

Chapter 6 — Steel Superstructures

6.1—OVERVIEW

This chapter provides guidance on the use of steel superstructures.

For locations where limited clearance is available, a steel superstructure can be a better choice because of shallower superstructure depth and better defined cambers on site. Welded steel plate girders webs are cut according to the plan camber so there is very little uncertainty in the camber compared to precast/prestressed concrete girders.

6.2—MATERIALS

6.2.1—Grades

All structural steel used for bridge superstructure construction shall be one of the following ASTM A709 grades:

- Grade 50W
- Grade HPS 70W
- Grade HPS 100W (Bridge Division approval required)

Grade 36 steel is essentially obsolete, as it comes from the same melt as grade 50 steel, and it is recertified down to grade 36 standards. This applies to all rolled shapes and plates. Therefore, it is advantageous to design for 50 ksi yield and take advantage of the higher strength properties.

6.2.2—Uses

For superstructure applications, uncoated Grade 50W steel is the most common material.

In suitable environments, uncoated weathering steel develops a thin oxidation layer, also called a surface patina, that is small grained (dense) and tightly adherent to the base metal. Once this oxidation layer builds to a sufficient thickness and density, the exposure of the base metal to moisture and oxygen is blocked and further corrosion is greatly reduced.

While the use of weathering steel can have a benefit with respect to long-term maintenance costs, it can also have a benefit in first cost. The first-cost savings come not only from saving the cost of paint, but also savings in the surface treatment and handling of the finished girder in the fabrication process.

Steel girders may utilize Grade HPS 70W for hybrid girders; Grade HPS 100W shall not be used without approval from Bridge Division.

Steel grades with $F_y \leq 70$ ksi show definite yield plateaus with similar ductility. However, the Grade HPS 100W steel does not have a clearly defined yield plateau and shows slightly lower ductility compared to the lower strength grades. It is for this reason that permission must be obtained before using Grade HPS 100W in designs. Grade HPS 50W material is more expensive and does not offer any design advantages over Grade 50W material. Therefore, Grade HPS 50W material shall not be utilized for design.

The limited number of mills may also have adverse effects on the delivery schedule. If HPS is considered for a design, potential suppliers should be contacted to discuss availability and cost-effectiveness. For preliminary checks Garrell & Hopper have compiled a guide to ensuring the requested plate size is manufactured by an adequate number of mills (2023). HPS may come at a premium price due to the limited number of mills that are rolling it.

Do not use Grade 50 steel for superstructure applications.

There is minimal cost difference between Grade 50 and Grade 50W steel, therefore weathering steel should always be used for superstructures even if they are painted.

Hot dip galvanization is a common coating method for new structures where components are small enough to fit into the tanks. This includes short spans as well as components such as separators where galvanization is simpler than painting.

6.2.3—Painting of Weathering Steel

6.2.3.1—Zone Painting

The following locations shall be zone painted on all steel girders:

- Girder ends (at abutments and near expansion joints, if provided) whether encased in concrete or not. The steel shall be painted for a length of at least 1.5 times the web depth with a minimum length of 5 ft., measured from centerline of bearing. Provide details as shown in Figure 6.2 and Figure 6.3 in the plans.
- The full length of exterior girders when open rails are utilized at the deck edge. Provide the detail shown in Figure 6.1 in the plans.
- The full length of median side exterior girders on twin structures with 30 ft. or less clear distance between edges of decks. This applies for both open and closed railings. Provide the detail shown in Figure 6.1 in the plans.

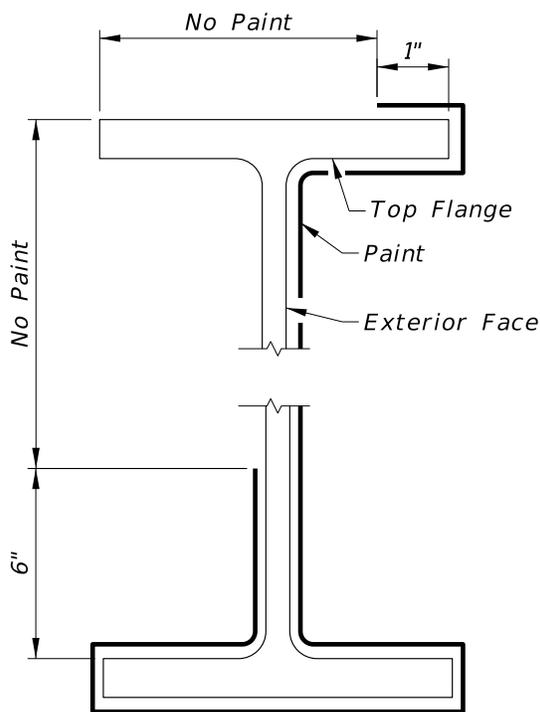


Figure 6.1—Exterior Girder Zone Coat Detail

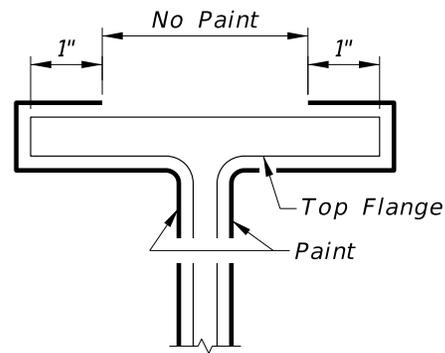
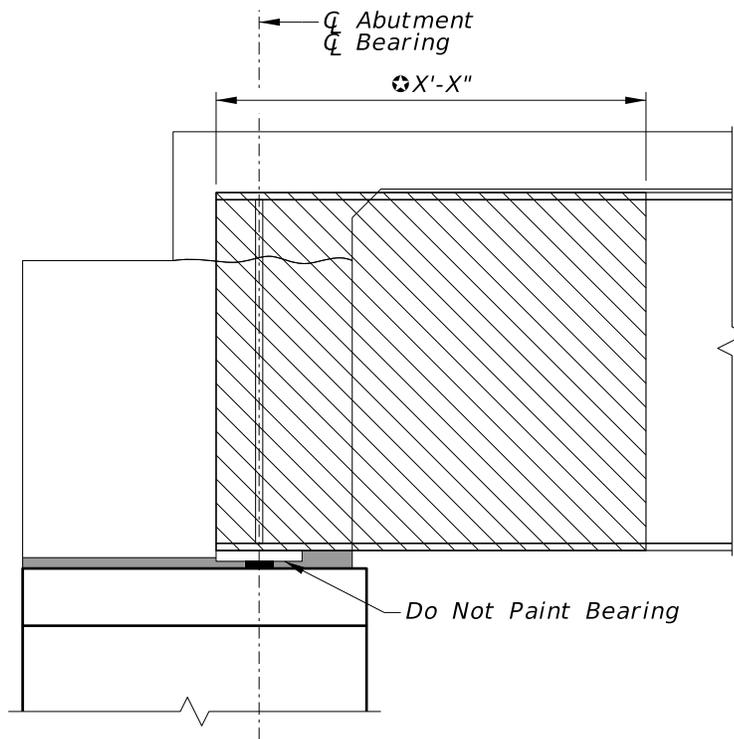


Figure 6.2—Top Flange Paint Detail



⊗ Limits of "PAINTING STRUCTURE (ZONE COAT) AT STA. XX+XX.XX" Apply paint to all girder faces within the limits indicated at Abutment No. 1 & Abutment No. 2 (see top flange detail). At least first coat of system shall be applied prior to turndown construction.

Figure 6.3—Girder End Zone Coat Detail

For localized painting of new and existing structural steel (Zone Coat) and existing bearings, existing piling and miscellaneous steel, the special provision "Localized Painting of Steel" must be included in the Contract Documents.

6.2.3.2—Full Painting

Full painting of a weathering steel superstructure will be determined on a case-by-case basis with input from Bridge Division. Painting requirements, if any, shall be determined during the preliminary design phase and noted in the BDS.

This section describes painting of new steel superstructures. See §7.5 for guidelines on painting existing steel

Paint should be applied to weathering steel superstructures when certain unfavorable conditions exist that may inhibit formation of the protective patina. These unfavorable conditions may consist of the following:

- Grade separations in tunnel-like conditions.
- Low-level water crossings.
- Industrial areas where concentrated chemical fumes may drift directly onto the bridge.

When bridges meet these criteria, consideration should be given to using painted weathering steel for the whole bridge or only the specific regions that are the most vulnerable (zone painting at the exterior girders, or over the roadway between bolted field splices).

Painting of new structural steel structures (complete structures) shall be in accordance with Section 709 of the *Standard Specifications*.

6.2.4—Fracture Toughness Requirements

The Temperature Zone Designation for Charpy V-Notch testing for steel elements used in Nebraska must comply with Zone 2.

Steel girder layouts will indicate the compression and tension zones for the purpose of Charpy V-Notch testing. Annotation for tension (T) or fracture critical (F) along with the temperature zone shall be added to the plans along with the ASTM A709 grade designation.

The cell shown in “[Figure 6.8—CHARP1 Cell \(Steel Girder Library\)](#)” shall be used on welded plate girder structures and the cell shown in “[Figure 6.10—CHARP2 Cell \(Steel Girder Library\)](#)” shall be used on rolled beam girder structures.

Particular consideration shall be given to painting weathering steel at grade separations with “tunnel-like” conditions when one or more of the following conditions are present:

- Vertical clearance is 20 feet or less because these bridges are more susceptible to “tunnel-like” conditions.
- Bridges over interstates or expressways in urban corridors, since deicer treatment in these areas is typically more concentrated.
- Heavy Trucks $\geq 10\%$ under the bridge, since trucks generate more misting with deicers than cars.
- Posted speed limit is 55 mph or greater for the roadway being crossed, since higher speeds generate more misting with deicers.

Painting weathering steel at locations other than roadway grade separations should also be considered when the following conditions are present:

- When the distance between OHW and the bottom of the girders is less than three feet.
- Railroad grade separations where excessive diesel exhaust may be generated when the locomotive is under heavy load. Examples of where this might happen include negotiating steep longitudinal grades or starting from a stopped condition.

The coating systems used for painting new and existing structural steel are different. New structural steel will receive a shop-applied primer coat, and the rest of the coating system will be applied in the field after erection. The coating systems used for existing steel are all applied in the field and change based on the exposure conditions of the items being painted.

Refer to LRFDBDS Article 6.6.2.1 for determining which members require Charpy V-Notch testing.

The cells may need to be adjusted for structures where additional members qualify as main tension members (e.g. curved structures).

6.2.5—Bolts, Nuts and Washers

ASTM F3125, Gr. A325 (Type 1 or Type 3) high strength bolts shall be used.

For joining galvanized members, use Type 1 galvanized bolts. When joining painted or unpainted weathering steel applications, use Type 3 bolts.

Stainless steel bolts are not allowable for structural applications.

Type 1 bolts can be galvanized, while Type 3 bolts use weathering steel and do not required galvanization.

There are currently no stainless steel bolts available that meet the strength and ductility requirements of ASTM F3125.

Nuts and hardened steel washers must comply with the ASTM requirements referenced in the current edition of the *Nebraska Standard Specifications for Highway Construction*.

Pairs of beveled washers should be specified when the angle formed between the nut or bolt head contact surface and the member is 20:1 or steeper. Plate washers must be used for round holes larger than oversized or slotted holes. Plate washers must have a minimum thickness of $\frac{5}{16}$ in. and made of the same material as the connected element.

6.2.6—Welds

The tensile strength of the weld material must be compatible the tensile strength of the base material. The AWS D1.5 table used to match electrodes with base metal is partially reproduced in [Table 6.1](#) for use in design.

Table 6.1—Welding Electrode Compatibility

Base Metal	Electrode Designation	Electrode Tensile Strength (F_{EXX})
Grades 36, 50, 50W, and HPS 50W	E70XX	70 ksi
Grade HPS 70W	E90XX	90 ksi
Grade HPS 100W	E110XX	110 ksi

6.3—GIRDER ERECTION PLANS

For bridges where any of the following occurs, girder erection plans shall be required to be submitted by the Contractor through the shop drawing process.

- Span lengths greater than 200 ft.
- Girders with horizontal curvature.
- Field assembled tied arch, truss, or other non-typical spans.

6.4—CONNECTIONS

6.4.1—Welded Connections

Field welded connections shall be avoided.

Welding previously galvanized surfaces shall be avoided due to the damage incurred on the galvanizing. If previously galvanized surfaces are welded, they shall be repaired using zinc rich paint in accordance with the *Standard Specifications* Section 1061, Repair Method 2.

Tensile strength for weld material is matched to base material strength. Additional information can be found in §6.2.6.

6.4.2—Bolted Connections

The preferred bolt size for all connections is $\frac{7}{8}$ in. diameter. For primary members, larger diameter bolts may be used. For secondary members only, $\frac{3}{4}$ in. diameter bolts may be used as a minimum when required by geometry.

For primary member connections, only standard size holes shall be used. For secondary member connections, oversized holes may be used in one ply only.

For cross-frame connection bolts, the bolts shall be designed with threads included in the shear plane.

The bolts for these types of connections are typically very short, making it impossible to keep the threads out of the shear plane.

Standard Note #093 shall be placed on the front sheet and lists which thread conditions shall be used.

6.4.3—Field Splices

Welded field splices are not allowed. Only bolted field splices shall be used.

Field splices shall be located and dimensioned as follows:

- Locate field splice in constant web depth area.
- Locate field splices as close as possible to the points of dead load contraflexure (at Strength I load combination).
- Girder clear gap shall not be less than $\frac{1}{16}$ in. nor shall it exceed $\frac{1}{4}$ in., and web center shall be used for alignment details (not shown in the plans).
- The number of different splice plate thicknesses specified for a bridge shall be minimized.

Field splices shall be designed as listed below:

- A minimum of eight bolts shall be used for each half of a flange splice on all girders.
- All field splices shall be designed as slip critical connections.
- Class B surface condition shall be used for splice design for weathering steel girders blast cleaned and for painted surfaces with zinc primer only.
- Flange splice bolt threads shall be excluded from the shear plane. Web splice bolt threads shall be designed as included in the shear planes.

Field welding often produces poor results due to the limited ability to control environmental conditions.

All primers listed on the Approved Product List for Paint Systems - New Structural Steel meet Class B surface conditions.

The web and its splice plates may be thin enough that threads will be included in the shear plane. It is therefore more conservative to always consider them as included in the shear plane.

6.5—COMPOSITE DESIGN AND SHEAR CONNECTORS

6.5.1—Composite Girder Design

All steel girders shall be designed for composite action in both the positive and negative moment regions. Stud-type shear connectors shall be used to create the composite connection between the concrete deck and the steel girder.

6.5.2—Shear Connectors

End welded stud-type shear connectors shall be $\frac{7}{8}$ in. diameter, installed in rows of three or more.

Standard shear connector height shall be 5 in. Variations in flange and haunch thickness may require modifications to shear connector height. Designers shall ensure that the bottom of the shear connector head extends above the bottom mat of longitudinal reinforcement in the deck at all locations.

Shear connector spacing shall not exceed 24 in. longitudinally, except that they may be omitted to clear flange splice plates that are 42 in. or less in length. For flange splice plates with a length greater than 42 in., shear connectors shall be attached to those splice plates.

Shear connectors applied to flange splice plates shall be installed in rows of two or more, and shall provide a minimum of 3 stud diameters between centerline of shear connector and centerline of bolt.

6.6—I-SECTION FLEXURAL MEMBERS

6.6.1—General Design Considerations

Girders shall be spaced at a maximum of 15 ft. center to center. For deck overhang limitations, see §9.2.3.

The use of cover plates is not allowed for new designs.

In accordance with LRFDBDS Articles 6.10.1.1.1.b and c., only the reinforcing steel in the deck shall be considered to resist flexure in negative moment regions. In the positive moment regions, the entire transformed concrete deck section shall be considered to resist flexure.

W-sections at slab thickenings near discontinuities may be allowed to have rows of only 2 shear connectors when space is limited.

LRFD BDS 8th edition modified maximum pitch to allow up to 48 in. for members having depths greater than 48 in. Bridge Division maintains the 24 in. maximum to align with previous codes.

This is to ensure that the weld around the stud does not interfere with the bolts. For the typical $\frac{7}{8}$ in. stud diameter, the minimum distance can be taken as $2\frac{1}{2}$ in. for ease of detailing.

Adding cover plates to girders is generally not economical and can present fatigue problems. Cover plates can be considered for retrofit situations when no other options are available but are not allowable for new designs.

6.6.2—Stiffeners

6.6.2.1—Bearing Stiffeners

Bearing stiffeners shall be provided on both sides of the web on all I-girders, and shall extend as close as practical to the outer edges of the flanges. In the case where top and bottom flanges are of different widths, the bearing stiffener shall extend to the edge of the wider flange and shall be clipped at 45° for the narrower flange. Bearing stiffeners shall be installed perpendicular to the web.

- Bearing stiffeners shall be fillet welded to the web and to both the top and bottom flange for all bridges.
- For straight bridges, the bearing stiffeners shall be tight fit at the top flange and milled (or ground) to bear at the bottom flange.
- For horizontally curved bridges, the bearing stiffeners shall be milled (or ground) to bear at both flanges.
- At the fabricator's options, the milled (or ground) bearing stiffener connection(s) may be welded with a complete penetration weld in lieu of a fillet weld for straight bridges. This is not permitted for horizontally curved bridges.

The minimum thickness for bearing stiffeners shall be $\frac{1}{2}$ in.

Designers shall give consideration to multiple bearing stiffeners at long spans to accommodate large movements.

6.6.2.2—Intermediate Connection Plate Stiffeners

Intermediate connection plate stiffeners shall be rigidly attached to both top and bottom flanges using welds. Intermediate connection plate stiffeners shall be aligned perpendicular to the web.

The minimum intermediate connection plate stiffener size is $\frac{1}{2}$ in. \times 5 in.

Refer to §6.6.6 for details.

6.6.2.3—Intermediate Stiffeners

Intermediate stiffeners are stiffeners without separator connections used solely for shear strengthening of girders.

Intermediate stiffeners shall be aligned perpendicular to the web.

The minimum intermediate stiffener size is $\frac{3}{8}$ in. \times 5 in.

Although there are instances, such as rolled beam girders or the use of integral abutments where bearing stiffeners may not be required by design, bearing stiffeners must still be provided at all support locations.

This is to limit the distortion of the top flange.

This attachment is required because differential movement of adjacent girders at the diaphragm location can cause distortion fatigue due to out-of-plane bending in the girder web if the stiffeners are not rigidly attached.

Connection plates at cross-frames or diaphragms also serve as intermediate shear stiffeners. However, their main purpose is to make the connection between the girder web and flanges and the cross-frames or diaphragms. There tends to be twisting at these connection locations, especially with large substructure skews and horizontal curvature in the girders. Therefore, connection plates are required to have a minimum thickness of $\frac{1}{2}$ in.

A minimum intermediate stiffener thickness of $\frac{3}{8}$ in. is adequate to stiffen the web for shear resistance and to develop post-buckling tension field action.

6.6.2.3.1—Straight Girders

Intermediate stiffeners need only be placed on one side of the web. For exterior girders place all stiffeners on the inside face of the web. For interior girders alternate the face of the web on which the stiffeners are placed.

When the design indicates a tension and compression flange at the stiffener location, the stiffener shall be fillet welded to the compression flange and cut back 2 in. from the tension flange.

When the design indicates both flanges are within tension zones at the stiffener location, the stiffener shall be detailed as a tight fit to both flanges but not welded to the flanges.

6.6.2.3.2—Horizontally Curved Girders

When the design indicates a tension and compression flange at the stiffener location, the stiffener shall be fillet welded to both flanges and need only be placed on one side of the web.

When the design indicates both flanges are within tension zones at the stiffener location, stiffeners shall be placed on both sides of the web and detailed as a tight fit to both flanges but not welded to the flanges.

6.6.3—Drip plates

Drip plates shall be provided on the upgrade side of all supports at the outside bottom flange of exterior girders. Drip plate details and locations relative to supports shall be in accordance with the standard details.

Both sides of intermediate supports count as upgrade on a haunched girder.

6.6.4—Rolled Beam Girders

6.6.4.1—General Design Considerations

For continuous spans up to approximately 120 ft., rolled beam girders (W-sections) are generally the most economical steel superstructure type. Typical practice is to use heavier section at negative moment regions and a lighter section at positive moment regions, with splices near the dead load contraflexure points.

Typical rolled beam piece lengths range up to 80 ft., though segments of certain piece sizes may be available up to approximately 120 ft. Designers should consult with fabricators to determine the longest practical segment lengths for the section sizes being considered.

When end spans are short compared to adjacent interior spans, consideration should be given to extending the heavier negative moment section to the abutments.

When changing section sizes for rolled beams, it is preferable to stay within a beam series. Matching up sections at field splices is more straightforward when the adjoining pieces are from the same beam series.

Rolled beam sections up to 36 in. depth are generally readily available up to approximately the W36x256 size. "Jumbo" sections of 36 in. depth heavier than W36x256, as well as 40 in. and 44 in. depth are available from a smaller group of producers and on a less-frequent rolling schedule. Designers should review the [availability list on AISC's website](#) and consult with fabricators to determine the availability of larger sizes.

Welded splices for rolled beam girder sections are not allowed.

Welded splices for rolled beam girder sections are not allowed in the shop or in the field. It is very difficult for fabricators and contractors to achieve quality complete joint penetration welds over the entire cross section of the rolled beam. The junction between the web and flanges is especially difficult in this regard.

6.6.4.2—Camber and Blocking

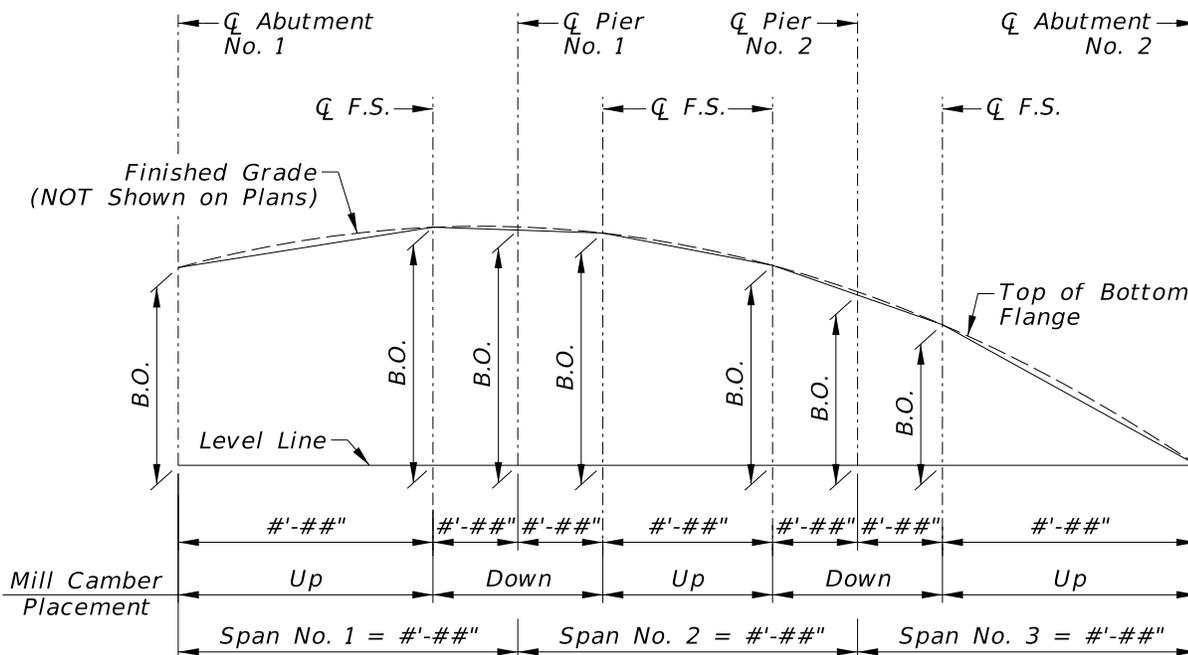
Roller beam girders are rarely cambered. Instead, the direction of mill camber is specified on the blocking diagram.

Mill camber is the natural camber resulting from the rolling and cooling process inherent in steel manufacturing. Tolerances for mill camber are described in ASTM A6.

A blocking diagram must be provided on the girder sheets. On the blocking diagram:

- Provide ordinates at all supports (abutments and intermediate supports), where the superstructure is discontinuous, and at all field splices. The blocking ordinates will match the grade offsets at the field splice locations.
- Specify the orientation (upward or downward) of the mill camber on the blocking diagram. Provide upward mill camber in spans and downward mill camber over interior support.
- Since a straight line is assumed between the blocking ordinates, the girder seat elevations at the intermediate supports may need to be adjusted accordingly.

This will counteract dead load deflections as much as possible.



ROLLED BEAM BLOCKING DIAGRAM

Figure 6.4—Rolled Beam Girder Blocking Diagram

Tables indicating the blocking ordinates, camber ordinates (if required) and deflections for shims, must be shown in the Plans. Blocking ordinates shall be reported in decimal feet to the nearest thousandth of a foot. Camber ordinates and deflections for shims shall be reported in decimal inches to the nearest hundredth of an inch.

Bridge Division approval is required prior to specifying cambering of rolled beam girders. Camber for rolled beam girders should only be considered when a shim thickness over 2 in. is anticipated due to vertical roadway curvature combined with dead load deflection. Maximum camber for rolled beam girders shall be $L/240$ where 'L' is the length of a field section.

Since camber is not generally allowed for rolled beam girders, camber ordinates will not typically be provided on the plans. Should camber be required, camber ordinates must be provided at quarter points of the field section(s) requiring camber

6.6.4.3—Shims

Plans must indicate the expected deck thickness (including shim) at all supports using the cell shown in "Figure 6.11—SHIM RB Cell (Bridge Deck Library)". The minimum deck thickness (including shim) for rolled beams is the distance from the top of the thickest top flange to the finished grade. The minimum shim must provide $1/2$ in. clear between the top flange and the bottom of the bridge deck. This provides a $1/2$ in. construction tolerance and a minimum shim at these locations. Field splice top plates may be embedded in the deck no more than 1 in.

6.6.4.4—Field Splice Elimination

For rolled beam girders only, and if approved by the Engineer, Contractors may eliminate any field splice by extending the heavier section without any change in contract price or quantities. Designers shall include Standard Note #088 on the Front Sheet of the Bridge Plans. Changes in the plans will be shown in the Shop Plans.

This change is only allowed for rolled beams as they are not cambered by the fabricator and thus the change is easier to accommodate during the shop drawing review process.

6.6.5—Welded Plate Girders

AISC/NSBA collaboration have created Standard Plans for Steel Bridges for single and continuous spans. These designs can be used as a starting point for sizing welded plate girders.

6.6.5.1—General Design Considerations

Parallel flange girder designs are preferred. Where site conditions, span lengths or aesthetics dictate, haunched girders may be used.

For routine steel girder design use a single web thickness and depth for the entire length of the girder and only vary flange sizes at the bolted field splices.

6.6.5.2—Field Sections

The field section lengths of girders shall be determined by the designer through a thorough evaluation of site conditions, economy and hauling considerations. Local fabricators currently have a maximum piece length limitation of 145 ft.

For girder lengths approaching this limit, the designer must ensure that a hauling permit can be secured for the proposed length and weights before final design can proceed.

6.6.5.3—Special Considerations for Horizontally Curved Girders

The following special considerations must be taken into account by the designer:

- Cross-frames, diaphragms, and their connections shall be designed as primary members.
- Shear connectors shall be designed for the effects due to curvature, as described in LRFD BDS Article 6.10.10.4.2., in addition to the combined effects of shear in the longitudinal direction produced by flexure and by shear in the lateral direction produced by cross-frame forces.
- Bearings shall be aligned along chords extending from fixed bearings to expansion bearings. See §14.2 for additional information.
- Suggested preliminary cross-frame spacing as listed in Table 6.2

Refer to AISC/NSBA website for latest version of the Standard Plans: <https://www.aisc.org/nsba/design-and-estimation-resources/standard-bridge-plans/>. The AISC/NSBA collaboration has generally found that there is little to no cost saving by using deeper sections than the minimum depths given by LRFDBDS 2.5.2.6.3.

Haunched girder designs are generally less economical than parallel flange designs. Haunched girders are typically designed and fabricated in two profiles: parabolic haunches and straight tapered haunches. Parabolic haunches are favored due to aesthetics and provide a profile that more closely matches moment gradients in negative flexural regions. Straight tapered haunches are favored by fabricators due to simplified cutting, bending and welding requirements. Where haunched girders are utilized, both profiles are acceptable, and the designer should choose the profile based on project needs.

The designer is advised that superloads require special permits. A superload is defined as a vehicle with a gross weight over 201 kips (201,000 pounds), a length greater than 160 ft., or a width greater than 16 ft. Therefore, a girder or group of girders with a weight of approximately 147 kips or greater, or a length of approximately 144 ft. or greater, would require vehicular transportation which is classified as a superload. Girders requiring a superload to transport should be avoided where possible.

For state projects, NDOT Bridge Office will coordinate with the NDOT permit office to determine if hauling permits can be secured.

Girders that must accommodate a horizontally curved roadway alignment will typically be welded plate girders as these can be fabricated to match the curvature of the roadway. Special design considerations are important for these types of bridges since the load paths for horizontally curved girders are slightly different. Cross-frames or diaphragms are necessary for controlling torsion and overall geometry in curved steel girder bridges.

Although the LRFDBDS does not specify maximum cross-frame or diaphragm spacings, some historic guidelines for horizontally curved girders have performed well over the years. The suggested spacings indicated are a good starting point for the design process.

Table 6.2—Preliminary Cross-frame or Diaphragm Spacing for Curved Girders

Centerline Radius of Bridge (ft)	Suggested Maximum Spacing(ft)
≤ 200	15
$200 < R \leq 500$	17
$500 < R \leq 1000$	20
> 1000	25

6.6.5.4—Flange Plates

The following guidelines will provide a more cost-effective welded plate girder and shall be followed as much as possible:

- Use maximum flange plates thickness of $2\frac{1}{2}$ in.
- Select flange width in 2 in. increments.
- Select flange plate thicknesses in $\frac{1}{8}$ in. increments up to 2 in. thickness and $\frac{1}{4}$ in. increments beyond.
- Use minimum flange size of $\frac{3}{4}$ in. \times 12 in.
- Minimize the number of different flange plate thicknesses used on a given bridge. For projects with multiple bridges, consider all bridges together if they are to be manufactured by the same fabricator. Consider a minimum order weight of 10 tons for any plate thickness per structure.
- Do not reduce the flange cross-sectional area by more than approximately one-half of the area of the heavier flange plate to facilitate stress flow at the transition of any type of splice.
- Detail a shop welded flange splice only when the reduced size represents a savings of approximately 1,200 lb. of plate per splice.
 - If the weight of steel saved is sufficient, reduce the flange thickness, but maintain the flange width. Thickness transitions are easier to fabricate than width transitions for welded splices.
 - If a flange width transition of 2 in. or more must be made at a splice, the corners of the wider flange must be clipped at a $2\frac{1}{2}$:1 transition. Change in flange thickness at butt welds must also include a $2\frac{1}{2}$ H:1V transition at the end of the thicker flange plate.
 - If a transition of both width and thickness must be provided at any location, shift the thickness transition at a minimum of 3 in. away from the width transition. This makes it much easier to fit run-off tabs needed to make the butt weld and improves the ability to properly test the weld.

Fabricators may have difficulty obtaining thicker plates. Consider using Grade HPS 70W when plates thicker than $2\frac{1}{2}$ in. of Grade 50W are needed for a more cost-effective girder.

Refer to the LRFDBDS for other flange proportioning requirements related to flexural members and constructibility.

AASHTO/NSBA Steel Bridge Collaboration provides a more complex calculation in their publication (2020).

Fabrication costs can be reduced if welded flange plate transitions do not change both thickness and width at the same location. It is less expensive for most fabricators to order wide plates, make a single butt splice weld, and strip-cut flanges to the desired width from the larger welded assembly. If the fabricator cannot prepare large flange plates, each flange must be custom cut and welded individually. Therefore, the designer should attempt to use constant width flanges in regions where flange transitions are commonly used, such as in negative moment regions.

When a flange must be bent about its weak axis, a minimum bend radius of 50 ft. is required.

For girder stability during shipping, erection and deck placement, the top flange width shall meet the following requirement:

$$b \geq L/85$$

where

- b = the top flange width
- L = the shipping length

6.6.5.5—Web Plates

The following guidelines will provide a more cost-effective welded plate girder and shall be followed as much as possible:

- Web depth should be selected in 2 in. increments.
- Web thickness should be selected in $1/16$ in. increments with a minimum thickness of $7/16$ in.
- If designs have variable web thickness, changes in thickness are only allowed at bolted field splices. Changes in web thickness are not allowed at web butt splices.
- If variable web depth is used, the web shall have a constant depth for at least 3 ft. 6 in. at the supports.
- Provide a partially stiffened web per the following guidelines, as this is generally the most efficient design:
 - Determine an initial web thickness based on an unstiffened web plate. Reduce the web plate thickness by $1/16$ in. and add stiffeners as needed, resulting in a partially stiffened web. Further iterations of web reduction can be used if needed to arrive at an economical design.
 - Longitudinal stiffeners should be avoided in all designs, but may be considered for web depths greater than 120 in. if approved by Bridge Division.

In some cases, flanges need to be bent due to girder profiles. An example is the bottom flange at the intermediate support on a haunched girder. The minimum bend radius of 50 ft. is a rule of thumb and bend radii should be discussed with fabricators to confirm their capabilities.

Designers need to be aware of the flange proportioning requirements in the LRFDBDS, as well as the $b \geq L/85$ requirement in this section. These proportioning requirements are important for minimizing potential construction problems such as out-of-plane distortions of the compression flange and web during deck placement.

Designers should be aware that in some cases a Contractor may wish to splice two field sections together prior to erection. Good practice would be to identify such locations and proportion flanges accordingly.

Refer to the LRFDBDS for other web proportioning requirements related to flexural members and constructibility.

Standard Note #089 should be placed on the front sheet to allow fabricators the option of the unstiffened web where practical.

Longitudinal stiffeners can result in significant savings in web material. However, there are many undesirable details associated with longitudinal stiffeners that both increase fabrication costs and result in potential fatigue issues. A clear need for longitudinal stiffeners must be demonstrated before being considered as a viable component of a web design.

6.6.5.6—Hybrid Girders

Designers may consider the use of hybrid girders for positive moment areas, negative moment areas or the entire girder if the preliminary design of the superstructure indicate a need for using higher strength steel (HPS 70W) for one or both of the flanges. Steel Grade HPS 100W shall not be used without approval from Bridge Division.

Due to the added cost of HPS material, it should not be used for components such as stiffeners, connection plates and cross-frames where it is not required by design.

6.6.5.7—Girder Fit Condition

Table 6.3 describes which fit condition should be utilized for various bridge geometries.

Standard Note #098 or Standard Note #099 is to be provided in the General Notes section of the plans based on the selected fit condition.

Studies have shown that hybrid designs using Grade HPS 70W material for the flanges and Grade 50W material for the webs are more economical than homogeneous designs using Grade HPS 70W material for both the flanges and the webs. Web designs are typically controlled by slenderness concerns, which are related to geometry rather than yield strength. Therefore, a reduction in overall girder weight is generally not achieved when using higher strength material in girder webs.

Table 6.3 summarizes the information found in the LRFDBDS Article 6.7.2 and from NSBA (NSBA Technical Subcommittee et al., 2016).

Table 6.3—Welded Plate Girder Fit Conditions

Bridge Geometry	Fit Condition
Straight bridge with all supports skewed up to 20° from normal	No Load Fit (NLF)
Straight bridge with one or more supports skewed more than 20° from normal	Steel Dead Load Fit (SDLF)
Horizontally curved bridge with $(L/R)_{MAX} < 0.2$	Steel Dead Load Fit (SDLF)
Horizontally curved bridge with $(L/R)_{MAX} \geq 0.2$	No Load Fit (NLF)

Where:

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

R = radius of the centerline of the bridge (ft)

The designer is responsible for determining the proper fit condition and for stating the selected fit condition in the General Notes section of the plans. The steel fabricator is responsible for detailing the cross-frame connections such that they account for the selected fit condition.

6.6.5.8—Camber and Blocking

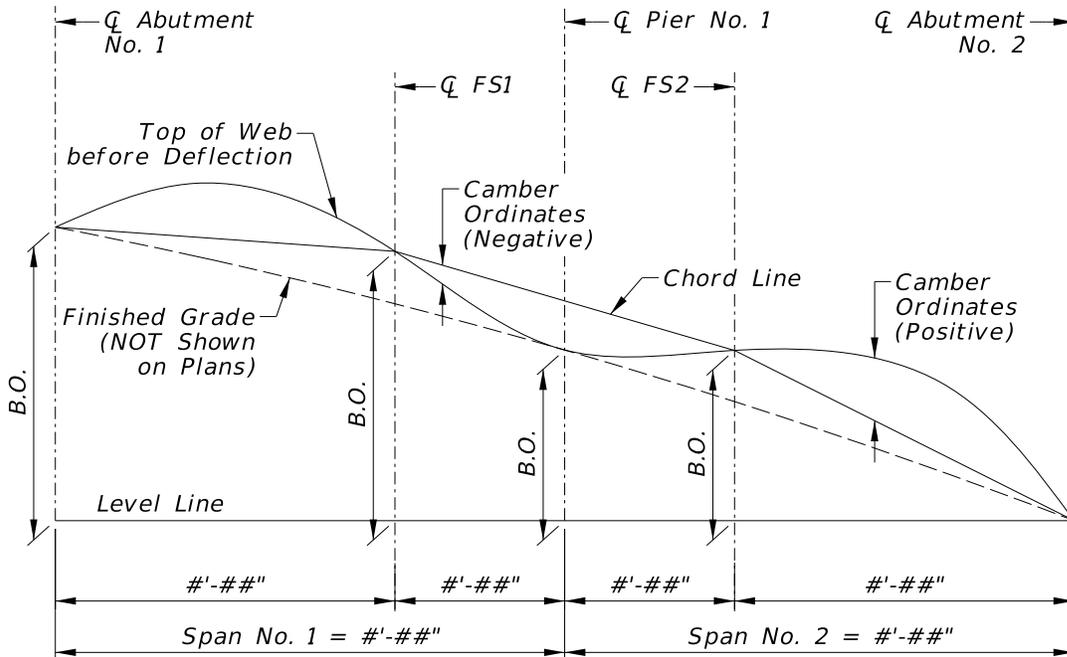
Welded plate girders shall have camber cut into the webs to compensate for dead load deflection and roadway vertical profile. Camber is not used for the control of live load deflections.

Concrete dead load deflections are influenced by the deck pouring sequence, especially with long spans of approximately 200 ft or more. Therefore, concrete dead load deflections and resultant cambers reported on the plans must be developed from an analysis that matches the deck pouring sequence depicted in the plans.

Deck pouring sequence and the potential for variable stiffness due to hardened concrete has a direct effect on dead load deflections and by extension camber values. This is especially true with longer spans (approximately 200 ft and longer). The deck pouring sequence must therefore be consistent with the analysis used to determine girder deflections and camber values. If an alternate deck pouring sequence is proposed by the Contractor, it must be evaluated with these effects in mind and prior to submittal of steel girder shop drawings.

A Camber and Blocking Diagram must be provided on the girder sheets as outlined below.

The blocking diagram must provide Blocking Ordinates (B.O.) at all supports and at all field splices. The Blocking Ordinates will match the grade offsets at the field splice locations. Blocking Ordinates will be used to fabricate the girder in the shop and to assemble the sections in the field.



CAMBER AND BLOCKING DIAGRAM

Figure 6.5—Welded Plate Girder Camber and Blocking Diagram

Camber ordinates (C.O.) are given at span tenth points and field splice locations. After total dead load deflections are applied to the superstructure, the top of web line shall be parallel to the finished grade. Future surfacing deflections should only be included if the wearing surface is installed at the time of initial construction.

Tables indicating the blocking ordinates, camber ordinates and deflections for shims, must be shown in the Plans. Blocking ordinates shall be reported in decimal feet to the nearest thousandth of a foot. Camber ordinates and deflections for shims shall be reported in decimal inches to the nearest hundredth of an inch.

Deflections must be, at minimum, broken into two components: steel deflections (girder self-weight), and concrete deflections (deflections due to all components above the top of the girder, which can include, but not be limited to, concrete deck, barriers, medians, sidewalks, wearing courses and fencing).

Generally, a Camber and Blocking Diagram with a single set of Camber/Deflection and Blocking Data Tables will be adequate. However, if camber ordinates in adjacent girders differ by more than 1/4 in. at tenth point locations, camber and blocking ordinates will be required for each individual girder.

6.6.5.9—Shims

The shim for welded plate girders is shown as the distance from the top of the web line to the finished grade. The top of web line should be established so that the top of the thickest top flange plate is $\frac{1}{2}$ in. below the bottom of the bridge deck. This provides a $\frac{1}{2}$ in. construction tolerance and a minimum shim at these locations. Field splice top plates may be embedded in the deck by no more than 1 in.

Plans must indicate the expected shim at the supports as shown in the cell "Figure 6.9—SHIM WP Cell (Bridge Deck Library)".

6.6.6—Cross-frames and Diaphragms

6.6.6.1—General Design Considerations

Both cross-frames and diaphragms can be used as bracing members in steel superstructures, subject to the limitations provided in this chapter. To simplify the text, the generic term "separator" is used to mean either cross-frames or diaphragms.

The LRFDBDS does not stipulate maximum spacing for separators. The Bridge Division follows this practice of not mandating maximum spacing. However, for curved bridges, suggested maximum separator spacing is provided in §6.6.5.3. Regardless of spacing, designers must analyze separators and their effects on the girders themselves at all stages of construction and in the final condition.

Separators shall be placed perpendicular to girders (radial lines for horizontally curved bridges), except that at abutments and piers, separators shall be placed along the skew of the support. For bridges with skews of 20° or less, the perpendicular layout is preferred, but the intermediate separators may be placed parallel to the substructure skew to simplify detailing.

For girders with wide flanges (greater than 24 in. wide) or with significant cross-slopes, designers shall consider the minimum haunch at the flange tip, not just at girder centerline. Designers shall also consider requesting surveyed top of girder data at each flange tip, similar to NU girders.

Historically, the AASHTO design specifications have limited the longitudinal separator spacing to a maximum of 25 ft. Over the years, bridges have performed well under this limitation. The LRFDBDS does not specify a limit on separator spacing, rather requiring the designer to design the girders for the unbraced length corresponding to the separator spacing. However, the intent was not to encourage overly large spacings, rather to eliminate "extra" separators that would be added solely to meet an arbitrary spacing limit. Designers are still encouraged to use the maximum 25 ft spacing as a guide when laying out separators on a straight bridge.

When substructure skews are 20° or less, skewed intermediate separators (those not at abutments or piers) are permissible. However, skewed intermediate separators have been shown to induce significant localized lateral bending near support locations, and can cause issues with girder fit-up at erection due to a combination of differential deflections and rotations. Therefore, designers are encouraged to use perpendicular separators at all intermediate locations, even when the skew is 20° or less for all structures.

Separator layouts for bridges with skews greater than 20° must be analyzed to account for unintended stiff transverse load paths. Designers must consider modifications to separator layouts that will reduce transverse stiffness in certain areas.

Separator layouts can inadvertently create stiff transverse load paths, especially for bridges with one or more of the following characteristics:

- skews greater than 20°
- variable skews
- simply supported spans
- poor span balance in continuous spans

Stiff transverse load paths can produce dramatically increased separator forces and can result in potential fit-up difficulties. Therefore, consideration must be given to implementing strategies that reduce the transverse stiffness of the bridge, particularly in the vicinity of supports. Options to accomplish this goal include placement of intermediate cross-frames in discontinuous lines perpendicular to the girders, selectively removing certain separators, and/or staggering the separators in adjacent bays between the girders.

Designers are encouraged to review the NSBA publication *Skewed and curved Steel I-Girder Bridge Fit for guidance on separator layouts for skewed bridges* (2016). In particular, §4.4 contains pertinent information for these separator layouts.

6.6.6.2—Cross-frames

The K-type cross-frame is the preferred type for all girder depths 48 in. and greater. However, if the ratio of girder spacing (S) to girder depth (D) is 1.0 or less (i.e. $S/D \leq 1.0$), then X-type cross-frames could be considered.

Where x-frame type cross-frames are used on new construction, diagonals shall be sized to meet compression and tension requirements. For retrofit or existing structure analysis the cross-frame may be analyzed as a tension only system if required.

The main benefit of K-type cross-frames is that if welds are designed on only one side of the gusset plates (see detail below), then the assemblies will not need to be removed from their fabrication jigs and inverted to complete the welding. Minimizing handling reduces overall cost. Where it is not possible to design the cross-frame with single sided welds, double sided welds such as [Figure 6.6](#) are permitted. All around fillet welds are not feasible in this case therefore shall not be used.

Double angles should be avoided in cross-frames due to the likelihood of trapping water and debris between the back-to-back legs, which increases the potential for corrosion. WTs can be substituted for angles when additional strength or stiffness is required by design. Designers are cautioned to select WT shapes rather than MT shapes or ST shapes due to greater availability.

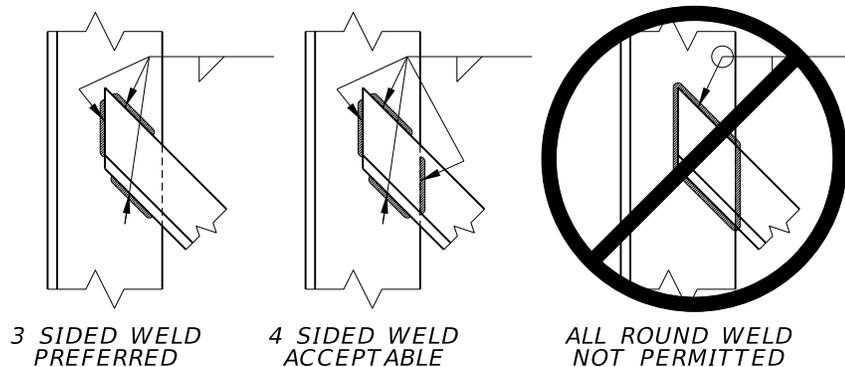


Figure 6.6—Cross-frame Welding

An inverted K-frame with the diagonals connecting to the girders near the bottom flange should also be considered for the cross-frames at supports. This configuration better transfers lateral loads into the bearings.

Ideally, cross-frame diagonals should be close to 45° where possible. Cross-frame diagonals shall have a minimum angle of 30° measured from the diagonal to a horizontal line.

Oversize holes may be used in one ply of the cross-frame to girder connections for straight bridges. For horizontally curved bridges, standard holes must be used in all plies of the cross-frame to girder connections.

End cross-frames shall be provided at deck discontinuities such as expansion joints located at the ends of units. These deck discontinuities and expansion joint hardware must be supported on the top chords of the end cross-frames. For these end cross-frames, channels are preferred, but W-sections may be used with coped flanges if additional strength or flange width is needed.

The intersection of the braces in a x-frame shall not be considered a bracing point for buckling analysis.

Wherever possible designers should orient cross-frame members made from angles so that the horizontal leg of the angle is at the top. This prevents debris accumulation and reduces corrosion potential.

6.6.6.2.1—Standard Cross-frame

Minimum cross-frame member sizes and connection details are provided in [Figure 6.7](#). Designers may use this cross-frame detail without further analysis for typical bridges with all the following characteristics:

- Girders without horizontal curvature
- Cross-frame spacing not exceeding 25 ft.
- Girder spacing not exceeding 10 ft.
- Girder web depths not exceeding 6 ft.
- Four or more girders in the cross section
- Spans not exceeding 150 ft.
- Support skew angle not exceeding 20°
- Skew index not exceeding 0.3

Standard holes in all cross-frame connections for horizontally curved bridges help to maintain the geometry of the structure during and after erection.

Reserved for Future Use

Figure 6.7—Standard Cross-frame Detail

6.6.6.3—Diaphragms

For girders webs less than 48 in. deep, diaphragms are preferred over cross-frames.

For shallow girder depths less than 30 in., standard rolled channels should be used instead of bent plate separators. Miscellaneous channels (MC shapes) should be avoided due to reduced availability.

The minimum bent plate separator thickness shall be $\frac{3}{8}$ in. Consideration should be given for thicker bent plate separators if girder spacing exceeds 10 ft or diaphragm depth exceeds 5 ft.

Bolt spacing for diaphragms shall be as close as possible to the maximum spacing for sealing bolts (LRFD BDS Article 6.13.2.6.2).

Diaphragms can be used for girder depths exceeding 48 in. However, in most cases, cross-frames are lighter and more economical for these deeper girder sections.

6.7—BOX AND TUB GIRDERS

Reserved for future use.

Although the BDM does not yet address other types of structural steel superstructure than I girder design, it also does not preclude the use of other systems, such as box and tub girders, where applicable.

6.8—FABRICATION REVIEWS

Fabricator submittals such as shop drawings and RFIs for welded plate girders contain certain items that need special focus during review. Particular attention must be given to the following.

- Shop welded flange butt splices shall be placed to avoid the maximum moment regions.
- Shop welded web butt splices shall be placed to avoid regions of high shear. A region of high shear is defined as a distance that is within 10% of the span length away from a support.
- Flange and web butt splice locations do not conflict with stiffeners or connection plates.
- Connections are detailed for the proper girder fit condition.
- Working points for cross-frames are maintained in the designed locations.
- Oversize holes at cross-frames shall not be allowed on horizontally curved girders since these members are used to control the superstructure geometry throughout construction of the bridge.

Eliminating butt welded flange transitions between field splices by extending the larger flange size may be acceptable to the Engineer of Record during fabricator review process as the changes to the deflections are typically small. Eliminating field sections by extending the larger section will have much greater impacts (such as requiring a plan revision) and will not typically provide a better value for the Department even if the fabricator may save fabrication costs. The cost of these changes to the Department need to be carefully evaluated against the potential cost savings before this change is approved.

Oversize holes may be permitted in phased construction or widenings to accommodate differential deflections at the phase line. They are allowed in a single ply only, but never for horizontally curved girders.

6.9—REFERENCES

AASHTO/NSBA Steel Bridge Collaboration. (2020). *Guidelines to Design for Constructability and Fabrication* (Nº G12.1-2020). <https://www.aisc.org/media/f5oop4mb/g121-2020-guidelines-to-design-for-constructability.pdf>

Garrell, C., & Hopper, T. (October 2023). Steel Plate Availability for Highway Bridges. *Modern Steel Construction*, 63(10), 16–21. <https://www.aisc.org/modern-steel/archive/october-2023/>

NSBA Technical Subcommittee, Beckmann, F., Chavel, B., Coletti, D., Cooper, J., Grubb, M., Frank, K., Glen Fraser, Kozy, B., Medlock, R., Murray, G., Nguyen, T., Russo, F., Wallace, D., Walsh, S., White, D., & Yadlosky, J. (August 2016). *Skewed and Curved Steel I-Girder Bridge Fit* (Nº B013-20-FULL). <https://www.aisc.org/media/2fkle45e/skewed-and-curved-i-girder-bridge-fit-full-2016-revision.pdf>

6.10—REFERENCED CELLS

<p>CHARPY IMPACT TEST REQUIREMENTS FOR MAIN TENSION MEMBERS</p>
<p><i>For the purpose of impact test the following material shall be classified as main tension members:</i></p> <p style="text-align: center;"> <i>All flange plates marked thus: ★</i> <i>All web plates</i> <i>All field splice plates</i> </p>

Figure 6.8—CHARP1 Cell (Steel Girder Library)

<p>CHARPY IMPACT TEST REQUIREMENTS FOR MAIN TENSION MEMBERS</p>
<p><i>For the purpose of impact test the following material shall be classified as main tension members:</i></p> <p style="text-align: center;"> <i>All girders</i> <i>All field splice plates</i> </p>

Figure 6.10—CHARP2 Cell (Steel Girder Library)

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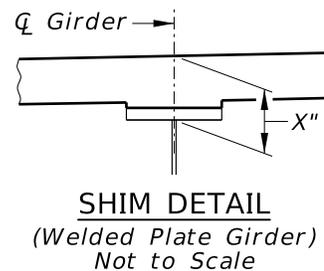


Figure 6.9—SHIM WP Cell (Bridge Deck Library)

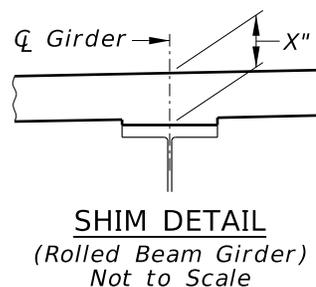


Figure 6.11—SHIM RB Cell (Bridge Deck Library)

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