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Chapter 7

STEEL

Turner Bridge, Turner

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7 STEEL

7.1 General

7.1.1 Blocking and Camber

7.1.1.1 Bottom-of-Slab Elevations

A Blocking Table containing bottom-of-slab elevations should be provided on the design plans for bridges having concrete slabs on steel girders. Bottom-of-slab elevations are used to set formwork after steel erection. No adjustment of beam dead loads is needed to calculate these elevations. For the calculation, the fluid and superimposed dead load deflections are added to the in-service bottom-of-slab elevations. These results are shown in the Blocking Table on the plans.

The Contractor will use the bottom-of-slab elevations in the Blocking Table to set the deck forms relative to the steel at each blocking point (bottom–of-slab elevation point), which will result in the correct profile of the constructed deck after deflection has occurred. The bottom-of-slab elevations are tabulated at regular intervals (generally 10 feet) for each span, beginning at the centerline of bearings. The last space in a span may be 10 feet or less.

7.1.1.2 Blocking Detail

The appropriate note from Appendix D Standard Notes Structural Steel should document the theoretical blocking used at each support to establish the position of the slab relative to the steel girders. The theoretical blocking is defined as the theoretical distance between the bottom of slab and the top of web for welded girders, and as the theoretical distance between the bottom of slab and top of flange for rolled beams, disregarding any cover plates. This definition avoids confusion caused by the abrupt changes in measured blocking thicknesses at cover plates and changes in plate girder top flange thickness. Refer to Figure 7-1.

It is necessary to use blocking because the profile of the steel will vary from the theoretical profile. Without the blocking, the steel could encroach into the bottom of the slab, or the slab profile would have to deviate from the design profile. The theoretical blocking on new construction should be established to provide an actual minimum of 1-1/2” clear distance between the bottom of slab and the top of steel point. The top of steel point is defined as top of flange on welded shapes and top of any cover plate on rolled shapes, but does not include any splice plates. Included in this 1-1/2” is 1 inch used in design to compute section properties of composite beams or girders, plus an additional 1/2“ to compensate for construction tolerances. In a connection with a splice plate thickness greater than 1 inch, the reference ordinate to the splice may be reduced to insure that the splice plate will not protrude into the slab. This is addressed again in Section 7.1.1.3 Camber Diagram.

In cases where span lengths exceed 100 feet, the clear distance between the reference point and the bottom of slab should be increased from 1-1/2” to 2-1/2” to further compensate for allowable fabrication tolerances. This clear distance should be increased even more, up to 3” or 3-1/2” when continuous spans with multiple placements are likely, to compensate for construction tolerances. However, this large clear distance may result in excessive blocking of greater than 1-1/2” at supports of long span continuous bridges. Another alternative to increasing the clear distance to this level is to arbitrarily flatten the camber of long girders by up to 1 inch, by detailing the camber diagram flatter than predicted by dead load deflections (refer to Section 7.1.1.3 Camber Diagram). This will minimize excessive blocking at supports. This will also increase the mid-span blocking measured in the field, will reduce the projection of shear studs into the deck, and may result in a sag in the steel on bridges not designed on a crest curve. These limitations should be carefully analyzed by the Structural Designer to determine the best fit blocking and camber for each bridge.

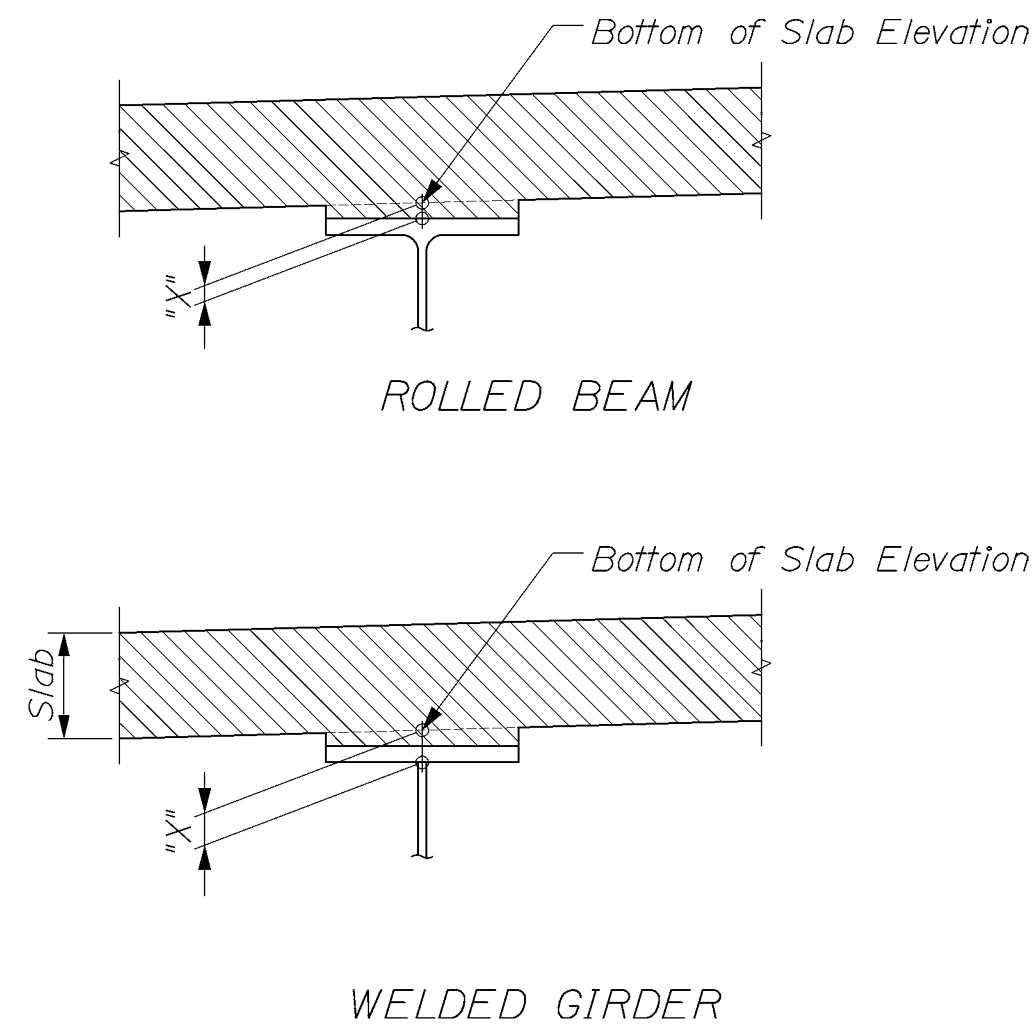


Figure 7-1 Theoretical Blocking Details

7.1.1.3 Camber Diagram

A camber diagram should be provided on the plans for all steel bridges except for simple span rolled beams. Camber is utilized to compensate for steel, fluid, and superimposed dead load deflections in order to attain the finished grades shown on the plans, and to avoid a sag in the steel or excessive blocking at mid-span. In the case of continuous span steel bridges, reference ordinates are shown from a level reference line to the beam at each support and field splice. Refer to Figure 7-2 for guidance. In accordance with design assumptions, the ordinates to the field splices are computed for the beam as if it were fabricated and erected in a weightless state. Some fabricators work with the steel lying on its side (weightless), and other fabricators work with the steel in a vertical position, resulting in some deflection from the weight of the steel. The former method is more in conformance with design assumptions, but both methods seem to work adequately. Reference ordinates to splice points may need to be adjusted as discussed in Section 7.1.1.2 Blocking Detail to insure that the splice plates have an actual clearance to the bottom of slab of at least 1 inch.

Camber ordinates for welded girders, as well as rolled beams that will be heat cambered, are computed for an equal number of spaces from support to support for simple spans, and from the centerline of support to the centerline of splice for continuous spans. The spaces should be about 10 feet. Only an ordinate at the mid-length point should be specified for rolled beams that will be cold cambered; no other ordinates are needed or specified. When specifying camber for rolled beams, use the "Limits and Tolerances for Mill Cambering" as outlined in the AISC Manual of Steel Construction to the greatest extent as possible.

If the camber required for design is greater than can be achieved with cold cambering, then a combination of cold cambering and heat cambering, or heat cambering alone can be used. Camber should not be specified between 0 inches and the lower cold cambering limits.

In any case, Natural Mill Camber should not be used as an actual reliable quantity to give the camber required by design. The limits of camber, shown in the AISC Manual of Steel Construction and in ASTM Standard A6/A6M, are straightness tolerances for acceptance of as-rolled material only. All rolled beams are straightened at the mill after rolling. However, if the desired camber is between 0 inches and the lower limits for cold cambering, the Structural Designer should provide a blocking arrangement that can tolerate 0 inches of camber, and then specify Natural Mill Camber (either up or down as necessary) on the plans.

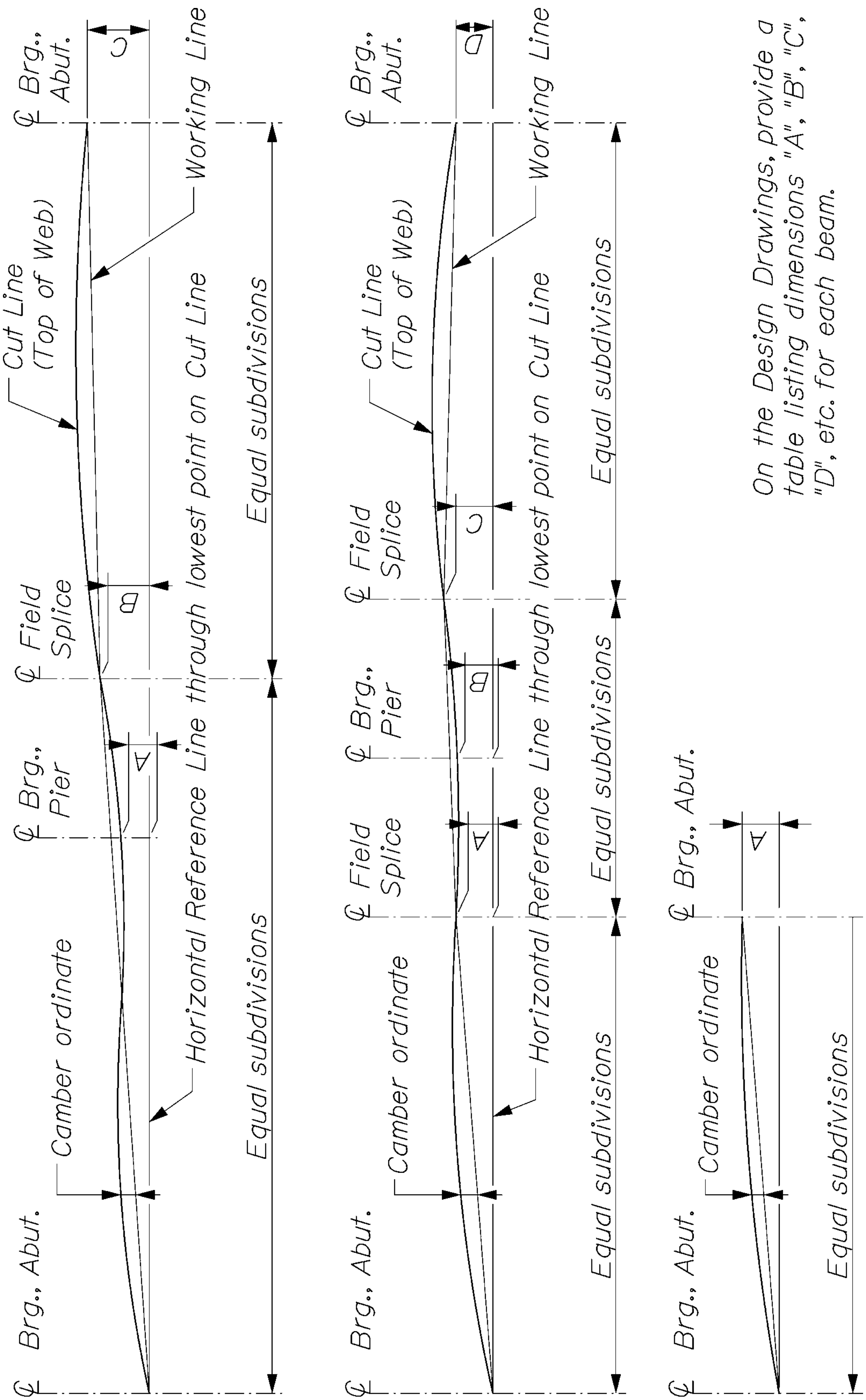


Figure 7-2 Camber Details

7.1.2 Section Properties

When designing positive moment regions in beams with composite concrete decks, section properties should be computed assuming a haunch dimension of 1 inch and an equivalent transformed width of deck.

Negative moment regions of beam and slab bridges should be designed as non-composite. However, double studs at 2 foot centers should be installed throughout the area in lieu of additional connectors as indicated in AASHTO LRFD Section 6.10 - Shear Connectors, except that studs should not be installed in areas where the allowable fatigue stresses would be exceeded if studs were installed. The section properties used in the analysis should be that of the steel section alone.

7.1.3 Constructability

Structural Designers should be familiar with constructability issues, and incorporate good practices in their designs. An excellent resource is the AASHTO/NSBA website at http://www.steelbridge.org/.

7.2 Materials

7.2.1 Structural Steel

Unpainted ASTM A709 Grade 50W steel (weathering steel) should be used for structures over water, except when such structures have open roadway joints or are located in a coastal, salt spray, or heavy industrial area. Unpainted ASTM A709 Grade 50W steel may be used for structures over railroads and highways except for narrow depressed roadways and similar situations that create tunnel-like conditions.

Weathering steel is resistant to only certain types of atmospheric corrosion. Weathering steel will not develop a protective oxide coating if it remains wet more than 60% of the time. Also, an excessive amount of contaminants in the air or the presence of salt will prevent the oxide coating from forming. For more information on this subject, refer to FHWA Technical Advisory (1989).

Painted, metallized, or galvanized ASTM A709 Grade 50 steel may be used where weathering steel is inappropriate, but only if a concrete superstructure is not a feasible alternative. Refer to Section 7.2.3 for coating requirements.

H-Piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel. Pipe piles used for bridge foundations should conform to the requirements of ASTM A252 Grade 2 or

Grade 3 with either straight or spiral butt-welded seams. Lap welded seams are not allowed.

7.2.2 Higher Strength Bridge Steel

This section will be written in the future.

7.2.3 Coatings

7.2.3.1 New Steel

In areas where the basic design criteria restricts the use of unpainted ASTM A709 Grade 50W steel, or in cases where a painted steel system is desired, a shop-applied, three-coat, zinc-rich coating system should be used with some field touch-up to repair any erection damage. The MaineDOT Standard Specifications do not address painting of structural steel; therefore, a Supplemental Specification needs to be provided in the PS&E package when a painted steel system is to be used.

If a painted steel system is desired, the Structural Designer should specify Type 1 bolts galvanized in accordance with ASTM A153. When unpainted weathering steel is used, only Type 3 bolts should be used, which are always plain.

The Contractor must select a coating system from the Northeast Protective Coating Committee (NEPCOAT) Qualified Products List (QPL). This list may be found through MaineDOT's QPL website: http://www.state.me.us/mdot/planning/products/product.htm. The Structural Designer should consult with the coatings technical resource personnel to discuss the appropriate use of the specification.

7.2.3.2 Existing Steel

When developing a field paint project, the Structural Designer must bear in mind certain environmental and safety considerations that will require the containment of the blast medium used to remove the existing coatings and blasted material. These situations may result in a decrease in underclearance, requiring that provisions for maintenance of traffic and/or sequencing of operations be described in a Special Provision. Existing utility companies should be contacted through the Utility Coordinator to determine if there is a need for protecting any utility during construction. As with new steel, a NEPCOAT pre-qualified system must be used.

7.2.3.3 Galvanizing

Galvanizing is typically used on small plates, bridge railings, joint armor, and accessories. A top coat may be specified for aesthetic considerations. Galvanizing main beams may be considered for relatively short span rolled beams, but only if weathering steel is not an option. Approval from the Engineer of Design must be obtained for galvanizing main beams.

7.2.4 Availability

There are a limited number of steel suppliers of various shapes that satisfy the Buy America requirement included in the majority of MaineDOT contracts. This may lead to issues relating to excessive lead time for particular components.

Only one mill (Nucor-Yamato) in the United States produces all W40 shapes and the heavier W36 and W30 shapes. This mill and one other (TXI Chaparral) produce the remaining W36, W33, and W30 shapes. Lead time for W shapes is from 12 to 14 weeks.

Currently, these two mills also supply all HP shapes in the United States. Lead time for these shapes is from 8 to 12 weeks.

Through the information provided in Table 7-1, the Structural Designer should confirm the required lead time for the designed shapes. This will affect the planned construction schedule and necessary advertising date.

Table 7-1 Steel Fabricators

Company	Location	Website
Nucor-Yamato	Blytheville, AR	www.nucoryamato.com
TXI Chaparral	Midlothian, TX	www.chaparralsteel.com/

7.2.5 Bolts, Nuts, and Washers

Bolted field splices and other structural applications should use 7/8” diameter ASTM A325 High Strength Bolts. ASTM A490 bolts should not be used unless approved by the Engineer of Design.

7.2.6 Welds

Welds should be designed in accordance with the applicable AASHTO LRFD standard. Welds for cover plates, plate girders, and bearing stiffeners should be shown on the plans.

7.2.7 Field Splices

Bolted field splices should be designed as slip-critical. Uncoated weathering steel should be designed for Class B (slip coefficient 0.55) faying surfaces. For painted surfaces, refer to the approved coating list for the appropriate slip coefficient. The Structural Designer should not indicate the thickness of filler plates for splices on the plans. Allowable construction tolerances may affect these thicknesses, which are easily adjusted by the fabricator.

7.3 Economy

The Structural Designer should keep in mind that a design utilizing the least material is not necessarily the most economical design, since material cost represents only about one third of the total fabricated cost of a welded girder. The bulk of the cost lies in fabrication, shop fit-up, delivery, and field erection. Simplification and repetition of details, reduction of fabrication and welding operations, and ease of handling and erection are often better means to achieve cost savings.

As a general rule, unstiffened webs should be used for depths of 50 inches and below. For web depths over 50 inches, unstiffened or partially stiffened webs should be used. To determine an optimum number of intermediate stiffeners for a partially stiffened web, a cost of $150 to $200 per stiffener can be assumed.

At least 800 pounds of flange material must be saved to justify the introduction of a shop flange splice. Normally, the most economical design results when the flange sizes are carried through the entire positive moment section. It may or may not be cost effective to transition flange sizes in the negative moment section. If a flange transition is specified, the thickness and not the width should be varied, since a uniform flange width allows welding of an entire slab of steel rather than individual pieces.

The number of beams used in a structure should be determined by taking into account the following:

- Traffic may need to be maintained over the structure during a future redecking. The number and spacing of the beams should allow for future staged construction of a new deck.
- No structure should have less than four beams.
- The maximum beam spacing is limited to 15 feet.
- A cost comparison should be done between the different number of beams under consideration, using the procedure discussed in Section 2.2.7 Cost Comparison for Number of Beams. Included in the cost analysis should be any increase

in grade required by greater beam depths. If there is no appreciable cost difference among the different numbers of beams being considered, the greater number should be used to provide increased redundancy in the structure.

7.4 Design Requirements

7.4.1 Welded Girders

To facilitate handling, the unsupported length of member/compression flange width (L/b) ratio preferably should not exceed 90. If using an L/b ratio of 90 results in an uneconomical design, a ratio of up to 100 may be used with permission from the Engineer of Design.

Flanges for welded beams should be proportioned to give a flange width/flange thickness (b/t) ratio between 12 and 20 with a preferred ratio of 16. The minimum flange width and thickness should be 12 inches and 3/4”, respectively. These limits are set to avoid either a very thin wide flange that will distort when welded to the web, or a very thick narrow flange that would be uneconomical to purchase or laterally unstable.

The Structural Designer must verify that the design does not incorporate the need for transverse butt-welded joints in areas where such are not allowed. Restrictions on the location of these splices are given in Appendix D Standard Notes Structural Steel. Locations of transverse butt-welded splices for flanges and webs are typically not shown on the contract drawings, but are located by the fabricator. The Structural Designer must also verify that the stress range in areas where the fabricator is allowed to make transverse butt-welded splices meet AASHTO LRFD fatigue criteria.

7.4.2 Rolled Beams

The use of cover plates should be avoided on rolled beams. Increasing the size of the rolled beam itself is usually more cost effective, considering the expensive shop fit-up and reduced fatigue life of cover plate attachment.

> *Commentary: Most fabricators believe that when a cover plate is required, a welded plate girder will be more cost-effective. The camber can be cut into the web (eliminating heat-cambering) and the flanges can be sized exactly to meet design requirements.*

If a cover plate is required, it should be designed to be a minimum of 1/2” thick and no thicker than the flange to which it is welded. In order to facilitate welding, cover plates should be smaller than the flange width by a minimum of twice the required weld size plus 1/8”. For example, for a 12 inch wide flange and cover plate requiring a 5/16" fillet weld, the maximum cover plate width would be 11-1/4". Refer to Example 7-1.

Example 7-1 Maximum Cover Plate Width

Given: Flange Width = 12 inch
Required Fillet Weld = 5/16"

$$12 \cdot in - \left(2 \times \frac{5}{16} \cdot in\right) - \frac{1}{8} \cdot in = 11\frac{1}{4} \cdot in$$

7.4.3 Fabrication Considerations

The Designer should consult with the Bridge Quality Assurance Team for information pertaining to fabrication requirements. Some specific issues of interest may be weldment design, fabrication tolerances, shipping limitations and cost, fabricator limitations, appropriate material usage, and protective coatings.

Fracture critical members should be avoided, if possible. These members are non-redundant, and require an increase of 5% on the load side of the equation. They may need more frequent maintenance inspections. They may also be more expensive due to welding procedure requirements, base metal and weld testing, and construction inspection.

Weight considerations of the individual components should be carefully considered. By limiting maximum weight of fabricated continuous girder units, more steel fabrication shops can compete for the work. A brief review of the lifting capacity potential of fabrication shops may be prudent.

The Structural Designer should provide an optional field splice for simple span bridges exceeding 120 feet, or for even shorter spans where there is a challenging delivery route or constricted bridge site. A complete design of the optional splice should be shown on the plans.

On single span rolled beam structures with a camber of 3 inches or greater, fabricators should be given the option of fabricating welded plate girders in place of the rolled beams shown on the plans. The fabricator is responsible for determining the plate thicknesses based upon the depth and moment of inertia of the rolled section. This should be shown on the plans using the note given in Appendix D Standard Notes Structural Steel.

A beam stress diagram should be included on the plans of all continuous steel structures. It should indicate where the flanges are subject to tensile stress or stress reversal.

On rehabilitation projects that may require welding to existing steel, the Structural Designer must consider the weldability of the existing base metal. The Bridge Quality Assurance Team is an excellent resource for this information.

7.4.4 Diaphragms and Cross Frames

Diaphragms must be located within 5 feet of each point of dead load contraflexure on multiple span continuous structures with more than 250 yd^3 of deck concrete. These diaphragms will allow for the construction of a slab joint. Diaphragms at these locations must be marked with an asterisk (*) and the appropriate note from Appendix D Standard Notes Structural Steel must be included on the plans. Cross frames and diaphragms should be specified on the plans in accordance with Standard Details 504 (15-22).

7.4.4.1 Instructions for Use of Standard Details

When selecting a diaphragm or cross frame, the span is defined as the distance between beams as measured along the diaphragm or cross frame. For beam depths and/or spans not covered in the following tables, the Structural Designer may make exceptions to the above limitations in special cases, providing that the design criteria are adequately satisfied. For cases where the design criteria are not adequately satisfied, the Structural Designer should design a special diaphragm or cross frame meeting the design requirements.

A. Slab Ends and Slab Joints

The diaphragms intended for use at slab ends and slab joints are listed in Table 7-2.

Table 7-2 End Diaphragms

	Beam Type	**Beam Depth**	**Maximum Span (ft)**
Type A1	Rolled	Any	12
Type A2	Rolled	Any	15
Type B	Welded	3’-0” to 3’-8”	20
Type C1	Welded	3’-8” to 3’-11”	15
Type C2	Welded	3’-11” to 4’-6”	20
Type D	Welded	4’-6” to 9’-0”	15

Commentary: The maximum span for all types except Type D is controlled by the moment strength of the member. The members were assumed to be simply supported beams loaded with two 20 k wheel loads plus impact 4 feet apart. The moment capacity of skewed diaphragms can be reassessed with the distance between wheels increased as a function of the skew angle, if warranted by special case. Two rows of bolts are considered necessary to carry moment induced by lateral loads and to ensure adequate stability during construction. The maximum span for Type D is controlled by the l/r ratio of the bottom lateral (l/r $\leq$ 140). The beam depths for all types except Type D are sized to meet the depth requirements specified in AASHTO LRFD Section 6.7.4 - Diaphragms and Cross Frames.

B. Intermediate Locations

The cross frames intended for intermediate locations are listed in Table 7-3. To ensure that the intersection point of Types G or J does not act like a hinge during construction, Type H or K shall be used in the exterior bays.

Table 7-3 Intermediate Cross Frames

	Beam Type	Beam Depth	Maximum Span (ft)
Type E	Rolled	Any	10'-6"
Type F	Rolled	Any	12'
Type G	Welded	3'-0" to 3'-8"	13'-6"
Type H	Welded	3'-0" to 3'-8"	15'
Type J	Welded	3'-8" to 5'-0"	15'
Type K	Welded	3'-8" to 5'-0"	15'
Type L	Welded	5'-0" to 9'-0"	15'
Type M	Welded	5'-0" to 9'-0"	15'

Commentary: The maximum span for these types is controlled by the l/r ratio of the members (l/r $\leq$ 140). The bolts in each connection are limited to the number considered to be adequate to transfer lateral loads to the slab and to distribute vertical loads during construction to insure stability.

7.4.5 Stiffeners and Diaphragm Connection Plates

7.4.5.1 General

For rolled beam designs, bearing stiffeners at end bearings and intermediate stiffeners should be used only when required by AASHTO

LRFD. At bearings other than end bearings, bearing stiffeners should always be used whether required by AASHTO LRFD or not.

For welded girder designs, bearing stiffeners should be used at all bearings. At bearings other than end bearings, a minimum of 2 intermediate stiffeners should be used at either side of the bearing stiffeners for a total of 4. Additional intermediate stiffeners should be used where required by AASHTO LRFD. On exterior beams, the intermediate stiffeners should be placed on the interior face of the web.

At bearings other than end bearings, on the fascia side of both rolled and welded exterior beams, either a single bearing stiffener placed at the centerline of bearing or two bearing stiffeners placed symmetrically on either side of the centerline of bearing should be used, at the option of the Structural Designer. If two bearing stiffeners are used, they must be a minimum of 8 inches apart to allow adequate access for welding. On the interior face of exterior beams, a stiffener layout as shown in Detail D or E in Figure 7-4 should be used as applicable.

7.4.5.2 Effect of Skew

Intermediate diaphragms and corresponding connection plates should be skewed on bridges with a 20° skew or less. On bridges with more than a 20° skew, the intermediate diaphragms and connection plates should be kept normal to the beams and arranged in a staggered pattern.

End bearing stiffeners should be skewed on bridges with a 30° skew or less, and used as diaphragm connection plates (refer to Detail A, Figure 7-3). On bridges with skews between 30° and 45°, the end bearing stiffeners should be kept normal to the beams with skewed connection plates for the end diaphragm connections (refer to Detail B, Figure 7-3). When the skew exceeds 45°, the end bearing stiffeners should be kept normal to the beams with bent connection plates for the end diaphragm connections (refer to Detail C, Figure 7-3).

When used as diaphragm connection plates, bearing stiffeners at all bearings other than end bearings should be skewed on bridges with a 45° skew or less (refer to Detail D, Figure 7-4). At all bearings other than end bearings on bridges with skews greater than 45°, two bearing stiffeners should be placed normal to the beams, and two bent stiffener plates used for the diaphragm connections (refer to Detail E, Figure 7-4).

Any bearing stiffener not used as a diaphragm connection plate should be kept normal to the beam centerline.

Any dimension shown as indeterminate on Details A through E (e.g. “6 inches minimum”, “as required”, etc.) must be determined by the Structural Designer and shown on the design drawings.

Note 7 in Standard Detail 504 (22) calls for the connection plates and stiffeners to be welded to the web with a fillet weld on both sides. If this weld arrangement is used, the dimensions shown as 6 inches minimum on Detail D and E in Figure 7-4 must be detailed on the design drawings as not less than 8 inches. If the dimension must be between 8 inches and 6 inches, the weld arrangement shown on Detail D must be detailed on the design drawings. If the skew is greater than 45°, the weld arrangements as shown on Detail C, Figure 7-3 and Detail E, Figure 7-4 must be used.

On beams with wide flanges and large skews, the Structural Designer should consider increasing the width of the bearing sole plate so that no part of a bearing stiffener group (e.g. Detail D) is more than the flange plate thickness outside the sole plate.

7.4.6 Handhold Bars

Handhold bars should be used on all girder structures with web depths of at least 6 feet that are not easily accessible for inspection by the under-bridge crane. The crane can reach a maximum of 25 feet under a bridge deck. The handhold bars should be placed on both sides of interior girders and on the inside only of exterior girders. Refer to Standard Details 504 (23-24) for more information. The connection at girder ends should be made 10 feet plus or minus from the centerline of bearings. This will allow the web to resist the tension force and discourage the public from climbing the girders.

Stiffeners should be a minimum of 1/2” thick where the handhold bar is terminated or spliced. Termination and splicing of handhold bars should occur only at stiffeners.

> *Commentary: The minimum thickness of the stiffener requirement is based on tests conducted at the University of Maine. Testing showed that thinner plates would be stressed beyond yield.*

The angle clip detail is unable to resist or transmit the design forces. This may require adding stiffeners to both sides of the web, even if stiffeners are required on only one side.

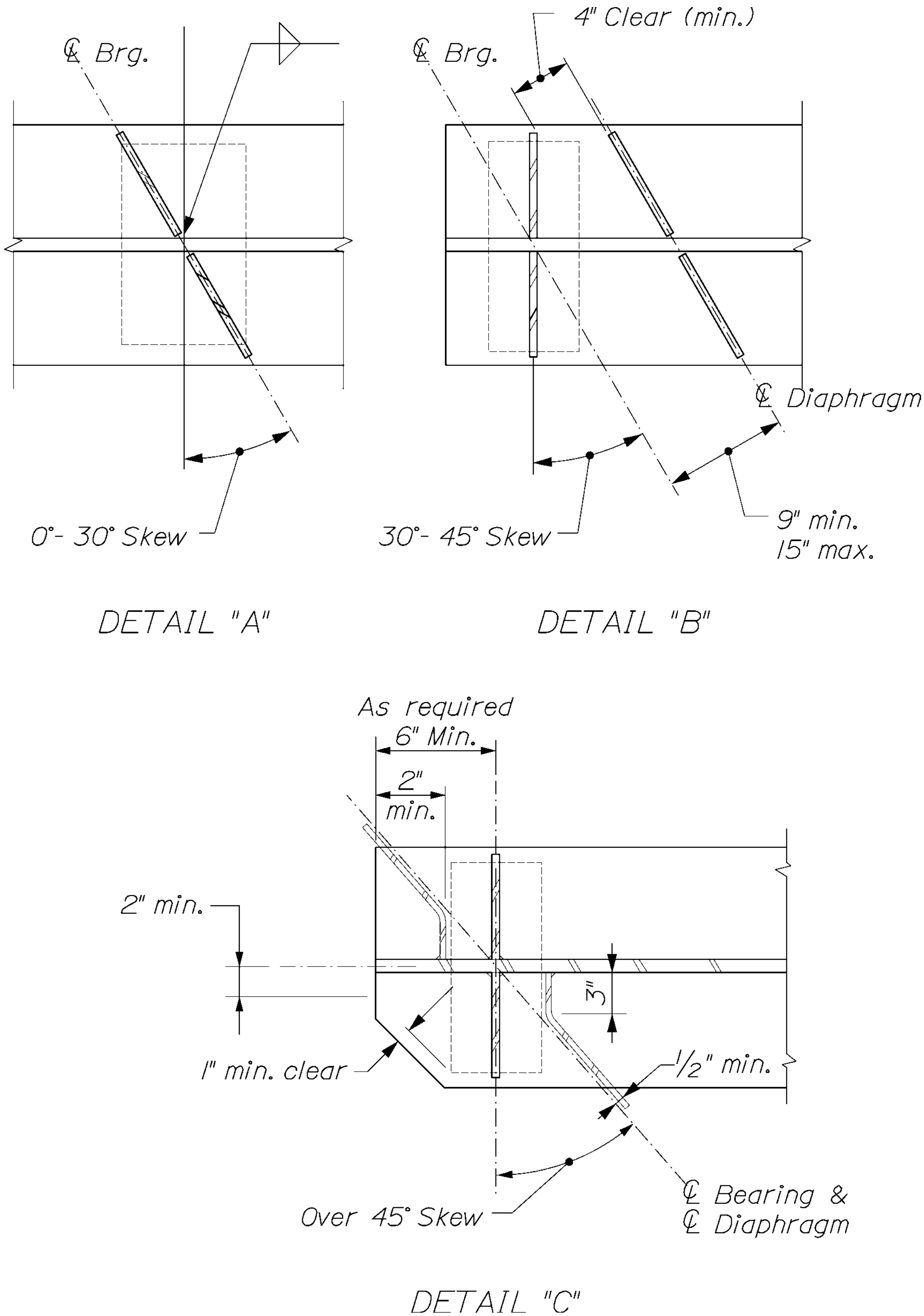
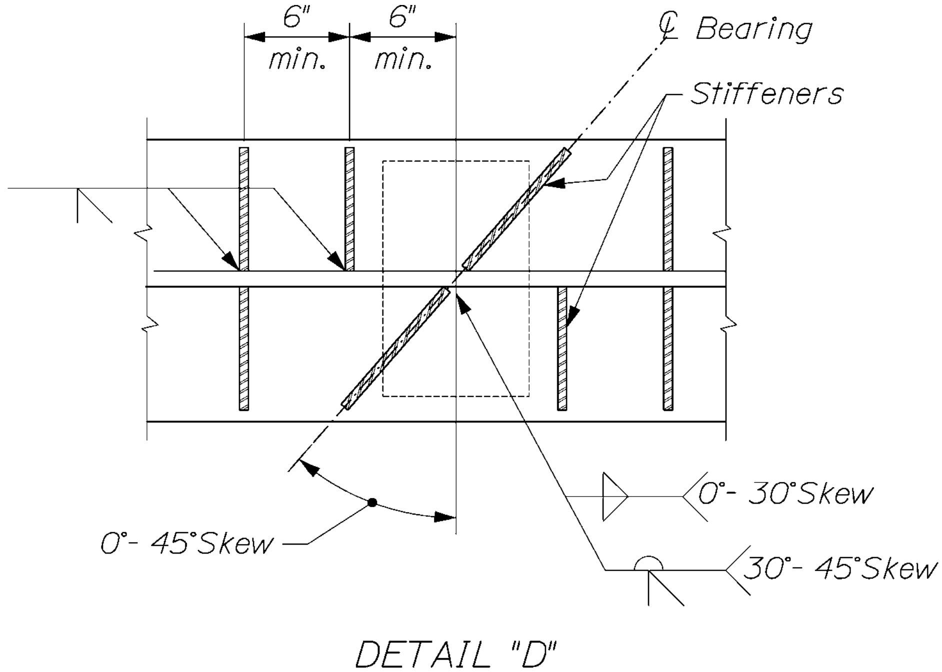


Figure 7-3 Stiffener/Connections Plate Details A Through C



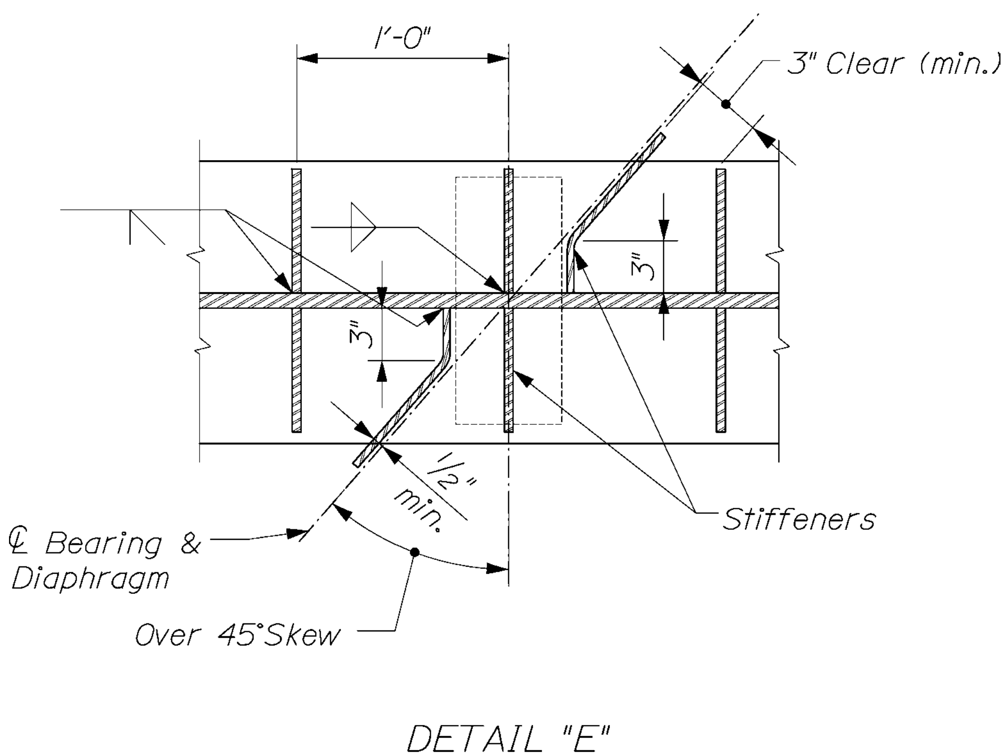


Figure 7-4 Stiffener/Connections Plate Details D and E

7.4.7 Slab Overhang Limits

In order to prevent excessive torsional deflections in beams during placement of the deck concrete, the slab overhang should not exceed the applicable value from Table 7-4. For overhangs exceeding the limits of Table 7-4, a torsional analysis of the exterior beam should be completed. Torsional analysis of the exterior beam should also be completed on all deck replacement and widening projects. As part of the shop drawing submittal, the Bridge Quality Assurance Team will complete a torsional analysis of the exterior beam for construction loading.

Table 7-4 Slab Overhang Limits

Beam Spacing	Maximum Overhang is the lesser of:
Less than 9’-0”	3’-0” or depth of beam
9’-0” to 10’-6”	1/3 of the beam spacing or depth of beam
Greater than 10’-6”	3’-6” or depth of beam

Note: Table 7-4 is for use on straight bridges. Maximum overhang for bridges with curved fascias is limited to 3’-6”, or depth of beam plus 6”, whichever is less.

7.4.8 Composite Design

All new steel girder bridges should be designed as composite structures. Non-composite section properties should be used for negative moment sections without transforming the rebar in the deck.

References

AASHTO, 1978, and Interims through 1986, *Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members*

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