

6. STEEL STRUCTURES

Structural steel, in the form of rolled steel beams or welded plate girders, is used for bridge superstructures. In rare instances integral pier caps or substructures will be designed using steel. This section is intended to provide general design guidance and information on detailing practices. In addition, a design example for a two-span plate girder superstructure is included.

6.1 Materials

Structural Steels

Steel bridges are fabricated and constructed with steel elements that are produced at two different types of steel mills; shape mills and plate mills. In addition to different products, the grades of steel available from each type of mill differ slightly.

Shape mills produce bars, angles, tubes, pipes, channels, "W" sections (wide flange), "S" sections (American Standard), and piling that satisfy a variety of material specifications. Standard mill lengths available for these sections range from 30 to 60 feet. With sufficient quantities and sufficient lead time, longer lengths may be available. AISC's *Modern Steel Construction* yearly January issue provides information on different shapes available domestically from various mills. The designer shall check the availability of shapes before specifying their use in a structure.

Plate mills produce flat sections that are used to fabricate plate girders, connections, gusset plates, etc. Plate steel is also produced in a number of different material specifications. Larger plate mills have a width limitation of 150 inches. The maximum available plate length varies by mill and cross sectional dimensions of the plate.

The LRFD Specifications identify a number of steels that can be incorporated into bridge structures. They are identified in Tables 6.4.1-1 and 6.4.2-1 of the LRFD Specifications with both AASHTO and ASTM designations. Weathering steels have a "W" appended to the grade designation (e.g. 50W, 70W, 100W). Note that the AASHTO and ASTM designations are not identical.

Use weathering steel (Mn/DOT 3309, 3316, or 3317) for rolled beams, plate girders, and diaphragms on all steel bridges.

The AASHTO Specifications require additional tests (Charpy testing) to verify the toughness of the material. Mn/DOT Spec. 3308 requires this testing be conducted for steel incorporated into major structural components. Mn/DOT Spec. 2471.2 lists the specification numbers for

standard structural metals used in bridge projects. Structural steel of primary members shall satisfy the toughness requirements for Zone 3. Shape sections, other than pipe and tubing, are typically available in 36, 50, or 50W grades. Steel plate is available in 36, 50, and 50W grades. High Performance Steel (HPS) is available in Grades HPS50W and HPS70W. The typical steels for Mn/DOT bridge designs are Grade 50W and HPS70W. Hybrid sections may be the most cost effective, with the bottom flange in positive moment regions and both flanges near piers consisting of HPS70W, and all other steel 50W.

Additional information regarding steel shapes and plates may be found in *Properties of Bridge Steels*, Vol. I, Chapter 3, Highway Structures Design Handbook, May 1994.

The unit weight of steel is 0.490 kcf.

The coefficient of thermal expansion for steel is 6.5×10^{-6} in/in-°F.

[6.4.3]

Bolts, Nuts, and Washers

For most steel bridge applications ASTM A325 high strength bolts per Mn/DOT 3391.2B are to be used. The LRFD specifications also include ASTM A490 high strength bolts. Due to reduced availability and higher cost, A490 bolts should not be used without first consulting the Bridge Design Engineer. A490 bolts cannot be sold with plating, galvanizing, or mechanical zinc coating, so their use as field bolts is problematic.

For applications where strength is not the primary design consideration, ASTM A307 bolts per Mn/DOT Spec. 3391.2A may be used. See Mn/DOT Spec. 3391 for additional information on fasteners.

Additional fastener information may be found in *Mechanical Fasteners for Steel Bridges*, Vol. I, Chapter 4A, Highway Structures Design Handbook, April 1996.

Dimensional and weight information for bolts, nuts, and washers is provided in Appendix Figures 6-A1 through 6-A5.

Shear Connectors (Stud Welded Fasteners)

The material requirements for shear connectors are listed in Mn/DOT Spec. 3391. They shall satisfy ASTM material requirements, have a yield strength of 50 ksi, and an ultimate tensile strength of 60 ksi.

Welds

A variety of welding processes and materials are available to fabricators for different weld types. In most cases, designers need not concern themselves with the welding process selected by the fabricator.

Typically, only fillet welds and full penetration welds are permitted. Designs using partial penetration weld details can only be used with approval from the Fabrication Methods Engineer. Base weld designs on E70 filler material.

With the exception of pile splices, shear connectors, and railroad ballast plate splices, field welding is not used or permitted. Additional information on welding can be obtained from the Structural Metals Unit or Fabrication Methods Unit of the Bridge Office.

Additional references are the ANSI/AASHTO/AWS *Bridge Welding Code - D1.5*, and *Welding of Steel Bridges*, Vol. I, Chap 15, Highway Structures Design Handbook.

Appendix Figures 6-A6 through 6-A8 contain information on the proper construction of weld symbols and the proper application of the symbols to different types of details.

Bearings

Steel plates used in the fabrication of bearings shall meet Mn/DOT Spec. 3306, 3309, or 3310. Bearings made from castings shall satisfy ASTM A148, GR. 80-40.

Paint Systems

Use of weathering steel in Minnesota bridges has proven effective against continuous corrosion and section loss. Therefore, painting of weathering steel will only be considered under the following conditions:

- Near expansion joints (within 7 feet of the joint).
- Low level water crossings.
- Wide grade separations that may create tunnel-like conditions with persistent high humidity.
- Where warranted as an aesthetic treatment (limited to the outside surface and bottom flange of fascia beams).

All steel bridges will be reviewed during the preliminary design process to identify whether painting is required.

Preferred practice for painting is to have the primer applied in the fabrication shop and the intermediate and top or finish coat applied in the

field. Use the inorganic zinc-rich paint system (Mn/DOT Spec. 2479) for new steel designs.

For maintenance painting projects, use the organic zinc-rich paint system (Mn/DOT Spec.2478).

6.2 General Dimensions and Details

[2.5.2.6.3]

As a rule-of-thumb for the preliminary design of continuous structures, try a steel section depth of $0.033L$, where L is the span length. In no case should the steel section be less than $0.0285L$, unless approved by the Bridge Design Engineer. Typically a member taller than the minimum will be the most efficient. The most efficient depth of girder will vary with span and girder spacing. For large structures, perform a web depth study to arrive at the optimal girder height.

Locate field splices at or near points of dead load contraflexure.

Provide adequate spacing (2'-0" minimum) between field splices and diaphragm connection plates and stiffeners.

Identify "Area A" on the beam or girder plan sheets. "Area A" is the portion of the top flange that is in tension due to total dead load. Identifying the tension flange is important for a number of reasons:

- For complex bridge types or curved girders where a grid or 3-D analysis is needed, a bolted tab plate connection must be used to connect connection plates to tension flanges. A fillet weld is used for the connection to the compression flange. See Details B402 (bolted diaphragms), B407 (cross frame diaphragms), B408 (cross frame diaphragms for curved beams), and B410 (bolted stiffener to flange detail).
- During fabrication, identification of the tension flange is needed to complete the nondestructive testing requirements of MN/DOT Spec. 2471.
- During construction, the contractor is allowed per Mn/DOT Spec. 2402.3D to weld screed rail supports to the top flange of steel girders, except in "Area A".

For straight girders and those with slight curvature that meet the criteria given in LRFD Article 4.6.1.2.4b, connection plates may be connected to both the tension and compression flange using a fillet weld. Note that Standard Detail B407 allows for the use of either a welded or bolted connection. Because of its lower cost, the welded connection is preferred. The designer should first check the fatigue limit state at the diaphragm stiffener connections to determine if a welded detail is acceptable. If

stresses in the flange are too high to permit the welded detail, the designer should consider increasing the flange thickness to lower the stress range for fatigue or moving the diaphragm. Compare costs between the bolted option with initial flange thickness and the welded option with a thicker flange to determine the most economical option to show on the plan.

[C6.7.4.1]

The LRFD Specifications do not explicitly give a maximum diaphragm spacing as was previously given in the Standard Specifications. Diaphragms are used for bracing the compression flange and the diaphragm spacing is used to determine allowable compressive stresses.

Choose the diaphragm spacing in the positive moment area based on the maximum allowed for the bracing of the top compression flange during construction of the deck (typically 25 to 30 feet). In the negative moment area, the resistance of the bottom compression flange is based on the diaphragm spacing. The spacing in negative moment regions is usually 15 to 20 feet.

When choosing the distance from the centerline of bearing to the end of beam, use a minimum of nine times the web thickness.

For a steel superstructure, identify the following items on the framing plan:

- beam spacing
- staging
- distance between diaphragms along each beam
- type of diaphragms used in different locations
- centerline of bearing at piers and abutments
- Working Line and Working Points
- beam marks (B1, B2, etc.)
- type and location of bearings
- the location of intermediate stiffeners

The plate girder details shall identify plate sizes, length of plate segments, location of "Area A", spacing of shear studs, sole plate size, bearing and intermediate stiffener size, connection plate size, splice location and type, a table showing top of field splice elevations, and all pertinent notes. Standard notes are contained in Appendix 2-H of Section 2.

Structural steel plans and details must clearly describe the material to be used for each structural steel component. Even for projects where structural steel is paid for on a lump sum basis, provide informational

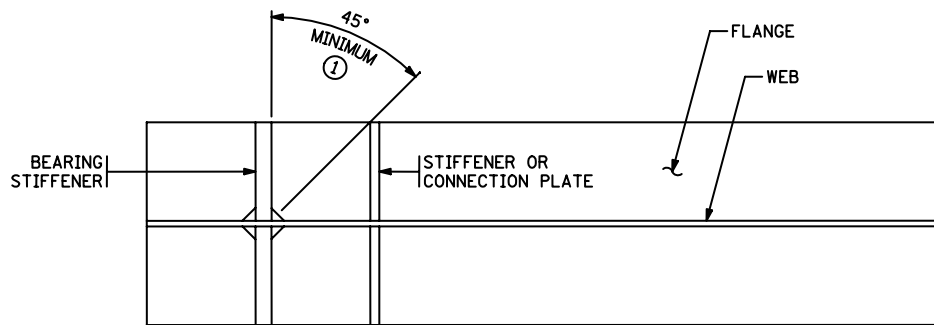
quantities in the plan set to quantify the amounts of different steels to be used. This is particularly true for projects with fracture critical members. Fracture critical members are fabricated to a higher quality standard to reduce the potential for defects and thus are more expensive. Do not specify members as fracture critical unless necessary and appropriate.

Structural steel quantities are computed by finding the weight of steel beams or girders, diaphragms, cross frames, and all other plates (e.g., sole and gusset plates). Increase the calculated weight by 1½% to account for the weight of steel for welds and bolt stick-through when computing structural steel quantities.

Designers should provide simple details that are easily fabricated and do not sacrifice the integrity of the bridge. Details that trap water or produce an environment that is conducive to corrosion should be avoided. In addition, details with inadequate clearances are difficult to fabricate and erect.

[AISC Steel Construction Manual, 13th Edition, Table 7-16]

The equipment used to weld and bolt steel pieces together requires room to operate. The *AISC Manual of Steel Construction* contains tables with minimum clearances for bolted connections. Figure 6.2.1 describes minimum clearances that must be provided for welded components.



PLAN VIEW

① WHEN DETAILING WELDED ATTACHMENT, ALLOW FOR A MINIMUM OPENING OF 45° FOR WELDING ACCESSIBILITY.

Figure 6.2.1

Show the type of weld to be used for each connection in the plans. The welding code (AWS-D1.5) specifies the minimum size of fillet welds. Identify the required weld size only if the minimum weld size does not have adequate capacity.

All connection details for lateral wind bracing systems shall be bolted.

For box pier caps and tub girders, make access holes and manholes through diaphragms as large as possible and locate for ease of passage. The minimum opening is 2'-0" x 2'-6". Provide an access door near each end of box piers for inspection purposes. Locate the door for ladder access off the roadway, if possible, and hinge the door to swing away from traffic. Place access doors in the side of the box where protected from superstructure runoff and in the bottom of the box where exposed. Use Mn/DOT Detail B942 for the door. Door frames shall be bolted to box.

Where single conduits pass through steel diaphragms and require a passage hole with a diameter greater than 3 inches, reinforce the opening with a section of pipe or curved steel plate.

6.3 General Design Philosophy

In general, structural steel superstructures are shallower and lighter than concrete superstructures. In addition to long span and specialty structures, steel superstructures should be considered where foundations are expensive or where a change in superstructure height has significant cost implications on the approaches.

Design girders to be composite with the concrete deck throughout the entire girder length. Provide shear connectors, in the form of shear studs, in both positive and negative moment areas and over field splices.

Stools are used with steel superstructures to provide a construction tolerance for the profile of the deck. The stools shall have vertical edges that are flush with the edges of the top flange. For plate girders the stool is defined as the distance between the bottom of the deck and the top of the web. For rolled beams the stool height is defined as the distance between the bottom of the deck and the bottom of the top flange. Stool heights are to be given at the centerline of the beam. The minimum height for the concrete portion of the stool is 1½ inches. This minimum is measured at the edge of the flange taking into account the cross slope of the deck. At field splices check that the top plates do not penetrate the bottom of the deck.

During design, it may be assumed that the dead load of the steel beam or girder is 15% larger than that computed using only the flanges and web. This is a reasonable estimate for the weight of stiffeners, diaphragms or cross frames, and connections.

Use of the moment redistribution provisions of LRFD Appendix B is not allowed.

The maximum nominal flexural resistance F_n allowed by Mn/DOT for design of steel beams is equal to F_y . Do not exceed the moment at first yield.

Use a limited number of thicknesses when sizing stiffeners and connection plates. To permit two lines of bolts, connection plates must be a minimum of 7" in width.

For steel superstructures with uplift at the abutments, the end diaphragms and/or counterweight shall be cast prior to deck construction and the deck shall be cast beginning at the abutment with the greatest uplift. Temporary tie-downs at the abutments may be necessary for the deck pour.

Provide bent plate diaphragms (Mn/DOT Detail B402) for the following cases:

- rolled beam superstructures
- plate girders with depths less than 40 inches
- beam depth to lateral spacing ratio less than 0.40

In other cases, use cross frame diaphragms (Mn/DOT Detail B407).

Railroad bridges designed in accordance with the AREMA Specifications may have slightly different criteria than AASHTO for high strength bolts, pin bolts, and welding. Check with the railroad in question for specific criteria.

6.3.1 Shear Connectors **[6.10.7.4.1]**

Provide $\frac{7}{8}$ inch diameter stud connectors that extend a minimum of 2 inches above the bottom of the deck and a maximum of 3 inches below the top of the deck. Studs must be applied in the field after girder erection.

6.3.2 Fatigue

Fatigue cracks are generally classified as either load induced or displacement induced. Load and stress limits are placed on members to minimize load induced fatigue cracks from forming. Proper detailing practices are used to prevent displacement induced fatigue cracks.

Designers must check connections for fatigue resistance. For all Trunk Highway bridges, check details for an infinite fatigue life level regardless of ADT level.

Figure 6.3.2.1 identifies the appropriate fatigue category to be used for typical plate girder connections. Check all fatigue categories that apply. For discussion of "Area A" and welding vs. bolting connection plates to the tension flange, see Article 6.2 of this manual.

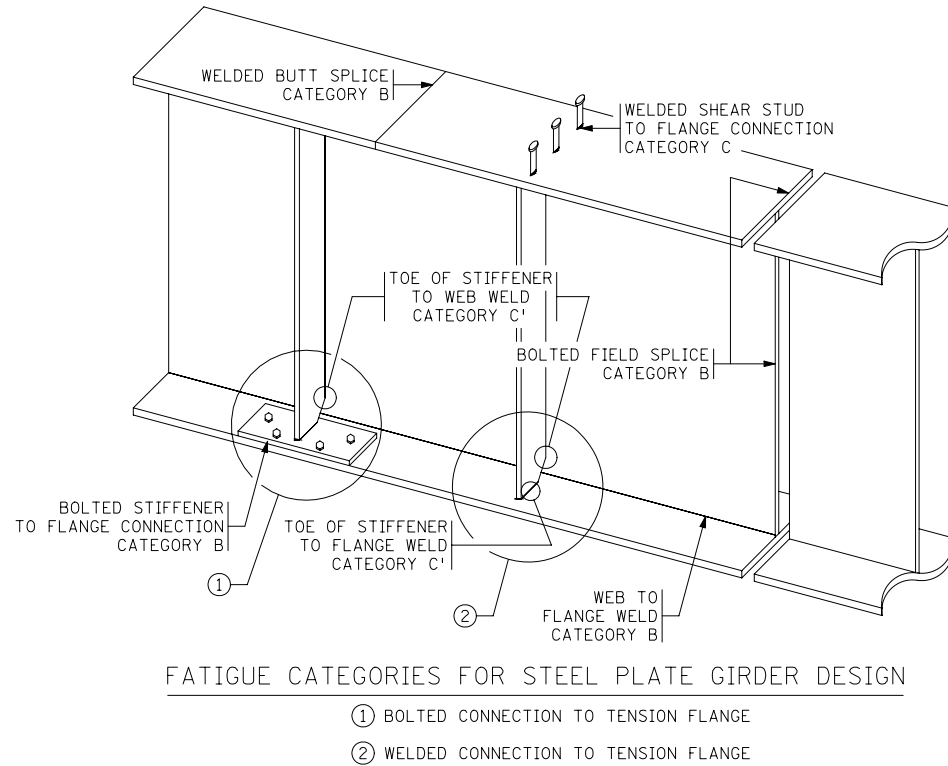


Figure 6.3.2.1

[6.10.8.1.1]

Detailing practices that prevent displacement induced fatigue cracks from forming include coping stiffeners and terminating welds slightly before reaching the end of an element. Tops and bottoms of transverse stiffeners and connection plates are typically coped $1\frac{1}{2}$ inches from face of web and $2\frac{1}{2}$ inches from face of flange. (See Mn/DOT Detail B411.)

6.3.3 Deflections

To ensure that bridges are constructed with a proper vertical profile, the deflections associated with selfweight, deck placement, and composite superstructure dead loads shall be presented in the plan set. Split the dead load deflection into two categories: selfweight (including diaphragms), and dead load due to deck and all superimposed loads (excluding future wearing course). Display deflections in feet with a precision of three decimal places.

Live load deflection shall be limited to $L/800$ for typical bridges and $L/1000$ for bridges carrying pedestrians. Calculate the live load distribution for deflection by taking the number of lanes times the multiple presence factor and divide by the number of beams. The multiple presence factor used with this calculation shall not be less than 0.85.

6.3.4 Camber

For most steel bridges, camber is fabricated into the beam to match the profile grade and offset the deflections due to applied dead loads. In some cases, residual camber (extra camber added for architectural reasons) is also added to eliminate the possible appearance of a sag in a span.

For rolled beams introducing camber can be a relatively expensive operation. It is usually accomplished with cold bending and/or with heat straightening techniques. Camber rolled beams for bridges only if the dead load deflection exceeds the maximum mill tolerance for camber, which is equal to $1/8$ inch per 10 feet of length. If the deflection is less, state that the beam shall be placed "natural camber up".

Plate girders shall always be cambered. Vertical cambers are introduced by cutting the web plates into the desired profile. Horizontal curvature is introduced (if necessary) by cutting flange plates with the proper horizontal shape. During fabrication, the web and flanges are attached to each other to produce a member with the proper geometric characteristics.

Camber for vertical curvature, anticipated dead load deflections, and residual camber (if required). Do not include the deflection due to future wearing course (FWC). Provide residual camber only in girders with straight grades with lengths in excess of 100 feet. Use approximately $1\frac{1}{2}$ inches of residual camber for a 100 foot span. Increase the residual camber by $1/8$ inch for each 10 foot change in span length. Use a maximum residual camber of $2\frac{1}{2}$ inches.

Choose a stool height that will be constant throughout the length of the girder for girders without residual camber. For girders where residual camber is used, the stool height will vary. For these situations, the stool height will have its largest value at substructure locations and smaller values near midspan. In no case is the thickness of concrete in the stool to be less than $1\frac{1}{2}$ inches.

The following procedure may be used to develop a camber diagram:

- 1) The camber curve, a line located at the bottom of top flange for rolled beams and at the top of web for plate girders, defines the cambered shape of the member. Geometric camber, dead load camber, and residual camber (if required) are the components that make up the camber curve. Start by determining the geometric camber profile due to the vertical geometry of the roadway. To do this:
 - Calculate profile grade elevations at tenth points along the member as well as at field splice and/or point of contraflexure locations.
 - Calculate top of deck elevations at centerline of member by adjusting for cross slopes and offset from profile grade.
 - Calculate geometric camber profile by subtracting the deck thickness and stool height.
- 2) Determine total dead load (minus future wearing course) deflections for the member. Downward deflections are considered negative and upward deflections are considered positive. The dead load camber profile is the opposite sign (downward +, upward -) of the total dead load deflections.
- 3) If there is no residual camber, add the dead load camber profile to the geometric camber profile to get the final camber curve.
- 4) If residual camber is required, calculate residual camber profile assuming a maximum value at midspan and parabolic distribution over the rest of the span. Then, add the residual camber profile and the dead load camber profile to the geometric profile to get the final camber curve.
- 5) Establish a horizontal line at the substructure centerline of bearing with the lowest camber curve elevation.
- 6) Establish chord lines, which are defined as straight lines between the end of each beam segment at the camber curve.
- 7) Determine all vertical and horizontal dimensions to be entered on camber diagram to the nearest $\frac{1}{8}$ inch. These include:
 - Dimensions from horizontal line to camber curve at all support points, field splices, and contraflexure points.
 - Dimensions from horizontal line to camber curve for at least three points intermediate to those in the first bullet for each curved section. Locate one of these points at the point of maximum camber within the curved section.
 - Maximum camber dimensions from chord line to camber curve for each curved section.
 - Span lengths, segment lengths, and dimensions from supports to field splices and contraflexure points.
 - Locations of camber dimensions along the member.

Refer to Figure 6.3.4.1 for a sample camber diagram.

6.4 Rolled Beams

Rolled beams may be difficult to obtain in lengths over 90 feet. Check with the Fabrication Methods Unit prior to incorporating beams with lengths over 90 feet into a design.

Use rolled beam sections with a minimum flange width of 14 inches. This will allow four lines of bolts to be used in the field splice design.

6.5 Plate Girders

For shipping purposes, limit the length between field splices to 145 feet.

Select plate thickness in $\frac{1}{16}$ inch increments for thicknesses up to 1 inch. For thicknesses between 1 and $2\frac{1}{2}$ inches, use $\frac{1}{8}$ inch increments. Use $\frac{1}{4}$ inch increments for thicknesses between $2\frac{1}{2}$ and 4 inches.

In general, additional web thickness increases shear capacity. An increase in web height or flange area increases moment capacity and reduces live load deflections.

In general, follow these guidelines in plate size selection for plate girders:

Flanges**[C6.10.3.4]**

For plate girder flanges, the minimum size is $\frac{3}{4}$ " x 14". The 14 inch flange permits four lines of bolts for field splices. Limit the ratio of segment length to top flange width to 85 for stability during shipping and erection.

The change in flange area at butt weld splices shall not exceed 100%. In general, it is economical to provide a butt splice if 1000 to 1200 pounds or more of steel can be saved. Further discussion on this issue can be found in Article 1.5.2 of *G12.1 - 2003 Guidelines for Design for Constructibility*.

Where practical, keep the bottom flange at a constant width over the entire girder length. Top flanges should be kept at a constant width within each field piece. Use a 24 inch radius for the taper detail where flanges need to be tapered in width. When changing the flange width and thickness at a butt splice, first taper the width and then taper the thickness. If changing the top flange width at a field splice, do not taper the flange width.

[6.13.6.1.5]

When thick fill plates are required at field splices, additional rows of bolts will be required to transfer the force to the member.

Web

For web plates the minimum thickness is $\frac{1}{2}$ inch. The $\frac{1}{2}$ inch web reduces the potential for warping during fabrication. Select maximum web height while still meeting clearance requirements.

When choosing a web thickness, first determine the thickness at which no intermediate stiffeners are required for shear. Reduce the web thickness in $\frac{1}{16}$ inch increments and calculate the weight of web steel saved per stiffener added. Generally, it is economical to reduce the web thickness when the weight of web steel saved per stiffener added is greater than 1000 pounds.

Web thickness changes are allowed at field splices. For a thickness change of $\frac{1}{16}$ inch, detail the web splice with a 16 gauge fill plate on one side of the web only.

Longitudinal stiffeners should only be considered for girders over 84 inches deep. Terminate longitudinal stiffeners at a low stress point with a fatigue resistant detail. Generally, detail longitudinal stiffeners as continuous through transverse and bearing stiffeners.

6.5.1 High Performance Steel Girders

Use of High Performance Steel (HPS) Grade HPS 70W ($F_y = 70$ ksi) may be an economical alternative to 50 ksi steel. Typically, a hybrid design that utilizes HPS steel for the bottom flange in positive moment areas and both flanges in negative moment areas is most economical.

6.6 Horizontally Curved Steel Girders

The 2005 Interim Specifications unified Section 6 to include both straight and horizontally curved steel bridges. This article highlights some of the issues particular to curved steel design.

[6.10.6-6.10.8]**Flexure**

The preliminary depth and girder spacing shown in the Preliminary Plan is determined using a straight line girder analysis with a maximum bending stress limit of $0.85F_y$. Use the Preliminary Plan to develop a framing plan for review with the Design Unit Leader and Bridge Design Engineer. Once the framing plan has been approved, analyze the bridge as a system using an appropriate structural analysis program.

Design considerations unique to horizontally curved steel girders include:

[4.6.1.2.4b]

- The span, radius, and skew of the girder determine whether the curvature must be considered in the analysis.
- Curved steel girders are always considered noncompact in the positive moment region. Therefore the maximum nominal bending stress is F_y .
- Use of Appendix A or Appendix B is not allowed.
- Lateral flange bending stresses due to torsion must be taken into account. As a result, curved steel plate girder bridges usually have wider flanges than straight steel bridges.
- Horizontal curvature causes a variable load distribution that increases from inside to outside of the curve. Theoretically, flange and web sizes could be different for each girder. The designer must consider the economic benefits associated with grouping plate sizes. In other words, consider grouping the girders, using identical flange sizes for the fascia and first interior beam, the second and third interior beam, etc. Also consider carefully whether to incrementally increase the web depth from inside to outside of the curve. This practice may cause the outside beams to become too stiff, drawing too much moment to the outside fascia beam.

[6.10.9]

Shear

Web shear capacity is treated the same for both straight and horizontally curved steel girders.

[6.7.4]

Diaphragms

Diaphragms are considered primary structural members in curved bridges. Intermediate diaphragms may be either cross frame or bent plate type (Detail B408 or B402).

Use the following guidance for design and detailing of diaphragms:

- Maximum diaphragm spacing given in the *LRFD Bridge Design Specifications* is the lesser of $R/10$ or 30 feet for curved steel girders. Mn/DOT more conservatively limits the diaphragm spacing to 25 feet.

[6.13.1]

- Design diaphragms and their connections for the factored forces and moments determined by analysis. Unlike beam splices, design for higher loads is not required. If the analysis software allows, include the deck in the analysis model to reduce diaphragm stresses.

- Generally, provide radial lines of diaphragms which extend across the entire width of the bridge. Diaphragms may be discontinued near the obtuse corner of skewed abutments or over skewed piers to provide flexibility between supports and adjacent members.
- Place cross frame members such that their lines of action intersect at the center of gravity of the bolt group or produce the smallest possible moment on the bolt group that connects the gusset plate to the connection plate. If the lines of action must move, balance the forces such that moments on the connection are minimized.
- Due to the high cost of WT shapes, use angles for all cross frame members. Place all angles with the vertical leg projection down to prevent debris from collecting in the angle.
- Due to the high cost of bolting, connect angle members to the gusset plate with welds. Also, use a connection at the intersection of cross frame diaphragm diagonals. Use all-around welds to prevent moisture and debris from collecting between members. Consider weld details for diaphragms as Fatigue Category E, but allow a fatigue overstress of 15% per Mn/DOT policy.
- Design welds for the shear force and moment (caused by the member eccentricity) in the plane of the weld. Neglect the out-of-plane moment caused by the vertical eccentricity of the angle. When designing welds, check if the minimum weld size is adequate and increase as needed. The weld length may also be increased depending on the connection geometry.
- Design the gusset plate for the moment induced by the connected members. A reduction in the height of the gusset plate may be required as the full gusset plate may not aid in resisting the applied forces. Also, check the axial stress induced in the gusset plate by the connected members.
- Design cross frame members for factored axial forces. When in compression, slenderness of the members shall be computed using an effective length factor $K = 1.0$.

[4.6.2.5]

[6.10.10]**Miscellaneous**

The design of shear connectors in horizontally curved steel beams accounts for shear forces produced in the longitudinal direction by beam bending, and shear forces produced in the radial direction by the cross frames. The available curved girder analysis programs do not clearly specify the direction of cross frame forces under fatigue loadings. Since this makes it difficult to obtain an accurate net fatigue force range, cross frame forces can conservatively be added for simplicity.

Design curved steel girder splices for vertical bending, lateral bending and shear. Composite section properties shall be used.

Calculate camber based on dead load deflection, vertical curvature, and residual camber (if required). Compute deflections assuming the deck is poured in a 7 inch lift followed by a 2 inch wearing course.

For shipping purposes, limit the length between field splices to 100 feet when the offset from the chord connecting the ends is between 3 and 6 feet. A shipping length of 145 feet can be used when the offset is less than 3 feet. Check with the Fabrication Methods Engineer for specific situations.

For unusual circumstances, consider requiring the contractor to use erection shoring. Prior to using this design assumption, discuss the project with the Bridge Design Engineer and the Regional Bridge Construction Engineer.

Full assembly should be considered for curved steel superstructures. Discuss use of full assembly vs. special assembly with the Fabrication Methods Engineer or Structural Metals Engineer before specifying on the Plan.

**6.7 Box or Tub
Girders**

Box or tub girders have rarely been used in Minnesota, but may be an economical choice for longer span bridges. Typically, they are trapezoidal in shape, with two top flanges, two webs, and a single wide bottom flange. The top flanges have shear connectors attached to them that are used to develop composite action with a cast-in-place deck. Once the deck is in place the closed shape of the cross section is effective in carrying torsional loads in addition to flexural loads.

Ensure that the structure has adequate capacity prior to the development of composite action with the deck. The lateral bracing system for the top flanges must be considered during construction.

6.8 Bolted Connections and Splices

Bolted connections are used mainly in field splices, diaphragms, and metal railings.

Check details to ensure that there are no bolting access or assembly problems.

Splices

Use $\frac{7}{8}$ inch diameter ASTM A325 bolts. The standard bolt pattern is a 3 inch grid with edge distances of $1\frac{1}{2}$ inch.

Use a maximum gap equal to $\frac{3}{8}$ inch between the ends of spliced beams. Provide a minimum of 3 inches from the inside of the inside flange splice plates to the center of the first row of bolts in the web splice.

The change in flange area at bolted splices shall not exceed 100%. The splice plates must be of the same steel as the elements being connected. The minimum thickness of splice plates is $\frac{5}{16}$ inches.

Design bolted field splices as slip-critical connections. Assume that a Class B surface coating or condition is available for slip resistance (Slip Coefficient 0.50).

Include the standard plan note concerning field splice elevations on the detailed drawings.

Where splice plates are $\frac{3}{8}$ inch thick or greater, connections may be designed assuming threads are excluded from the shear plane.

**6.9 Two-Span
Plate Girder Design
Example**

This example illustrates the design of a two-span welded plate girder bridge with a 20 degree skew. The bridge is on a constant grade of 1.5% and has two equal spans of 175'-0". Mn/DOT standard details and drawings for diaphragms (B402 and B407) and railings (Fig. 5-397.117) should be referenced when reviewing this example. This example includes the detail design of a typical interior girder at the critical sections in flexure and shear for AASHTO HL-93 loading. Design of the stiffeners, end diaphragm, shear connectors, and field splice is also included. Fatigue is checked at critical locations.

The superstructure consists of five girders spaced at 11'-4" centers. Girders are designed to act compositely with a 9¹/₂ inch deck. A 1/2 inch of wear is assumed and a deck thickness of 9 inches is used for composite section properties.

**A. Materials and
Geometry**

The following material and geometric parameters are used in this example:

Concrete (deck and overlay)

Dead load unit weight $w_c = 0.150$ kcf

Compressive strength $f'_c = 4$ ksi

Elastic modulus $E_c = 3644$ ksi

Steel

Dead load unit weight $w_{st} = 0.490$ kcf

Yield strength $F_y = 50$ ksi

Tensile strength $F_u = 70$ ksi

Elastic modulus $E_s = 29,000$ ksi

[6.10.1.1.1b]

Composite Section Properties

Short-term modular ratio = $n = 8$

Long-term modular ratio = $3n = 24$

Average Daily Truck Traffic 2000 per day

The overall geometry for the example is presented in Figures 6.9.1 and 6.9.2. Girder geometry is presented in the next section (see Figures 6.9.3 and 6.9.4) where section properties are assembled.

A typical section for the bridge is shown in Figure 6.9.1. The deck is supported on five lines of girders. The girders are spaced on 11'-4" centers and the roadway is 48'-0" wide (two 12'-0" traffic lanes and two 12'-0" shoulders). A Type F-rail is provided on each side of the bridge.

The framing of the superstructure is presented in Figure 6.9.2. The structure has a 20 degree skew. Due to the symmetric span arrangement, only a half-framing plan is provided. Rolled beam end diaphragms are located at the abutments. Cross frames are used for interior diaphragms.

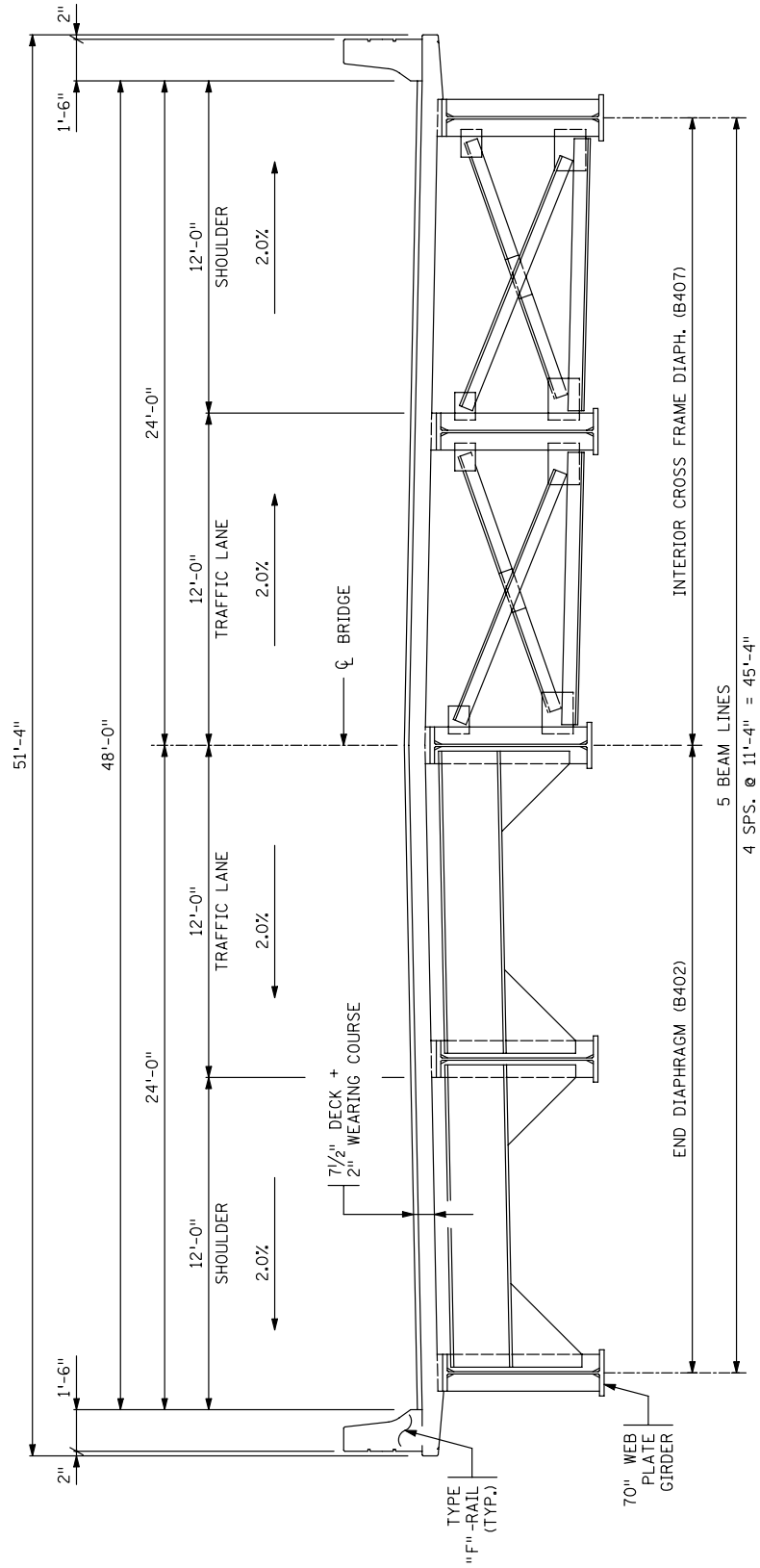


Figure 6.9.1

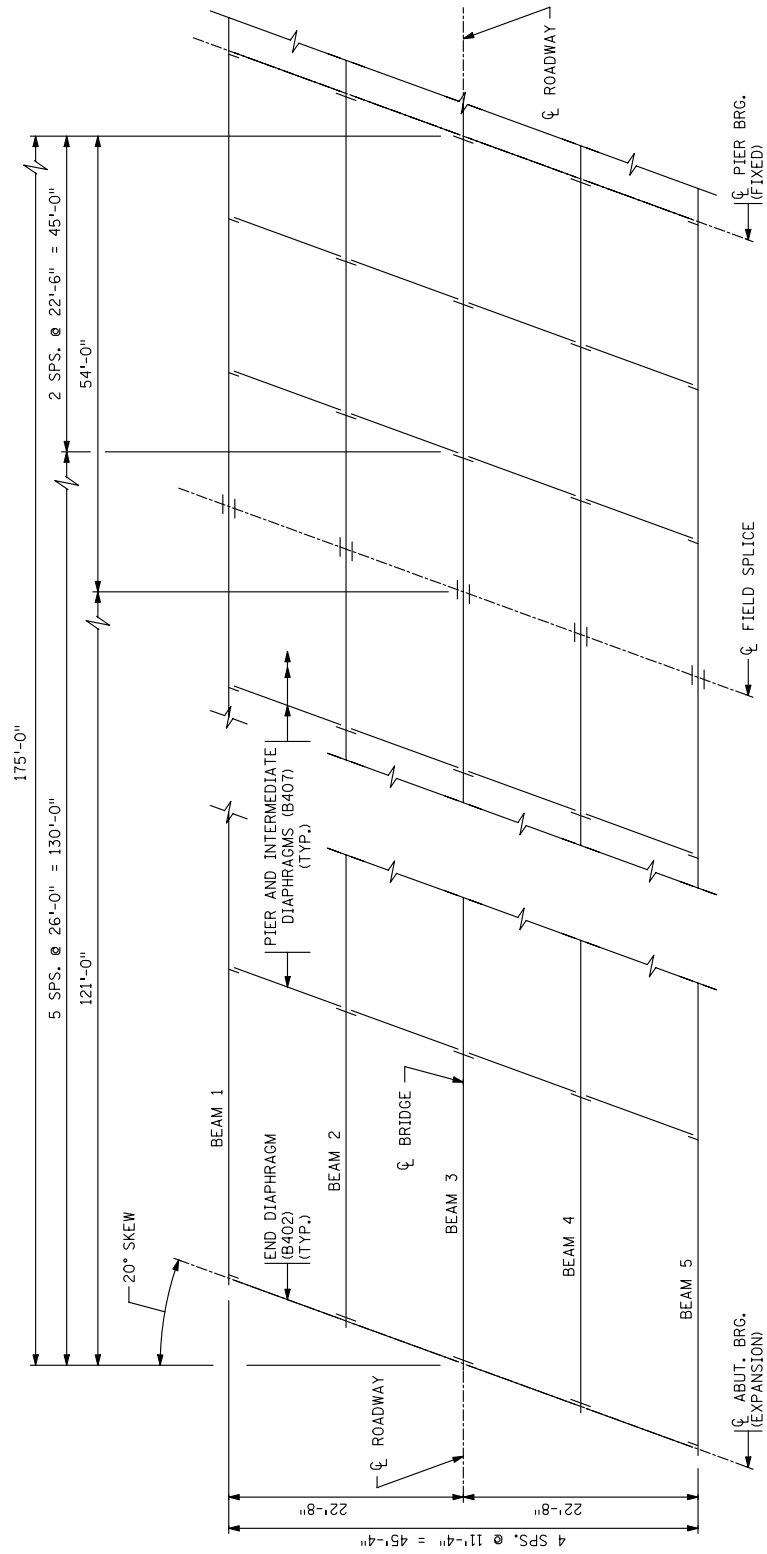


Figure 6.9.2
Partial Framing Plan

B. Determine Cross Section Properties**Non-Composite Section Properties**

The minimum depth of the steel girder (see Section 6.2 of this manual) in a continuous span is $0.0285L$. Mn/DOT typically considers a preliminary depth of $0.033L$.

For $L = 175$ ft:

$$0.0285 \cdot L = 59.9 \text{ in}$$

$$0.033 \cdot L = 69.3 \text{ in}$$

A member deeper than the minimum is usually the most economical. Adequate clearance is assumed available for the example, so try a 70 inch deep web.

[6.10.2.1.1]

Webs without longitudinal stiffeners must be proportioned such that:

$$\frac{D}{t_w} \leq 150$$

Then, the minimum web thickness t_w is:

$$t_w \geq \frac{D}{150} = \frac{70}{150} = 0.47 \text{ in}$$

Section 6.5 of this manual requires a minimum web thickness of $1/2$ inch.

Try a web thickness of $5/8$ inch.

[6.10.2.2]

The minimum flange width b_f and flange thickness t_f is:

$$b_f \geq \frac{D}{6} = \frac{70}{6} = 11.7 \text{ in}$$

$$t_f \geq 1.1t_w = 1.1(0.625) = 0.6875 \text{ in}$$

The minimum flange size specified by Section 6.5 of this manual is $3/4$ " x 14".

[C6.10.3.4]

For stability during shipping and erection the minimum compression flange width b_{fc} is:

$$b_{fc} \geq \frac{L}{85} = \frac{121 \cdot (12)}{85} = 17.1 \text{ in}$$

Based on experience and the minimum proportions given above, preliminary web and flange plate sizes were chosen and are shown in Figure 6.9.3. The girder is symmetric about the pier with a 1" x 20" top flange and a 1¹/₄" x 22" bottom flange in the positive moment region. In the negative moment region, the top flange is 1³/₄" x 20" near the field splice and 3¹/₄" x 20" over the pier. The bottom flange is 1³/₄" x 22" near the field splice and 3¹/₄" x 22" over the pier. For the web, a ⁵/₈" x 70" plate is used throughout.

[6.10.2.2]

Two additional flange proportion checks are required:

$$\frac{b_f}{2t_f} \leq 12.0 \quad \text{and} \quad 0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10.0$$

Make each check using the most critical section:

For 1" x 20" flange, $\frac{b_f}{2t_f} = \frac{20}{2 \cdot (1.0)} = 10.0 < 12.0$ OK

For Design Section 1,

$$I_{yc} = \frac{1.00 \cdot (20)^3}{12} = 666.7 \text{ in}^4$$

$$I_{yt} = \frac{1.25 \cdot (22)^3}{12} = 1109.2 \text{ in}^4$$

$$\frac{I_{yc}}{I_{yt}} = \frac{666.7}{1109.2} = 0.60 \quad 0.1 < 0.60 < 10.0 \quad \underline{\text{OK}}$$

The non-composite section properties of the girder are provided in Table 6.9.1.

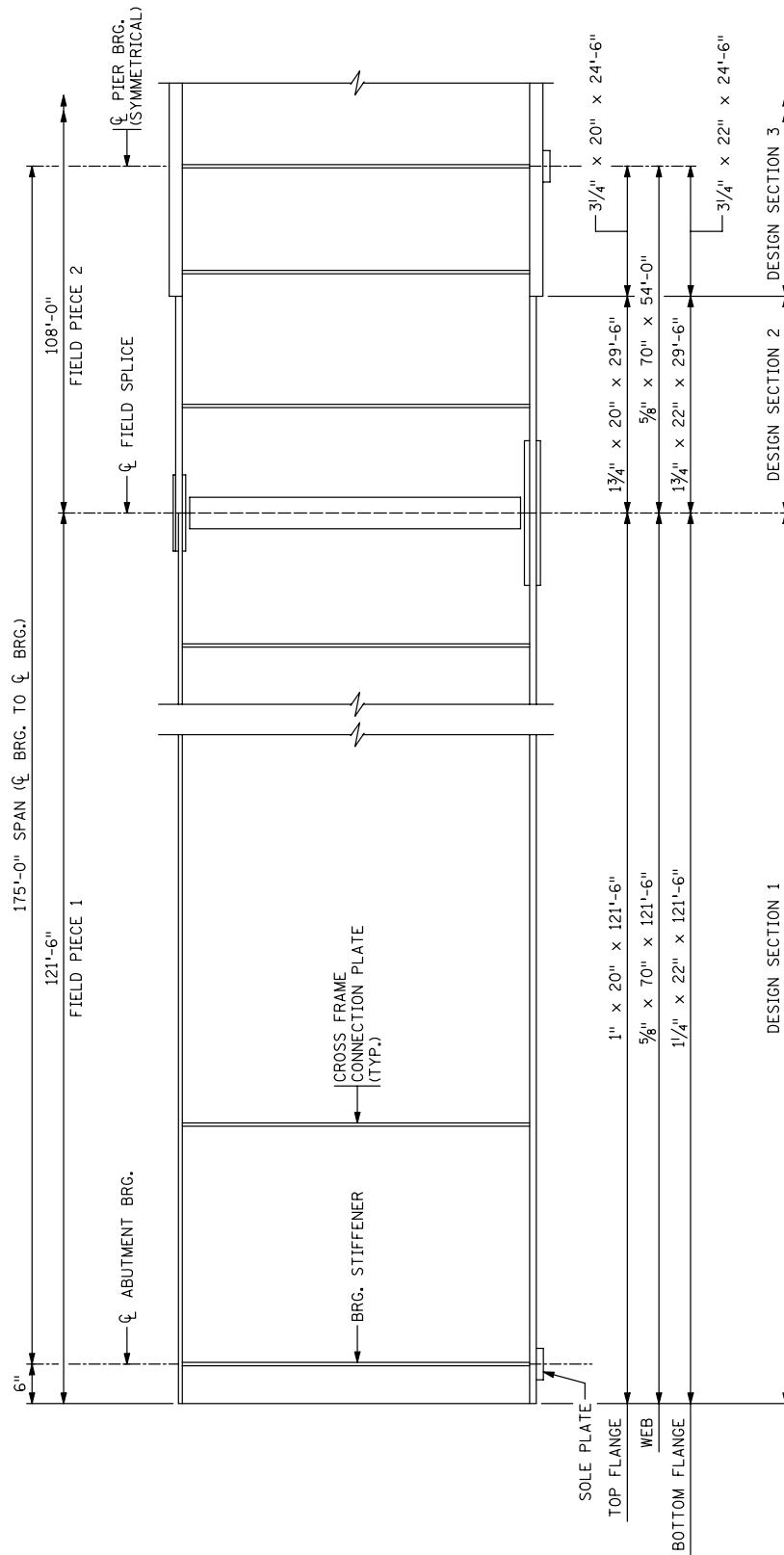


Figure 6.9.3
Preliminary Beam Layout – Half Elevation

Table 6.9.1
Non-Composite Section Properties

Parameter	Design Section 1 *	Design Section 2 **	Design Section 3 ***
d_{nc} (in)	72.25	73.50	76.50
A (in ²)	91.25	117.25	180.25
I (in ⁴)	77,179	112,345	200,770
y_t (in)	38.96	37.82	39.57
y_b (in)	33.29	35.68	36.93
S_t (in ³)	1981	2970	5074
S_b (in ³)	2318	3149	5437

* Design Section 1 is from abutment to field splice

** Design Section 2 is from field splice to flange butt splice

*** Design Section 3 is section over pier

Effective Flange Width

For simplicity, and in order to be conservative, the beams are designed assuming the full 9¹/₂ inches of deck thickness is placed in a single pour instead of the actual two pours.

For section property computations the deck thickness is reduced by ¹/₂ inch to account for wear.

[4.6.2.6]

The width of deck b_{eff} assumed to act compositely with the girder and resist external loads is the smallest of three values:

$$b_{eff} = 0.25 \cdot (\text{Effective span length}) = 0.25 \cdot (0.7 \cdot 175) \cdot 12 = 368 \text{ in}$$

or

$$b_{eff} = 12 \cdot (\text{Deck thickness}) + 0.5 \cdot (\text{Top flange width})$$

$$= 12 \cdot 9 + 0.5 \cdot 20 = 118 \text{ in} \quad \text{GOVERNS}$$

or

$$b_{eff} = \text{Average beam spacing} = 136 \text{ in}$$

Positive Moment Region Composite Section Properties

Using the modular ratios provided earlier ($n = 8$, $3n = 24$) results in:

[6.10.1.1.1b]

Transformed b_{eff_n} for transient, short-term loads

$$b_{eff_n} = \frac{118}{8} = 14.75 \text{ in}$$

Transformed $b_{\text{eff}_{3n}}$ for permanent, long-term loads

$$b_{\text{eff}_{3n}} = \frac{118}{24} = 4.92 \text{ in}$$

Choose a stool height for the girder, which is defined as the distance from the bottom of the deck to the top of the web. Because the top flange thickness varies along the girder length, the concrete portion of the stool will vary. The minimum required thickness of the concrete portion of the stool is $1\frac{1}{2}$ " at the edge of the flange. Therefore, the stool height is dependent on the thickest top flange plate, which is located at the pier.

The largest top flange plate is $3\frac{1}{4}$ " x 20" and the deck cross slope is 2%.

Then, the minimum required concrete portion of the stool height along the girder centerline is:

$$\text{stool}_{\text{min conc}} = 1.50 + 0.02 \cdot 0.5 \cdot 20 = 1.70 \text{ in} \quad \text{Use} = 1.75 \text{ in}$$

The minimum required total stool height along the girder centerline at the pier is:

$$\text{stool}_{\text{min pier}} = 3.25 + 1.75 = 5.0 \text{ in}$$

This bridge is on a straight grade, so residual camber is required, which "eats" into the stool at midspan. Therefore, check the minimum required stool height at midspan also to see if it governs.

The required residual camber is:

$$\text{camber}_{\text{res}} = 1.50 + \left(\frac{175 - 100}{10} \right) \cdot 0.125 = 2.44 \text{ in}$$

$$\text{Use camber}_{\text{res}} = 2.50 \text{ in}$$

The top flange plate at midspan is 1" x 20". Then, the minimum required total stool height along the girder centerline at midspan is:

$$\text{stool}_{\text{min midspan}} = 1.00 + 1.75 + 2.50 = 5.25 \text{ in} \quad \underline{\text{GOVERNS}}$$

Choose a stool height of 5.25 inches.

For calculation of the girder section properties, use the minimum concrete stool height t_{cstool} equal to 1.75 inches.

See Figure 6.9.4 and Table 6.9.2 for the composite sections and computed properties used for design in the positive moment region. Note that only Design Sections 1 and 2 fall in the positive moment region.

Negative Moment Region Composite Section Properties

[6.10.1.1.1c]

For negative moment regions, the section assumed effective in resisting external loads is the steel girder section plus the reinforcement within an effective width of the slab.

[6.10.1.7]

In negative moment regions, the longitudinal reinforcing steel in the deck is approximately 1% of the area of the deck. Two thirds of this steel is to be placed in the top mat of reinforcement. Referring to Figure 9.2.1.7, the area of steel within the effective flange width is:

Top mat: #16 bars @ 18" with 2- #19 bars in between

$$A_{\text{stop}} = (0.31 + 2 \cdot 0.44) \cdot \left(\frac{118}{18} \right) = 7.80 \text{ in}^2$$

Bottom mat: #13 bars @ 6"

$$A_{\text{sbot}} = 0.20 \cdot \left(\frac{118}{6} \right) = 3.93 \text{ in}^2$$

The top mat is located 3.50 inches from the top of the deck (based on 3 inches clear, $\frac{1}{2}$ inch wear, and #16 transverse bars) and the bottom mat is located 1.88 inches from the bottom (based on 1 inch clear, and #16 transverse bars). See Figure 6.9.4 and Table 6.9.2 for the composite sections and computed properties used for design in the negative moment region.

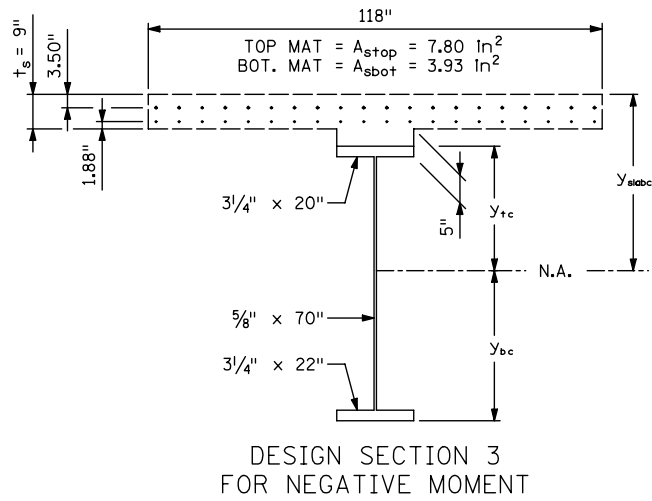
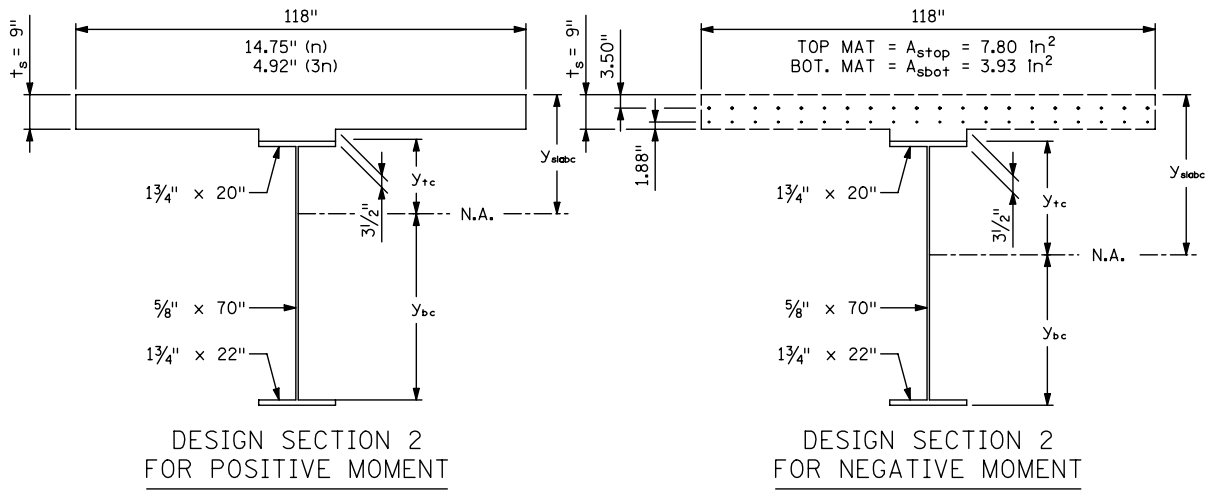
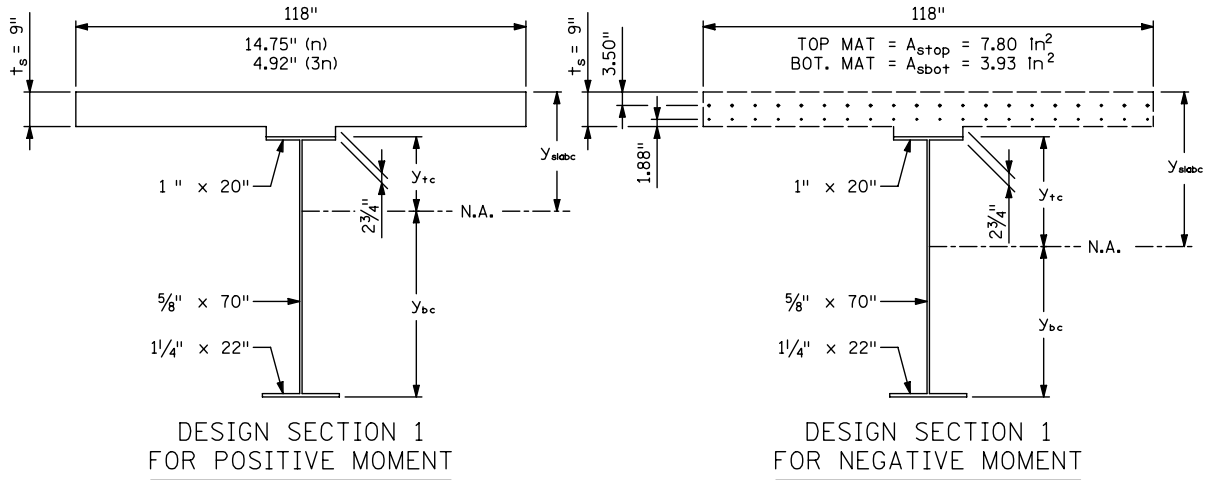


Figure 6.9.4

**Table 6.9.2
Composite Section Properties for Design**

Parameter	Design Section 1 for Positive Moment *		Design Section 1 for Negative Moment *	Design Section 2 for Positive Moment **		Design Section 2 for Negative Moment **	Design Section 3 for Negative Moment ***
	n	3n		n	3n		
A_c (in ²)	228.38	136.96	102.98	254.38	162.96	128.98	191.98
I_c (in ⁴)	189,316	139,281	98,220	235,171	176,064	132,855	223,679
Y_{slabc} (in)	22.67	34.68	44.58	24.91	36.26	44.58	47.53
Y_{tc} (in)	11.92	23.93	33.83	14.16	25.51	33.83	36.78
Y_{bc} (in)	60.33	48.32	38.42	59.34	47.99	39.67	39.72
S_{tc} (in ³)	15,882	5820	2903	16,608	6902	3927	6082
S_{bc} (in ³)	3138	2882	2556	3963	3669	3349	5631

* Design Section 1 is from abutment to field splice
 ** Design Section 2 is from field splice to flange butt splice
 *** Design Section 3 is section over pier

**C. Select Applicable
Load Combinations
and Load Factors
[1.3.3-1.3.5]
[3.4.1]**

The following load multipliers will be used for this example.

$\eta_D = 1.00$
 $\eta_R = 1.00$
 $\eta_I = 1.00$

Standard HL-93 loading will be used. The load combinations considered applicable to the design example are identified below:

STRENGTH I: $1.25 \cdot DC + 1.75 \cdot LL$

Primary applications include:

- maximum bottom flange stress in positive moment location
- maximum top and bottom flange stress in negative moment locations

STRENGTH IV: $1.5 \cdot DC$

Primary applications include:

- maximum bottom flange stress in positive moment location
- maximum top and bottom flange stress in negative moment locations

SERVICE II: $1.0 \cdot DC + 1.3 \cdot LL$

Corresponds to the overload provisions in the AASHTO Standard Specifications pertaining to yield control and to slip-critical connections.

FATIGUE: $0.75 \cdot LL_{\text{range}}$

Checks to limit the potential for fatigue cracks to form in a structure.

[3.4.2]

CONSTRUCTION LOAD COMBINATION: $1.25 \cdot DC_{\text{temp}} + 1.5 \cdot LL_{\text{temp}}$

During erection, the girder will need to resist stresses associated with the steel section alone. In addition, the need for diaphragms or cross frames will be determined at this stage.

Due to the continuous configuration, maximum and minimum (γ_p) load factor values will be used.

IM = 15% when evaluating fatigue and fracture

IM = 33% when evaluating all other limit states

**D. Live Load
Distribution Factors
(LLDF) for Moment
[4.6.2.2.2b]
[C4.6.2.2.1-1]**

1. Interior Beam Moment LLDFs

For LRFD Table 4.6.2.2.1-1, a Type (a) superstructure describes the structural system used in this example. Per LRFD Table 4.6.2.2.2b-1, the approximate distribution equations can be used if these geometric constraints are met:

Type (a) Cross Section Range of Applicability Limits for Flexure

Parameter	Design Example	Minimum	Maximum
Beam Spacing (S)	11.33'	3.5'	16.0'
Slab Thickness (t_s)	9.0"	4.5"	12"
Number of Beams (N_b)	5	4	-
Span Length (L)	175'	20'	240'

[4.6.2.2.1]

In addition to S, t_s , and L, the distribution equations for live load moment area also based on K_g , a longitudinal stiffness parameter defined as:

$$K_g = n \cdot (I + A \cdot e_g^2)$$

where n is the modular ratio, I is the non-composite girder moment of inertia, A is the non-composite area of the girder, and e_g is the distance between the centers of gravity of the non-composite girder and the deck.

Positive Moment Region

For the positive moment region,

$$I = 77,179 \text{ in}^4$$

$$A = 91.25 \text{ in}^2$$

$$t_s = 9.0 \text{ in}$$

$$y_t = 38.96 \text{ in}$$

$$e_g = \text{concrete stool} + \frac{t_s}{2} + y_t = 1.75 + \frac{9.0}{2} + 38.96 = 45.21 \text{ in}$$

$$K_g = 8 \cdot (77,179 + 91.25 \cdot 45.21^2) = 2.11 \times 10^6 \text{ in}^4$$

Moment LLDF for one design lane loaded:

**[Table
4.6.2.2.2b-1]**

$$\begin{aligned} gM &= 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.06 + \left(\frac{11.33}{14}\right)^{0.4} \left(\frac{11.33}{175}\right)^{0.3} \left(\frac{2.11 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.477 \text{ lanes/girder} \end{aligned}$$

Moment LLDF for two or more design lanes loaded:

$$\begin{aligned} gM &= 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.075 + \left(\frac{11.33}{9.5}\right)^{0.6} \left(\frac{11.33}{175}\right)^{0.2} \left(\frac{2.11 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.739 \text{ lanes/girder} \end{aligned} \quad \text{GOVERNS}$$

Moment LLDF for fatigue:

**[3.6.1.1.2]
[3.6.1.4]**

The design fatigue truck is a single lane vehicle that does not include the multiple presence factor. The tabulated approximate distribution factor equations for moment include the multiple presence factors. Therefore, remove the single lane multiple presence factor (1.2) from the LLDF for one lane loaded to get the fatigue LLDF.

$$gM_f = \frac{0.477}{1.2} = 0.398 \text{ lanes/girder}$$

Negative Moment Region

For the negative moment region (defined as between the dead load contraflexure points) use a single LLDF based on the largest negative moment section (located over the pier).

$$I = 200,770 \text{ in}^4$$

$$A = 180.25 \text{ in}^2$$

$$t_s = 9.0 \text{ in}$$

$$y_t = 39.57 \text{ in}$$

$$e_g = \text{concrete stool} + \frac{t_s}{2} + y_t = 1.75 + \frac{9.0}{2} + 39.57 = 45.82 \text{ in}$$

$$K_g = 8 \cdot (200,770 + 180.25 \cdot 45.82^2) = 4.634 \times 10^6 \text{ in}^4$$

Moment LLDF for one design lane loaded:

[Table
4.6.2.2.2b-1]

$$\begin{aligned} gM &= 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.06 + \left(\frac{11.33}{14}\right)^{0.4} \left(\frac{11.33}{175}\right)^{0.3} \left(\frac{4.634 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.512 \text{ lanes/girder} \end{aligned}$$

Moment LLDF for two or more design lanes loaded:

[Table
4.6.2.2.2b-1]

$$\begin{aligned} gM &= 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12 \cdot L \cdot (t_s)^3}\right)^{0.1} \\ &= 0.075 + \left(\frac{11.33}{9.5}\right)^{0.6} \left(\frac{11.33}{175}\right)^{0.2} \left(\frac{4.634 \cdot 10^6}{12 \cdot 175 \cdot (9)^3}\right)^{0.1} \\ &= 0.793 \text{ lanes/girder} \end{aligned} \quad \underline{\text{GOVERNS}}$$

Moment LLDF for fatigue:

$$gM_f = \frac{0.512}{1.2} = 0.426 \text{ lanes/girder}$$

[4.6.2.2.2d]

2. Exterior Beam Moment LLDFs

Table 4.6.2.2.2d-1 contains the approximate distribution factor equations for exterior beams. Check the value of d_e to ensure they are valid.

$$d_e = 3.00 - 1.67 = 1.33 \text{ ft} \quad (\text{see Figure 6.9.5})$$

$$-1.0 \text{ ft} < 1.33 \text{ ft} < 5.5 \text{ ft} \quad \underline{\text{OK}}$$

Moment LLDF for one design lane loaded:
 Use the lever rule and refer to Figure 6.9.5.

Exterior beam reaction or distribution factor is:

$$W_1 = W_2 = 0.5 \text{ lanes}$$

$$gM = \left[\frac{0.5 \cdot (11.33 - 0.67) + 0.5 \cdot (11.33 - 6.67)}{11.33} \right] \cdot 1.2$$

$$= 0.811 \text{ lanes/girder}$$

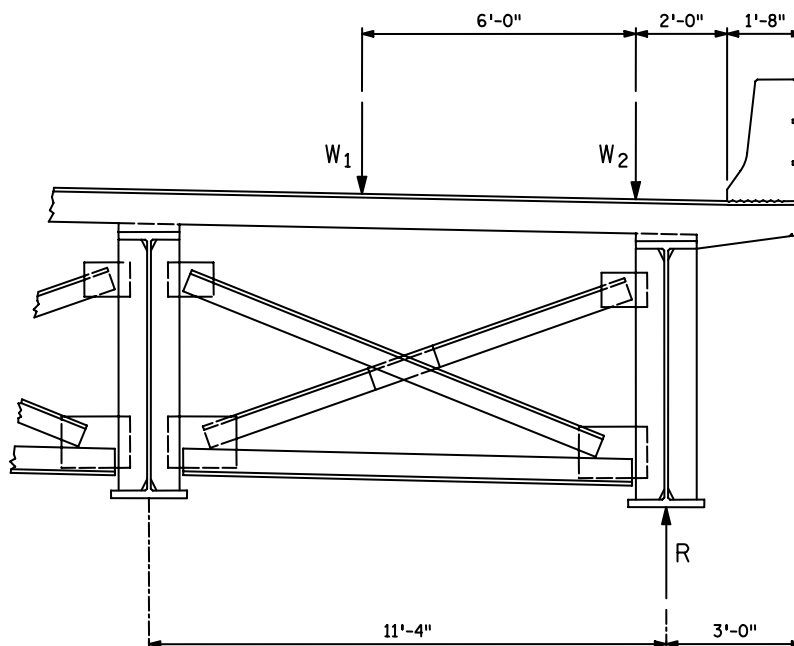


Figure 6.9.5

Moment LLDF for two or more design lanes loaded:

[Table
 4.6.2.2.2d-1]

$$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{1.33}{9.1} = 0.916$$

$$gM = e \cdot gM_{int} = 0.916 \cdot 0.739 = 0.677 \text{ lanes/girder for pos. moment}$$

$$gM = e \cdot gM_{int} = 0.916 \cdot 0.793 = 0.726 \text{ lanes/girder for neg. moment}$$

Moment LLDFs for fatigue:

$$gM_f = \frac{0.811}{1.2} = 0.676 \text{ lanes/girder}$$

3. Skew Reduction Factor for Moment

[4.6.2.2.2e]

The framing plan is skewed 20 degrees. Although there is no modification to the moments for skew until the skew angle is 30 degrees or greater, note that Mn/DOT has set this factor to 1.0 for all steel bridges in order to boost the load rating.

E. Live Load Distribution Factors (LLDF) for Shear

1. Interior Beam Shear LLDFs

Check range of applicability for use of the simplified distribution equations.

[Table 4.6.2.2.3a-1]

Type (a) Cross Section Range of Applicability Limits for Shear

Parameter	Design Example	Minimum	Maximum
Beam Spacing (S)	11.33'	3.5'	16.0'
Slab Thickness (t_s)	9.0"	4.5"	12"
Number of Beams (N_b)	5	4	-
Span Length (L)	175'	20'	240'
Pos. Mom. Long. Stiffness (K_g)	2.110×10^6	10,000	7.0×10^6
Neg. Mom. Long. Stiffness (K_g)	4.634×10^6	10,000	7.0×10^6

[4.6.2.2.3a]

All parameters for the design example are within permissible limits.

[Table 4.6.2.2.3a-1]

Shear LLDF for one design lane loaded:

$$gV = 0.36 + \frac{S}{25} = 0.36 + \frac{11.33}{25} = 0.813 \text{ lanes/girder}$$

Shear LLDF for two or more design lanes loaded:

$$gV = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 = 0.2 + \frac{11.33}{12} - \left(\frac{11.33}{35}\right)^2 = 1.039 \text{ lanes/girder}$$

[4.6.2.2.3b]

2. Exterior Beam Shear LLDFs

Shear LLDF for one design lane loaded:

Use the lever rule, which results in the same factor that was computed for flexure.

$$gV = 0.811 \text{ lanes/girder}$$

Shear LLDF for two or more design lanes loaded:

$$e = 0.6 + \frac{d_e}{10} = 0.6 + \frac{1.33}{10} = 0.733$$

$$gV = e \cdot gV_{int} = 0.733 \cdot 1.039 = 0.762 \text{ lanes/girder}$$

[4.6.2.2.3c]

3. Skew Correction Factor for Shear

There is a modification to the shear at the obtuse corners for bridges with skewed lines of support. This example has a skew angle of 20 degrees.

**[Table
4.6.2.2.3c-1]**

Type (a) Cross Sections Range of Applicability Limits for Skew Correction (Shear)

Parameter	Design Example	Minimum	Maximum
Skew Angle (θ)	20 degrees	0 degrees	60 degrees
Beam Spacing (S)	11.33'	3.5'	16.0'
Number of Beams (N_b)	5	4	-
Span Length (L)	175'	20'	240'

$$CF = 1.0 + 0.2 \cdot \left(\frac{12 \cdot L \cdot (t_s)^3}{K_g} \right)^{0.3} \cdot \tan(\theta)$$

$$= 1.0 + 0.2 \cdot \left(\frac{12 \cdot 175 \cdot (9)^3}{2.110 \cdot 10^6} \right)^{0.3} \cdot \tan(20)$$

$$= 1.07 \text{ lanes/girder at the abutment}$$

$$CF = 1.0 + 0.2 \cdot \left(\frac{12 \cdot 175 \cdot (9)^3}{4.634 \cdot 10^6} \right)^{0.3} \cdot \tan(20)$$

$$= 1.05 \text{ lanes/girder at the pier}$$

For simplicity, only the larger correction factor will be used to modify the live load distribution factors for shear. The adjusted shear distribution factors are:

Interior Girders

For one lane loaded:

$$gV = 0.813 \cdot 1.07 = 0.870 \text{ lanes/girder}$$

For two or more design lanes loaded:

$$gV = 1.039 \cdot 1.07 = 1.112 \text{ lanes/girder}$$

For fatigue:

$$gV_f = \frac{0.870}{1.2} = 0.725 \text{ lanes/girder}$$

Exterior Girders

For one lane loaded:

$$gV = 0.811 \cdot 1.07 = 0.868 \text{ lanes/girder}$$

For two or more design lanes loaded:

$$gV = 0.762 \cdot 1.07 = 0.815 \text{ lanes/girder}$$

For fatigue:

$$gV_f = \frac{0.868}{1.2} = 0.723 \text{ lanes/girder}$$

**Table 6.9.3
Distribution Factor Summary (Lanes/Girder)**

Girder/Force Component		One Lane LLDF	Multiple Lane LLDF	Governing LLDF
Interior Girder	+ Moment	0.477	0.739	0.739
	- Moment	0.512	0.793	0.793
	Shear	0.870	1.112	1.112
	+ Fatigue Moment	0.398		0.398
	- Fatigue Moment	0.426		0.426
	Fatigue Shear	0.725		0.725
Exterior Girder	+ Moment	0.811	0.677	0.811
	- Moment	0.811	0.726	0.811
	Shear	0.868	0.815	0.868
	+ Fatigue Moment	0.676		0.676
	- Fatigue Moment	0.676		0.676
	Fatigue Shear	0.723		0.723

F. Calculate Force Effects

Axial loads generated as a result of creep, shrinkage, and thermal movements will not be considered for the design of the girders. These loads are considered in the bearing and substructure design examples.

From this point forward only the design of an interior girder subject to dead load and HL-93 live loads will be presented.

[6.10.1.5]

Unfactored bending moments, shears, and reactions at different locations along the girder are presented in Tables 6.9.5 through 6.9.12. They are based on applying the loads as follows:

- DC1 loads are applied to a continuous beam model with varying non-composite section properties (see Table 6.9.1).
- DC2 loads are applied to a composite continuous beam model consisting of the steel girder plus the concrete deck where a modular ratio of 3n is used for the section properties.
- Live loads are applied to a composite continuous beam model consisting of the steel girder plus the concrete deck with a modular ratio of n.

Table 6.9.4 presents the areas and moments of inertia used for analysis.

**Table 6.9.4
Composite Section Properties for Analysis**

Parameter	Design Section 1 *		Design Section 2 **		Design Section 3 ***	
	n	3n	n	3n	n	3n
A _c (in ²)	228.38	136.96	254.38	162.96	317.38	225.96
I _c (in ⁴)	189,316	139,281	235,171	176,064	364,078	277,092

* Design Section 1 is from abutment to field splice

** Design Section 2 is from field splice to flange butt splice

*** Design Section 3 is section over pier

DC1 consists of the following loads: girder selfweight, concrete deck and wearing course, stool, and form loads. Note that Mn/DOT includes the wearing course load with DC1 loads (not DW). A 15% detail factor (based on the selfweight of the girder) is used to account for the dead load of connection and cross frame elements. A 0.010 ksf load is considered during construction to account for the weight of deck formwork.

$$w_{\text{beam}} = \frac{A_{\text{beam}}}{144} \cdot 0.490 \cdot (1.15) = \begin{pmatrix} 0.357 \text{ k/ft - Section 1} \\ 0.459 \text{ k/ft - Section 2} \\ 0.705 \text{ k/ft - Section 3} \end{pmatrix}$$

$$\begin{aligned}
 w_{\text{deck}} &= (\text{Area Deck} + \text{Area Stool}) \cdot 0.150 \\
 &= \left(11.33 \cdot \frac{9.5}{12} + \frac{5.25 - \text{top flange}}{12} \cdot \frac{20}{12} \right) \cdot 0.15 \\
 &= \begin{pmatrix} 1.434 \text{ k / ft} - \text{Section 1} \\ 1.418 \text{ k / ft} - \text{Section 2} \\ 1.387 \text{ k / ft} - \text{Section 3} \end{pmatrix}
 \end{aligned}$$

$$w_{\text{forms}} = 0.010 \cdot 11.33 = 0.113 \text{ k/ft}$$

A 0.020 ksf temporary construction live loading is also considered. It is assumed to be acting full length on a single span concurrent with wet concrete placement. In Table 6.9.6, DC_{const} consists of girder selfweight, form load, and one span of concrete. LL_{const} consists of one span of construction liveload.

DC2 consists of long-term dead loads, barrier, and future wearing course (FWC). Note that Mn/DOT uses a FWC of 0.020 ksf and includes the FWC load with DC2 loads (not DW).

$$w_{\text{barrier}} = 0.439 \text{ k/ft} \cdot \frac{2 \text{ barriers}}{5 \text{ girders}} = 0.176 \text{ k/ft}$$

$$w_{\text{fwc}} = 0.020 \text{ ksf} \cdot \frac{48}{5} = 0.192 \text{ k/ft}$$

The field splice is located 121 feet from the abutment bearing, approximately 0.69 of the span. This location was chosen as the nearest even foot along the span to the noncomposite dead load inflection point during the initial sizing.

All of the DC1 loads presented in the example include the 0.010 ksf load associated with the formwork. This increases the strength design loads by 2% but greatly simplifies the calculations. In reality, the load is applied to the non-composite section but is removed from the composite section. The actual stresses are also dependent on the pour sequence for the deck.

In the following tables, Girder Point 0.0 is the centerline of bearing at the abutment. Girder Point 1.0 is centerline of bearing at the pier. Due to the symmetry of the span arrangement, only data for Girder Points 0.0 to 1.0 is provided for most loads. However, due to the asymmetric loading

considered during construction, values are provided for both spans in Tables 6.9.6 and 6.9.13.

Table 6.9.5
Dead Load Bending Moments (Unfactored)

Girder Point	DC1 Moment (k-ft)				DC2 Moment (k-ft)		
	Girder	Slab and Stool	Forms	Total	Barrier	FWC	Total
0.0	0	0	0	0	0	0	0
0.1	329	1297	102	1728	163	178	341
0.2	549	2154	170	2873	273	297	570
0.3	659	2573	202	3434	328	357	685
0.4	659	2552	202	3413	329	359	688
0.5	550	2092	165	2807	277	302	579
0.6	331	1194	94	1619	170	186	356
0.691 ⁽¹⁾	32	-13	-1	18	26	28	54
0.7	-2	-145	-11	-158	10	11	21
0.742 ⁽²⁾	-186	-853	-67	-1106	-75	-82	-157
0.8	-470	-1922	-152	-2544	-204	-222	-426
0.807 ⁽³⁾	-510	-2066	-163	-2738	-221	-242	-463
0.860 ⁽⁴⁾	-819	-3197	-252	-4268	-358	-391	-749
0.871 ⁽⁵⁾	-894	-3458	-272	-4624	-390	-425	-815
0.9	-1092	-4134	-326	-5552	-472	-515	-987
0.936 ⁽⁶⁾	-1364	-5029	-396	-6790	-581	-633	-1214
1.0	-1918	-6778	-535	-9231	-794	-866	-1660

⁽¹⁾ Field splice

⁽²⁾ Second diaphragm away from pier

⁽³⁾ Midway point between first and second diaphragms away from pier

⁽⁴⁾ Flange butt splice

⁽⁵⁾ First diaphragm away from pier

⁽⁶⁾ Midway point between centerline of pier and first diaphragm away from pier

For this design example, the LRFD 6.10.3.2 constructibility checks use the values provided in Table 6.9.6.

Table 6.9.6
Construction Load Bending Moments (Unfactored)

Girder Point	DC _{const} Moment (kip-ft)			LL _{const} Moment (kip-ft)
	Girder	Slab and Stool	Forms	
0.0	0	0	0	0
0.1	329	1677	102	2108
0.2	549	2902	170	3621
0.297 ⁽¹⁾	657	3663	202	4522
0.3	659	3679	202	4540
0.365 ⁽²⁾	670	3957	205	4832
0.4	659	4003	202	4864
0.446 ⁽³⁾	623	4003	189	4815
0.5	550	3879	165	4594
0.6	331	3305	94	3730
0.691 ⁽⁴⁾	32	2385	-1	2416
0.7	-2	2279	-11	2266
0.8	-470	808	-152	186
0.860 ⁽⁵⁾	-819	-289	-252	-1360
0.9	-1092	-1109	-326	-2527
1.0	-1918	-3471	-535	-5924
1.1	-1092	-3124	-326	-4542
1.2	-470	-2777	-152	-3399
1.3	-2	-2430	-11	-2443
1.309 ⁽⁴⁾	32	-2400	-1	-2369
1.4	331	-2082	94	-1657
1.5	550	-1736	165	-1021
1.6	659	-1390	202	-529
1.7	659	-1041	202	-180
1.8	549	-695	170	24
1.9	329	-347	102	84
2.0	0	0	0	0

⁽¹⁾ Second diaphragm away from abutment

⁽²⁾ Midway point between second and third diaphragm away from abutment

⁽³⁾ Third diaphragm away from abutment

⁽⁴⁾ Field splice

⁽⁵⁾ Flange butt splice

The truck train generated the controlling negative bending moment over the pier. The distance between trucks in the train is variable but can be no less than 50 feet. The largest moment was obtained when the distance between the last axle of the first truck and the first axle of the second truck was 119 feet. The truck train multiplier was increased from 0.90 to 1.05 based on the Memo to Designers (2005-01).

Table 6.9.7 contains positive and negative live load moments due to truck, lane, and truck train loading.

Table 6.9.7
Live Load Design Moments per Lane (Unfactored)

Girder Point	Pos. M.* (kip-ft)	Neg. M.** (kip-ft)	Governing LL Type for Negative Moment
0.0	0	0	(Truck + IM) + Lane
0.1	2064	-329	(Truck + IM) + Lane
0.2	3510	-658	(Truck + IM) + Lane
0.3	4428	-987	(Truck + IM) + Lane
0.4	4783	-1316	(Truck + IM) + Lane
0.5	4648	-1645	(Truck + IM) + Lane
0.6	4073	-1973	(Truck + IM) + Lane
0.691 ⁽¹⁾	3182	-3113	(Truck Train + IM) + Lane
0.7	3085	-3153	(Truck Train + IM) + Lane
0.742 ⁽²⁾	2565	-3358	(Truck Train + IM) + Lane
0.8	1784	-3642	(Truck Train + IM) + Lane
0.807 ⁽³⁾	1696	-3694	(Truck Train + IM) + Lane
0.860 ⁽⁴⁾	1025	-4041	(Truck Train + IM) + Lane
0.871 ⁽⁵⁾	902	-4174	(Truck Train + IM) + Lane
0.9	625	-4584	(Truck Train + IM) + Lane
0.936 ⁽⁶⁾	341	-5254	(Truck Train + IM) + Lane
1.0	0	-6905	(Truck Train + IM) + Lane

* Positive M = (1.33 x Truck) + Lane

** Negative M = maximum of (1.33 x Truck) + Lane or 1.05 x [(1.33 x Truck Train) + Lane]

⁽¹⁾ Field splice

⁽²⁾ Second diaphragm away from pier

⁽³⁾ Midway point between first and second diaphragms away from pier

⁽⁴⁾ Flange butt splice

⁽⁵⁾ First diaphragm away from pier

⁽⁶⁾ Midway point between centerline of pier and first diaphragm away from pier

Table 6.9.8 lists the fatigue moment range at various girder points when the fixed axle fatigue truck is run across the structural model.

Table 6.9.8
Live Load Fatigue Moments per Lane (Unfactored)

Girder Point	Fatigue Moment Range (kip-ft) *
0.0	0
0.1	1248
0.2	2136
0.3	2691
0.4	2978
0.5	3053
0.6	2959
0.691 ⁽¹⁾	2691
0.7	2659
0.8	2209
0.860 ⁽²⁾	1908
0.9	1759
1.0	1567

* Fatigue Moment Range = $1.15 \cdot (\text{Fatigue Truck Positive M} - \text{Fatigue Truck Neg. M})$

⁽¹⁾ Field Splice

⁽²⁾ Flange Butt Splice

Table 6.9.9 presents the unfactored dead load shear forces at different girder locations for different load components.

**Table 6.9.9
Dead Load Shear (Unfactored)**

Girder Point	DC1 Shear (kips)				DC2 Shear (kips)		
	Girder	Slab and Stool	Forms	Total	Barrier	FWC	Total
0.0	22	87	7	115	11	12	23
0.1	16	62	5	82	8	8	16
0.2	9	36	3	49	5	5	10
0.3	3	11	1	15	1	2	3
0.4	-3	-14	-1	-18	-1	-2	-3
0.5	-9	-39	-3	-51	-4	-5	-9
0.6	-16	-64	-5	-85	-8	-8	-16
0.691 ⁽¹⁾	-22	-87	-7	-116	-11	-11	-22
0.7	-23	-89	-7	-119	-11	-11	-22
0.742 ⁽²⁾	-26	-100	-8	-134	-12	-13	-25
0.8	-31	-114	-9	-154	-14	-15	-29
0.860 ⁽³⁾	-37	-129	-10	-176	-16	-17	-33
0.871 ⁽⁴⁾	-38	-132	-10	-180	-16	-17	-33
0.9	-41	-139	-11	-191	-17	-18	-35
1.0	-53	-163	-13	-230	-20	-22	-42

- ⁽¹⁾ Field splice
- ⁽²⁾ Second diaphragm away from pier
- ⁽³⁾ Flange butt splice
- ⁽⁴⁾ First diaphragm away from pier

Table 6.9.10 contains the dead load reactions at Abutment, (Girder Point 0.0) and Pier (Girder Point 1.0). The reactions at Girder Point 1.0 are larger than the shear at Girder Point 0.0 because the reaction includes the load from both spans.

**Table 6.9.10
Dead Load Reactions (Unfactored)**

Girder Point	DC1 Reaction (kips)	DC2 Reaction (kips)
0.0	115	23
1.0	459	83

Table 6.9.11 contains the live load shear extremes for the various live load components. Per LRFD Article 3.6.1.3.1, truck train loading is not to be used for shear.

Table 6.9.11
Live Load Design Shear per Lane and Fatigue Shear (Unfactored)

Girder Point	Truck + Lane Positive Shear* (kips)	Truck + Lane Negative Shear* (kips)	Fatigue Truck Shear Range** (kips)
0.0	137	-19	78
0.1	116	-20	67
0.2	95	-31	57
0.3	75	-47	56
0.4	58	-63	58
0.5	43	-80	60
0.6	30	-98	62
0.691 ⁽¹⁾	20	-114	64
0.7	19	-115	65
0.742 ⁽²⁾	15	-122	66
0.8	10	-132	69
0.860 ⁽³⁾	6	-142	71
0.871 ⁽⁴⁾	5	-144	71
0.9	4	-149	72
1.0	0	-166	76

* = (1.33 · (Truck Shear)) + Lane Shear

** = 1.15 · (Fatigue Truck Positive V - Fatigue Truck Negative V)

⁽¹⁾ Field splice

⁽²⁾ Second diaphragm away from pier

⁽³⁾ Flange butt splice

⁽⁴⁾ First diaphragm away from pier

Table 6.9.12 presents the live load reactions at the abutment (Girder Point 0.0) and the pier (Girder Point 1.0). Similar to the dead load reactions presented in Table 6.9.10, the reactions at Girder Point 1.0 are larger than the shear at Girder Point 0.0 because the reaction includes the load from both spans. Per LRFD Article 3.6.1.3.1 the truck train loading needs to be considered for reactions at interior supports.

Table 6.9.12
Live Load Reactions per Lane (Unfactored)

Girder Point	LL + IM Reaction (kips)	LL Only Reaction (kips)
0.0	137	115
1.0	341*	294

* HL-93 Truck Train + Lane Reaction governs

Table 6.9.13 presents shear values due to construction loads.

Table 6.9.13
Construction Load Shear (Unfactored)

Girder Point	DC _{const} Shear (kips)				LL _{const} Shear (kips)
	Girder	Slab+Stool	Forms	Total	
0.0	22	108	7	137	18
0.1	16	82	5	103	13
0.2	9	57	3	69	9
0.3	3	32	1	36	4
0.4	-3	6	-1	2	1
0.5	-9	-20	-3	-32	-3
0.6	-16	-45	-5	-66	-8
0.691 ⁽¹⁾	-22	-69	-7	-98	-11
0.7	-23	-71	-7	-101	-11
0.8	-31	-97	-9	-137	-15
0.860 ⁽²⁾	-37	-112	-10	-159	-18
0.9	-41	-122	-11	-174	-19
1.0 Left	-53	-148	-13	-214	-23
1.0 Right	53	20	13	86	3
1.1	41	20	11	72	3
1.140 ⁽²⁾	37	20	10	67	3
1.2	31	20	9	60	3
1.3	23	20	7	50	3
1.309 ⁽¹⁾	22	20	7	49	3
1.4	16	19	5	40	3
1.5	9	20	3	32	3
1.6	3	20	1	24	3
1.7	-3	20	-1	16	3
1.8	-9	20	-3	8	3
1.9	-16	20	-5	-1	3
2.0	-22	20	-7	-9	3

⁽¹⁾ Field splice

⁽²⁾ Flange butt splice

The checks in this example begin with the strength checks on the preliminary layout. Designers should be aware that deflections may control the design. The deflection checks for this example are presented in Section M.

**G. Flexure –
Investigate
Strength Limit
State**

At the strength limit state the girder is designed to carry factored dead and live loads. The resisting section in the positive moment regions is the girder plus deck composite section. In the negative moment regions, resistance is provided by the girder plus deck reinforcement composite section.

**G.1 Design
Section 1 –
Positive Moment**

The maximum factored positive moment M_u is at $0.4L = 70.0$ ft from each abutment.

$$\begin{aligned} M_u &= 1.25 \cdot (3413 + 688) + 1.75 \cdot (4783) \cdot 0.739 \\ &= 5126 + 6186 = 11312 \text{ kip-ft} \end{aligned}$$

The maximum factored stresses are at $0.4L$ for top and bottom flanges. Refer back to Tables 6.9.1 and 6.9.2 for section properties and Tables 6.9.5 & 6.9.7 for moments.

For top flange:

$$\begin{aligned} f_{\text{buc}} &= 1.25 \left(\frac{M_{\text{DC1}}}{S_t} + \frac{M_{\text{DC2}}}{S_{\text{tc}(3n)}} \right) + 1.75 \left(\frac{M_{\text{LL+I}}}{S_{\text{tc}(n)}} \right) (\text{LLDF}) \\ &= 1.25 \left(\frac{3413}{1981} + \frac{688}{5820} \right) (12) + 1.75 \left(\frac{4783}{15882} \right) (12) (0.739) \\ &= 32.3 \text{ ksi} \end{aligned}$$

For bottom flange:

$$\begin{aligned} f_{\text{but}} &= 1.25 \left(\frac{M_{\text{DC1}}}{S_b} + \frac{M_{\text{DC2}}}{S_{\text{bc}(3n)}} \right) + 1.75 \left(\frac{M_{\text{LL+I}}}{S_{\text{bc}(n)}} \right) (\text{LLDF}) \\ &= 1.25 \left(\frac{3413}{2318} + \frac{688}{2882} \right) (12) + (1.75) \left(\frac{4783}{3138} \right) (12) (0.739) \\ &= 49.3 \text{ ksi} \end{aligned}$$

Since the bridge only has a minor skew, and the beam is an interior beam, lateral bending does not need to be considered.

$$f_\ell = 0.0$$

The procedure for evaluating the flexural strength of a girder in accordance with the LRFD Specifications is quite involved. To clarify the steps involved, flow charts are included in LRFD Appendix C, Article C6.4. Follow the procedure shown in LRFD Figures C6.4.4-1 and C6.4.5-1.

The span under consideration is continuous, but Mn/DOT does not allow design using the moment redistribution provisions of LRFD Appendix B. In addition, Mn/DOT does not permit exceeding the moment at first yield for all sections at positive moment region. Therefore, there is no need to check section compactness criteria and the check will be made in accordance with LRFD 6.10.7.2 (see LRFD Figure C6.4.5-1).

Compression Flange in Positive Flexure

[6.10.7.2.2]

The nominal flexural resistance of the compression flange shall be taken as:

$$F_{nc} = R_b R_h F_{yc}$$

[6.10.1.10.2]

First determine R_b :

[6.10.2.1.1]

$$\frac{D}{t_w} = \frac{70}{0.625} = 112 < 150 \quad \underline{OK}$$

The section is composite in positive flexure. Therefore web load-shedding factor $R_b = 1.0$.

[6.10.1.10.1]

The section is homogenous. Therefore hybrid factor $R_h = 1.0$.

$$F_{yc} = 50 \text{ ksi}$$

$$\text{Then, } F_{nc} = (1.0) (1.0) (50.0) = 50 \text{ ksi}$$

Compression flange shall satisfy:

[6.10.7.2.1]

$$f_{buc} \leq \phi_f F_{nc} \quad \phi_f = 1.0$$

$$f_{buc} = 32.3 \text{ ksi} < (1.0) (50.0) = 50.0 \text{ ksi} \quad \underline{OK}$$

Tension Flange in Positive Flexure

[6.10.7.2.2]

The nominal flexure resistance of tension flange shall be taken as:

$$F_{nt} = R_h F_{yt}$$

The section is homogenous, so $R_h = 1.0$

$$F_{yt} = 50.0 \text{ ksi}$$

$$F_{nt} = (1.0) (50.0) = 50.0 \text{ ksi}$$

Tension flange shall satisfy:

[6.10.7.2.1]

$$f_{but} + \frac{1}{3} f_{\ell} \leq \phi_f F_{nt}$$

$$f_{but} = 49.3 \text{ ksi} \quad f_{\ell} = 0.0 \text{ ksi} \quad \phi_f = 1.0$$

$$f_{but} + \frac{1}{3} f_{\ell} = 49.3 + 0.0 = 49.3 \text{ ksi} < (1.0) (50.0) = 50.0 \text{ ksi} \quad \text{OK}$$

The positive moment section has adequate flexural strength.

[6.10.7.3]

Check ductility of the section:

[D6.1]

To determine D_p use Appendix D from Section 6. The figure for load components for positive bending sections is presented in Figure 6.9.6.

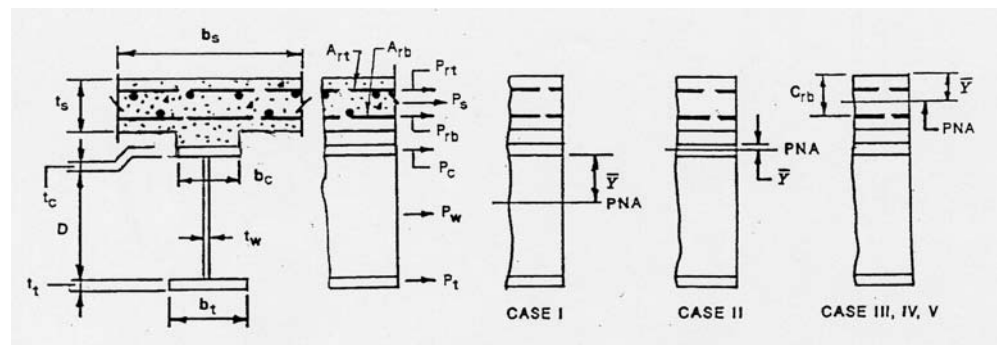


Figure 6.9.6

To simplify computations neglect the P_{rt} and P_{rb} terms.

$$P_c = \text{Force in the top flange} = 50 \cdot 1.0 \cdot 20 = 1000 \text{ kips}$$

$$P_w = \text{Force in the web} = 50 \cdot 0.625 \cdot 70 = 2188 \text{ kips}$$

$$P_t = \text{Force in the bottom flange} = 50 \cdot 1.25 \cdot 22 = 1375 \text{ kips}$$

$$P_s = \text{Force in the slab} = 0.85 \cdot 4 \cdot (9 \cdot 118 + 1.75 \cdot 20) = 3730 \text{ kips}$$

Begin by checking Case I (PNA in the web of the girder).

$$P_t + P_w = 1375 + 2188 = 3563 \text{ kips}$$

$$P_c + P_s = 1000 + 3730 = 4730 > 3563$$

Therefore, the PNA is not in the web.

Try Case II (PNA in the top flange)

$$P_t + P_w + P_c = 1375 + 2188 + 1000 = 4563 \text{ kips}$$

$$P_s = 3730 < 4563 \text{ kips}$$

Therefore, the PNA is in the top flange. Use the equation in LRFD Table D6.1-1 to locate the position of the PNA in the top flange.

$$\bar{y}_{tfl} = \frac{t_c}{2} \cdot \left(\frac{P_w + P_t - P_s}{P_c} + 1 \right) = \frac{1.0}{2} \cdot \left(\frac{2188 + 1375 - 3730}{1000} + 1 \right) = 0.42 \text{ in}$$

$$D_t = t_s + t_{cstool} + d_{nc} = 9 + 1.75 + 72.25 \text{ in} = 83.0 \text{ in}$$

$$D_p = t_s + t_{cstool} + \bar{y}_{tfl} = 9 + 1.75 + 0.42 = 11.17 \text{ in}$$

$$0.42 \cdot D_t = 0.42 \cdot 83.0 = 34.86 \text{ in}$$

$$D_p = 11.17 \text{ in} < 34.86 \text{ in} \quad \underline{OK}$$

G.2 Design
Section 3 –
Negative Moment

First, determine the maximum factored stresses at pier for the top and bottom flange. Referring back to Tables 6.9.1 and 6.9.2 for section properties and Tables 6.9.5 and 6.9.7 for moments:

For top flange:

$$\begin{aligned} f_{\text{but}} &= 1.25 \cdot \left(\frac{M_{\text{DC1}}}{S_t} + \frac{M_{\text{DC2}}}{S_{\text{tc}}} \right) + 1.75 \cdot \left(\frac{M_{\text{LL+I}}}{S_{\text{tc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left(\frac{9231}{5074} + \frac{1660}{6082} \right) \cdot 12 + 1.75 \cdot \left(\frac{6905}{6082} \right) \cdot 12 \cdot 0.793 = 50.3 \text{ ksi} \end{aligned}$$

For bottom flange:

$$\begin{aligned} f_{\text{buc}} &= 1.25 \cdot \left(\frac{M_{\text{DC1}}}{S_b} + \frac{M_{\text{DC2}}}{S_{\text{bc}}} \right) + 1.75 \cdot \left(\frac{M_{\text{LL+I}}}{S_{\text{bc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left(\frac{9231}{5437} + \frac{1660}{5631} \right) \cdot 12 + 1.75 \cdot \left(\frac{6905}{5631} \right) \cdot 12 \cdot 0.793 = 50.3 \text{ ksi} \end{aligned}$$

Since bridge only has minor skew, and it is an interior beam, no lateral bending needs to be considered

$$f_\ell = 0.0 \text{ ksi}$$

Next, determine flexural resistance of top and bottom flanges. Refer to the flow chart shown in LRFD Figure C6.4.4.-1. Mn/DOT does not use the optional provisions of Appendix A, so there is no need to check the web slenderness ratio and flange inertia ratio of LRFD Article 6.10.6.2.3. Our check will be made in accordance with LRFD 6.10.8. (See LRFD Figure C6.4.6-1.) Begin with the compression (bottom) flange, which is discretely braced. The flexural resistance of the compression flange F_{nc} is the smaller of the local buckling resistance $F_{\text{nc}(\text{FLB})}$ and the lateral torsional buckling resistance $F_{\text{nc}(\text{LTB})}$. First, check local buckling resistance.

[6.10.8.2.2]

$$\lambda_f = \frac{b_{\text{fc}}}{2t_{\text{fc}}} = \frac{22}{2 \cdot (3.25)} = 3.38$$

[C6.10.8.2.2]

For $F_{\text{yc}} = 50 \text{ ksi}$, $\lambda_{\text{pf}} = 9.2$

Then, $\lambda_f < \lambda_{\text{pf}}$ and compression flange is compact.

For a compact compression flange, flexural resistance F_{nc} is

$$F_{\text{nc}(\text{FLB})} = R_b R_h F_{\text{yc}}$$

[6.10.1.10.2]

For a negative moment section, R_b is dependent on the web slenderness ratio $\frac{2 \cdot D_c}{t_w}$.

[D6.3.1]

For composite negative moment sections, D_c is based on the section consisting of the steel girder plus the longitudinal reinforcement.

Then, $D_c = y_{bc} - t_{fc} = 39.72 - 3.25 = 36.47$ in

$$\frac{2 \cdot D_c}{t_w} = \frac{2 \cdot (36.47)}{0.625} = 116.7$$

[C6.10.1.10.2]

For $F_y = 50$ ksi, $\lambda_{rw} = 137$. Therefore,

$$\frac{2 \cdot D_c}{t_w} < \lambda_{rw} \text{ and } R_b = 1.0.$$

The girder is homogeneous (not a hybrid), so $R_h = 1.0$

The flange local buckling resistance is:

$$F_{nc(\text{FLB})} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

[6.10.8.2.3]

Now, determine the lateral torsional buckling resistance.

At the pier, the unbraced length L_b is:

$$L_b = 22.5 \text{ ft} = 270 \text{ in}$$

This is to be compared with the compact bracing limit L_p .

$$L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$$

$$r_t = \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{D_c t_w}{3 b_{fc} t_{fc}}\right)}} = \frac{22}{\sqrt{12 \cdot \left(1 + \frac{36.47 \cdot 0.625}{3 \cdot 22 \cdot 3.25}\right)}} = 6.04 \text{ in}$$

$$\text{Then, } L_p = 1.0 \cdot (6.04) \cdot \sqrt{\frac{29,000}{50}} = 145.5 \text{ in} < 270 \text{ in}$$

Therefore, $L_b > L_p$, so check noncompact bracing limit L_r .

$$L_r = \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}}$$

$$F_{yr} = 0.7 \cdot F_{yc} = 0.7 \cdot (50.0) = 35.0 \text{ ksi}$$

$$\text{Then, } L_r = \pi \cdot (6.04) \cdot \sqrt{\frac{29,000}{35.0}} = 546.2 \text{ in} > 270 \text{ in}$$

Therefore, $L_p < L_b < L_r$

$$\text{And, } F_{nc(LTB)} = C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

C_b , moment gradient modifier, can be calculated as follows (refer to Figure 6.9.7 and LRFD C6.4.10):

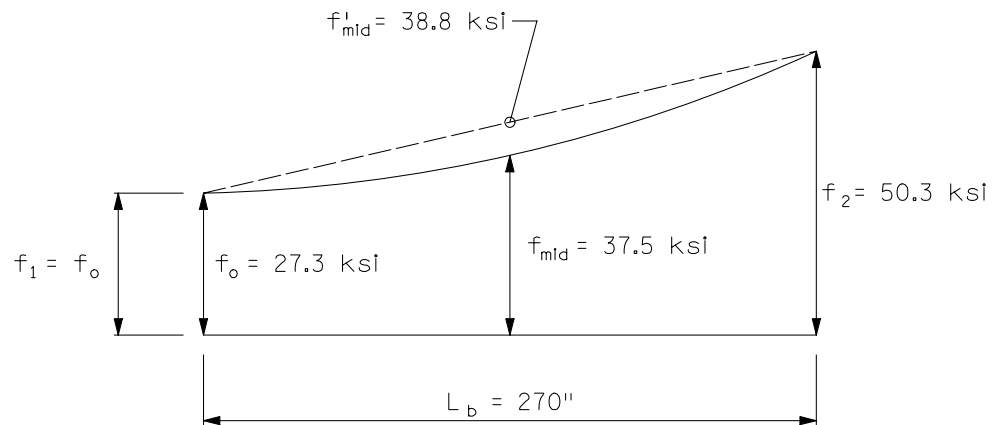


Figure 6.9.7

f_2 is the compression stress at centerline of pier diaphragm:

$$f_2 = 50.3 \text{ ksi}$$

f_o is the compression stress at first brace point (diaphragm) away from pier:

$$\begin{aligned} f_o &= 1.25 \left(\frac{4624}{5437} + \frac{815}{5631} \right) (12) + (1.75) \left(\frac{4174}{5631} \right) (12) (0.793) \\ &= 27.3 \text{ ksi} \end{aligned}$$

f_{mid} is the compression stress at the point midway between the centerline of pier and first brace point away from pier:

$$\begin{aligned} f_{\text{mid}} &= 1.25 \left(\frac{6790}{5437} + \frac{1214}{5631} \right) (12) + (1.75) \left(\frac{5254}{5631} \right) (12) (0.793) \\ &= 37.5 \text{ ksi} \end{aligned}$$

Check if moment envelope is concave between f_2 and f_o . The stress at the middle of the unbraced length, f'_{mid} , assuming a linear variation between f_2 and f_o is:

$$f'_{\text{mid}} = \frac{f_2 + f_o}{2} = \frac{50.3 + 27.3}{2} = 38.8 \text{ ksi} > 37.5 \text{ ksi}$$

Therefore, moment envelope is concave and $f_1 = f_o = 27.3 \text{ ksi}$

$$\frac{f_1}{f_2} = \frac{27.3}{50.3} = 0.54$$

$$\begin{aligned} C_b &= 1.75 - 1.05 \left(\frac{f_1}{f_2} \right) + 0.3 \left(\frac{f_1}{f_2} \right)^2 \\ &= 1.75 - (1.05) (0.54) + (0.3) (0.54)^2 \\ &= 1.27 < 2.3 \quad \quad \quad \underline{\text{OK}} \end{aligned}$$

Then,

$$\begin{aligned} F_{\text{nc(LTB)}} &= 1.27 \cdot \left[1 - \left(1 - \frac{35}{1.0 \cdot 50} \right) \left(\frac{270 - 145.5}{546.2 - 145.5} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0 \\ &= 57.6 \text{ ksi} > 50 \text{ ksi} \end{aligned}$$

Therefore,

$$F_{\text{nc(LTB)}} = 50.0 \text{ ksi}$$

The compression flange flexural resistance F_{nc} is the smaller of $F_{\text{nc(FLB)}}$ and $F_{\text{nc(LTB)}}$.

$$F_{\text{nc}} = F_{\text{nc(LTB)}} = 50.0 \text{ ksi}$$

Then, $\phi F_{\text{nc}} = 1.0 \cdot 50.0 = 50.0 \text{ ksi}$

$$f_{\text{buc}} + \frac{1}{3} f_{\ell} = 50.3 + 0.0 = 50.3 \text{ ksi} \approx 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

[6.10.8.1.3]

Now, consider the tension (top) flange, which is continuously braced by the deck in its final state.

$$\text{Then, } \phi F_{\text{nt}} = \phi \cdot (R_h \cdot F_{yt}) = 1.0 \cdot (1.0 \cdot 50.0) = 50.0 \text{ ksi}$$

$$f_{\text{but}} = 50.3 \approx 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

G.3 Design**Section 2 –****Negative Moment**

First, determine the maximum factored stresses at 0.860L (flange butt splice location) for the top and bottom flange. Referring back to Tables 6.9.1 and 6.9.2 for section properties and Tables 6.9.5 and 6.9.7 for moments:

For top flange:

$$\begin{aligned} f_{\text{but}} &= 1.25 \cdot \left(\frac{M_{\text{DC1}}}{S_t} + \frac{M_{\text{DC2}}}{S_{\text{tc}}} \right) + 1.75 \cdot \left(\frac{M_{\text{LL+I}}}{S_{\text{tc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left(\frac{4268}{2970} + \frac{749}{3927} \right) \cdot 12 + 1.75 \cdot \left(\frac{4041}{3927} \right) \cdot 12 \cdot 0.793 = 41.6 \text{ ksi} \end{aligned}$$

For bottom flange:

$$\begin{aligned} f_{\text{buc}} &= 1.25 \cdot \left(\frac{M_{\text{DC1}}}{S_b} + \frac{M_{\text{DC2}}}{S_{\text{bc}}} \right) + 1.75 \cdot \left(\frac{M_{\text{LL+I}}}{S_{\text{bc}}} \right) \cdot (\text{LLDF}) \\ &= 1.25 \cdot \left(\frac{4268}{3149} + \frac{749}{3349} \right) \cdot 12 + 1.75 \cdot \left(\frac{4041}{3349} \right) \cdot 12 \cdot 0.793 = 43.8 \text{ ksi} \end{aligned}$$

Since bridge only has minor skew, and it is an interior beam, no lateral bending needs to be considered:

$$f_{\ell} = 0.0 \text{ ksi}$$

Next, determine the flexural resistance of top and bottom flanges. Refer to the flow chart shown in LRFD Figure C6.4.4.-1. Mn/DOT does not use the optional provisions of Appendix A, so there is no need to check the web slenderness ratio and flange inertia ratio of LRFD Article 6.10.6.2.3. The check will be made in accordance with LRFD 6.10.8. (See LRFD Figure C6.4.6-1) Begin with the compression (bottom) flange, which is discretely braced. The flexural resistance of the compression flange F_{nc} is the smaller of the local buckling resistance $F_{\text{nc(FLB)}}$ and the lateral torsional buckling resistance $F_{\text{nc(LTB)}}$.

First, check local buckling resistance.

$$[6.10.8.2.2] \quad \lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{22}{2 \cdot (1.75)} = 6.29$$

$$[C6.10.8.2.2] \quad \text{For } F_{yc} = 50 \text{ ksi, } \lambda_{pf} = 9.2$$

Then, $\lambda_f < \lambda_{pf}$ and compression flange is compact.

For a compact compression flange, flexural resistance F_{nc} is

$$F_{nc(\text{FLB})} = R_b R_h F_{yc}$$

$$[6.10.1.10.2] \quad \text{For a negative moment section, } R_b \text{ is dependent on the web slenderness ratio, } \frac{2 \cdot D_c}{t_w}.$$

$[D6.3.1]$ For composite negative moment sections, D_c is based on the section consisting of the steel girder plus the longitudinal reinforcement.

$$D_c = y_{bc} - t_{fc} = 39.67 - 1.75 = 37.92 \text{ in}$$

$$\frac{2 \cdot D_c}{t_w} = \frac{2 \cdot (37.92)}{0.625} = 121.34$$

$[C6.10.1.10.2]$ For $F_y = 50$ ksi, $\lambda_{rw} = 137$. Therefore,

$$\frac{2 \cdot D_c}{t_w} < \lambda_{rw} \text{ and } R_b = 1.0.$$

The girder is homogeneous so $R_h = 1.0$

The flange local buckling resistance is:

$$F_{nc(\text{FLB})} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

$[6.10.8.2.3]$ Now, determine the lateral torsional buckling resistance.

At the flange butt splice, the unbraced length L_b is:

$$L_b = 22.5 \text{ ft} = 270 \text{ in}$$

This is to be compared with the compact bracing limit L_p .

[6.10.8.2.3-4]

$$L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$$

[6.10.8.2.3]

$$r_t = \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{D_c t_w}{3 b_{fc} t_{fc}}\right)}} = \frac{22}{\sqrt{12 \cdot \left(1 + \frac{37.92 \cdot 0.625}{3 \cdot 22 \cdot 1.75}\right)}} = 5.79 \text{ in}$$

$$\text{Then, } L_p = 1.0 \cdot (5.79) \cdot \sqrt{\frac{29,000}{50}} = 139.4 \text{ in} < 270 \text{ in}$$

Therefore, $L_b > L_p$, so check noncompact bracing limit L_r .

$$L_r = \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}}$$

$$F_{yr} = 0.7 \cdot F_y = 0.7 \cdot (50.0) = 35.0 \text{ ksi}$$

$$\text{Then, } L_r = \pi \cdot (5.79) \cdot \sqrt{\frac{29,000}{35.0}} = 523.6 \text{ in} > 270 \text{ in}$$

Therefore, $L_p < L_b < L_r$

$$\text{And, } F_{nc(LTB)} = C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}}\right) \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_{yc}$$

C_b , moment gradient modifier, can be calculated as follows:

f_2 is the compression stress at first brace point (diaphragm) off the pier:

$$f_2 = 27.3 \text{ ksi}$$

f_o is the compression stress at second diaphragm away from pier:

$$f_o = 1.25 \left(\frac{1106}{3149} + \frac{157}{3349} \right) (12) + 1.75 \left(\frac{3358}{3349} \right) (12) (0.793)$$

$$= 22.7 \text{ ksi}$$

f_{mid} is the compression stress at the point midway between the first and second diaphragm away from pier:

$$f_{mid} = 1.25 \left(\frac{2738}{3149} + \frac{463}{3349} \right) (12) + 1.75 \left(\frac{3694}{3349} \right) (12) (0.793)$$

$$= 33.5 \text{ ksi}$$

$$\frac{f_{mid}}{f_2} = \frac{33.5}{27.3} = 1.23 > 1 \quad \text{Therefore, } C_b = 1.0$$

$$F_{nc(LTB)} = 1.0 \cdot \left[1 - \left(1 - \frac{35}{1.0 \cdot 50} \right) \left(\frac{270 - 139.4}{523.6 - 139.4} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0$$

$$= 44.90 \text{ ksi}$$

The compression flange flexural resistance F_{nc} is the smaller of $F_{nc(FLB)}$ and $F_{nc(LTB)}$.

$$F_{nc} = F_{nc(LTB)} = 44.90 \text{ ksi}$$

Then, $\phi F_{nc} = 1.0 \cdot 44.90 = 44.90 \text{ ksi}$

$$f_{buc} + \frac{1}{3} \cdot f_\ell = 43.8 < 44.9 \text{ ksi} \quad \underline{OK}$$

[6.10.8.1.3]

Now consider the tension (top) flange, which is continuously braced by the deck in its final state.

Then

$$\phi F_{nt} = \phi \cdot (R_h \cdot F_{yt}) = 1.0 \cdot (1.0 \cdot 50.0) = 50.0 \text{ ksi} > 41.6 \text{ ksi} \quad \underline{OK}$$

**G.4 Design
Section 1 - Negative
Moment**

Design Section 1 was checked for negative moment following the same procedure used for Design Section 2 and found to be adequate.

**G.5 Design
Section 2 - Positive
Moment**

Design Section 2 was checked for positive moment following the same procedure used for Design Section 1 and found to be adequate.

**H. Flexure –
Investigate
Constructibility**

The capacity of the girders must be evaluated during construction, prior to composite action carrying the loads. For this example, the check consists of placing selfweight and formwork on both spans, while deck dead loads and a 20 psf construction live load is placed on one span.

**H.1 Design
Section 1 -Positive
Moment**

Load factors for this check are based on the values provided in LRFD Article 3.4.2, where 1.25 is used on dead loads and 1.5 is used on live loads. The maximum factored construction moment for Section 1 occurs at 0.4L:

$$\begin{aligned} M_{u\text{temp}} &= 1.25 \cdot DC_{\text{temp}} + 1.5 \cdot LL_{\text{temp}} \\ &= 1.25 \cdot (4864) + 1.5 \cdot (618) = 7007 \text{ k-ft} \end{aligned}$$

For top flange:

$$f_{\text{buc}} = \frac{M_{u\text{temp}}}{S_t} = \frac{(7007) \cdot 12}{1981} = 42.4 \text{ ksi}$$

For bottom flange:

$$f_{\text{but}} = \frac{M_{u\text{temp}}}{S_b} = \frac{(7007) \cdot (12)}{2318} = 36.3 \text{ ksi}$$

The flange lateral bending stress, f_ℓ , is assumed equal to 0.0 ksi.

The flow chart to evaluate the constructibility of the girder is shown in Appendix C6.4, Figure C6.4.1-1.

Begin by checking compression (top) flange. During construction, the top flange is discretely braced.

[6.10.3.2.1]

Check flange nominal yielding:

$$f_{\text{buc}} + f_{\ell} \leq \phi_f \cdot R_h \cdot F_{yc}$$

$$\phi_f = 1.0 \quad R_h = 1.0 \quad F_{yc} = 50.0 \text{ ksi}$$

$$\phi_f \cdot R_h \cdot F_{yc} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

$$f_{\text{buc}} + f_{\ell} = 42.4 + 0.0 = 42.4 \text{ ksi} < 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

Check flexural resistance:

$$f_{\text{buc}} + \frac{1}{3} \cdot f_{\ell} \leq \phi_f \cdot F_{nc}$$

$$\phi_f = 1.0$$

F_{nc} shall be taken as the smaller of the local buckling resistance and the lateral torsional buckling resistance.

[6.10.8.2.2]

Local buckling resistance:

$$\lambda_f = \frac{b_{fc}}{2 \cdot t_{fc}} = \frac{20}{2 \cdot (1.0)} = 10.0$$

[C6.10.8.2.2]

$$\lambda_{pf} = 9.2, \text{ for } F_{yc} = 50.0 \text{ ksi}$$

Then, $\lambda_f > \lambda_{pf}$ and compression flange is noncompact.

$$F_{nc(\text{FLB})} = \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

$$F_{yr} = \text{smaller of } \begin{cases} 0.7 \cdot F_{yc} = 0.7 \cdot (50.0) = 35.0 \text{ ksi} \\ F_{yw} = 50.0 \text{ ksi} \end{cases}$$

$$\text{But minimum } F_{yr} = 0.5 \cdot F_{yc} = 25.0 \text{ ksi}$$

Therefore, $F_{yr} = 35.0 \text{ ksi}$

$$\lambda_{rf} = 0.56 \cdot \sqrt{\frac{E}{F_{yr}}} = 0.56 \cdot \sqrt{\frac{29,000}{35}} = 16.1$$

When checking constructability, $R_b = 1.0$

The girder is homogeneous, so $R_h = 1.0$

$$F_{nc(FLB)} = \left[1 - \left(1 - \frac{35}{1.0 \cdot 50} \right) \left(\frac{10.0 - 9.2}{16.1 - 9.2} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0 = 48.3 \text{ ksi}$$

[6.10.8.2.3]

Lateral Torsional Buckling Resistance:

$$L_b = 26 \text{ ft} = 312 \text{ in}$$

$$L_p = 1.0 \cdot r_t \cdot \sqrt{\frac{E}{F_{yc}}}$$

$$D_c = y_t - t_{fc} = 38.96 - 1.0 = 37.96 \text{ in}$$

$$r_t = \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{D_c t_w}{3 b_{fc} t_{fc}} \right)}} = \frac{20}{\sqrt{12 \cdot \left(1 + \frac{37.96 \cdot 0.625}{3 \cdot 20 \cdot 1.0} \right)}} = 4.89 \text{ in}$$

$$L_p = 1.0 \cdot (4.89) \cdot \sqrt{\frac{29,000}{50.0}} = 117.8 \text{ in}$$

$$L_r = \pi \cdot r_t \cdot \sqrt{\frac{E}{F_{yr}}} = \pi \cdot (4.89) \cdot \sqrt{\frac{29,000}{35.0}} = 442.2 \text{ in}$$

Therefore, $L_p < L_b < L_r$

$$\text{Then } F_{nc(LTB)} = C_b \cdot \left[1 - \left(1 - \frac{F_{yr}}{R_h \cdot F_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_{yc}$$

First determine C_b :

f_2 is the compression stress at the third diaphragm away from the abutment:

$$f_2 = \frac{[(1.25)(4815) + 1.5(618)](12)}{1981} = 42.1 \text{ ksi}$$

f_0 is the compression stress at the second diaphragm away from the abutment:

$$f_0 = \frac{[(1.25)(4522) + (1.5)(566)] \cdot 12}{1981} = 39.4 \text{ ksi}$$

f_{mid} is the compression stress at the midway point between the second and third diaphragm away from the abutment:

$$f_{\text{mid}} = \frac{[(1.25)(4832) + (1.5)(612)] \cdot 12}{1981} = 42.1 \text{ ksi}$$

$$f_{\text{mid}} = f_2$$

Therefore, $C_b = 1.0$

$$\begin{aligned} F_{\text{nc(LTB)}} &= 1.0 \cdot \left[1 - \left(1 - \frac{35.0}{1.0 \cdot 50.0} \right) \left(\frac{312 - 117.8}{442.2 - 117.8} \right) \right] \cdot 1.0 \cdot 1.0 \cdot 50.0 \\ &= 41.0 \text{ ksi} \end{aligned}$$

Then, $F_{\text{nc}} = 41.0 \text{ ksi}$

$$\phi_f \cdot F_{\text{nc}} = 1 \cdot 41.0 = 41.0 \text{ ksi} < 42.4 \text{ ksi} \quad (3.4\% \text{ overstress})$$

The top flange compression stress is greater than 3% over the factored resistance. The 20 psf construction live load was placed over the entire first span for simplicity and to be conservative. Since this is an unlikely loading case and the overstress is only at 3.4%, by engineering judgement the flexural resistance is considered adequate.

Check web bend buckling. First, determine the nominal elastic web bend buckling resistance.

[6.10.1.9.1]

$$F_{\text{crw}} = \frac{0.9 \cdot E \cdot k}{\left(\frac{D}{t_w} \right)^2} \quad \text{not to exceed the smaller of } R_h F_{yc} \text{ and } F_{yw} / 0.7.$$

$$k = 9.0 \cdot \left(\frac{D}{D_c} \right)^2 = 9.0 \cdot \left(\frac{70}{37.96} \right)^2 = 30.6$$

Substituting values into Equation 6.10.1.9.1-1 results in

$$F_{\text{crw}} = \left[\frac{0.9 \cdot 29,000 \cdot 30.6}{\left(\frac{70}{0.625} \right)^2} \right] = 63.7 \text{ ksi}$$

$$R_h F_{yc} = 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

$$\frac{F_{yw}}{0.7} = \frac{50.0}{0.7} = 71.4 \text{ ksi}$$

Set limit at $F_{crw} = 50.0 \text{ ksi}$

[6.10.3.2.1]

$$\phi_f F_{crw} = 1.0 \cdot 50.0 = 50.0 \text{ ksi} > 42.4 \text{ ksi} \quad \underline{\text{OK}}$$

[6.10.3.2.2]

Now check tension (bottom) flange:

$$f_{bu} + f_\ell \leq \phi_f R_h F_{yt}$$

$$\phi_f = 1.0 \quad R_h = 1.0 \quad F_{yt} = 50.0 \text{ ksi}$$

$$\phi_f R_h F_{yt} = 1.0 \cdot 1.0 \cdot 50.0 = 50.0 \text{ ksi}$$

Assuming $f_\ell = 0.0 \text{ ksi}$ for constructibility check

$$f_{but} + f_\ell = 36.3 \text{ ksi} < 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

**H.2 Design
Sections 2 & 3 -
Negative Moment**

The sections for negative moment were checked following the same procedure and found to be adequate.

**I. Investigate the
Service Limit State
[6.10.4]**

Overload provisions control the amount of permanent deflection. Refer to the flow chart shown in LRFD Figure C6.4.2-1. The Service II load combination shall apply.

Flange Stress Limitations

Mn/DOT limits the maximum flange stress to F_y under the Strength Limit State. LRFD Article 6.10.4.2.2 limits the maximum flange stress to $0.95 F_y$ under the Service II load combination:

$$1.0 f_{DC} + 1.3 f_{LL} \leq 0.95 F_y$$

Dividing through by 0.95,

$$1.05 f_{DC} + 1.37 f_{LL} \leq F_y$$

Compare the load factors above with those of the Strength Limit State. By inspection, you can see that the flanges will pass this check due to the smaller load factors associated with the Service II load combination.

Bend Buckling Resistance

For the composite section in positive flexure,

[6.10.2.1.1]

$$\frac{D}{t_w} = \frac{70}{0.625} = 112 < 150 \quad \text{no checking of bend buckling required}$$

For the composite section in negative flexure, the compression flange stress due to the Service II Loads, f_c , shall satisfy LRFD Equation 6.10.4.2.2-4.

Compression stress of bottom flange at Section 3:

$$f_c = (1.0) \left(\frac{9231}{5437} + \frac{1660}{5631} \right) (12) + (1.3) \left(\frac{6905}{5631} \right) (12) (0.793) = 39.1 \text{ ksi}$$

Compression stress of bottom flange at Section 2:

$$f_c = (1.0) \left(\frac{4268}{3149} + \frac{749}{3349} \right) (12) + (1.3) \left(\frac{4041}{3349} \right) (12) (0.793) = 33.9 \text{ ksi}$$

[6.10.1.9.1]

$$F_{crw} = \frac{0.9 \cdot E \cdot k}{\left(\frac{D}{t_w} \right)^2}$$

For Section 3:

$$k = 9.0 \cdot \left(\frac{D}{D_c} \right)^2 = 9.0 \cdot \left(\frac{70}{36.47} \right)^2 = 33.2$$

Substituting values to arrive at the limiting stress results in

$$\left[\frac{0.9 \cdot 29,000 \cdot 33.2}{\left[\frac{70}{0.625} \right]^2} \right] = 69.1 \text{ ksi}$$

For Section 2:

$$k = 30.7 \text{ and } \frac{0.9 \cdot E \cdot k}{\left(\frac{D}{t_w}\right)^2} = 63.9 \text{ ksi}$$

The upper limit is capped by the smaller of

$$R_h F_{yc} = 50.0 \text{ and } \frac{F_{yw}}{0.7} = 71.4 \text{ ksi. So, } F_{crw} = 50.0 \text{ ksi.}$$

For Section 3:

$$f_c = 39.1 \text{ ksi} < F_{crw} = 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

For Section 2:

$$f_c = 33.9 \text{ ksi} < F_{crw} = 50.0 \text{ ksi} \quad \underline{\text{OK}}$$

The web bend-buckling resistance is adequate.

J. Investigate the Fatigue Limit State [6.10.5]

Although LRFD 6.6.1.2.3 states that only details with fatigue resistance Category C or lower resistance need to be evaluated during design, details that are classified as Category B' and above should be checked.

J.1 Fatigue Loading [3.6.1.4]

The HL-93 truck is used to generate the fatigue loads that are used to evaluate different components of a design. For fatigue, the HL-93 truck has a fixed rear axle spacing of 30 feet. In addition, a load factor of 0.75 is applied to calibrate the stresses to those observed in field studies. The dynamic load allowance for fatigue loading is 15%. Distribution for fatigue is equal to the one design lane loaded distribution, with the multiple presence factor removed (if approximate equations are used for one lane loaded).

[6.6.1.2.2]

$$\gamma \cdot (\Delta f) \leq (\Delta F)_n$$

$$\gamma = \text{load factor for fatigue} = 0.75$$

$$(\Delta f) = \text{live load stress range}$$

$$(\Delta F)_n = \text{nominal fatigue resistance}$$

J.2 Check Largest Stress Range Location

The unfactored fatigue moments in Table 6.9.8 are multiplied by the fatigue load factor (0.75) and the appropriate distribution factor to arrive at the design moment ranges for fatigue. In Table 6.9.14 the stresses at the positive flexure section are computed by dividing the design moment range by the composite (n) section modulus assuming the deck is effective for both positive and negative moment.

Designers should note that the fatigue distribution factor for the exterior girder is significantly larger (0.676 versus 0.398/0.426) than that of the interior girders.

**Table 6.9.14
Fatigue Range**

Girder Point	Fatigue			
	* Fatigue Moment Range Per Lane (kip-ft)	** Factored Design Moment Range (kip-ft)	Top Stress Range (kip)	Bottom Stress Range (kip)
0.0	0	0	0.00	0.00
0.1	1248	373	0.28	1.42
0.2	2136	638	0.48	2.44
0.3	2691	803	0.61	3.07
0.4	2978	889	0.67	3.40
0.5	3053	911	0.69	3.48
0.6	2959	883	0.67	3.38
0.691 ⁽¹⁾	2691	860	0.65	3.29
0.7	2659	850	0.61	2.57
0.8	2209	706	0.51	2.14
0.860 ⁽²⁾	1908	610	0.44	1.85
0.9	1759	562	0.37	1.05
1.0	1567	501	0.33	0.94

* Includes 15% Dynamic Load Allowance

** Girder Point 0.1 – 0.691: (Fatigue Moment Range) × 0.75 × 0.398

Girder Point 0.691 – 1.0: (Fatigue Moment Range) × 0.75 × 0.426

⁽¹⁾ Field Splice

⁽²⁾ Flange butt splice

J.3 Check Fatigue Details

[6.6.1.2]

[Table 6.6.1.2.3-1]

[Table 6.6.1.2.5-3]

For this example, the details that should be investigated for fatigue are: the welded flange butt splices, the web to flange welds, the toe of stiffener to web welds, the toe of stiffener to flange welds, and the shear stud to top flange welds. Fatigue at the bolted field splice should also be

investigated and will be considered later. Details subject to stress ranges less than $1/2$ the infinite life fatigue threshold are assumed to have infinite life. The $1/2$ factor accounts for the probability that some vehicles larger than the HL-93 fatigue truck will cross the bridge.

The worst case detail for fatigue is the shear stud to top flange weld which is a Category C detail. Category C details have a constant amplitude fatigue threshold $(\Delta F)_{TH}$ of 10.0 ksi. From Table 6.9.14 the largest flange stress range is 3.48 ksi. This value is below $1/2$ of the constant amplitude fatigue threshold (5.0 ksi). Therefore, all of the details have an infinite fatigue life.

J.4 Fatigue Requirements for Web

[6.10.5.3]

To control out-of-plane flexing of the web under repeated live loading the following constraints are placed on webs.

Interior panels of webs with transverse stiffeners, with or without longitudinal stiffeners, shall satisfy $V_u \leq V_{cr}$, where V_u is the maximum elastic shear stress in the web due to unfactored permanent load and factored fatigue load.

The live load used for this check is twice that presented in LRFD Table 3.4.1-1.

The computations for the shear buckling resistance of the web in shear is based on the following equation:

[6.10.9.3.3]

$$V_{cr} = CV_p$$

$$V_p = 0.58 \cdot F_{yw} (D)(t_w) = 0.58 \cdot (50.0) \cdot (70)(0.625) = 1268.8 \text{ kip}$$

C is the ratio of shear buckling stress to shear yield strength.

Since the transverse stiffener spacing exceeds $3D=17.5$ ft, the web is unstiffened.

$$\frac{D}{t_w} = \frac{70}{0.625} = 112$$

$$k = 5$$

[6.10.9.2]

$$1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 1.40 \cdot \sqrt{\frac{29,000 \cdot 5.0}{50.0}} = 75.4 < \frac{D}{t_w}$$

Then,

[Eqn. 6.10.9.3.2-6]

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = \frac{1.57}{(112)^2} \cdot \left(\frac{29,000 \cdot 5.0}{50.0}\right) = 0.363$$

$$V_{cr} = CV_p = 0.363 \cdot (1268.8) = 460.6 \text{ kip}$$

Table 6.9.15
Shear Fatigue

Girder Point	DC1 Shear (kips)	DC2 Shear (kips)	Fatigue (per lane, no impact)		* V_u (kips)
			Minimum Shear (ksi)	Maximum Shear (ksi)	
0.0	115	23		69	224
0.1	82	16		58	171
0.2	49	10		48	119
0.3	15	3		39	67
0.4	-18	-3	-30		-59
0.5	-51	-9	-38		-108
0.6	-85	-16	-47		-160
0.691 ⁽¹⁾	-116	-22	-54		-206
0.7	-119	-22	-56		-211
0.8	-154	-29	-64		-263
0.860 ⁽²⁾	-176	-33	-68		-294
0.9	-191	-35	-70		-314
1.0	-230	-42	-76		-367

* DC1 + DC2 + (Maximum Shear or Minimum Shear) × 0.725 × 0.75 × 2.0 × 1.15

⁽¹⁾ Field Splice

⁽²⁾ Flange butt splice

The shear stresses at all girder points are well below the 460.6 kip permitted. The web satisfies the shear fatigue checks.

K. Strength Limit State Shear Resistance [6.10.9]

Within the commentary to LRFD Article 6.10.9.1 a flow chart identifies the steps for the shear design of I-sections. A copy of the flow chart is provided below in Figure 6.9.8.

[6.10.9.1]

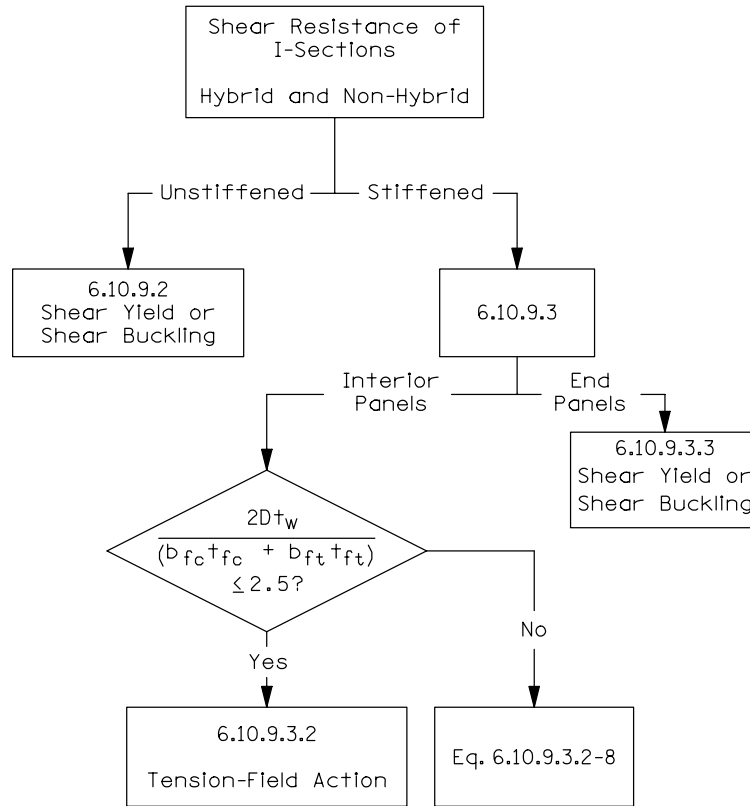


Figure 6.9.8

Determine the maximum shear capacity of the section with an unstiffened web and compare that to the required shear resistance.

[6.10.9.2]

$$V_n = C V_p$$

$$V_p = 0.58 \cdot F_{yw} \cdot (D)(t_w) = 0.58 \cdot (50.0) \cdot (70)(0.625) = 1268.8 \text{ kips}$$

C is dependent on the web depth to thickness ratio.

$$\frac{D}{t_w} = \frac{70}{0.625} = 112$$

[6.10.9.3.2]

The appropriate equation for C is selected based on how slender the web is:

for unstiffened web, $k = 5.0$

$$1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 1.40 \cdot \sqrt{\frac{29,000 \cdot 5.0}{50.0}} = 75.4 < \frac{D}{t_w}$$

Then,

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = \frac{1.57}{(112)^2} \cdot \left(\frac{29,000 \cdot 5.0}{50.0}\right) = 0.363$$

The capacity of the unstiffened web is:

$$V_n = 0.363 \cdot 1268.8 = 460.6 \text{ kips}$$

[6.5.4.2]

$$\phi_v = 1.0$$

$$V_r = \phi_v V_n = (1.0)(460.6) = 460.6 \text{ kips}$$

K.1 Pier Region

Assume the critical section for shear is at Girder Point 1.0. Based on Tables 6.9.3, 6.9.9, and 6.9.11, the factored shear force over the pier is:

$$V_{u(1.0)} = 1.25 \cdot (230 + 42) + 1.75 \cdot 166 \cdot 1.112 = 663 \text{ kips} > \phi V_n$$

The resistance of an unstiffened web is less than the demand of 663 kips; therefore, transverse stiffeners are required near the pier.

[6.10.9.1]

In order to qualify as a stiffened web, the maximum spacing for transverse stiffeners is three times the depth of the web:

$$3D = 3 \cdot (70) = 210 \text{ in} = 17.5 \text{ ft}$$

The diaphragms in the region of the pier are spaced at 22.5 feet. The diaphragm connection plates act as web stiffeners. Try adding a stiffener midway between the pier and the first diaphragm away from the pier.

Then $d_o = 11.25 \text{ ft} = 135 \text{ in}$

[6.10.9.3.2]

Compute k :

$$k = 5 + \frac{5}{\left[\frac{d_o}{D}\right]^2} = 5 + \frac{5}{\left[\frac{135}{70}\right]^2} = 6.3$$

$$1.40 \cdot \sqrt{\frac{E \cdot k}{F_{yw}}} = 1.40 \cdot \sqrt{\frac{29,000 \cdot 6.3}{50.0}} = 84.6 < 112$$

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = \frac{1.57}{\left(\frac{70}{0.625}\right)^2} \cdot \left(\frac{29,000 \cdot 6.3}{50.0}\right) = 0.457$$

For an interior panel at Girder Point 1.0,

$$\frac{2D \cdot t_w}{(b_{fc} \cdot t_{fc}) + (b_{ft} \cdot t_{ft})} = \frac{2(70)(0.625)}{(22)(3.25) + (20)(3.25)} = 0.64 < 2.5$$

Then,

$$V_n = V_p \cdot \left[C + \frac{0.87 \cdot (1 - C)}{\sqrt{1 + \left(\frac{d_o}{D}\right)^2}} \right]$$

$$= 1268.8 \cdot \left[0.457 + \frac{0.87 \cdot (1 - 0.457)}{\sqrt{1 + \left(\frac{135}{70}\right)^2}} \right] = 855.8 \text{ kips}$$

$$V_r = \phi_v V_n = (1.0) \cdot (855.8) = 855.8 \text{ kips} > 663 \text{ kips} \quad \underline{\text{OK}}$$

Check if an additional stiffener is required midway between the first and second diaphragm away from the pier by checking the shear at the first diaphragm (0.871 girder point).

$$V_{u(0.871)} = 1.25 \cdot (180 + 33) + 1.75 \cdot 144 \cdot 1.112 = 546.5 \text{ kips} > 460.6 \text{ kips}$$

Therefore an additional stiffener is needed.

Check if another stiffener is needed between the second and third diaphragm by checking the shear at the second diaphragm (0.742 girder point).

$$V_{u(0.742)} = 1.25 \cdot (134 + 25) + 1.75 \cdot (122) \cdot 1.112$$

$$= 436.2 \text{ kips} < 460.6 \text{ kips} \quad \underline{\text{OK}}$$

Therefore, stiffeners are not required between the second and third diaphragms away from the pier.

K.2 Abutment Region

From previous calculations, the capacity of the unstiffened web is
 $V_r = 460.6 \text{ kips}$

[6.10.9.2-1]

At Girder Point 0.0 the shear demand is:

$$V_{u(0.0)} = 1.25 \cdot (115 + 23) + 1.75 \cdot (137) \cdot 1.112 = 439.1 < 460.6 \text{ kips}$$

The web has adequate capacity at the abutment without stiffeners.

K.3 Transverse Stiffener Design

[6.10.11.1]

Ideally the size of the stiffener should be coordinated with the cross frame connection plates. Fabrication of the girder will be simplified if only one plate size and thickness is welded to the web at non-bearing locations. In addition, transverse stiffeners and diaphragm connection plates should be detailed with widths that are in $\frac{1}{4}$ inch increments. This provides the fabricator additional flexibility. They can either cut the stiffeners and connection plates out of large mill plate or utilize standard flat bar stock.

Transverse stiffeners are required near the pier. Mn/DOT Detail B411 (Stiffener Details) addresses the constraints placed on stiffeners in LRFD Article 6.10.11.1.1.

The dimensions of transverse stiffeners are required to fall within geometric constraints based on section depth, flange width, and projecting element thickness.

[6.10.11.1.2]

Begin with the projecting width constraint:

$$b_t \geq 2.0 + \frac{D}{30} = 2.0 + \frac{70}{30} = 4.33 \text{ in}$$

Try a single 8" x $\frac{1}{2}$ " stiffener.

$$b_t = 8 \text{ in} \qquad t_p = 0.50 \text{ in}$$

Check flange width constraint:

$$16.0 \cdot t_p = 16.0 \cdot 0.50 = 8.0 \text{ in} \quad \underline{\text{OK}}$$

$$0.25 \cdot b_f = 0.25 \cdot (22) = 5.5 \text{ in} < 8.0 \text{ in} \quad \underline{\text{OK}}$$

[6.10.11.1.3]

In addition to good aspect ratios, stiffeners must also have an adequate moment of inertia.

$$J = 2.5 \cdot \left(\frac{D}{d_o} \right)^2 - 2.0 = 2.5 \cdot \left(\frac{70}{135} \right)^2 - 2.0 = -1.33 < 0.5$$

Therefore, $J = 0.5$.

For a transverse stiffener spacing of 135 inches, the shear buckling resistance of the web V_{cr} is:

$$V_{cr} = CV_p = 0.457 \cdot 1268.8 = 579.8 \text{ kips} < 663 \text{ kips}$$

Because the factored shear force V_u (1.0) is greater than V_{cr} , the required stiffness I_t of the stiffeners shall satisfy:

$$\text{Min. } I_t \geq \frac{D^4 \rho_t^{1.3}}{40} \cdot \left(\frac{F_{yw}}{E} \right)^{1.5}$$

$$\rho_t = \text{larger of } \begin{cases} F_{yw}/F_{crs} \\ 1.0 \end{cases}$$

$$F_{crs} = \frac{0.31 \cdot E}{\left(\frac{b_t}{t_p} \right)^2} \leq F_{ys} = \frac{0.31 \cdot 29,000}{\left(\frac{8}{0.5} \right)^2} = 35.1 \text{ ksi} < F_{ys} = 50 \text{ ksi}$$

$$\frac{F_{yw}}{F_{crs}} = \frac{50}{35.1} = 1.42$$

$$\rho_t = 1.42$$

$$\text{Min. } I_t \geq \frac{(70.0)^4 \cdot (1.42)^{1.3}}{40.0} \cdot \left(\frac{50.0}{29,000} \right)^{1.5} = 67.8 \text{ in}^4$$

The stiffener moment of inertia taken about the edge in contact with the web is:

$$\text{Actual } I_t = \frac{1}{3} \cdot 0.5 \cdot 8^3 = 85.3 > 67.8 \text{ in}^4 \quad \underline{\text{OK}}$$

**K.4 Bearing
Stiffener Design
[6.10.11.2.1]**

For welded plate girders, bearing stiffeners are needed at both the abutments and piers.

Abutment Bearing

The reaction to be carried by the bearing stiffeners is:

$$R_u = 1.25 \cdot (115 + 23) + 1.75 \cdot (137) \cdot 1.112 = 439 \text{ kips}$$

Similar to transverse stiffeners, there are constraints on the geometry of bearing stiffeners.

The bearing stiffeners should extend close to the outside edges of the narrower flange, which is 20 inches in width. Try a 1" x 9" wide bearing stiffener on each side of the web.

Begin by checking the projecting width.

[6.10.11.2.2]

$$0.48 \cdot t_p \cdot \sqrt{\frac{E}{F_{ys}}} = 0.48 \cdot (1.0) \cdot \sqrt{\frac{29,000}{50}} = 11.56 \text{ in} > 9.00 \text{ in} \quad \underline{\text{OK}}$$

The bearing resistance check is based on the net area of steel in contact with the flange. Assume a 1¹/₂ inch cope at the bottom of the stiffener in accordance with Mn/DOT Detail B411.

[6.10.11.2.3]

$$(R_{sb})_n = 1.4 \cdot A_{pn} \cdot F_{ys} = 1.4 \cdot [1.00 \cdot (9.0 - 1.5) \cdot 2] \cdot (50.0) = 1050 \text{ kips}$$

$$\phi_b \cdot (R_{sb}) = 1.0 \cdot (1050) = 1050 \text{ kips} > 439 \text{ kips} \quad \underline{\text{OK}}$$

[6.10.11.2.4]

Now check the axial resistance of the bearing stiffeners.

[6.10.11.2.4b]

The stiffeners will act like a column while supporting the bearing reaction. The effective section consists of the stiffeners, plus $9t_w$ (thickness of the girder web) on each side of the stiffeners (see Figure 6.9.9).

The area for this column is:

$$A = 1.0 \cdot 9.0 \cdot 2 + 11.25 \cdot 0.625 = 25.03 \text{ in}^2$$

The moment of inertia about the girder web is:

$$I = \frac{1}{12} \cdot (11.25 - 1.0) \cdot 0.625^3 + \frac{1}{12} \cdot 1.0 \cdot 18.625^3 = 538.6 \text{ in}^4$$

The radius of gyration is:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{538.6}{25.03}} = 4.64 \text{ in}$$

Check the width/thickness limits of Article 6.9.4.2

[6.9.4.2]

$$k \cdot \sqrt{\frac{E}{F_y}} = 0.45 \cdot \sqrt{\frac{29,000}{50.0}} = 10.84$$

$$\frac{b}{t} = \frac{9.0}{1.0} = 9.0 < 10.84 \quad \text{OK}$$

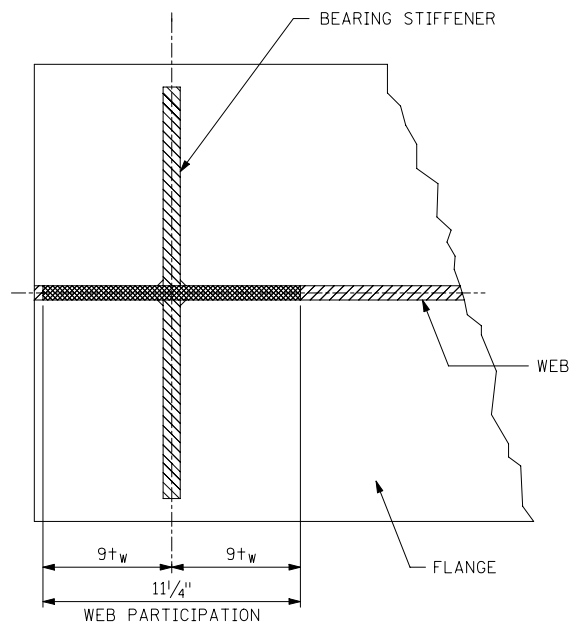


Figure 6.9.9
"Column" for Bearing Stiffener at Abutment

[6.9.3]

Check slenderness ratio:

[6.10.11.2.4a]The effective length ($K\ell$) of the column is $0.75D = 0.75 \cdot (70) = 52.5$ in

$$\frac{K\ell}{r} = \frac{52.5}{4.64} = 11.3 < 120 \quad \underline{\text{OK}}$$

[6.9.4.1]

Determine factored axial resistance:

$$\lambda = \left(\frac{K\ell}{r \cdot \pi} \right)^2 \cdot \frac{F_{ys}}{E} = \left(\frac{52.5}{4.64 \cdot \pi} \right)^2 \cdot \frac{50.0}{29,000} = 0.02$$

Since $\lambda < 2.25$,

$$P_n = 0.66^\lambda \cdot F_y \cdot A_s = 0.66^{0.02} \cdot (50.0) \cdot (25.03) = 1241 \text{ kips}$$

[6.9.2.1]

$$P_r = \phi_c P_n = (0.90) \cdot (1241) = 1117 \text{ kips} > 439 \text{ kips} \quad \underline{\text{OK}}$$

Therefore, use a pair of 1" x 9" bearing stiffeners at the abutments. Using the same design procedure, a pair of 1¹/₂" x 9" bearing stiffeners were found adequate to carry the factored pier reaction of 1341 kips.

**K.5 Shear
Resistance During
Construction
[6.10.3.3]**

The web is to be investigated for the sum of factored permanent loads and factored construction loads applied to the non-composite section during construction. The web shall satisfy $V_u < \phi_v V_{cr}$. The normal shear resistance for this check is limited to the shear yielding or shear buckling resistance per Article 6.10.9.3.3. Using the same procedure used above, calculations show that the web has adequate capacity during construction.

**L. Design Shear
Connectors
[6.10.10]**

Shear connectors are to be placed along the full length of the girder, including negative moment regions, because the girder is designed as composite for negative moment.

Shear connectors are designed to satisfy fatigue constraints after which a strength check is performed. Assume that ⁷/₈ inch diameter shear connectors will be used.

The minimum transverse spacing for connectors is 4.0 stud diameters. For ⁷/₈ inch diameter studs, this translates into a minimum spacing of

3¹/₂ inches. The minimum clear distance from a stud to the edge of a flange is 1.0 inch. With a 20 inch top flange width, the maximum number of stud spaces placed in a line across the flange is:

$$\frac{20 - 2 \cdot (1) - 0.875}{3.5} = 4.9 \text{ spaces}$$

Five studs across the flange is permissible, but use four shear studs at each location.

The studs must extend a minimum of 2 inches into the deck and have a minimum of 3 inches of cover. At midspan, the amount of concrete stool is 1.75 inches. At the pier, the amount of concrete stool is 2 inches. Choose a stud height of 5 inches.

**L.1 Fatigue Limit
State**

[6.10.10.1.2]

[6.10.10.2]

The pitch P (longitudinal spacing) of each set of studs shall satisfy:

$$\text{Max } p \leq \frac{n \cdot Z_r}{V_{sr}}$$

The shear fatigue resistance of an individual connector is based on the number of fatigue cycles anticipated: $Z_r = \alpha d^2 \geq \frac{5.5}{2} d^2$.

where $\alpha = 34.5 - 4.28 \cdot \log N$

The lower bound corresponds to the resistance for a stud subjected to approximately 26,200,000 cycles.

For sections near pier:

$$\text{ADTT} = 2000 \quad \text{Design Life} = 75 \text{ years}$$

From LRFD Table 6.6.12.5-2, use 1.5 cycles per truck passage.

Then

$$N = 1.5 \cdot 2000 \cdot 365 \cdot 75 = 82,125,000 \text{ cycles} > 26,200,000 \text{ cycles}$$

The lower bound governs:

$$Z_r = \frac{5.5}{2} \cdot d^2 = \frac{5.5}{2} \cdot 0.875^2 = 2.11 \text{ kips}$$

For sections away from pier:

From Table 6.6.12.5-2, 1.0 cycles per truck passage shall be used.

$$N = 1.0 \cdot 2000 \cdot 365 \cdot 75 = 54,750,000 \text{ cycles} > 26,200,000 \text{ cycles}$$

The lower bound governs:

$$Z_r = 2.11 \text{ kips}$$

V_{sr} is to be computed as follows:

$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2}$$

For a straight span, F_{fat} may be taken as 0.

$$\text{Then } V_{sr} = V_{fat} = \frac{VQ}{I}$$

The inertia values are taken from Table 6.9.2:

For the positive moment region, $I = 189,316 \text{ in}^4$.

For the negative moment region, $I = 132,855 \text{ in}^4$ (value for the smaller negative moment section).

Now compute the "Q" values. For the positive moment region:

$$Q = \frac{1}{n} \cdot b_{eff} \cdot t_s \cdot \left(y_{tc} + t_{cstool} + \frac{t_s}{2} \right) = \frac{1}{8} \cdot 118 \cdot 9 \cdot \left(11.92 + 1.75 + \frac{9}{2} \right) = 2412 \text{ in}^3$$

For the negative moment region, only the area of steel in the concrete deck is considered.

$$d_{r_{avg}} = \frac{(7.80)(5.5) + (3.93)(1.88)}{(7.80 + 3.93)} = 4.29 \text{ inches from bottom of deck}$$

$$Q = (A_{rt} + A_{rb}) \left(y_{tc} + t_{cstool} + d_{r_{avg}} \right) = (7.80 + 3.93) (33.83 + 1.75 + 4.29) \\ = 468 \text{ in}^3$$

Knowing n , Z_r , I , and Q leaves the pitch to be a function of the fatigue shear force range V_f . For the positive moment region

$$\text{Max } p \leq \frac{n \cdot Z_r \cdot I}{V_f \cdot Q} = \frac{4 \cdot 2.11 \cdot 189,316}{V_f \cdot 2412} = \frac{662}{V_f}$$

For the negative moment region the required pitch is

$$\text{Max } p \leq \frac{n \cdot Z_r \cdot I}{V_f \cdot Q} = \frac{4 \cdot 2.11 \cdot 132,855}{V_f \cdot 468} = \frac{2396}{V_f}$$

Table 6.9.16
Shear Connector Spacing For Fatigue

Girder Point	* Fatigue Shear LLDF	** Factored Fatigue Shear Range V_f (kips)	Max p (Positive) (in)	Max p (Negative) (in)	*** Max p Limit (in)
0.0	0.725	42	16		24
0.1	0.725	36	18		24
0.2	0.725	31	21		24
0.3	0.725	31	22		24
0.4	0.725	31	21		24
0.5	0.725	33	20		24
0.6	0.725	34	20		24
0.7	0.725	35	19		24
0.8	0.725	37		64	24
0.9	0.725	39		61	24
1.0	0.725	41		58	24

* See Table 6.9.3

** $0.75 \cdot \text{LLDF} \cdot \text{Fatigue Truck Shear Range}$ from Table 6.9.11

*** Per LRFD 6.10.10.1.2, the maximum limit for spacing of shear connectors is 24 inches and minimum limit is $6_d = 6 \cdot 0.875 = 5.25$ in

By inspection, the negative moment fatigue requirements are satisfied if 2 studs are placed on a 20 inch spacing.

L.2 Strength Limit State
[6.10.10.4]

In addition to fatigue, adequate studs are needed to ensure that the cross sections can generate the flexural resistance computed earlier.

The factored shear resistance of a single shear connector Q_r , shall be taken as:

$$Q_r = \phi_{sc} Q_n$$

$$\phi_{sc} = \text{resistance factor} = 0.85$$

[6.5.4.2]

[6.10.10.4.3]

The nominal resistance of a shear connector Q_n is:

$$Q_n = 0.5 \cdot A_{sc} \cdot \sqrt{f'_c \cdot E_c} = 0.5 \cdot 0.60 \cdot \sqrt{4 \cdot 3644} = 36.2 \text{ kips}$$

$$\text{But, } Q_n \leq A_{sc} \cdot F_u = 0.60 \cdot 60 = 36 \text{ kips}$$

$$Q_r = \phi Q_n = 0.85 \cdot 36 = 30.6 \text{ kips}$$

The minimum number of shear connectors, n , shall be taken as:

[6.10.10.4.1]
$$n = \frac{P}{Q_r}$$

where, P = total nominal shear force (Article 6.10.10.4.2).

[6.10.10.4.2] For the region between the point of maximum positive LL + I moment and the abutment:

$$P = \sqrt{P_p^2 + F_p^2}$$

F_p may taken as 0.0 for straight spans.

P_p is taken as the lesser of the capacity of the deck or the capacity of the steel section.

The capacity of the deck is:

$$P_{1P} = 0.85 \cdot f'_c \cdot b_s \cdot t_s = 0.85 \cdot 4 \cdot 118 \cdot 9 = 3611 \text{ kips}$$

The capacity of the steel section is:

$$\begin{aligned} P_{2P} &= F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc} \\ &= 50 \cdot (70 \cdot 0.625 + 22 \cdot 1.25 + 20 \cdot 1.0) = 4563 \text{ kips} \end{aligned}$$

So, $P_p = P_{1P} = 3611 \text{ kips}$

$$P = P_p = 3611 \text{ kip}$$

$$n = \frac{P}{Q_r} = \frac{3611}{30.6} = 118 \text{ studs}$$

For the region between the point of maximum positive LL + I moment and the centerline of an adjacent interior support:

$$P = \sqrt{P_T^2 + F_T^2}$$

F_T may be taken as 0.0 for straight spans.

$$P_T = P_p + P_n$$

P_n is total longitudinal shear force in the concrete deck over an interior support taken as the lesser of either:

$$\begin{aligned} P_{1n} &= F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc} \\ &= 50 \cdot (70 \cdot 0.625 + 20 \cdot 3.25 + 22 \cdot 3.25) = 9013 \text{ kips} \end{aligned}$$

or

$$P_{2n} = 0.45 \cdot f'_c \cdot b_s \cdot t_s = 0.45 \cdot 4.0 \cdot 118 \cdot 9.0 = 1912 \text{ kips}$$

So, $P_n = P_{2n} = 1912 \text{ kips}$

$$P_T = P_n + P_p = 1912 + 3611 = 5523 \text{ kips}$$

$$P = P_T = 5523 \text{ kips}$$

$$n = \frac{P}{Q_r} = \frac{5523}{30.6} = 180 \text{ studs}$$

The final details for the shear studs need to satisfy the constraints of both the fatigue design and the strength design. After reviewing the constraints, the layout provided in Figure 6.9.16 was chosen.

M. Investigate the Field Splice Design [6.13]

Several items need to be considered when locating and designing field splices for steel girders. Typically, splices are located near inflection points to minimize the flexural resistance required of the connection. In addition, designers need to ensure that adequate clearance is provided to transverse stiffeners, cross frame connection plates, etc.

As a general rule, designers should limit the number of plate thicknesses used in a splice. The splice used for this example has four plate thicknesses ($\frac{3}{8}$ ", $\frac{1}{2}$ ", $\frac{5}{8}$ ", and $\frac{3}{4}$ ") used for the splice and fill plates.

The number of limit states and loading conditions to consider in the design of a splice is significant. Construction, Service II (permanent deflection), Fatigue, and Strength limit states should all be evaluated. In most cases, the Strength limit state will dictate the plate sizes and the

number of bolts. The bolted connections used in the splice are Category B details.

Typically, three splice plates are used for each flange and two splice plates are used for the web. This permits all of the bolts to function in double shear and minimizes the number of bolts required.

As the size of splice plates are considered, it is prudent to look at the change in plate sizes on both sides of the splice. The thickness of fill plates can be determined prior to any design of the connection. For this example, the top flange on the left is 1" x 20" and on the right is 1³/₄ x 20". The fill plate for the top flange splice will have a thickness of ³/₄". The web on both sides of the splice is ⁵/₈". No fill plate will be necessary for the web. The bottom flange on the left is 1¹/₄" x 22". The bottom flange on the right is 1³/₄" x 22". A ¹/₂" fill plate will be required for the bottom flange splice. Using splice plates with a ³/₄" or ¹/₂" thickness will minimize the number of plate thicknesses required for the splice.

The splice will be designed as a slip-critical connection. The bolted connections will be proportioned:

- 1) to provide shear and bearing resistance under the governing strength limit state
- 2) to prevent slip at the Service II limit state
- 3) to have adequate fatigue resistance

The resistance of the bolts will be designed based on threads excluded from the shear plane for plates which are ³/₈ inch thick or greater.

The loads at the location of the splice are shown in Table 6.9.17.

Table 6.9.17
Loads at Girder Point 0.69 (Unfactored)

Component	Moment (k-ft)	Shear (k)
DC1	18	-116
DC2	54	-22
Pos. M LL + I	2351	22
Neg. M LL + I	-2469	-126
Pos. M DC _{CONST}	2416	-98
Neg. M DC _{CONST}	-2369	49
Pos. M LL _{CONST}	369	-11
Neg. M LL _{CONST}	-371	3
Fatigue LL + I Range	2691	64

The loads are applied to the non-composite, short-term composite (n), and long-term composite (3n) cross sections. Table 6.9.18 shows the section properties used for the splice design.

To arrive at design stresses for the splice plates, the loads are applied to the appropriate section. The stresses from the load components are then factored to arrive at design stresses. Table 6.9.19 lists the unfactored component stresses and the factored design stresses for the flanges and the web. Flange splices are based on mid-flange stresses. Web splices can conservatively be based on mid-flange stresses or can use the stresses at the top and bottom of the web. The strength of the splice is based on the capacity of the smaller girder framing into the connection. For this example, the positive moment section is the smaller capacity member.

Table 6.9.18
Section Properties for Splice Design

Parameter	Design Section 1			
	Non-composite	Positive Moment		Neg. Moment
		Long-Term Composite (3 · n)	Short-Term Composite (n)	
Moment of Inertia (in ⁴)	77,179	139,281	189,316	98,220
y_t	38.96	23.93	11.92	33.83
y_b	33.29	48.32	60.33	38.42
Top Flange Thickness (in)	1	1	1	1
Bottom Flange Thickness (in)	1.25	1.25	1.25	1.25
S_{mid} top flange	2007	5945	16,578	2947
S_{top} web	2033	6074	17,337	2992
S_{bottom} web	2409	2959	3204	2642
S_{mid} bottom flange	2363	2920	3171	2599

Table 6.9.19
Flexural Stress Components at Splice

Loading	Stress at Mid-depth of ⁽¹⁾ Top Flange (ksi)	Stress at Mid-depth of ⁽¹⁾ Bottom Flange (ksi)
DC1	-0.11	0.09
DC2	-0.11	0.22
Pos. M (LL + I)	-1.70	8.90
Neg. M (LL + I)	10.05	-11.40
Pos. M DC _{const}	-14.45	12.27
Neg. M DC _{const}	14.16	-12.03
Pos. M LL _{const}	-2.21	1.87
Neg. M LL _{const}	2.22	-1.88
Pos. M Strength I	-3.25	15.96
Neg. M Strength I	17.39	-19.67
Pos. M Service II	-2.43	11.88
Neg. M Service II	12.85	-14.51
Pos. M Service II Const.	-17.32	14.70
Neg. M Service II Const.	17.05	-14.47
Fatigue LL + I Range ⁽²⁾	0.65	3.29

⁽¹⁾ Positive number denotes tension stress.

⁽²⁾ Factored stress range.

M.1 Controlling Flange ***[C6.13.6.1.4c]***

At the strength limit state, the controlling flange is defined as the flange with the maximum ratio of factored flexure stress to factored resistance. Table 6.9.19 indicates that the bottom flange is the controlling flange for both positive and negative live load of the Strength I Limit State. The splice must be capable of resisting both positive and negative live load moment conditions.

Bottom Flange Splice Plates

When the combined area of the inner splice plates is within 10 percent of the area of the outer splice plate, both the inner and outer splice plate may be designed for one-half the flange design force. In addition, the area of the outer plate and the sum area of the inner plates each need to have a cross sectional area which is approximately half that of the flange. Try an outer splice plate that is $\frac{5}{8}$ " x 22". Try inner splice plates that are $\frac{3}{4}$ " x 10".

Gross area of inner splice plates:

$$A_{g_in} = 2 \cdot 0.75 \cdot 10 = 15.00 \text{ in}^2$$

Gross area of outer splice plate:

$$A_{g_out} = 0.625 \cdot 22 = 13.75 \text{ in}^2$$

The difference in area is 8%. Therefore equal distribution of the flange force to the splice plates can be assumed. Note that if the areas of the inner and outer splice plates had differed by more than 10%, the splice plate design force would be calculated by multiplying the flange design force by the ratio of the area of the splice plate under consideration to the total area of the splice plates.

[6.13.6.1.4c]

Load Case I - Positive Live Load for Strength I Limit State

Bottom Flange is in tension.

The flange splice shall be designed to provide a minimum design resistance equal to the greater of:

$$F_{cf} = \frac{\left[\frac{f_{cf}}{R_h} + \alpha \cdot \phi_f \cdot F_{yf} \right]}{2}$$

or

$$F_{cf} = 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf}$$

From Table 6.9.19

$$f_{cf} = 15.96 \text{ ksi}$$

$$F_{cf} = \frac{\left[\frac{15.96}{1} + 1 \cdot 1 \cdot 50 \right]}{2} = 33.0 \text{ ksi}$$

or

$$F_{cf} = 0.75 \cdot 1 \cdot 1 \cdot 50 = 37.50 \text{ ksi} \quad \underline{\text{GOVERNS}}$$

Design force, $T_{tcfdes} = F_{cf} \cdot A_e$

Bottom flange is a tension flange under positive live load moment.

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}} \right) A_n$$

where,

$$\phi_u = 0.8$$

$$F_u = 70.0 \text{ ksi}$$

$$\phi_y = 0.95$$

$$F_y = 50 \text{ ksi}$$

7/8" diameter bolts will be used for the splice design. For A_n calculation, assume 6 bolts per row with a 1" diameter.

$$A_n = (1.25)(22 - 6 \cdot 1) = 20.00 \text{ in}^2$$

$$A_e = \frac{(0.8)(70)}{(0.95)(50)} (20.00) = 23.58 \text{ in}^2 < A_g = 27.5 \text{ in}^2$$

$$T_{\text{tcfdes}} = (37.50) (23.58) = 884.3 \text{ kips}$$

Maximum Tension Design force for inner plates and outer plate:

$$T_{\text{in_tcfdes}} = T_{\text{out_tcfdes}} = \frac{884.3}{2} = 442.2 \text{ kips}$$

[6.8.2.2]

The factored tensile resistance, P_r , shall be taken as the lesser of the following:

Yield on the gross section: $P_{r1} = \phi_y F_y A_g$

[6.13.5.2]

Fracture on the net section: $P_{r2} = \phi_u F_u A_n U$ where $A_n < 0.85 A_g$ and $U = 1$

For Outer Splice Plate:

$$A_g = (22) (0.625) = 13.75 \text{ in}^2$$

Assuming 6 holes per row with a 1" diameter:

$$A_n = [22 - (6) (1)] (0.625) = 10.00 \text{ in}^2 < 0.85 \cdot A_g = 11.69 \text{ in}^2$$

$$P_{r1} = \phi_y \cdot F_y \cdot A_g = (0.95) (50) (13.75) = 653.1 \text{ kips}$$

$$P_{r2} = \phi_u \cdot F_u \cdot A_n \cdot U = (0.8) (70) (10.00) (1) = 560.0 \text{ kips} \quad \underline{\text{GOVERNS}}$$

$$P_r = 560.0 \text{ kips} > 442.2 \text{ kips} \quad \underline{\text{OK}}$$

For Inner Splice Plates:

$$A_g = (2) (10) (0.75) = 15.00 \text{ in}^2$$

$$A_n = (2) [10 - (3) (1)] (0.75) = 10.50 \text{ in}^2 < 0.85 \cdot A_g = 12.75 \text{ in}^2$$

$$P_{r1} = \phi_y \cdot F_y \cdot A_g = (0.95) (50) (15) = 712.5 \text{ kips}$$

$$P_{r2} = \phi_u \cdot F_u \cdot A_n \cdot U = (0.8) (70) (10.50) (1) = 588.0 \text{ kips} \quad \underline{\text{GOVERNS}}$$

$$P_r = 588.0 \text{ kips} > 442.2 \text{ kips} \quad \underline{\text{OK}}$$

[6.13.6.1.4c]

Load Case 2 - Negative Live Load at Strength I Limit State

Bottom Flange is in compression.

The flange splice shall be designed to provide a minimum design resistance equal to the greater of:

$$F_{cf} = \frac{\left[\frac{|f_{cf}|}{R_n} + \alpha \phi_f F_{yf} \right]}{2}$$

or

$$F_{cf} = 0.75 \cdot \alpha \cdot \phi_f \cdot F_{yf}$$

From Table 6.9.19, $f_{cf} = -19.67 \text{ ksi}$

$$F_{cf} = \frac{\left[\frac{-19.67}{1.0} + (1.0) (1.0) (50.0) \right]}{2} = 34.8 \text{ ksi}$$

or

$$F_{cf} = (0.75) (1.0) (1.0) (50.0) = 37.5 \text{ ksi} \quad \underline{\text{GOVERNS}}$$

Design force: $T_{cfdes} = F_{cf} \cdot A_e$

$$A_e = A_g \text{ for compression flange}$$

$$T_{cfdes} = (37.5) (22) (1.25) = 1031.3 \text{ kips}$$

Since the combined area of the inner splice plates is within 10% of the area of the outer splice plate, both the inner and outer splice plate can be designed for one-half the flange design force.

Maximum Compression design force for inner plate and outer plate is taken as:

$$T_{in_ccfdes} = T_{out_ccfdes} = \frac{1031.3}{2} = 515.7 \text{ kips}$$

The factored resistance of splice plates subjected to compression, R_r , shall be taken as:

$$R_r = \phi_c F_y A_s$$

Where:

A_s = gross area of the splice plate

$$\phi_c = 0.9$$

For Outer Splice Plate:

$$R_r = (0.9) (50) (22) (0.625) = 618.8 \text{ kips} > 515.7 \text{ kips} \quad \underline{OK}$$

For Inner Splice Plate:

$$R_r = (0.9) (50) (10.0) (0.75) (2) = 675.0 \text{ kips} > 515.7 \text{ kips} \quad \underline{OK}$$

Load Case 3 - Fatigue

The fatigue detail category for a bolted connection is Category B. The splice is to be designed for infinite fatigue life and the Category B constant amplitude fatigue threshold $(\Delta F)_{TH}$ is 16 ksi.

$$\text{Then } \Delta F_n = 0.5 \cdot (\Delta F)_{TH} = 0.5 \cdot 16 = 8.0 \text{ ksi}$$

$$\text{Actual range } \gamma \cdot (\Delta f) = 3.29 \text{ ksi} < 8.0 \text{ ksi} \quad \underline{OK}$$

Bolt Shear Resistance

Now compute the resistance of a $7/8$ " diameter A325 bolt and determine the number of bolts for the bottom flange splice plate. The filler plate for the bottom flange is $1/2$ inch. Consequently, the fillers need to be extended or the capacity of the bolts reduced. For this example, the capacity of the bolts will be reduced (using LRFD Equation 6.13.6.1.5-1). Use a filler plate that is as wide as the flange. A_p is the smaller of:

$$A_p = (1.25) (22) = 27.5 \text{ in}^2 \quad \underline{GOVERNS}$$

[6.13.6.1.5]

or

$$A_p = (2)(10)(0.75) + (22)(0.625) = 28.75 \text{ in}^2$$

Fill plate reduction factor:

$$\gamma = \frac{A_r}{A_p} = \frac{(22) \cdot (0.5)}{28.75} = 0.40$$

$$R = \left[\frac{(1 + \gamma)}{(1 + 2 \cdot \gamma)} \right] = \left[\frac{(1 + 0.40)}{(1 + 2 \cdot 0.40)} \right] = 0.78$$

[6.13.2.7]

The shear resistance of a $\frac{7}{8}$ " diameter A325 bolt without threads in the shear plane with reduction of a filler is:

$$R_n = 0.48 \cdot A_b \cdot F_{ub} \cdot N_s \cdot R = 0.48 \cdot 0.601 \cdot 120 \cdot 2 \cdot 0.78 = 54.0 \text{ kips}$$

$$\phi_s R_n = 0.80 \cdot 54.0 = 43.2 \text{ kips}$$

The maximum design force for Strength I, $T_{ccfdes} = 1031.3$ kips

The number of bolts, N, required on the fill plate side of the connection is:

$$N = \frac{T_{ccfdes}}{\phi_s \cdot R_n} = \frac{1031.3}{43.2} = 23.9 \text{ bolts}$$

Use 4 rows of 6 bolts on each side of the splice.

[6.13.2.9]

Bolt Bearing Resistance

Check the bearing on the smaller flange plate (element carrying the double shear load).

$$\text{Average design force of each bolt} = \frac{1031.3}{24} = 43.0 \text{ kips}$$

Bolts will be spaced at 3 inches with an edge distance of 1.5 inches.
Clear distance between holes = $3 \cdot 1 = 2.0$ inches

Clear end distance = $1.5 - 0.5 = 1.0 \text{ in} < 2.0d$

$$R_n = 1.2 \cdot L_c \cdot t \cdot F_u = 1.2 \cdot \left(1.5 - \frac{1.0}{2} \right) \cdot 1.25 \cdot 70 = 105.0 \text{ kips}$$

$$R_r = \phi_{bb} \cdot R_n = 0.8 \cdot 105.0 = 84.0 \text{ kips} > 43.0 \text{ kips} \quad \underline{\text{OK}}$$

[6.13.2.8]**Bolt Slip Resistance**

Slip resistance of bolts in a slip-critical connection shall be taken as:

$$R_n = K_h K_s N_s P_t$$

where:

$$K_h = 1.0 \quad K_s = 0.5 \quad N_s = 2 \quad P_t = 39.0 \text{ kips}$$

$$R_n = 1.0 \cdot 0.5 \cdot 2 \cdot 39.0 = 39.0 \text{ kips/bolt}$$

[6.13.2.2]

$$R_r = R_n = 39.0 \text{ kips/bolt}$$

LRFD 6.13.6.1.4a requires that connections be proportioned to prevent slip during construction as well as under service loads.

Based on Table 6.9.19, maximum design force on gross section of Service II load combination shall be taken as:

$$F_{II} = (14.70) (22) (1.25) = 404.3 \text{ kips}$$

Average design force of each bolt:

$$= \frac{404.3}{24} = 16.8 \text{ kips} < 39.0 \text{ kips} \quad \underline{\text{OK}}$$

[6.13.4]**Block Shear Rupture Resistance**

All tension connections must be investigated to ensure that adequate connection material is provided to develop the factored resistance of the connection.

Check block shear on a transverse section through the smaller flange plate with the design force for the flange.

Possible block shear failure mode 1 on the inner and outer splice plates is shown below in Figure 6.9.10.

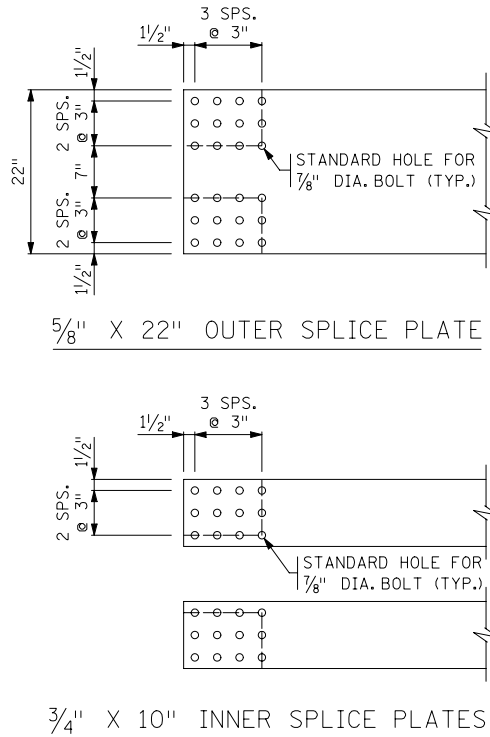


Figure 6.9.10

Block Shear Failure Mode 1 – Bottom Flange Splice Plates

Check Outer Splice Plate

A_{tn} is the net area along the planes resisting the tensile stress:

$$A_{tn} = 2 \cdot [1.5 + 2 \cdot 3 - (2.5)(1.00)] \cdot 0.625 = 6.25 \text{ in}^2$$

A_{vn} is the net area along the planes resisting the shear stress:

$$A_{vn} = 2 \cdot [1.5 + 3 \cdot 3 - 3.5(1.00)] \cdot 0.625 = 8.75 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{6.25}{8.75} = 0.71 > 0.58$$

$$R_r = \phi_{bs} \cdot (0.58 \cdot F_y A_{vg} + F_u A_{tn})$$

where:

$$\phi_{bs} = 0.8$$

A_{vg} is the gross area along the plane resisting shear stress:

$$A_{vg} = 2 (1.5 + 3 \cdot 3) (0.625) = 13.13 \text{ in}^2$$

$$R_r = 0.8 \cdot (0.58 \cdot 50 \cdot 13.13 + 70 \cdot 6.25) = 654.6 \text{ kips} > 442.2 \text{ kips} \quad \underline{OK}$$

Check inner splice plates

$$A_{tn} = 2 \cdot [1.5 + (2) (3) - (2.5)(1.0)] \cdot 0.75 = 7.50 \text{ in}^2$$

$$A_{vn} = 2 \cdot [1.5 + (3)(3) - 3.5(1.0)] \cdot 0.75 = 10.50 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{7.50}{10.50} = 0.71 > 0.58$$

$$A_{vg} = 2 \cdot (1.5 + 3 \cdot 3) \cdot 0.75 = 15.75 \text{ in}^2$$

$$R_r = \phi_{bs} \cdot (0.58 \cdot F_y A_{vg} + F_u A_{tn})$$

$$= 0.8 \cdot (0.58 \cdot 50 \cdot 15.75 + 70 \cdot 7.50) = 785.4 \text{ kips} > 442.2 \text{ kips} \quad \underline{OK}$$

The possible block shear failure mode 2 on the outer splice plate is shown below in Figure 6.9.11. Since the outer splice plate controlled for block shear failure mode 1, it can be seen that it will control for failure mode 2 also.

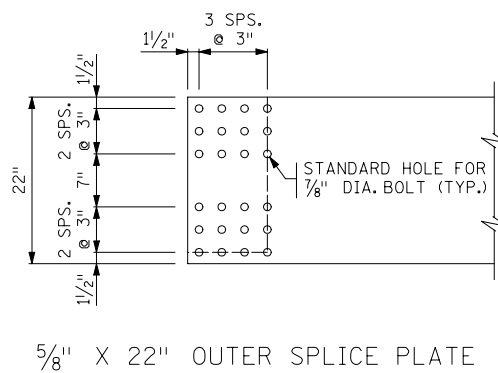


Figure 6.9.11

Block Shear Failure Mode 2 - Bottom Flange Splice Plates

For the outer splice plate,

$$A_{tn} = [1.5 + (4)(3) + (7) - (5.5)(1)] (0.625)$$

$$= 9.38 \text{ in}^2$$

$$A_{vn} = [1.5 + (3)(3) - (3.5)(1)] (0.625)$$

$$= 4.38 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{9.38}{4.38} = 2.14 > 0.58$$

$$A_{vg} = (1.5 + 3 \cdot 3) (0.625) = 6.56 \text{ in}^2$$

$$R_r = \phi_{bs} (0.58 F_y A_{vg} + F_u A_{tn})$$

$$= (0.8) [(0.58) (50.0) (6.56) + (70) (9.38)] = 677.5 \text{ kips} > 442.2 \text{ kips} \quad \underline{\text{OK}}$$

**M.2 Noncontrolling
Flange**
[6.13.6.1.4c]

Table 6.9.19 indicates that the top flange is the noncontrolling flange for both positive and negative live load for the Strength I Limit State.

The noncontrolling flange at the strength limit state shall be proportioned to provide a minimum design resistance for both positive and negative live load moments equal to the greater of:

$$F_{ncf} = R_{cf} \left| \frac{f_{ncf}}{R_h} \right| \quad \text{or} \quad 0.75 \alpha \phi F_{yf}$$

where:

$$R_{cf} = \left| \frac{F_{cf}}{f_{cf}} \right|$$

Load Case 1 - Positive Live Load for Strength I Limit State
Top flange is in compression
From Table 6.9.19:

$$f_{cf} = 15.96 \text{ ksi} \quad f_{ncf} = -3.25 \text{ ksi} \quad F_{cf} = 37.5 \text{ ksi}$$

$$R_{cf} = \left| \frac{F_{cf}}{f_{cf}} \right| = \frac{37.5}{15.96} = 2.35$$

$$F_{ncf} = (2.35) \cdot \left| \frac{-3.25}{1.0} \right| = 7.64 \text{ ksi}$$

or

$$F_{ncf} = (0.75) (1.0) (1.0) (50) = 37.5 \text{ ksi}$$

GOVERNS

Maximum compression design force in top flange is:

$$\begin{aligned} T_{\text{cnfdes}} &= F_{\text{ncf}} \cdot A_e, \text{ where } A_e = A_g \text{ for compression flange} \\ &= (37.5) (20) (1.0) = 750.0 \text{ kips} \end{aligned}$$

Load Case 2 - Negative Live Load for Strength I Limit State

Top flange is in tension.

From Table 6.9.19:

$$f_{\text{cf}} = -19.67 \text{ ksi}$$

$$f_{\text{ncf}} = 17.39 \text{ ksi}$$

$$F_{\text{cf}} = 37.5 \text{ ksi}$$

$$R_{\text{cf}} = \left| \frac{F_{\text{cf}}}{f_{\text{cf}}} \right| = \frac{37.5}{19.67} = 1.91$$

$$F_{\text{ncf}} = R_{\text{cf}} \left| \frac{f_{\text{ncf}}}{R_n} \right| = (1.91) \left| \frac{17.39}{1.0} \right| = 33.20 \text{ ksi}$$

or

$$F_{\text{ncf}} = (0.75) (1.0) (1.0) (50) = 37.5 \text{ ksi} \quad \underline{\text{GOVERNS}}$$

Effective area of top flange:

$$\begin{aligned} A_e &= \left(\frac{\phi_u F_u}{\phi_y F_{yt}} \right) A_n \\ &= \left(\frac{(0.8)(70)}{(0.95)(50)} \right) (1.0) (20 - (4)(1.0)) \\ &= 18.86 \text{ in}^2 < A_g = 20 \text{ in}^2 \end{aligned}$$

Maximum tension design force of top flange at splice location:

$$T_{\text{tnfdes}} = (37.5) (18.86) = 707.3 \text{ kips}$$

The design of the top flange splice is not included in this design example for brevity. However, the top flange splice is designed using the same procedures and methods presented in this example for the bottom flange splice. The size of the resulting top flange splice plates are as follows.

The outer plate is $1/2$ " x 20" (area = 10.00 in²) and the inner plates are $5/8$ " x 9" (area = 5.625 in² per plate).

M.3 Web Splice

[6.13.6.1.4b]

The web is designed to carry the entire factored vertical shear force. In addition, it must carry the moment due to the eccentricity of the shear force and the flexural moment which the web was assumed to carry. The flexural stresses in the web are resolved into flexural and axial (horizontal) components about mid-depth of the web. This allows the bolt group on each side of the splice to be designed for the vertical shear, the moment associated with the eccentricity of the vertical shear, the web flexural moment, and the resultant horizontal force in the web.

In this example, M_{uw} and H_{uw} are computed by conservatively using the stresses at the midthickness of the flanges. By utilizing the stresses at the midthickness of the flanges, the same stress values can be used for the design of both the flange and web splices, which simplifies the calculations. The design forces will be computed under the Strength I Limit State and Service II Limit State.

Strength I Limit State:

From Tables 6.9.9 and 6.9.11, the vertical shear force to be carried is:

For positive live load shear:

$$V_u = |0.9 \cdot [(-116) + (-22)] + 1.75 \cdot 1.112(20)| = 85.3 \text{ kips}$$

For negative live load shear:

$$V_u = |(1.25) [(-116) + (-22)] + 1.75 \cdot 1.112(-114)| = 394.3 \text{ kips} \quad \underline{\text{GOVERNS}}$$

The nominal shear resistance of the unstiffened web, V_n , is 460.6 kips.

Then

$$0.5\phi_v V_n = 230.3 \text{ kips} < 394.3 \text{ kips}$$

Therefore, the design shear force is taken as:

$$\begin{aligned} V_{uw} &= \frac{(V_u + \phi_v V_n)}{2} \text{ kips} \\ &= \frac{(394.3 + (460.6)(1.0))}{2} = 427.5 \text{ kips} \end{aligned}$$

Next, determine the design moment and the design horizontal force resultant.

Load Case 1 - Strength I Limit State With Positive Live Load

[C6.13.6.1.4b]

$$M_{uw} = \frac{t_w D^2}{12} \left| R_h F_{cf} - R_{cf} f_{ncf} \right|$$

where:

$$t_w = 0.625 \text{ in}$$

$$D = 70 \text{ in}$$

$$R_h = 1.0$$

$$F_{cf} = 37.5 \text{ ksi}$$

$$R_{cf} = 2.35$$

$$f_{ncf} = -3.25 \text{ ksi}$$

$$\begin{aligned} M_{uw} &= \frac{(0.625)(70)^2}{12} \left| (1.0)(37.5) - (2.35)(-3.25) \right| \left(\frac{1}{12} \right) \\ &= 960.0 \text{ k-ft} \end{aligned}$$

$$\begin{aligned} H_{uw} &= \frac{t_w D}{2} (R_h F_{cf} + R_{cf} f_{ncf}) \\ &= \frac{(0.625)(70)}{2} [(1.0)(37.5) + (2.35)(-3.25)] \\ &= 653.2 \text{ kips} \end{aligned}$$

Assume a horizontal bolt pitch of 3 inches and two vertical rows of bolts on each side of the splice. The eccentricity of the shear is the distance from the center of the bolt pattern to the center of the splice:

$$e_v = \frac{3}{2} + \frac{3.5}{2} = 3.25 \text{ in}$$

The moment associated with the vertical shear is:

$$M_v = e_v \cdot V_{uw} = 3.25 \cdot 427.5 \left(\frac{1}{12} \right) = 115.8 \text{ k-ft}$$

Total design moment:

$$M_{uw_pos} = 960.0 + 115.8 = 1075.8 \text{ k-ft}$$

The design forces for the web splice under positive live load condition are:

$$\begin{aligned}V_{uw} &= 427.5 \text{ kips} \\H_{uw} &= 653.2 \text{ kips} \\M_{uw_pos} &= 1075.8 \text{ k-ft}\end{aligned}$$

Load Case 2 - Strength I Limit State With Negative Live Load

$$\begin{aligned}t_w &= 0.625 \text{ in} \\D &= 70 \text{ in} \\R_h &= 1.0 \\F_{cf} &= -37.5 \text{ ksi} \quad (\text{compression}) \\R_{cf} &= 1.91 \\f_{ncf} &= 17.39 \text{ ksi} \\M_{uw} &= \frac{t_w D^2}{12} |R_h F_{cf} - R_{cf} f_{ncf}| \\&= \frac{(0.625)(70)^2}{12} |(1.0)(-37.5) - (1.91)(17.39)| \left(\frac{1}{12}\right) \\&= 1503.9 \text{ k-ft} \\H_{uw} &= \frac{t_w D}{2} (R_h F_{cf} + R_{cf} f_{ncf}) \\&= \frac{(0.625)(70)}{2} [(1.0)(-37.5) + (1.91)(17.39)] \\&= -93.7 \text{ kips} \\M_v &= 115.8 \text{ k-ft}\end{aligned}$$

Total design moment

$$M_{uw-neg} = 1503.9 + 115.8 = 1619.7 \text{ k-ft}$$

The design forces for the web splice under negative live load condition are:

$$\begin{aligned}V_{uw} &= 427.5 \text{ kips} \\H_{uw} &= -93.7 \text{ kips} \\M_{uw-neg} &= 1619.7 \text{ k-ft}\end{aligned}$$

Load Case 3 - Service II Limit State With Positive Live Load

From Tables 6.9.9 and 6.9.11, the factored shear with positive live load is:

$$V_{\text{pos}} = (1.0) [(-116) + (-22)] + (1.3) (1.112) (20) = -109.1 \text{ kips}$$

Determine the design moment and the design horizontal force resultant.

From Table 6.9.19:

[C6.13.6.1.4b]

$$f_s = 11.88 \text{ ksi}$$

$$f_{os} = -2.43 \text{ ksi}$$

$$M_{\text{ser-w}} = \frac{t D^2}{12} \left| 1.0 f_s - 1.0 f_{os} \right|$$

$$= \frac{(0.625)(70)^2}{12} \left| 1 \cdot (11.88) - 1 \cdot (-2.43) \right| \left(\frac{1}{12} \right) = 304.3 \text{ k-ft}$$

$$H_{\text{ser-w}} = \frac{t D}{2} (1.0 f_s + 1.0 f_{os})$$

$$= \frac{(0.625)(70)}{2} [1 \cdot (11.88) + 1 \cdot (-2.43)] = 206.7 \text{ k}$$

The moment from eccentricity of the shear:

$$M_v = V_{\text{ser-w}} \cdot e_v = (109.1) (3.25) \left(\frac{1}{12} \right) = 29.5 \text{ k-ft}$$

Total design moment:

$$M_{\text{ser-w}} = 304.3 + 29.5 = 333.8 \text{ k-ft}$$

The design force for the web splice under Service II Limit State with Positive Live Load is:

$$V_{\text{ser-w}} = 109.1 \text{ kips}$$

$$H_{\text{ser-w}} = 206.7 \text{ kips} \quad (\text{compression})$$

$$M_{\text{ser-w}} = 333.8 \text{ k-ft}$$

The design force values for the other Service II load cases are shown in Table 6.9.20.

Load Case 4 - Fatigue

The fatigue detail category for a bolted connection is Category B. The splice is to be designed for infinite fatigue life and the Category B constant amplitude fatigue threshold $(\Delta F)_{TH}$ is 16 ksi.

$$\text{Then } \Delta F_n = 0.5 \cdot (\Delta F)_{TH} = 0.5 \cdot 16 = 8.0 \text{ ksi}$$

$$\text{Max Actual range } \gamma \cdot (\Delta f) = 3.29 \text{ ksi} < 8.0 \text{ ksi} \quad \underline{\text{OK}}$$

Bolt Shear Resistance

The vertical shear and the horizontal force are assumed to be resisted equally by all bolts in the fastener group. The force carried by each of the bolts to resist flexure is assumed to be proportional to its distance from the center of the fastener group.

The force in each of the bolts can be found with the following equations:

$$R_{xA} = R_{xp} + R_{xm} = \frac{P_x}{nm} + \frac{M \cdot y_A}{I_p} \quad R_{yA} = R_{yp} + R_{ym} = \frac{P_y}{nm} + \frac{M \cdot x_A}{I_p}$$

$$R_A = \sqrt{R_{xA}^2 + R_{yA}^2}$$

$$I_p = \frac{nm}{12} [s^2(n^2 - 1) + g^2(m^2 - 1)]$$

where:

$$P_x = H_{uw} \quad P_y = V_{uw} \quad M = M_{uw}$$

x_A = x coordinate of bolt

y_A = y coordinate of bolt

I_p = polar moment of inertia of the bolt group

n = number of bolts per row

m = number of vertical rows of bolts

s = vertical pitch

g = horizontal pitch

[C6.13.6.1.4b]

Assume two vertical rows of 22 bolts on each side of the splice, a horizontal pitch g of 3 inches and a vertical pitch s of 3 inches. The bolts at the corners of the fastener group will be subject to the largest forces. Conservatively, the corner bolts will be checked only. The coordinates at the corners are $x = \pm 1\frac{1}{2}$ and $y = \pm 31.5$ inches.

Substituting values in the above equations, Table 6.19.20 shows the design force in the corner bolts under the Strength I Limit State and Service II Limit State.

Table 6.9.20
Design Force of the Corner Bolts

Parameter	Strength I		Service II		Service II Const.	
	M(+)	M(-)	M(+)	M(-)	M(+)	M(-)
M _{uw} (k-ft)	1075.8	1619.7	333.9	663.9	710.5	684.4
H _{uw} (kip) *	653.2	-93.7	206.7	-36.3	-57.3	56.4
V _{uw} (kip)	427.5	427.5	109.1	302.8	109.0	52.0
n	22	22	22	22	22	22
m	2	2	2	2	2	2
s (in)	3	3	3	3	3	3
g (in)	3	3	3	3	3	3
y (in)	31.5	-31.5	1.5	1.5	1.5	1.5
x (in)	1.5	1.5	31.5	-31.5	-31.5	31.5
I _p (in ²)	16038	16038	16038	16038	16038	16038
R _x (kip)	40.2	-40.3	12.6	-16.5	-18.0	17.4
R _y (kip)	10.9	11.5	2.9	7.6	3.3	1.9
R (kip)	41.7	41.9	12.9	18.2	18.3	17.5

* H_{uw} is a signed quantity, positive for tension and negative for compression.

The nominal shear resistance of a 7/8" diameter A325 bolt without threads in the shear plane and without fill plates used is:

[6.13.2.7]

$$\begin{aligned}
 R_n &= 0.48 A_b F_{ub} N_s \\
 &= (0.48) (0.601) (120) (2) \\
 &= 69.2 \text{ kips}
 \end{aligned}$$

$$\begin{aligned}
 R_r &= \phi R_n \\
 &= (0.8) (69.2) \\
 &= 55.4 \text{ kips}
 \end{aligned}$$

From Table 6.9.20, the maximum design force on the bolt at Strength I Limit State

$$R_{STRI} = 41.9 \text{ kips} < 55.4 \text{ kips} \quad \text{OK}$$

Bolt Slip Resistance

The nominal slip resistance of a $7/8$ " diameter A325 bolt is:

[6.13.2.8]

$$\begin{aligned} R_n &= K_h K_s N_s P_t \\ &= (1.0) (0.5) (2) (39.0) \\ &= 39.0 \text{ kips} \\ R_r &= R_n = 39.0 \text{ kips} \end{aligned}$$

From Table 6.9.20, the maximum design force on the bolt at Service II Limit State is:

$$R_{sII} = 18.3 \text{ kips} < 39.0 \text{ kips} \quad \text{OK}$$

Bolt Bearing Resistance

Nominal bearing resistance of interior or end bolt holes at the Strength I Limit State depends on the clear distance between the holes or clear end distance.

$$\text{Clear distance between holes} = 3 - 1 = 2" > 2d = 1.75"$$

$$\text{Clear end distance} = 1.75 - 0.5 = 1.25" < 2d = 1.75"$$

[6.13.2.9-2]

$$\begin{aligned} R_n &= 1.2 L_c t F_u \\ L_{c1} &= 3.0 - 1.0 = 2.0" && \text{for interior bolts} \\ L_{c2} &= 1.75 - 0.5 = 1.25" && \text{for end bolts} \end{aligned}$$

$$R_{n_int} = (1.2) (2.0) (0.625) (70) = 105 \text{ kips}$$

$$R_{n_end} = (1.2) (1.25) (0.625) (70) = 65.6 \text{ kips} \quad \text{GOVERNS}$$

$$\begin{aligned} R_r &= \phi_{bb} R_{n_end} \\ &= (0.8) (65.6) \\ &= 52.5 \text{ kips} \end{aligned}$$

From Table 6.9.20, maximum design force at Strength I Limit State

$$R_{STRI} = 41.9 \text{ kips} < 52.5 \text{ kips} \quad \text{OK}$$

The plates used in the web splice must have adequate resistance to carry the vertical shear. Two $3/8$ " thick plates are being used for the splice. Assume the plates are 66 inches tall ($21 \cdot 3 + 2 \cdot 1\frac{1}{2}$)

Gross area of the plates:

$$A_g = 66 \cdot 2 \cdot 0.375 = 49.50 \text{ in}^2$$

Assumed vertical shear resistance:

[6.13.5.3-2]

$$R_n = 0.58 \cdot F_y \cdot A_g = 0.58 \cdot 50 \cdot 49.50 = 1435.5 \text{ kips}$$

[6.13.5.3-1]

$$R_r = \phi_v R_n = (1.0) (1435.5) = 1435.5 \text{ kips} > V_{uw} = 427.5 \text{ kips} \quad \text{OK}$$

Block Shear Rupture Resistance

[6.13.4]

Check block shear failure mode shown in Figure 6.9.12 for web splice plates.

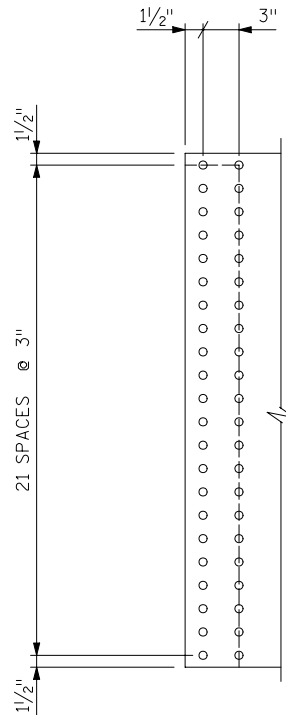


Figure 6.9.12

Net area along the plane resisting shear:

$$A_{vn} = (2) (66 - 1.5 - (21.5) (1.0)) (0.375) = 32.25 \text{ in}^2$$

Net area along the plane resisting tension:

$$A_{tn} = (2) (1.5 + 3.0 - (1.5) (1.0)) (0.375) = 2.25 \text{ in}^2$$

$$\frac{A_{tn}}{A_{vn}} = \frac{2.25}{32.25} = 0.07 < 0.58$$

$$R_r = \phi_{bs} (0.58 F_u A_{vn} + F_y A_{tg})$$

$$A_{tg} = (2) (1.5 + 3.0) (0.375) = 3.38 \text{ in}^2$$

$$R_r = (0.8) [(0.58) (70) (32.25) + (50) (3.38)]$$

$$= 1182.7 \text{ kips} > V_{uw} = 427.5 \text{ kips} \quad \text{OK}$$

At Strength I Limit State the flexure stress in the web splice plates shall not exceed $\phi_f F_y$:

[6.13.6.1.4b]

$$\sigma = \frac{H_{uw}}{A_g} + \frac{M_{uw}}{S_{pl}} \leq \phi_f F_y$$

Section modulus of the web splice plates is:

$$S_{pl} = 2 \cdot \frac{t_w \cdot D_{sp}^3}{12} \cdot \frac{1}{D_{sp}/2} = \frac{t_w \cdot D_{sp}^2}{3} = \frac{0.375 \cdot 66^2}{3} = 544.5 \text{ in}^3$$

$$\phi_f = 1.0$$

For positive liveload moment of Strength I limit State:

$$H_{uw} = 653.2 \text{ kips}$$

$$M_{uw} = 1075.8 \text{ k-ft}$$

$$\sigma = \frac{653.2}{49.50} + \frac{(1075.8)(12)}{544.5} = 36.9 \text{ ksi} < 50 \text{ ksi} \quad \text{OK}$$

For negative liveload moment of Strength I limit State:

$$H_{uw} = -93.7 \text{ kips}$$

$$M_{uw} = 1619.7 \text{ k-ft}$$

$$\sigma = \frac{93.7}{49.50} + \frac{(1619.7)(12)}{544.5} = 37.6 \text{ ksi} < 50 \text{ ksi} \quad \text{OK}$$

The assumed web splice details have adequate capacity. The field splice is detailed in Figure 6.9.13.

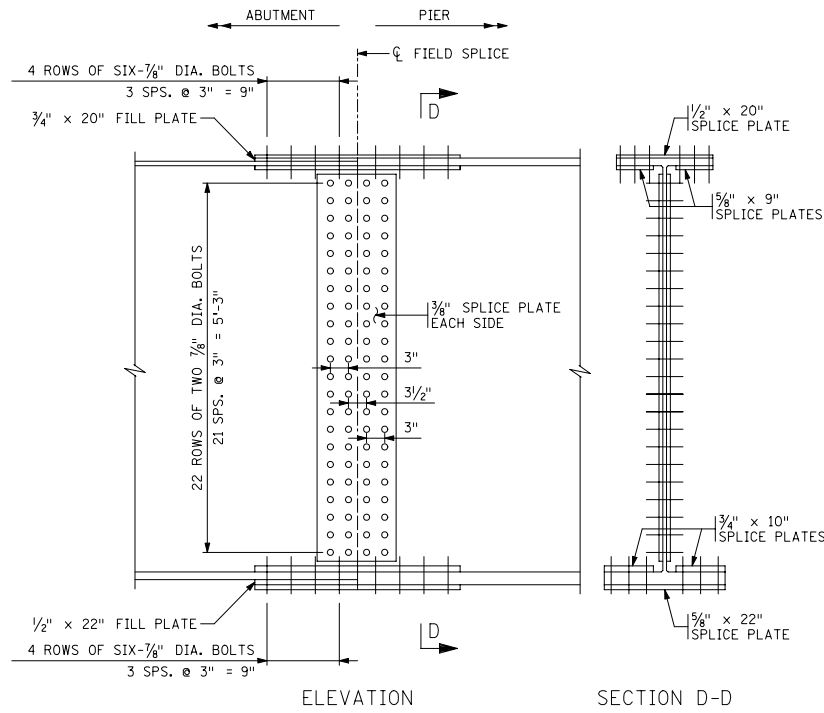


Figure 6.9.13

N. Investigate Deflection

Consider with the Service I load combination. No sidewalk or bicycle path is provided on the bridge. In accordance with Mn/DOT policy, the live load deflection limit is $L / 800$. The maximum deflection permitted for this example is:

$$L / 800 = 175 \cdot (12) / 800 = 2.63 \text{ in}$$

Two live loads are applied to the bridge and evaluated for the deflection check. Take the larger of:

[3.6.1.3.2]

- Design Truck alone
- 25% of Design Truck + Lane Loading

When computing deflections a separate distribution factor is used. It is simply the number of design lanes divided by the number of girders. Mn/DOT practice is to use a multiple presence factor for deflections of no less than 0.85 (See Section 3.4.2). For this example, the distribution factor is:

[2.5.2.6.2]

$$g_{\Delta} = \frac{\text{Number of Design Lanes}}{\text{Number of Girders}} \cdot \Delta_{MPF} = \frac{4}{5} \cdot 0.85 = 0.68$$

The maximum deflections (like the moments) are based on the composite section, including the deck in the negative regions. Including dynamic load allowance, the maximum deflections for a full lane or truck are:

$$\Delta_{\text{max}+l} = 1.45 \text{ inches at } 0.45 \text{ of span} < 2.63 \text{ in} \quad \text{OK}$$

O. Camber

To ensure that steel bridges have the proper profile after construction, steel girders are fabricated with camber. Camber is an adjustment to the vertical profile of a girder. Camber in the girder is made up of geometric camber, dead load camber, and residual camber (if required).

This bridge is on a straight grade, so it will not require any geometric camber.

The girders for this example will deflect 1.18 inches downward at the 0.4 Span Point due to their own weight. When the other DC1 dead loads (deck, stool) are added to the bridge, an additional 5.61 inches of downward deflection is estimated for the 0.4 Span Point. The addition of barriers will add an additional deflection of 0.61 inches downward at the same location. Summing these values results in an anticipated deflection of 7.40 inches. Deflections at 10th points along the span are provided for selfweight, other DC1 loads, and DC2 loads in Figure 6.9.14.

As previously calculated, a residual camber of 2.5 inches is also provided. The residual camber is provided to prevent the appearance of a sag in the span.

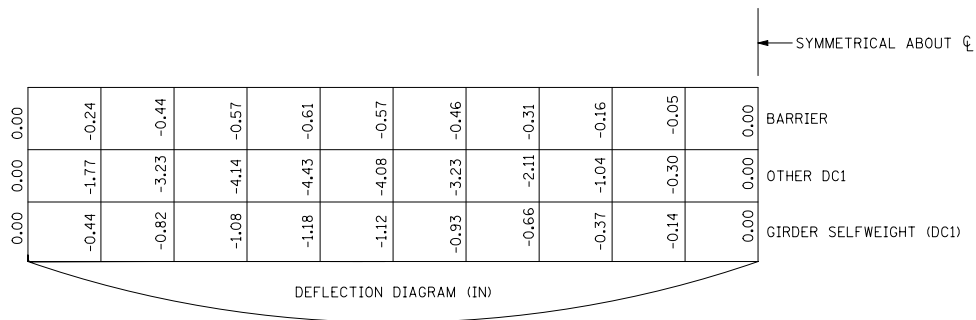


Figure 6.9.14

By following the procedure in 6.3.4, the camber diagram is developed and shown on Figure 6.9.15.

Figure 6.9.16 contains a half elevation of the girder that summarizes the design.

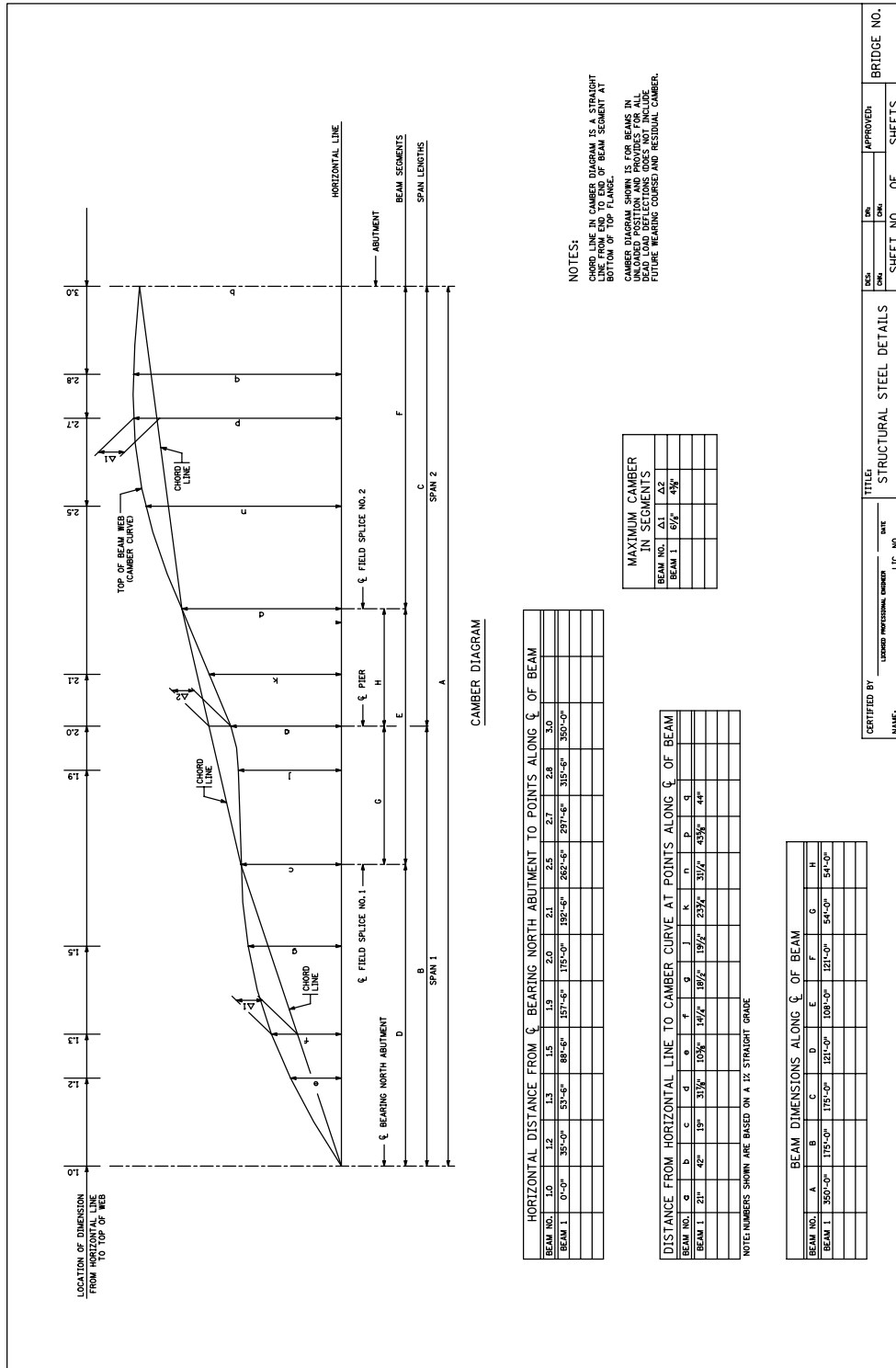


Figure 6.9.15

CERTIFIED BY: _____ DATE: _____
 LIC. NO.: _____
 TITLE: STRUCTURAL STEEL DETAILS
 APPROVED: _____ DATE: _____
 LIC. NO.: _____
 SHEET NO. ... OF ... SHEETS
 BRIDGE NO. _____

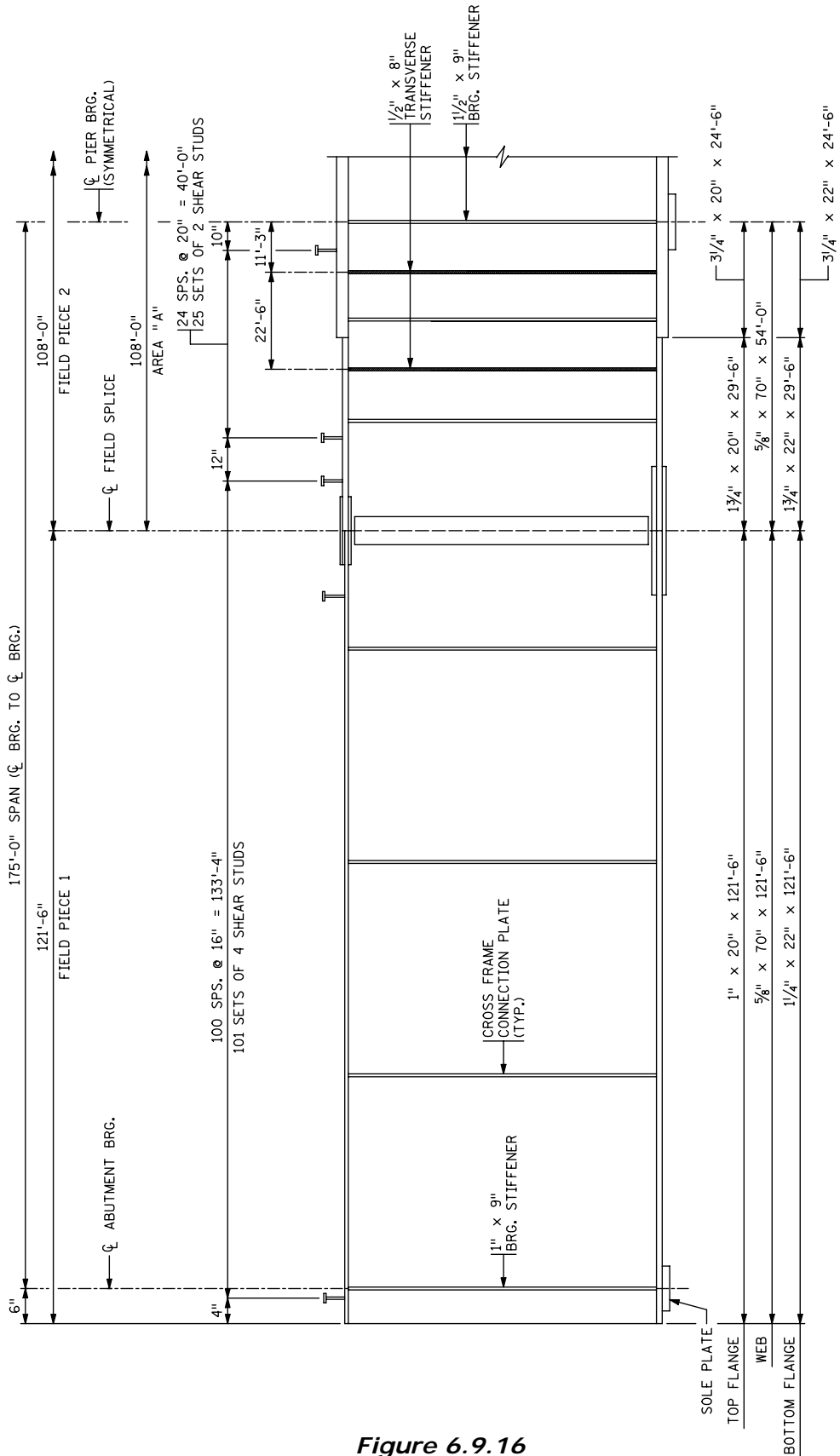


Figure 6.9.16
Half Elevation

P. End Diaphragm Design

The end diaphragm is used to support the end of the deck and to transfer wind load to the supports. It also is required to carry jacking loads if the bearings are replaced. Compared to the jacking loads and the dead and live loads, the wind loads for this example are relatively modest. The end diaphragm will be designed for two load combinations: Strength I where dead and live loads are carried on a simple non-composite span, and Strength I where dead loads and jacking loads are carried on simple span as well.

The design simple span length will be the distance between girders increased for the skew. See Figure 6.9.16.

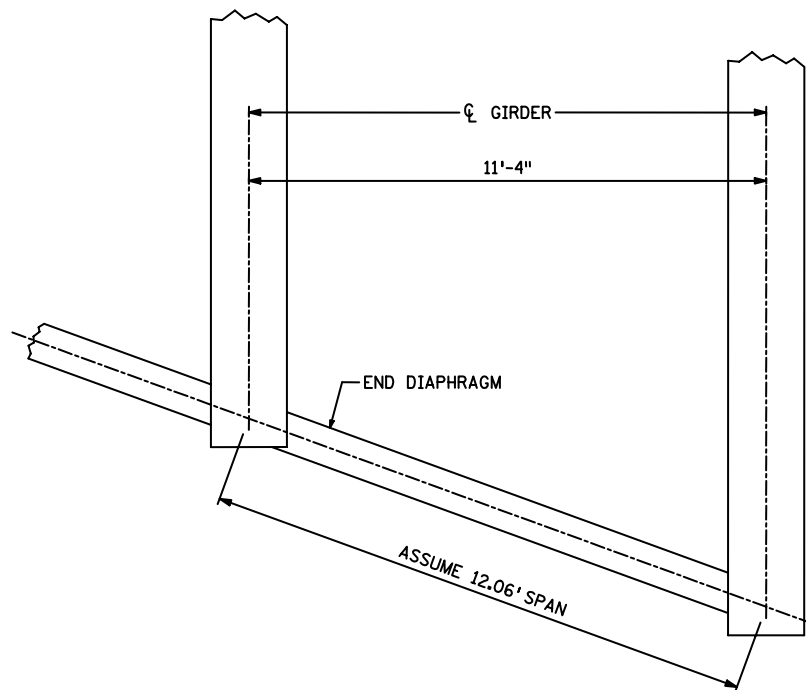


Figure 6.9.17
Length of End Diaphragm

Assume that the end diaphragm carries its own selfweight, the weight of a 2 foot strip of deck, and the additional weight of the thickened deck at the joint. For dead load purposes, assume the additional thickness is 4 inches and that it is 14 inches wide.

Assume 50 pounds per lineal foot for the weight of the beam and steel connections.

The assumed dead load per foot is:

$$w_d = 0.050 + 0.150 \cdot \left(2 \cdot \frac{9.5}{12} + \frac{14}{12} \cdot \frac{4}{12} \right) = 0.346 \text{ kips/ft}$$

Dead load shear is:

$$\frac{w_d \cdot L}{2} = \frac{0.346 \cdot 12.06}{2} = 2.1 \text{ kips}$$

Dead load moment is

$$\frac{w_d \cdot L^2}{8} = \frac{0.346 \cdot 12.06^2}{8} = 6.3 \text{ kips-ft}$$

Consider two live load cases, one where the lane of traffic is centered between the girders and a second one where one of the truck wheels is placed at the center of the diaphragm. The two cases are presented in Figure 6.9.18. This assumes two feet of lane load and includes dynamic load allowance on the wheel load. For Case 1, the live load is centered between the girders and the shear force is:

$$V = 21.3 + 0.064 \cdot 2 \cdot \frac{10}{2} = 21.9 \text{ kips}$$

The moment at mid span for this case is:

$$M \approx 21.3 \cdot 2.67 + \frac{0.064 \cdot 2 \cdot 12.06^2}{8} = 59.2 \text{ k-ft}$$

For Case 2, assume that the left wheel is just to the right of the interior girder. This will produce a conservative design shear. The shear force for this case is:

$$V = 21.3 + \frac{6.06}{12.06} \cdot 21.3 + \frac{8.06}{12.06} \cdot [8 \cdot 2 \cdot 0.064] = 32.7 \text{ kips}$$

The moment at mid span for this case is:

$$\begin{aligned} M &\approx \frac{6.06}{12.06} \cdot 21.3 \cdot 6.03 + \frac{8.06}{12.06} \cdot [0.064 \cdot 2 \cdot 8] \cdot 6.03 - [0.064 \cdot 2] \cdot \frac{(6.03)^2}{2} \\ &= 71.0 \text{ k - ft} \end{aligned}$$

[3.4.3.1]

Assuming lane closed during jacking operation, the shear force in the end diaphragms during jacking can be estimated from the abutment reactions for the DC1 and DC2 loads. Jacking forces have a 1.3 load factor. Assume that two jacks are used to lift each interior girder and that they are placed two feet away from the center of the girder to clear the bearings.

$$1.3V_{\text{jack}} = \frac{1.3 \cdot (\text{DC1} + \text{DC2})}{2} = \frac{1.3 \cdot (115 + 23)}{2} = 89.7 \text{ kips}$$

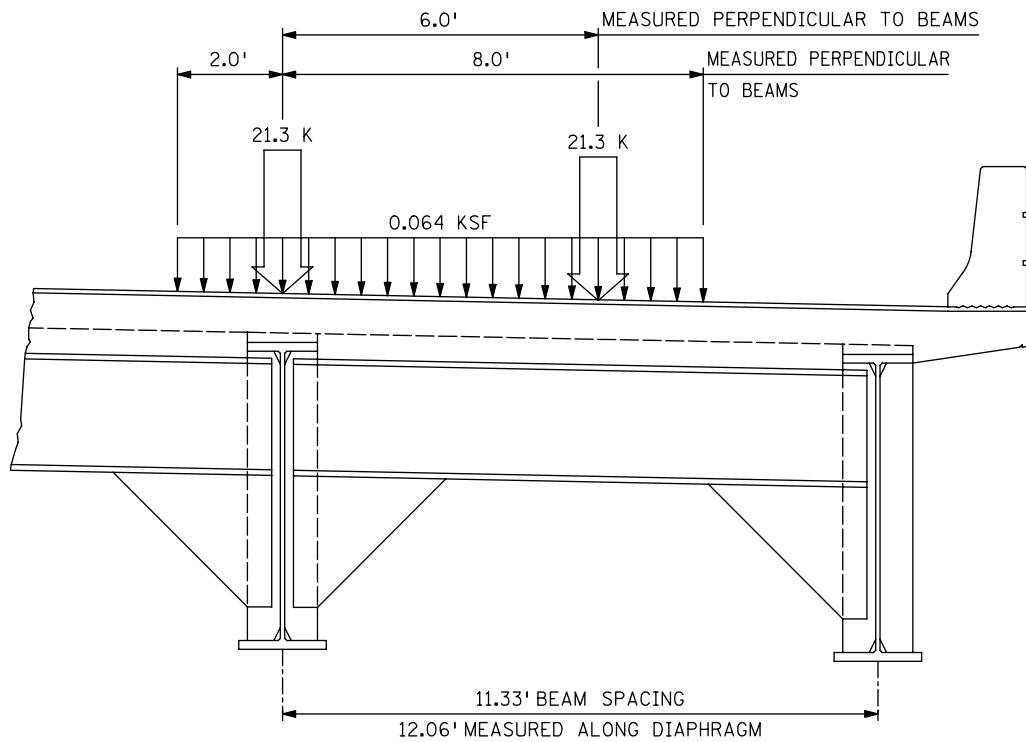
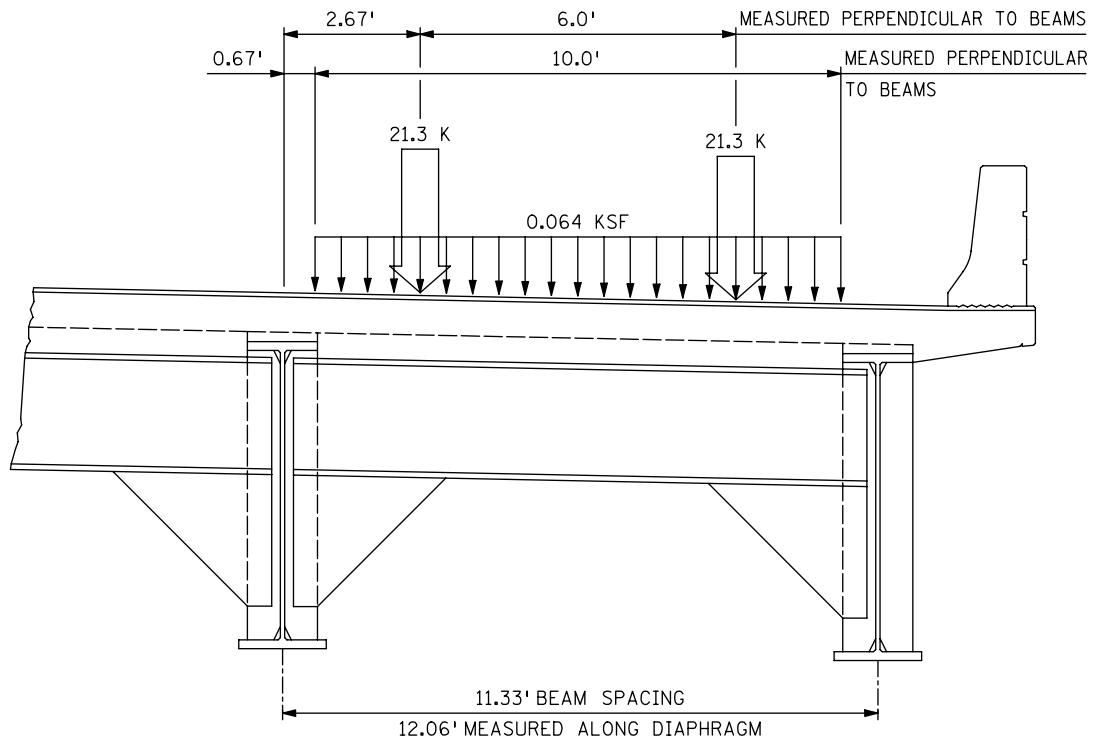


Figure 6.9.18
Live Load Placement on End Diaphragm

With each jack positioned two feet from the girder the moment at mid span in the end diaphragm is:

$$1.3M_{\text{jack}} = 89.7 \cdot 2 = 179.4 \text{ k-ft}$$

[6.10.9.2-2]

By inspection, the jacking operation governs the design of the end diaphragm. Begin by sizing a rolled beam based on shear capacity. Assume the rolled section will satisfy the slenderness ratio to permit that V_p equals 58% of the yield stress to be used.

$$\begin{aligned} V_r = V_u &= 1.25 V_{DL} + 1.3 V_{\text{jack}} \\ &= (1.25)(2.1) + 89.7 \\ &= 92.3 \text{ kips} \end{aligned}$$

$$V_r = \phi_v V_n = \phi_v C V_p$$

where:

$$\phi_v = 1.0$$

$$C = 1.0$$

Therefore,

$$V_r = (1.0)(1.0)(0.58)F_y A_w$$

$$A_w = \frac{V_r}{0.58 F_y} = \frac{92.3}{0.58 \cdot 50} = 3.18 \text{ in}^2 \text{ web area required}$$

Assume that the rolled beam can reach M_y . The required section modulus for the beam is:

$$S_{\text{required}} = \frac{(1.25)M_{DL} + (1.3)M_{\text{jack}}}{F_y} = \frac{(1.25 \cdot 6.3 + 179.4) \cdot 12}{50} = 44.9 \text{ in}^3$$

Based on review of section properties in the *AISC Manual of Steel Construction*, try a W12x40 section:

Area	A	=	11.8 in ²
Depth	D	=	12.0 in
Web Thickness	t _w	=	⁵ / ₁₆ in
Flange Width	b _f	=	8 in
Flange Thickness	t _f	=	¹ / ₂ in
Section Modulus	S	=	51.9 in ³
Radius of Gyration	r _t	=	1.9 in

Determine the nominal shear capacity with LRFD Article 6.10.9.2.

$$\frac{D}{t_w} = \frac{12 - 2 \cdot (0.5)}{0.31} = 35.5$$

$$1.12 \cdot \sqrt{\frac{E \cdot k}{F_y}} = 1.12 \cdot \sqrt{\frac{29,000 \cdot 5}{50}} = 60.3 > 35.5$$

Therefore, C = 1.0 and the shear capacity of the beam is:

[6.10.9.2-1]

$$V_n = C \cdot V_p = 1 \cdot 0.58 \cdot 50 \cdot [12 - 2 \cdot 0.5] \cdot 0.31 = 98.9 \text{ kips}$$

$$V_u = 1.3 V_{\text{jack}} + (1.25) V_{\text{DL}}$$

$$= 92.3 \text{ kips} < \phi_v V_n = (1.0) (98.9) = 98.9 \text{ kips} \quad \text{OK}$$

[6.10.8.2]

Check Compression flange flexural resistance

Local Buckling

$$\lambda_f = \frac{b_f}{2 \cdot t_f} = \frac{8}{2 \cdot 0.5} = 8 \leq \lambda_{pf} = 9.2$$

$$F_{nc} = F_{yc} = 50.0 \text{ ksi}$$

Section is subject to compact section.

Lateral Torsional Buckling

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} = (1.0) (1.93) \left(\sqrt{\frac{29,000}{50}} \right) = 46.5 \text{ in}$$

$$L_r = \pi r_{u t} \sqrt{\frac{E}{F_{yr}}} = (3.14) (1.93) \left(\sqrt{\frac{29,000}{35}} \right) = 174.0 \text{ in}$$

The distance between gusset plates is approximately 60 inches.

$$L_b = 60 \text{ in}$$

$$L_p < L_b < L_r$$

$$F_{nc} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h F_{yc}} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] R_b R_h F_{yc}$$

$$R_h = 1.0$$

$$F_{yr} = 0.7 F_{yc} = 35 \text{ ksi}$$

$$\begin{aligned} F_{nc} &= (1.0) \left[1 - \left(1 - \frac{35}{50} \right) \left(\frac{60 - 46.5}{174 - 46.5} \right) \right] (1.0) (1.0) (50) \\ &= 48.4 \text{ ksi} \end{aligned}$$

[6.10.8.2.3-6]

$$f_2 = 0 \quad \text{Therefore, } C_b = 1.0$$

[6.10.1.10.2-4]

$$\lambda_{rw} = 5.7 \sqrt{\frac{E}{F_{yc}}} = 137$$

In order to determine web load-shedding factor, R_b , the web shall satisfy:

[6.10.1.10.2-2]

$$\begin{aligned} \frac{2D_c}{t_w} &= \frac{2(12 - (0.5)(2))}{0.31} = 71 < \lambda_{rw} \\ R_b &= 1.0 \end{aligned}$$

Flexural Resistance

$$M_n = F_y \cdot S = (48.4) (51.9) = 2512 \text{ k-in} = 209.3 \text{ k-ft}$$

$$M_r = \phi M_n = (0.9) (209.3) = 188.4 \text{ k-ft}$$

$$M_u = 1.25 M_{DL} + 1.3 M_{jack}$$

$$= (1.25) (6.3) + 179.4$$

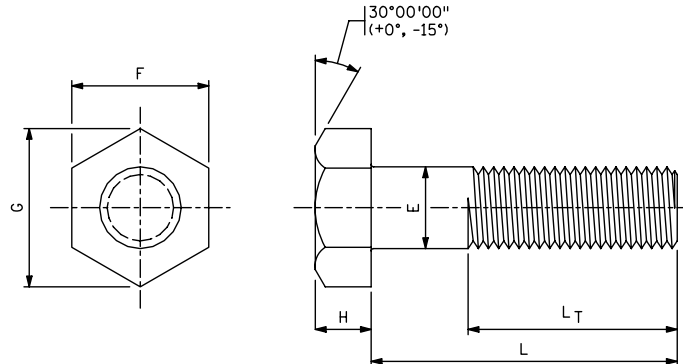
$$= 187.3 \text{ k-ft} < M_r$$

OK

Therefore, use a W12x40 for the end diaphragm.

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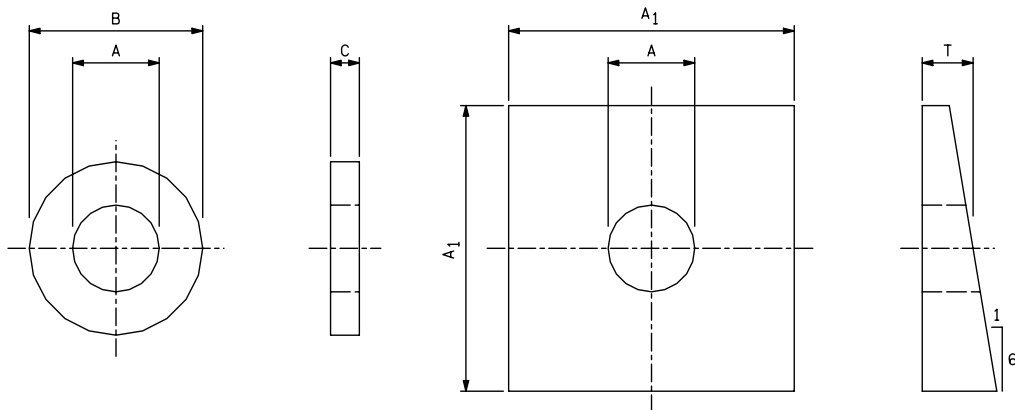
APPENDIX 6-A



NOMINAL BOLT DIAMETER	E	F				G		H			L _T	
	BODY DIA.	WIDTH ACROSS FLATS				WIDTH ACROSS CORNERS		HEIGHT			THREAD LENGTH FOR BOLT LENGTHS	
		MAX.	BASIC	MAX.	MIN.	MAX.	MIN.	BASIC	MAX.	MIN.	L ≤ 6"	L > 6"
1/2	0.515	7/8	0.875	0.850	1.010	0.969	1 1/32	0.364	0.302	1.250	1.500	
5/8	0.642	1 1/16	1.062	1.031	1.227	1.175	2 1/64	0.444	0.378	1.500	1.750	
3/4	0.768	1 1/4	1.250	1.212	1.443	1.383	1/2	0.524	0.455	1.750	2.000	
7/8	0.895	1 7/16	1.438	1.394	1.660	1.589	3 7/64	0.604	0.531	2.000	2.250	
1	1.022	1 5/8	1.625	1.575	1.876	1.796	4 3/64	0.700	0.591	2.250	2.500	
1 1/8	1.149	1 13/16	1.812	1.756	2.093	2.002	3/4	0.780	0.658	2.500	2.750	
1 1/4	1.277	2	2.000	1.938	2.309	2.209	2 7/32	0.876	0.749	2.750	3.000	
1 3/8	1.404	2 3/16	2.188	2.119	2.526	2.416	2 9/32	0.940	0.810	3.000	3.250	
1 1/2	1.531	2 3/8	2.375	2.300	2.742	2.622	1	1.036	0.902	3.250	3.500	
1 3/4	1.785	2 3/4	2.750	2.662	3.175	3.035	1 1/32	1.196	1.054	3.750	4.000	
2	2.039	3 1/8	3.125	3.025	3.608	3.449	1 1/32	1.388	1.175	4.250	4.500	
2 1/4	2.305	3 1/2	3.500	3.388	4.041	3.862	1 1/2	1.548	1.327	4.750	5.000	
2 1/2	2.559	3 7/8	3.875	3.750	4.474	4.275	1 21/32	1.708	1.479	5.250	5.500	
2 3/4	2.827	4 1/4	4.250	4.112	4.907	4.688	1 13/16	1.869	1.632	5.750	6.000	
3	3.081	4 5/8	4.625	4.475	5.340	5.102	2	2.060	1.815	6.250	6.500	

Figure 6-A1
Dimensions of Common Heavy Hex Structural Bolts
Mn/DOT 3391.2A (ASTM A307)

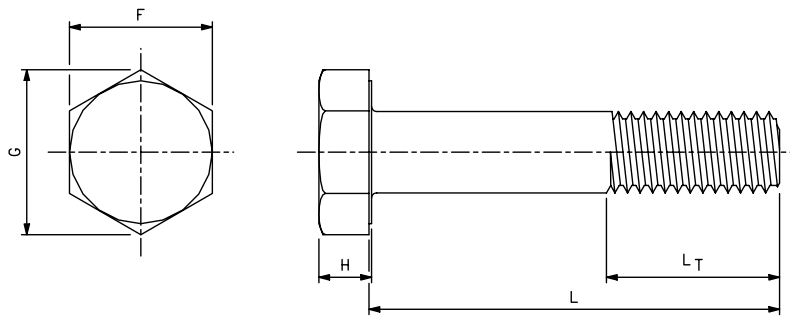
APPENDIX 6-A (Continued)



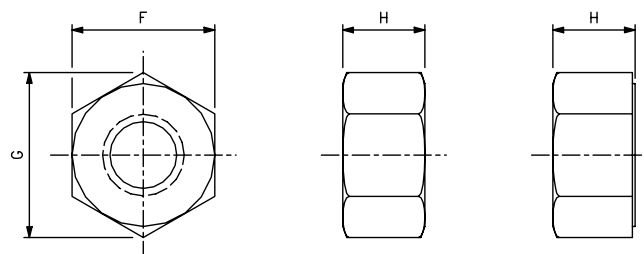
NOMINAL BOLT DIAMETER	FLAT CIRCULAR WASHERS				BEVELED WASHERS	
	A	B	C		A ₁	T
	INSIDE DIA.	OUTSIDE DIA.	THICKNESS		SIDE DIMENSION	MEAN THICKNESS
	NOM.	NOM.	MAX.	MIN.	NOM.	NOM.
1/2	0.531	1.062	0.177	0.097	1.750	0.312
5/8	0.688	1.312	0.177	0.122	1.750	0.312
3/4	0.812	1.469	0.177	0.122	1.750	0.312
7/8	0.938	1.750	0.177	0.136	1.750	0.312
1	1.125	2.000	0.177	0.136	1.750	0.312
1 1/8	1.250	2.250	0.177	0.136	2.250	0.312
1 1/4	1.375	2.500	0.177	0.136	2.250	0.312
1 3/8	1.500	2.750	0.177	0.136	2.250	0.312
1 1/2	1.625	3.000	0.177	0.136	2.250	0.312

Figure 6-A2
Washers for High Strength Structural Bolts
Mn/DOT 3391.2B (ASTM A325)

APPENDIX 6-A (Continued)



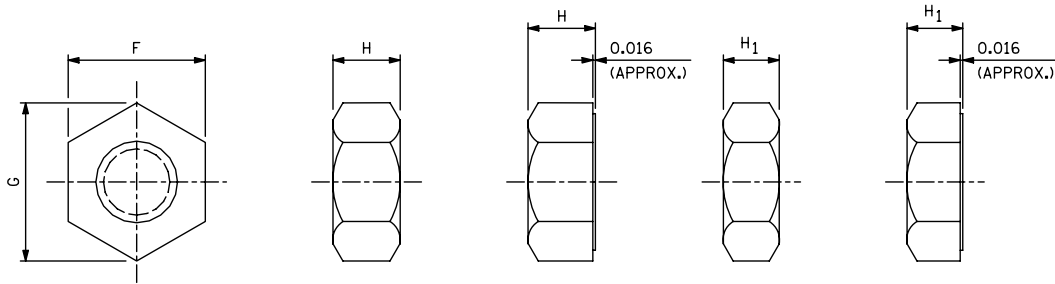
HIGH STRENGTH HEAVY HEX NUTS							
NOMINAL BOLT DIAMETER	F		G		H		L _T
	WIDTH ACROSS FLATS		WIDTH ACROSS CORNERS		HEIGHT		THREAD LENGTH
	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.	BASIC
1/2	0.875	0.850	1.010	1.969	0.323	0.302	1.00
5/8	1.062	1.031	1.227	1.175	0.403	0.378	1.25
3/4	1.250	1.212	1.443	1.382	0.483	.0455	1.38
7/8	1.438	1.394	1.660	1.589	0.563	0.531	1.50
1	1.625	1.575	1.876	1.796	0.627	0.591	1.75
1 1/8	1.812	1.756	2.093	2.002	0.718	0.658	2.00
1 1/4	2.000	1.938	2.309	2.209	0.813	0.749	2.00
1 3/8	2.188	2.119	2.526	2.416	0.878	.0810	2.25
1 1/2	2.375	2.300	2.742	2.622	0.974	0.902	2.25



HIGH STRENGTH HEAVY HEX NUTS						
NOMINAL BOLT DIAMETER	F		G		H	
	WIDTH ACROSS FLATS		WIDTH ACROSS CORNERS		THICKNESS	
	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.
1/2	0.875	0.850	1.010	1.969	0.504	0.464
5/8	1.062	1.031	1.227	1.175	0.631	0.587
3/4	1.250	1.212	1.443	1.382	0.758	0.710
7/8	1.438	1.394	1.660	1.589	0.885	0.833
1	1.625	1.575	1.876	1.796	1.012	0.956
1 1/8	1.812	1.756	2.093	2.002	1.139	1.079
1 1/4	2.000	1.938	2.309	2.209	1.251	1.187
1 3/8	2.188	2.119	2.526	2.416	1.378	1.310
1 1/2	2.375	2.300	2.742	2.622	1.505	1.433

Figure 6-A3
 High Strength Heavy Hex Structural Bolts and Nuts
 Mn/DOT 3391.2B (ASTM A325)

APPENDIX 6-A (Continued)



NOMINAL BOLT DIAMETER	F			G		H			H ₁		
	WIDTH ACROSS FLATS			WIDTH ACROSS CORNERS		THICKNESS HEAVY HEX NUTS			THICKNESS HEAVY HEX JAM NUTS		
	BASIC	MAX.	MIN.	MAX.	MIN.	BASIC	MAX.	MIN.	BASIC	MAX.	MIN.
1/4	1/2	0.500	0.488	0.577	0.556	15/64	0.250	0.218	11/64	0.188	0.156
3/16	9/16	0.562	0.546	0.650	0.622	19/64	0.314	0.280	13/64	0.220	0.186
3/8	11/16	0.688	0.669	0.794	0.763	23/64	0.377	0.341	15/64	0.252	0.216
7/16	3/4	0.750	0.728	0.866	0.830	27/64	0.441	0.403	17/64	0.285	0.247
1/2	7/8	0.875	0.850	1.010	0.969	31/64	0.504	0.464	19/64	0.317	0.227
9/16	15/16	0.938	0.909	1.083	1.037	35/64	0.568	0.526	21/64	0.349	0.307
5/8	1 1/16	1.062	1.031	1.227	1.175	39/64	0.631	0.587	23/64	0.381	0.337
3/4	1 1/4	1.250	1.212	1.443	1.382	47/64	0.758	0.710	27/64	0.446	0.398
7/8	1 1/8	1.438	1.394	1.660	1.589	55/64	0.885	0.833	31/64	0.510	0.458
1	1 5/8	1.625	1.575	1.876	1.796	63/64	1.012	0.956	35/64	0.575	0.519
1 1/8	1 13/16	1.812	1.756	2.093	2.002	17/64	1.139	10.79	39/64	0.639	0.579
1 1/4	2	2.000	1.938	2.309	2.209	17/32	1.251	1.187	23/32	0.751	0.687
1 3/8	2 3/16	2.188	2.119	2.526	2.416	11/32	1.378	1.310	25/32	0.815	0.747
1 1/2	2 3/8	2.375	2.300	2.742	2.622	15/32	1.505	1.433	27/32	0.880	0.808
1 5/8	2 9/16	2.562	2.481	2.959	2.828	19/32	1.632	1.556	29/32	0.944	0.868
1 3/4	2 3/4	2.750	2.662	3.175	3.035	123/32	1.759	1.679	31/32	1.009	0.929
1 7/8	2 5/8	2.938	2.844	3.392	3.242	127/32	1.886	1.802	11/32	1.073	0.989
2	3 1/8	3.125	3.025	3.608	3.449	131/32	2.013	1.925	13/32	1.138	1.050
2 1/4	3 1/2	3.500	3.388	4.041	3.862	213/64	2.251	2.155	113/64	1.251	1.155
2 1/2	3 3/8	3.875	3.750	4.474	4.275	229/64	2.505	2.401	129/64	1.505	1.401
2 3/4	4 1/4	4.250	4.112	4.907	4.688	245/64	2.759	2.647	137/64	1.634	1.522
3	4 5/8	4.625	4.475	5.340	5.102	261/64	3.013	2.893	145/64	1.763	1.643
3 1/4	5	5.000	4.838	5.774	5.515	33/16	3.252	3.124	113/16	1.876	1.748
3 1/2	5 3/8	5.375	5.200	6.207	5.928	37/16	3.506	3.370	115/16	2.006	1.870
3 3/4	5 3/4	5.750	5.562	6.640	6.341	311/16	3.760	3.616	21/16	2.134	1.990
4	6 1/8	6.125	5.925	7.073	6.755	315/16	4.014	3.862	23/16	2.264	2.112

Figure 6-A4

Dimensions of Common Heavy Hex Nuts and Heavy Hex Jam Nuts
Mn/DOT 3391.2A (ASTM A307)

APPENDIX 6-A (Continued)

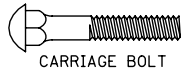
CARRIAGE BOLTS (LBS. PER HUNDRED) ①				
L (IN.)	DIAMETER			
	3/8"	1/2"	5/8"	3/4"
1	7.53	15.15		
1 1/4	8.13	16.24		
1 1/2	8.76	17.34	30.20	
1 3/4	9.52	18.44		
2	10.48	19.74	33.70	53.00
2 1/4	11.04	21.14		
2 1/2	11.80	22.44	37.80	58.30
2 3/4	12.56	23.84	39.90	
3	13.34	25.14	42.00	64.40
3 1/4	14.14	26.54		
3 1/2	14.84	27.84	46.20	70.40
3 3/4	15.64	29.14		
4	16.34	30.54	50.40	76.40
4 1/2	17.94	33.24	54.60	82.50
5	19.44	35.94	58.70	88.50
5 1/2	20.94	38.64	62.90	94.60
6	22.44	41.24	67.10	100.60
6 1/2	23.84	43.74	71.00	106.10
7	25.34	46.44	75.20	112.20
7 1/2	26.84	49.14	79.30	118.20
8	28.34	51.84	83.50	124.30
8 1/2	29.84	54.44		130.30
9	31.44	57.14	91.90	136.30
9 1/2	32.94	59.84		
10	34.44	62.54	100.30	149.30
11	37.44	67.94	108.70	
12	40.84	73.34	116.90	

STRUCTURAL BOLTS ①				
L (IN.)	DIAMETER			
	3/8"	1/2"	5/8"	3/4"
1	7.31	15.76	27.06	43.76
1 1/4	8.00	16.88	28.83	46.36
1 1/2	8.78	18.13	30.60	48.96
1 3/4	9.56	19.51	32.56	51.56
2	10.33	20.89	34.72	54.42
2 1/4	11.11	22.27	36.87	57.53
2 1/2	11.88	23.65	39.03	60.64
2 3/4	12.66	25.23	41.18	63.75
3	13.44	26.41	43.34	66.86
3 1/4	14.21	27.79	45.49	69.97
3 1/2	14.99	29.17	47.65	73.08
3 3/4	15.76	30.55	49.80	76.19
4	16.54	31.93	51.96	79.30
4 1/2	18.09	34.69	56.27	85.52
5	19.64	37.45	60.58	91.74
5 1/2	21.20	40.21	64.89	97.96
6	22.75	42.97	69.20	104.18
6 1/2	23.74	44.24	72.20	108.40
7	25.34	46.94	76.40	114.40
7 1/2	26.84	49.54	80.60	120.30
8	28.34	52.24	84.80	126.30
8 1/2	29.84	54.94	89.00	132.30
9	31.34	57.64	93.10	138.30
9 1/2	32.94	60.34	97.30	144.30
10	34.44	63.04	101.50	150.30
11	37.44	68.44	109.90	162.30
12	40.44	73.74	117.90	175.30

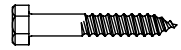
ROUND WASHERS ②				
BOLT DIA.	HOLE DIA.	THICK. (IN.)	O.S. DIA.	LBS. PER 100 PCS.
1/4"	5/16"	0.065	3/4"	0.67
3/8"	7/16"	0.083	1"	1.49
1/2"	9/16"	0.109	1 3/8"	3.85
5/8"	1 1/16"	0.134	1 3/4"	7.70
3/4"	1 3/16"	0.148	2"	11.00
7/8"	1 5/16"	0.165	2 1/4"	15.40
1"	1 1/16"	0.165	2 1/2"	18.80
1 1/8"	1 1/4"	0.165	2 3/4"	22.10
1 1/4"	1 3/8"	0.165	3"	26.10
1 3/8"	1 1/2"	0.180	3 1/4"	
1 1/2"	1 5/8"	0.180	3 1/2"	38.46

① AMERICAN STANDARD PLAIN (MEDIUM)

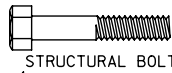
① CARRIAGE AND STRUCTURAL BOLTS INCLUDE NUTS. SEE SPEC. 3391.



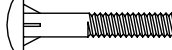
CARRIAGE BOLT



LAG SCREW



STRUCTURAL BOLT



DOME HEAD BOLT

NUTS (LBS. PER HUNDRED)		
BOLT DIA.	HEAVY HEX	HEAVY SQ.
1/4"	1.16	1.37
3/8"	3.14	3.74
1/2"	6.54	7.87
5/8"	11.9	14.3
3/4"	19.3	23.5
7/8"	29.7	36.2
1"	42.5	51.5
1 1/8"	59.2	72.4
1 1/4"	78.6	95.5
1 3/8"	102.0	125.0
1 1/2"	131.0	161.0

NAILS/SPIKES (LBS. PER HUNDRED)			
L (IN.)	SIZE	NAIL	SPIKE
1	2d	847	
1 1/4	3d	543	
1 1/2	4d	294	
1 3/4	5d	254	
2	6d	167	
2 1/4	7d	150	
2 1/2	8d	101	
2 3/4	9d	92	
3	10d	66	42
3 1/4	12d	61	39
3 1/2	16d	47	31
4	20d	29	22
4 1/2	30d	22	17
5	40d	17	13
5 1/2	50d	13	10
6	60d	10	9
7	5/16"		6
8	3/8"		4
9	3/8"		3
10	3/8"		3
11	3/8"		2

LAG SCREWS (LBS. PER HUNDRED)				
L (IN.)	DIAMETER			
	3/8"	1/2"	5/8"	3/4"
1	4.04			
1 1/4	4.67			
1 1/2	5.30	11.0		
1 3/4	5.93	12.1		
2	6.56	13.3	23.2	
2 1/2	7.82	15.5	26.8	41.8
3	9.10	17.8	30.4	47.1
3 1/2	10.4	20.0	34.0	52.3
4	11.6	22.3	37.6	57.5
4 1/2	12.9	24.6	41.2	62.7
5	14.1	26.9	44.7	68.0
5 1/2	15.4	29.1	48.3	73.2
6	16.6	31.4	51.9	78.4
6 1/2	17.9	33.6	55.5	83.6
7	19.1	35.9	59.0	88.9
7 1/2	20.4	38.2	62.6	94.1
8	21.6	40.5	66.2	99.3
9	24.1	45.0	73.4	110
10	26.6	49.5	80.6	120
11	29.1	54.0	87.8	131
12	31.6	58.5	95.0	141

DOME HEAD BOLTS (LBS. PER HUNDRED)				
L (IN.)	DIAMETER			
	1/2"	5/8"	3/4"	1"
10	130	130	201	
11	138	138	213	
12	146	146	225	323
13	155	155	237	339
14	163	163	249	356
15	171	171	261	372
16	179	179	273	389
17	188	188	285	405
18	196	196	297	422
19	204	204	309	438
20	212	212	321	455
21	221	221	333	471
22	229	229	345	488
23	237	237	357	504
24	245	245	369	520
25	254	254	381	537
26	262	262	393	553
27	270	270	405	569
28	278	278	417	586
29	287	287	429	602
30	295	295	441	619



STUD DETAIL

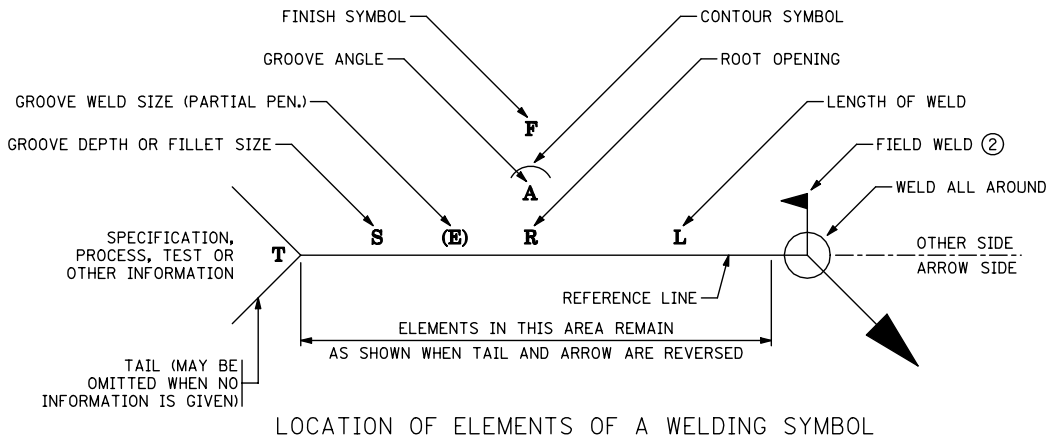
STUD SHEAR CONNECTORS (LBS. PER HUNDRED)					
STUD DIA.	3"	4"	5"	6"	7"
1/2"	21.0	27.0	33.0	39.0	45.0
5/8"	33.6	43.2	52.8	62.4	72.0
3/4"	49.0	61.5	74.0	86.5	99.0
7/8"	64.0	81.0	98.0	115.0	132.0

Figure 6-A5
Hardware Details

APPENDIX 6-A (Continued)

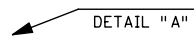
BASIC ARC WELD SYMBOLS								
LOCATION SIGNIFIANCE	FILLET ①	PLUG OR SLOT	SQUARE	V	GROOVE BEVEL ①	FLARE V	FLARE BEVEL ①	BACK OR BACKING
ARROW SIDE								
OTHER SIDE								
BOTH SIDES								

SUPPLEMENTARY SYMBOLS AND INITIALS						
WELD ALL AROUND	FIELD WELD	CONTOUR		FINISH	TESTS	PROCESSES (PER SPEC. 2471)
		FLUSH	CONVEX	"G" = GRINDING "C" = CHIPPING "M" = MACHINING	"RT" = RADIOGRAPHIC "UT" = ULTRASONIC "MT" = MAGNETIC PARTICLE "PT" = DYE PENETRANT	"GMAW" = GAS METAL ARC "FCAW" = FLUX CORED ARC "SMAW" = SHIELDED METAL ARC "SAW" = SUBMERGED ARC



GENERAL NOTES

THE USE OF SYMBOLS MAY BE OMITTED WHEN SPECIFICATIONS AND PROCEDURES TO BE FOLLOWED ARE PRESCRIBED ELSEWHERE. GENERAL NOTES MAY BE PLACED ON A DRAWING SUCH AS: "UNLESS OTHERWISE INDICATED, ALL FILLET WELDS ARE 5/16" SIZE." WHEN BASIC WELD SYMBOLS ARE INADEQUATE, USE REFERENCES SUCH AS:



THE SIDE OF THE JOINT TO WHICH THE ARROW POINTS IS THE ARROW (OR NEAR) SIDE AND THE OPPOSITE SIDE OF THE JOINT IS THE OTHER (OR FAR) SIDE.

BASIC SYMBOLS AS PER AMERICAN WELDING SOCIETY.

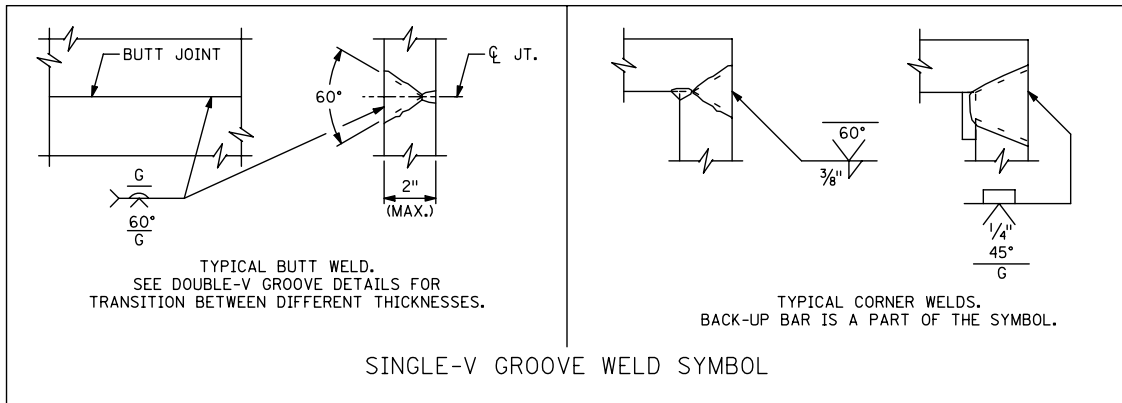
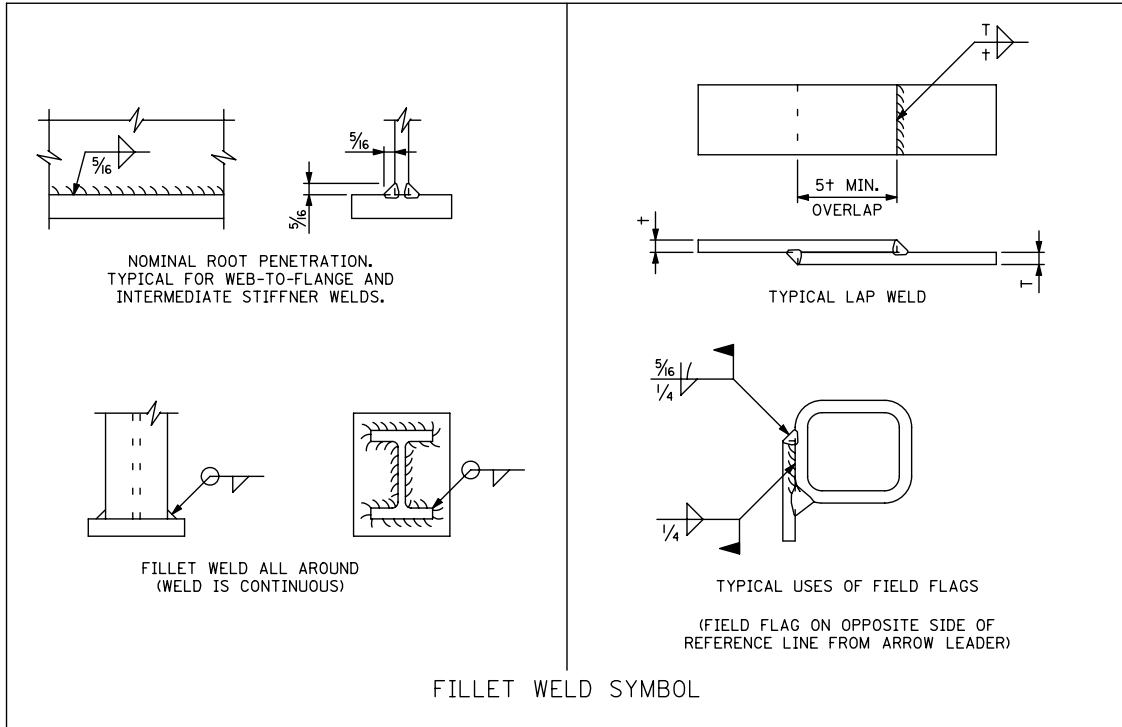
① VERTICAL LEG IS ALWAYS ON LEFT SIDE OF SYMBOL.

② FIELD WELD FLAG POINTS TOWARD TAIL OF WELD.

Figure 6-A6
Welding symbols and Notes

APPENDIX 6-A (Continued)

APPLICATIONS OF WELDING SYMBOLS



GENERAL NOTES (Continued)

ARROW SIDE AND OTHER SIDE WELDS ARE THE SAME SIZE UNLESS OTHERWISE SHOWN.

SYMBOLS APPLY BETWEEN ABRUPT CHANGES IN DIRECTION OF JOINT OR AS DIMENSIONED (EXCEPT WHERE ALL-AROUND SYMBOL IS USED).

ALL WELDS ARE CONTINUOUS AND OF THE USERS STANDARD PROPORTIONS UNLESS OTHERWISE SHOWN.

FILLET WELDS MAY BE SHOWN GRAPHICALLY, THUS:

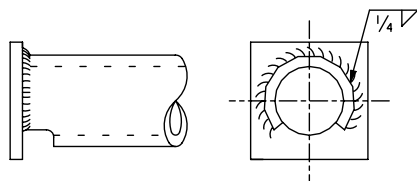


Figure 6-A7
Welding Notes and Joints

APPENDIX 6-A (Continued)

APPLICATIONS OF WELDING SYMBOLS (Continued)

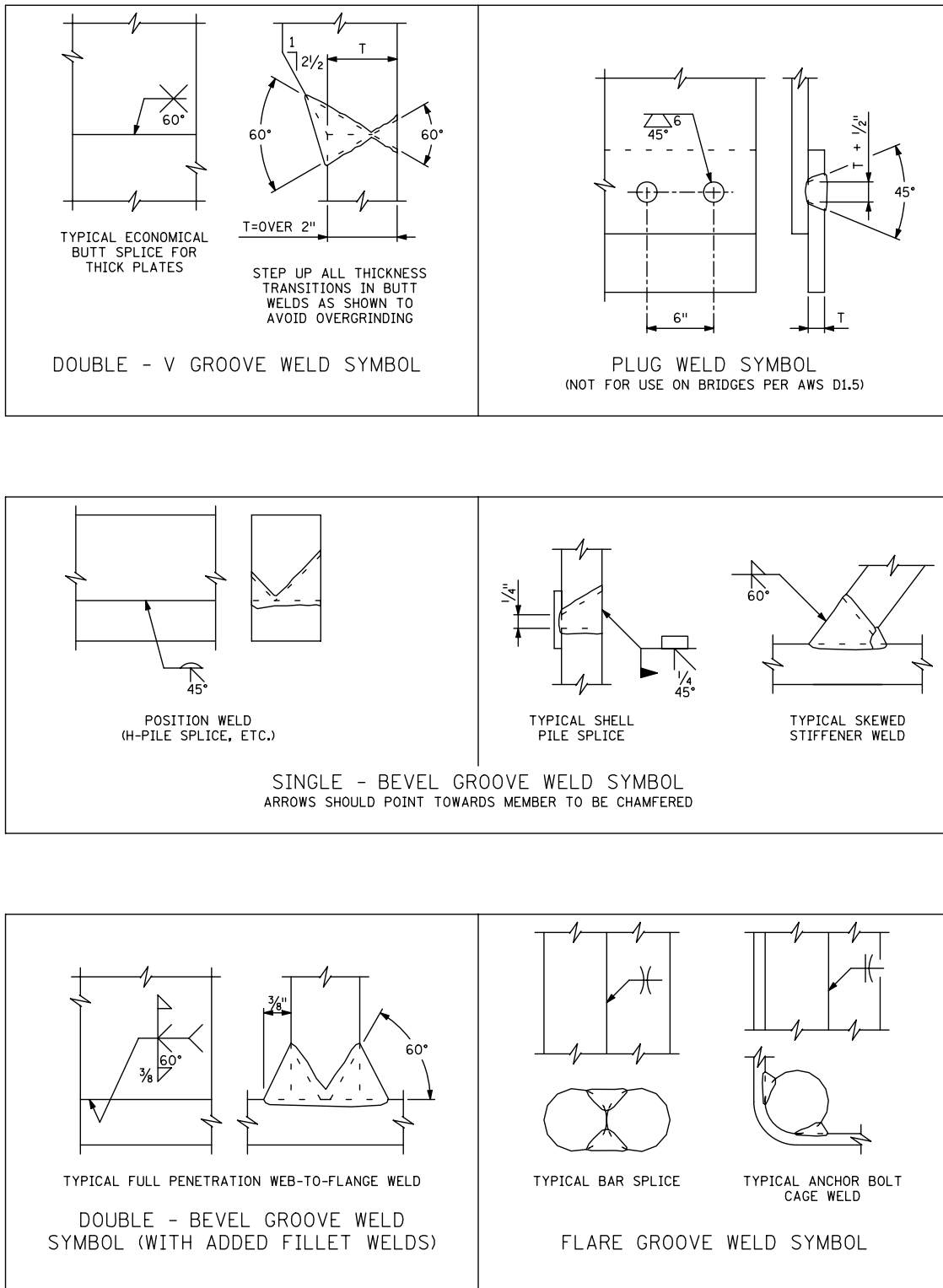


Figure 6-A8
Welding Joints