arrow Side or Other Side Significance</td>
<td>GSQN</td>
<td>GSQB</td>
<td>GEBN</td>
<td></td>
</tr>
</tbody>
</table>

---

STEEL WELDING

STANDARD WELDING SYMBOLS
### 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><img src="image1.png" alt="Butt Joint Diagram" /></td>
<td><img src="image2.png" alt="Lap Joint Diagram" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Corner Joint</th>
<th>T-Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image3.png" alt="Corner Joint Diagram" /></td>
<td><img src="image4.png" alt="T-Joint Diagram" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Edge Joint</th>
<th>Process Abbreviations</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image5.png" alt="Edge Joint Diagram" /></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 2.4-79, 71.</td>
</tr>
</tbody>
</table>

---

**STEEL WELDING STANDARD WELDING SYMBOLS**

**PART 2**
**DATE:** 27Mar2013
**SHEET** 9 of 10
**FILE NO.** 11.10-9
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.