CHAPTER 6 – STEEL STRUCTURES

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LIST OF FIGURES

6.4—MATERIALS

6.4.1—Structural Steels

The following shall supplement *A6.4.1*.

From Table 6.4.1-1, specify only Grades 36, 50, 50W, HPS 50W, and HPS 70W. Use of Grade HPS 100W shall be approved by the Bridge Design Engineer Administrator.

Unpainted weathering steel shall be used wherever possible. Use of painted steel shall be approved by the Bridge Design Engineer Administrator.

For plate girders, each field section shall use the same steel classification/yield strength for flanges and web, but the steel classification/yield strength may differ from field section to field section. Use of hybrid girders with different steel classification/yield strength for flanges and web shall be approved by the Bridge Design Engineer Administrator.

C6.4.1

The following shall supplement *AC6.4.1*.

Hybrid girders have been used in the past; however, steel pricing and design and fabrication considerations do not warrant general use. In cases where use is warranted due to unusual circumstances, the designer shall obtain prior approval from the Bridge Design Engineer Administrator.

6.4.3—Bolts, Nuts, and Washers

6.4.3.1—Bolts

The following shall supplement *A6.4.3.1*.

ASTM A490 bolts and bolt diameters greater than 1.125 inches shall not be used unless approved by the Bridge Design Engineer Administrator.

Use ASTM A325, Type 1 bolts for painted connections and Type 3 bolts for unpainted weathering steel connections.

If galvanization is required for the members, all elements of the connection except A490 bolts shall be galvanized. A490 bolts shall not be galvanized and shall be prepared in accordance with *Standard Specifications* when they are specified to connect galvanized parts.

Only one size bolt diameter shall be used for all structural connections of the same type on any specific project. A325 0.875 inch diameter is the most commonly used bolt, which shall be specified

C6.4.3.1

The following shall supplement *AC6.4.3.1*.

ASTM A490 bolts have greater carbon content and are not as ductile as ASTM A325 bolts. The limitations on bolt type and bolt diameter are based on experience that connections should be able to be designed without the need for higher strengths and larger bolt diameters.

Specifying the use of the same bolt diameter in structural connections of the same type on a project reduces the probability of bolts being placed incorrectly in critical connections. The whenever possible.

Bolts, nuts, washers, and appurtenant items shall also be in accordance with the *Standard Specifications*.

additional material and labor costs are insignificant.

6.4.3.5—Load Indicator Devices

The following shall supplement *A6.4.3.5.*

All bolting for structural steel bridge members shall utilize Direct Tension Indicator (DTI) for installation. With the approval of the Bridge Design Engineer Administrator, other load indicator devices may be considered when rehabilitating connections of existing members where DTI may not be the best installation method. Load indicator devices shall be specified on plans.

C6.4.3.5

The following shall supplement *AC6.4.3.5.*

DTI washers are an excellent alternative to turn-of-the-nut and calibrated torque wrench methods, which are also satisfactory when performed properly; DTI washers are considered easier with respect to installation and less prone to inspection errors.

One example where DTI may not be the best installation method is in the case of rehabilitation of riveted connections where the rivet/bolt holes may be oversized. However, DTIs can still be used effectively in this case by installing a structural plate washer between the hole and the DTI, provided that the plate washer is rigid and does not deform when the bolt is tensioned.

Should the contractor propose to use DTIs which incorporate a self-indicating feature (“squirter”) to signal sufficient bump compression, the engineer may permit their use, provided that pre-installation verification testing, installation, and inspection requirements are all performed in accordance with the *Louisiana Standard Specifications for Roads and Bridges* (latest edition), including using actual bump compression and feeler gage measurements to verify installation tension conformance.

6.6—FATIGUE AND FRACTURE CONSIDERATIONS

6.6.2—Fracture

The following shall supplement *A6.6.2.*

All main-load-carrying structural steel bridge members or portions of members subject to tension or stress reversal require longitudinal Charpy V-notch (CVN) testing.

The locations and lengths of those members or portions of members subject to tension or stress reversal where CVN testing is required shall be clearly stated on the contract plans.

All Fracture Critical Members (FCM) shall be clearly identified on contract plans.

6.7—GENERAL DIMENSION AND DETAIL REQUIREMENTS

6.7.2—Dead Load Camber

The following shall supplement *A6.7.2.*

Provide concrete deck placement sequences, deflections, and girder camber diagrams in contract plans.

The contract plans shall include the following note:

"Alteration of the plan specified deck placement sequence will require the approval of the Engineer of Record. If approved, the contractor shall either hire the original Engineer of Record or a new engineer to prepare the changes to the original design and plan details that are affected by the new deck placement sequences. In the case of hiring a new engineer, the qualifications of the engineer and supporting staff shall be submitted for approval. The revisions to the design and plan details shall be prepared in accordance with LADOTD QC/QA requirements and shall be submitted for verification. The contractor shall be responsible for all associated expenses."

For continuous span units, concrete deck in negative moment regions shall be poured last after a minimum of 7 days from concrete deck pouring of positive moment regions.

Individual deck pours shall be completed within 4 hours with a maximum concrete pouring rate of 60 cubic yard per hour. Higher rates shall be approved by the Bridge Design Engineer Administrator.

For the determination of deflections and camber, a design analysis using a grid, 3-D or finite element method shall be used for superstructure spans with skews greater than 20 degrees, for curved spans, flared spans and for spans with overhangs greater than 6 ft from the centerline of exterior girders.

C6.7.2

The following shall supplement *AC6.7.2.*

Deflections of spans with large skews and of curved spans are more sensitive to large center-to-center girder spacing and warrant utilizing refined analysis.

6.7.3—Minimum Thickness of Steel

The following shall supplement *A6.7.3.*

The minimum thickness of webs for plate girders and box girders shall be 0.5 inch. Web thickness shall be designated in 0.0625 inch increments up to 1.00 inch, inclusive. Web thickness greater than 1.00 inch shall be specified in 0.25 inch increments.

Web plate heights shall be specified in 1.00 inch increments.

The minimum thickness of flanges for plate girders and for top flanges of box girders shall be 0.75 inch. The minimum thickness of the bottom flange for box girders shall be 0.50 inch.

Flange plate widths for plate girders and for top flanges of box girders shall be a minimum of 12.00 inches. All flange plate widths shall be specified in 1.00 inch increments. Flange plate widths shall be designed and detailed as constant within individual field sections of girders (i.e., between field splice locations) when practical while varying the thickness.

The minimum thickness of stiffener plates shall be 0.50 inch.

The design and detailing of girder webs and flanges shall minimize differences in plate thicknesses such that a structural steel fabricator is not required to order small quantities of material.

C6.7.3

The following shall supplement *AC6.7.3.*

The minimum thickness and width dimensions given are selected in order to reduce distortion caused by welding and by heat-treatment for curving during fabrication. The minimum top flange plate width of 12.0 inches is to improve girder stiffness and lateral bracing during handling, shipping and erection.

For general reference in selecting plate dimensions during design and detailing, the availability of plate sizes varies from individual steel mills. The minimum width is typically 48 inches and the maximum width is typically 144 inches. The majority of steel fabricators order maximum width plates appropriate for a project and perform cutting in the shop for economy.

6.7.4—Diaphragms and Cross-Frames

6.7.4.1—General

The following supplements *A6.7.4.1*.

External diaphragms connecting adjacent plate or box girders shall be of either "X-frame" or "K-frame" configurations. All internal diaphragms for box girders shall be of "K-frame" configuration.

External diaphragms shall be bolted to girders at stiffener locations. Internal diaphragms for box girders may be bolted or welded to stiffeners during shop fabrication.

Detail diaphragms with bolted connections to stiffeners without separate connection plates.

C6.7.4.1

The following shall supplement *AC6.7.4.1*.

"K-frames" are best for internal diaphragms in box girders, as they allow better access for inspection. "X-frame" diaphragms, in general, are easier to fabricate and to connect during erection as opposed to "K-frames".

Problems can develop in stage construction as a result of differences in elevation between the Stage 1 deflected position and the undeflected position of the Stage 2 members before pouring the Stage 2 concrete. Deck alignment between Stage 1 and Stage 2 and crossframe connections between Stage 1 and Stage 2 girders require special considerations. Successfully implemented strategies to address this potential problem include the use of:

- At least three girders in either or both stages to reduce transverse movement during deck pour,
- A closure or construction pour between the two stages, or
- Only a top and bottom strut connecting girders between the two stages. Add cross bracing after the deck pour if deemed necessary.

Indicate web details clearly on the plans for either vertical no-load, steel dead-load, or full dead-load condition.

Where there are differential deflections between girders at the ends of crossframe connections, the girders will rotate transversely as (1) the dead load of the steel is applied, and (2) the concrete dead load is applied. This condition most commonly occurs in curved bridges and skewed bridges. This condition will affect the proper fit of subsections, field splices, and crossframe connections, and should be addressed by the designer.

Refer to *Standard Specifications* for shop assembly methods and other fabrication and erection requirements. Using shop assembly methods other than the ones specified in the *Standard Specifications* require special provisions. The primary reason for shop assembly is to ensure correct alignment for girder field splices.

For curved I-girders, crossframes are to be fabricated to fit the no-load condition. During field erection, girder segments will need to be adjusted or supported to make fit-up possible. This is not unreasonable, since curved girders are not self-supporting before crossframes are in place; however, the method results in out-of-plumb girders. For most cases, making theoretical compensation to arrive at plumb in final condition is not justified.

Highly skewed girders present difficult fit-up conditions. Setting screeds is also complicated because of differential deflections between neighboring girders. Design of crossframes and pier diaphragms must take into account twist and rotation of webs during construction. Often, slotted holes for crossframe connections can be used to allow settlement without undue web distortion. This situation should be carefully studied by grid or finite element analysis to determine amount and type of movement anticipated during construction. Details should be consistent. Unlike curved girders rotating away from plumb at midspan, girder webs for skewed construction should be kept plumb at piers.

Refer to AASHTO and National Steel Bridge Alliance (NSBA) Collaboration standards and documents for more information on steel girder design and detailing, fabrication and erection. These documents are posted on AISC/NSBA website and available for download.

6.7.5—Lateral Bracing

6.7.5.1—General

The following shall supplement *A6.7.5.1*.

Where lateral gusset plates are fillet welded to girder webs, the fatigue stress range in the girder is limited to Category E without transition radius or Category D with carefully made transition radius.

C6.7.5.1

The following shall supplement *AC6.7.5.1*.

In regions of high tension stress range, consider bolting gusset plates to the girder web.

6.7.5.3—Tub Section Members

The following supplements *A6.7.5.3*.

Box girders shall have an internal lateral bracing system inside the section, located as close as possible to the top flange without interfering with stay-in-place metal forms. Single diagonal members shall be used rather than "X-frame" diagonals, unless otherwise required by design.

C6.7.5.3

The following shall supplement *AC6.7.5.3*.

Single diagonal members that are spaced in opposite directions along the length of the box girder interior have proven to be satisfactory. "X-diagonal" frames are expensive and unnecessary unless extremely tight horizontal curvature is a factor.

6.7.7—Heat-Curved Rolled Beams and Welded Plate Girders

6.7.7.2—Minimum Radius of Curvature

The following shall supplement *A6.7.7.2*.

Curved welded plate girders shall be used for bridges having a horizontal radius of 1,200 feet or less. If rolled beams are desired, approval by the Bridge Design Engineer Administrator is required.

C6.7.7.2

The following shall supplement *AC6.7.7.2*.

Fabricators do not routinely heat-curve standard shapes. Consider 1,200 feet as a minimum horizontal radius for rolled beams.

6.7.7.3—Camber

The following shall supplement *A6.7.7.3*

Permanent girder deflections shall be shown in the contract plans in the form of camber diagrams and tables.

C6.7.7.3

The following shall supplement *AC6.7.7.3*

Camber curves shall be shown in the contract plans. Dimensions shall be given at tenth points (twentieth points for spans greater than 200 feet) and crossframe locations.

In order to place bearing stiffeners in the vertical position after bridge deck placement, it is necessary to show expected girder rotations at piers.

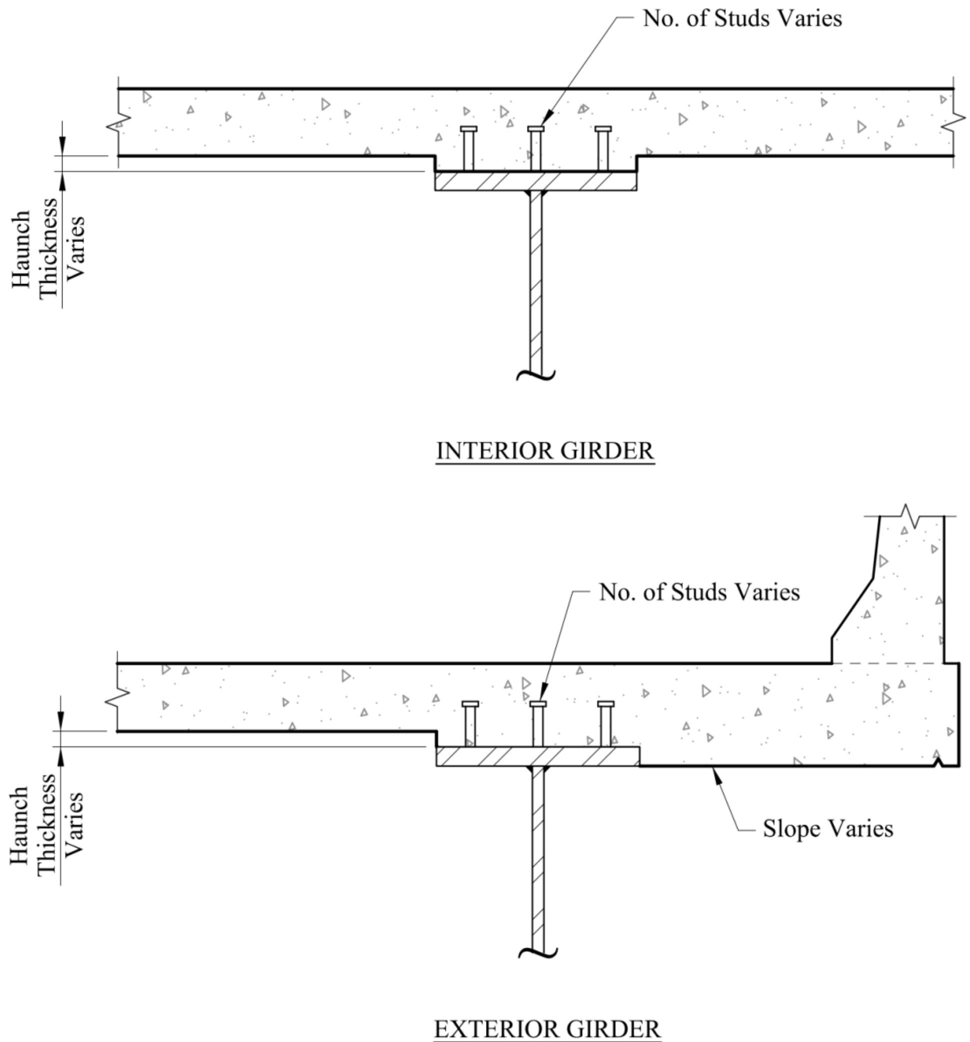
Since fabricated camber and girder erection have inherent variability, bridge deck form height is adjusted after steel has been set if necessary. Although a constant distance from top of web to top of deck is assumed, this distance will vary along the girders.

6.10—I-SECTION FLEXURAL MEMBERS

6.10.1—General

6.10.1.1—Composite Sections

Composite section shall be used for all steel girder designs unless noted in *D6.10.1.2*. The top flange in composite section shall be placed directly under the haunch as shown in figure below.



Composite Section Haunch Detail

6.10.1.1.1—Stresses

6.10.1.1.1a—Sequence of Loading

The following shall supplement *A6.10.1.1.1a.*

Design and detailing of shored construction shall not be used by the designer unless prior approval by the Bridge Design Engineer Administrator is granted.

C6.10.1.1.1a

The following shall supplement *AC6.10.1.1.1a.*

The LRFD Code states in part "...*While shored construction is permitted according to these provisions, its use is not recommended*..." and discusses reasons as to why. The term "shored construction", as used here, applies to shoring of steel girders during construction, such that the CIP concrete deck slab/girder is considered as a composite section resisting the weight of the steel girders and CIP deck when the shoring is removed after curing of the deck.

LADOTD concurs that this practice is not recommended and would allow shored permanent construction only in unique circumstances. Other states have utilized shoring of this type with prestressed concrete girders with success; however, the importance of the shoring being placed and maintained at critical loading and elevation levels is such that construction can be complicated with no easy method for correction if problems occur.

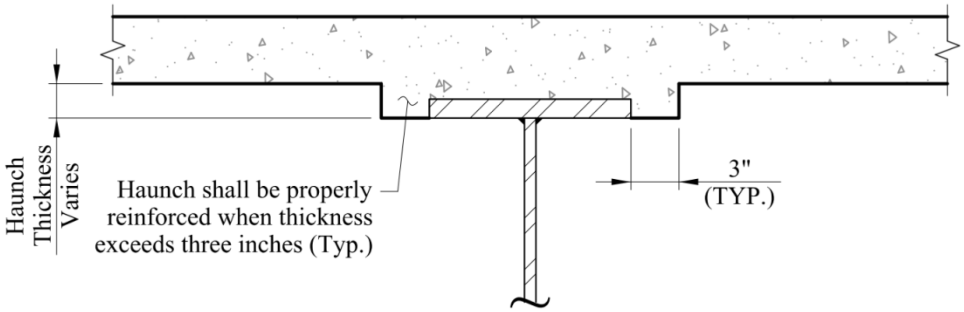
The temporary shoring of steel plate girders and box girders during construction until splice plates are connected is not applicable to this sub-article and is an accepted practice.

6.10.1.2—Noncomposite Sections

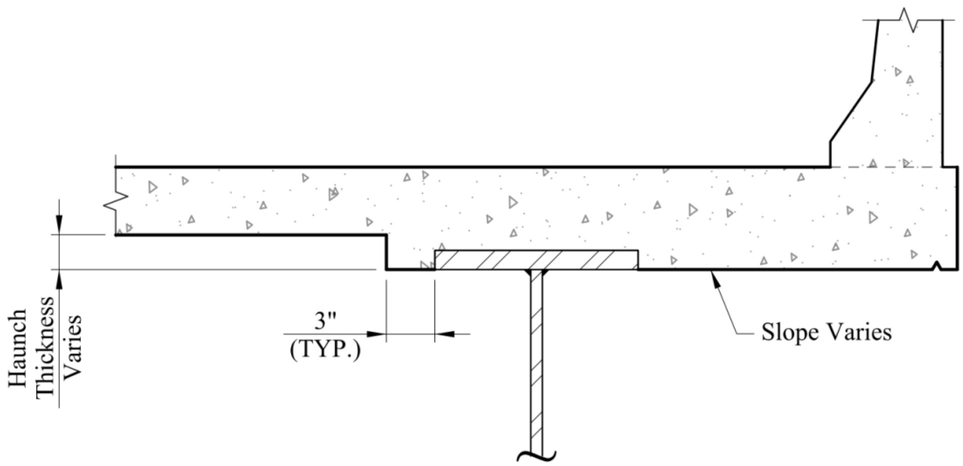
The following shall supplement A6.10.1.2.

Noncomposite section is not permitted in positive moment region, but it is allowed in negative moment region with the approval of the Bridge Design Engineer Administrator.

When noncomposite section is used, the top flange of girders shall be encased in concrete haunch as shown in the figure below.



INTERIOR GIRDER



EXTERIOR GIRDER

Noncomposite Section Haunch Detail

6.10.1.6 - Flange stresses and member bending moments

C6.10.1.6

The following shall supplement *AC6.10.1.6.*

Lateral bending stresses in discretely braced flanges shall be determined according to *A6.10.1.6* and *A6.10.3.2.2*. The top flange that is fully encased in concrete may be designed to be continuously braced by concrete. In this case, the lateral bending stresses shall be taken equal to zero. In addition to the simplified analysis procedure, the lateral bracing requirements can be reduced since the top flange is laterally restrained by the slab against live load effects.

6.10.10—Shear Connectors

6.10.10.1—General

The following shall supplement *A6.10.10.1*.

Shear connectors shall be either 3/4 inch or 7/8 inch diameter end-welded studs. Stud shear connector welding shall comply with the requirements of *ANSI/AASHTO/AWS D1.5, Bridge Welding Code, Section 7 "Stud Welding"*.

Other types of shear connectors shall not be used for new construction. For rehabilitation work, other types of shear connectors may be allowed with the approval of the Bridge Design Engineer Administrator.

The attachment method of shear connectors to steel girders (field weld or shop weld) should be contractors' choice; however, designers shall ensure that attachment methods for all shear connectors are shown on steel girder shop drawings.

C6.10.10.1

The following shall supplement *AC6.10.10.1*.

6.10.11—Stiffeners

6.10.11.1—Transverse Stiffeners

6.10.11.1.1—General

The following shall supplement *A6.10.11.1.1*.

Stiffeners used as connection plates shall be fillet welded to the compression flange on each side. Bolting is preferred for connecting to tension flanges or flanges subject to stress reversal, but welding is permitted provided fatigue is considered in the design.

Stiffeners not used as connection plates on straight girders shall be fillet welded to the compression flange on each side and shall be cut back from the tension flange and flanges subject to stress reversal.

C6.10.11.1.1

The following shall supplement *AC6.10.11.1.1*.

Transverse stiffeners used only as web shear stiffeners and located between stiffeners used as connection plates for cross frames or diaphragms are sometimes referred to as "intermediate stiffeners."

Welding of stiffeners to flanges is less expensive than bolting; however, any welding to a tension flange or flange subject to stress reversal results in reduction of allowable stresses and in fatigue considerations that must be accounted for in design. Potential future problems, due to inferior workmanship and inspection, even though slight in nature, warrant the slight increase in initial cost of using bolted connections at tension flanges.

Regarding design, for girder web depths up to 6 ft., the most economical design is to use intermediate stiffeners to satisfy shear requirements. It is recommended that at least 18 lbs. of web steel be saved for every one lb. of transverse stiffener steel that would be added to the girder. Optimum girder design is more sensitive to web plate thickness and number of stiffeners required rather than to the depth of the web plate. These are good rules of thumb, but may change over time. Designers should check AASHTO and National Steel Bridge Alliance (NSBA) publications for most updated information. These publications are posted on AISC/NSBA website and available for download.

6.10.11.2—Bearing Stiffeners

6.10.11.2.1—General

The following shall supplement *A6.10.11.2.1.*

Bearing stiffeners shall be designed to be vertical under full final dead loads which have been considered in the camber calculations.

6.10.11.3—Longitudinal Stiffeners

6.10.11.3.1—General

The following shall supplement *A6.10.11.3.1.*

Longitudinal stiffeners shall not be used for girders with web depth less than 8 feet unless approved by the Bridge Design Engineer Administrator.

C6.10.11.3.1

The following shall supplement *AC6.10.11.3.1.*

Use of longitudinal stiffeners for the design of girders with web depths less than 8 feet is not usually justified due to the direct labor cost for fit-up and welding of materials. It is usually more economical to design for thicker web plates than to use longitudinal stiffeners.

Longitudinally stiffened girders may become economical when girder web depths exceed 8 ft.; their use may be justified on deep, haunched girders or on widening of an existing structure.

6.11—BOX-SECTION FLEXURAL MEMBERS

6.11.1—General

6.11.1.2—Bearings

C6.11.1.2

The following shall supplement *AC6.11.1.2.*

The twisting of box girders needs to be considered if there is more than one bearing on either end of the box. Because of the rigidity of the boxes, provisions must be made to allow for field adjustments in the bearing height to account for any twisting that may occur.

6.11.11—Stiffeners

6.11.11.1—Web Stiffeners

The following shall supplement *A6.11.11.1*.

For box girders, bearing stiffeners shall be designed to be placed perpendicular to the bottom flange, regardless of vertical grade under final conditions.

C6.11.11.1

The following shall supplement *AC6.11.11.1*.

The design of stiffeners should always account for any placement such that the stiffener, as a critical compression member, is not completely vertical under final conditions.

6.13—CONNECTIONS AND SPLICES

6.13.1—General

The following shall supplement *A6.13.1*.

Field connections shall be bolted connections.

C6.13.1

The following shall supplement *AC6.13.1*.

Field welding can be performed successfully, but several other caveats exist, such as improperly grounding electrical connections to structural steel and the potential of weld splatter damaging tension flanges and other components. Bolted connection is more cost effective and generally safer to perform and inspect than field welding.

6.13.2—Bolted Connections

6.13.2.1—General

6.13.2.1.1—Slip-Critical Connections

The following shall supplement *A6.13.2.1.1*.

When evaluating the bearing capacity, threads shall be included in the shear plane.

6.13.2.1.2—Bearing-Type Connections

The following shall supplement *A6.13.2.1.2.*

Design all bearing-type connections with threads included in the shear plane unless it is not feasible for a specific connection.

Designing connections with threads excluded from the shear plane shall require approval of the Bridge Design Engineer Administrator.

The use of threads excluded in the bolt design shall be limited to bridge rehabilitation only. Bolts with threads, excluded from the design, must be indicated on the plans; dimension and the tolerance

C6.13.2.1.2

The following shall supplement *AC6.13.2.1.2.*

It is possible for bearing joints with threads included in the shear plane to have about 25 percent less capacity than those with threads excluded from the shear plane; therefore, it is prudent to design for the worst-case condition of bolt selection and placement.

for the threaded and non-threaded portions must be shown. A note shall be included in the plan to state that the contractor must ensure that no threaded portion shall be in the bearing surface of the connection.

6.13.2.3—Bolts, Nuts, and Washers

The following shall supplement *A6.13.2.3*.

All high-strength bolts shall have a standard hardened washer under the element that is turned in tightening.

If A490 bolt is approved for use, installation shall be in accordance with the *Standard Specifications*.

6.13.2.5—Size of Bolts

The following shall supplement *A6.13.2.5.*

Bolts for connecting primary and secondary members shall not be less than 0.75 inch in diameter. 0.875 inch diameter bolt is preferred. When practical, use same size bolts for all connections throughout the structure.

Bolt diameters greater than 1.125 inches shall not be used unless approved by the Bridge Design Engineer Administrator.

Refer to Article *A6.4.3.1* for additional criteria.

6.13.2.8—Slip Resistance

The following shall supplement *A6.13.2.8.*

Class “A” surface condition shall be used for the design of all bolted connections. Class "C" surface condition shall be used for galvanized surfaces. Design slip critical connections using a slip coefficient of no more than 0.33, regardless of the required surface Class specified on the project.

6.13.3—Welded Connections

6.13.3.1—General

The following shall supplement *A6.13.3.1*.

For complete joint penetration welded connections, do not detail a specific prequalified complete-joint penetration weld designation in the contract plans. The plans shall state that shop

C6.13.3.1

The following shall supplement *AC6.13.3.1*.

The steel fabricator should be allowed to select the complete penetration weld joint type, based upon their successful shop fabrication and inspection techniques.

drawings require weld symbols to be shown for review and approval.

In the contract plans, identify areas of flanges and other components that are subject to tension or stress reversal.

For widening or rehabilitation of existing structures, confer with the Bridge Design Engineer Administrator and Materials Office personnel for identification of existing base metals, if not attainable from existing plans or documentation.

Identification of tension and stress reversal areas enables inspection personnel to identify the scope and extent of weld testing, and the shop drawings should clearly designate such welds.

6.13.5—Connection Elements

6.13.5.1—General

The following shall supplement *A6.13.5.1.*

Connection plates shall be placed parallel to the skew for bridges with skew angles less than or equal to 20 degrees, and normal to the web for skews greater than 20 degrees. Transverse intermediate stiffeners that do not also serve as connection plates shall be placed normal to the web.

When transitioning the web plate thickness at a field splice, the web thickness increment shall be equal to at least 1/8 inch, such that 1/16 inch fill plates may be used on each side.

6.13.6—Splices

The following shall supplement *A6.13.6.*

When applicable, the following note shall be added to the contract plans:

“The contractor may propose alternate splice types and locations from those shown in the plans, all at no additional cost to the department and subject to review and approval of the Engineer of Record, prior to inclusion within the shop drawings.”

C6.13.6

The following shall supplement *AC6.13.6.*

Field splice locations are generally in low moment areas or where a section change is planned. Member lengths equal to or less than 115 ft. and/or of a member weight less than or equal to 100 kips are approximate maximum limits for individual pieces that can be handled efficiently during fabrication and erection.

Bolted field splices in continuous girders are good locations for changing the flange plate thickness and/or width, as this eliminates a welded butt splice.

When a girder is erected over a road open to traffic, consider locating the field splice outside of the traveled lanes in order to minimize disruption of traffic.

6.13.6.2—Welded Splices

The following shall supplement *A6.13.6.2.*

The cross-sectional area of the smaller plate at a flange transition shall not be less than 0.50 of the area of the larger flange plate.

6.16—REFERENCES

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