Bolted Field Splices for Steel Bridge Flexural Members

Overview & Design Examples

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BOLTED FIELD SPLICES FOR STEEL BRIDGE FLEXURAL MEMBERS

OVERVIEW & DESIGN EXAMPLES

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MARCH 2017

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1 INTRODUCTION

A splice is defined in AASHTO LRFD Article 6.2 as a group of bolted connections, or a welded connection, sufficient to transfer the moment, shear, axial force or torque between two structural elements joined at their ends to form a single, longer element. In steel bridge design, splices are typically used to connect girder sections together in the field; hence, the term field splices is often used.

The design of splices is covered in AASHTO LRFD Article 6.13.6. The design of bolted splices is covered in AASHTO LRFD Article 6.13.6.1, and the design of welded splices is covered in AASHTO LRFD Article 6.13.6.2. This document concentrates on the specifics related to the design of bolted field splices for steel-bridge flexural members, as outlined in AASHTO LRFD Article 6.13.6.1.3. The discussion includes an overview of the design procedure for bolted field splices for flexural members given in the 8th Edition AASHTO LRFD Bridge Design Specifications (2017), along with three design examples illustrating the application of the design procedure. Two of the design examples illustrate the application of the procedure to the design of bolted field splices for I-girder flexural members, and the last design example illustrates the application of the procedure to the design of a bolted field splice for a tub-girder flexural member.

A schematic of a typical bolted field splice for a flexural member is shown in Figure 1-1 (shown for an I-section). Bolted girder splices generally include top flange splice plates, web splice plates and bottom flange splice plates. In addition, if the plate thicknesses on one side of the joint are different than those on the other side, filler plates are used to match the thicknesses within the splice. For the flange splice plates, there is typically one plate on the outside of the flange and two smaller plates on the inside; one on each side of the web. For the web splice plates, there are two plates; one on each side of the web, with at least two rows of high-strength bolts over the depth of the web that are used to connect the splice plates to the member.

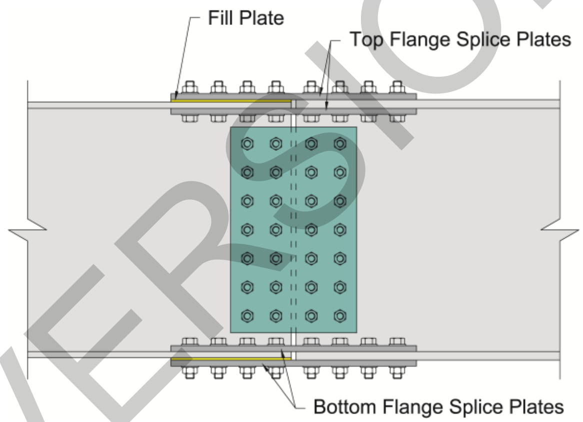


Fig. 1-1 Typical Bolted Field Splice for an I-Section Flexural Member

The AASHTO design procedure for the design of bolted splices for flexural members given in the 8th Edition LRFD Bridge Design Specifications (2017) is based upon designing the bolted flange and web splice connections for 100 percent of the individual design resistances of the flange and web; that is, the individual flange splices are designed for the smaller design yield resistance of the corresponding flanges on either side of the splice, and the web splice is designed for the smaller factored shear resistance of the web on either side of the splice. Therefore, the method satisfies the AASHTO design criteria since the web and flange splices have design resistances equal to the design resistances of their respective components. However, additional forces in the web connection may need to be considered if the flanges are not adequate to develop the factored design moment at the point of splice. No additional checks of the connection shear resistance are required.

The AASHO/AASHTO specifications required for many years that all splices and connections for primary members be designed for the average of the factored force effect at the point of splice or connection and the factored resistance of the member or element at the same point, but not less than 75 percent of the factored resistance of the member or element at the same point. This requirement is relatively straightforward when applied to a splice or connection for a truss member subject only to axial tension or compression since the stress is equal in the various components of the member. Application of this rule to the design of a bolted splice for a composite steel flexural member becomes more complex however since the stresses in the flanges are typically not equal, and the distribution of the stress in the web is a function of the loads applied to the composite and non-composite sections. In most designs, the factored resistance of the member controls the design of the bolted splice since the Engineer typically places the splice in a low-moment region near the point of dead-load contraflexure.

Experimental research at the University of Texas showed that a simpler method of design, on which the design procedure given in the 8th Edition AASHTO LRFD Specification is based in principle, produced a connection with adequate design resistance (Sheikh-Ibrahim and Frank, 1998 and 2001). The results showed that the web did not carry significant moment until the flange connection slipped. After the flange connection slipped, the web connection slipped and the force in the web did not increase until the flange bolts went into bearing and the flange yielded.

The efficacy of the 8th Edition AASHTO LRFD Specification design approach was further demonstrated through a detailed finite element analysis of the Design Example 2 connection in this document. The detailed finite element analysis of this particular connection was performed since it had the largest difference in the required number of web bolts between the new and old design approaches (Ocel, 2017).

As specified in AASHTO LRFD Article 6.13.6.1.3a, bolted splices in continuous spans should be made at or near points of permanent load contraflexure if possible. Splices located in areas of stress reversal near points of permanent load contraflexure are to be investigated for both positive and negative flexure in order to determine the governing condition.

Web and flange splices are not to have less than two rows of bolts on each side of the joint to ensure proper alignment and stability of the girder during construction. Also, oversize or slotted holes are not to be used in either the member or the splice plates at bolted splices to provide geometry control during erection before the bolts are tightened.

Bolted splice connections for flexural members are to be designed as slip-critical connections. Slip-critical connections are proportioned to prevent slip under Load Combination Service II and to provide bearing and shear resistance under the applicable strength limit state load combinations (AASHTO LRFD Article 6.13.2.1.1). In addition, bolted connections for flange and web splices in flexural members are proportioned to prevent slip during the casting of the concrete deck to provide geometry control (AASHTO LRFD Article 6.13.6.1.3a). The factored flexural resistance of the flanges at the point of splices at the strength limit state is to satisfy the applicable provisions of AASHTO LRFD Article 6.10.6.2 or 6.11.6.2.

AASHTO LRFD Eq. 6.10.1.8-1 provides a limit on the maximum major-axis bending stress permitted on the gross section of the girder, neglecting the loss of area due to holes in the tension flange. When a bolted splice is located at a section where the factored flexural resistance of the tension flange is less than or equal to the specified minimum yield strength of the flange, F_{yt}, Eq. 6.10.1.8-1 will not control and need not be checked since this equation is used to design the flange splices at the strength limit state. Eq. 6.10.1.8-1 will prevent a bolted splice from being located at a section where the factored flexural resistance of the section exceeds the moment at first yield, M_y, unless the stress in the tension flange at that section is limited to F_{yt}.

The nominal fatigue resistance of base metal at the gross section adjacent to slip-critical bolted connections is based on fatigue detail Category B assuming the bolts are installed in holes drilled full size or subpunched and reamed to size (*AASHTO LRFD* Table 6.6.1.2.3-1 – Condition 2.1), which is required for bolted girder splices. However, as mentioned in AASHTO LRFD Article C6.13.6.1.3a, fatigue will not control the design of the bolted splice plates for flexural members. The combined areas of the flange and web splice plates must typically equal or exceed the areas of the smaller flanges and web to which they are attached, and the flanges and web are usually checked separately for either equivalent or more critical fatigue category details. Therefore, fatigue of the splice plates will not need to be checked.

2 OVERVIEW OF DESIGN PROCEDURE

2.1 Flange Splice Design (AASHTO LRFD Article 6.13.6.1.3b)

2.1.1 Strength Limit State Design

2.1.1.1 General

The basis of the design method is to design each flange splice to develop the smaller full design yield resistance of the flanges on either side of the splice. The design yield resistance of each flange is calculated as (AASHTO LRFD Eq. 6.13.6.1.3b-1):

$$P_{fy} = F_{yf} A_e \qquad (2.1.1.1\text{-}1)$$

where F_{yf} is the specified minimum yield strength of the flange, and A_e is the effective flange area of the flange. The effective net area cannot exceed the gross area of the flange. This limit only applies to tension flanges, but is conservatively applied to both tension and compression flanges in the design method. A_e is calculated as follows (AASHTO LRFD Eq. 6.13.6.1.3b-2):

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yf}}\right) A_n \le A_g \qquad (2.1.1.1\text{-}2)$$

where:

ϕ_u = resistance factor for fracture of tension members as specified in AASHTO LRFD Article 6.5.4.2 = 0.80

ϕ_y = resistance factor for yielding of tension members as specified in AASHTO LRFD Article 6.5.4.2 = 0.95

A_n = net area of the flange under consideration determined as specified in AASHTO LRFD Article 6.8.3 (in.^{2})

A_g = gross area of the flange under consideration (in.^{2})

F_u = tensile strength of the flange under consideration determined as specified in AASHTO LRFD Table 6.4.1-1 (ksi)

Substituting the specified values of the resistance factors in Eq. 2.1.1.1-2 yields the following:

$$A_e = \left(\frac{\varphi_u F_u}{\varphi_y F_{yf}}\right) A_n = 0.84\left(\frac{F_u}{F_{yf}}\right) A_n \le A_g$$

(2.1.1.1-3)

The value of the coefficient, $0.84(F_u/F_y)$, in front of A_n in Eq. 2.1.1.1-3 for the various grades of steel is given in Table 2.1.1.1-1 below. The values are close to 1.0 for ASTM A709 Grade 50 and stronger steels; therefore, in most cases, the effective flange area will be less the gross area.

Table 2.1.1.1-1 – Coefficient in Front of A_n in Eq. 2.1.1.1-3

ASTM A709 Grade	F_y	F_u	$0.84\left(\frac{F_u}{F_y}\right)$
36	36	58	1.35
50	50	65	1.09
50W and HPS 50W	50	70	1.18
HPS 70W	70	85	1.02
HPS 100W	100	110	0.92

The load-shedding factor, R_b, and the hybrid factor, R_h, are not included in Eq. (2.1.1.1-1) since they are considered in the calculation to determine the flange sizes. The connections are designed to carry the full design yield resistance of the flanges.

For most girder splices, 4 rows of bolts in each flange are sufficient to meet the connection design requirements of the flange and continuity of the flange lateral stiffness. The net area of the flange assuming 4 bolts in a row across the flange without staggered bolt lines is:

$$\mathrm{A_n} = t_f\left\lfloor b_f - 4d_h \right\rfloor \qquad (2.1.1.1\text{-}4)$$

where:

t_f = flange thickness (in.)
b_f = flange width (in.)
d_h = diameter of standard-size bolt hole specified in AASHTO LRFD Table 6.13.2.4.2-1 (in.)

Note that for the following box sections:

- single box sections;
- multiple box sections in straight bridges not satisfying the requirements of AASHTO LRFD Article 6.11.2.3;
- single or multiple box sections in horizontally curved bridges; or
- single or multiple box sections with box flanges that are not fully effective according to the provisions of AASHTO LRFD Article 6.11.1.1,

the vector sum of the St. Venant torsional shear in the bottom flange and P_{fy} is to be considered in the design of the bottom-flange splice at the strength limit state. St. Venant torsional shears and longitudinal warping stresses due to cross-section distortion are typically neglected in top flanges of tub-girder sections once the flanges are continuously braced. Longitudinal warping stresses due to cross-section distortion do not need to be considered in the design of the bottom flange splices at the strength limit state since the flange splices are designed to develop the full design yield capacity of the flanges.

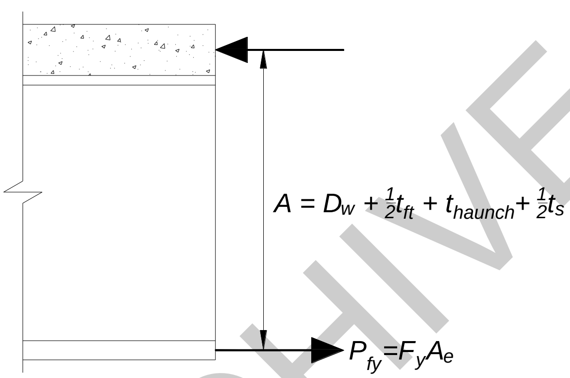
For straight girders and for horizontally curved girders, the effects of flange lateral bending need not be considered in the design of the bolted flange splices. Flange splices are designed to develop the full design yield capacity of the flange, which cannot be exceeded in the design of the flange for combined major-axis and lateral bending for constructibility and at the strength limit state.

2.1.1.2 Moment Resistance Check

The moment resistance provided by the flanges (neglecting the web and considering only the flange force) is next to be checked against the factored moment at the strength limit state at the point of splice. Should the factored moment exceed the moment resistance provided by the flanges, the additional moment is to be resisted by the web, as described further in Section 2.2.1.1 General. The moment resistance provided by the flanges is computed as follows:

Composite Sections Subject to Positive Flexure

For composite sections subject to positive flexure, the moment resistance provided by the flanges at the point of splice is computed as P_{fy} for the bottom flange computed from Eq. (2.1.1.1-1) times the moment arm, A, taken as the vertical distance from the mid-thickness of the bottom flange to the mid-thickness of the concrete deck including the concrete haunch (Figure 2.1.1.2-1):

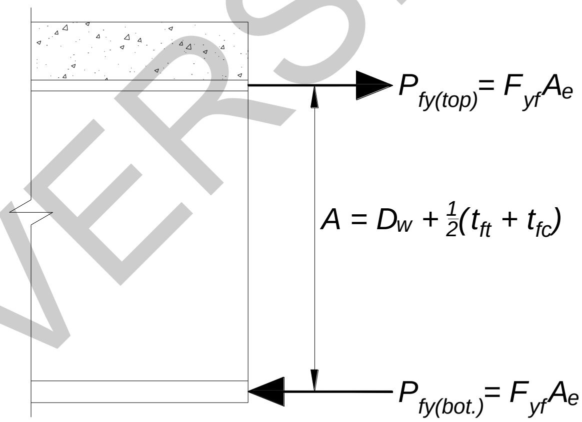


Note: Moment resistance is equal to P_{fy} for the bottom flange times the moment arm, A.

Fig. 2.1.1.2-1 Calculation of the Moment Resistance Provided by the Flanges at the Point of Splice for Composite Sections Subject to Positive Flexure

Composite Sections Subject to Negative Flexure and Noncomposite Sections

For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment resistance provided by the flanges at the point of splice is computed as P_{fy} for the top or bottom flange computed from Eq. (2.1.1.1-1), whichever is smaller, times the moment arm, A, taken as the vertical distance between the mid-thickness of the top and bottom flanges (Figure 2.1.1.2-2):



Note: Moment resistance is equal to $P_{fy(top)}$ or $P_{fy(bot.)}$, whichever is smaller, times the moment arm, A.

Fig. 2.1.1.2-2 Calculation of the Moment Resistance Provided by the Flanges at the Point of Splice for Composite Sections Subject to Negative Flexure and Noncomposite Sections

2.1.1.3 Flange Splice Bolts

The number of bolts required on one side of the flange splice at the strength limit state is found by dividing the design yield resistance of the flange, P_{fy}, by the factored shear resistance of the bolts determined as specified in AASHTO LRFD Article 6.13.2.2, including the reduction in the shear resistance of the bolts due to any needed fillers as specified in AASHTO LRFD

Article 6.13.6.1.4 (refer to Section 2.1.3 Filler Plates (AASHTO LRFD ARTICLE 6.13.6.1.4)). The resulting connections will develop the full design yield resistance of each flange (i.e. 100 percent of the design tension resistance of each flange). The factored shear resistance of the bolts, R_r, is calculated as the resistance factor for ASTM F3125 bolts in shear, ϕ_s = 0.80, times the nominal shear resistance of the bolts, R_n, computed as follows[1]:

- Where threads are excluded from the shear planes (AASHTO LRFD Eq. 6.13.2.7-1):

$$R_n = 0.56 A_b F_{ub} N_s \qquad (2.1.1.3\text{-}1)$$

- Where threads are included in the shear planes (AASHTO LRFD Eq. 6.13.2.7-2):

$$R_n = 0.45 A_b F_{ub} N_s \qquad (2.1.1.3\text{-}2)$$

where:

A_b = area of the bolt corresponding to the nominal diameter (in.^{2})

F_{ub} = specified minimum tensile strength of the bolt specified in AASHTO LRFD Article 6.4.3.1.1 (ksi)

N_s = number of shear planes per bolt (N_s = 2 since flange and web splices are symmetrical double-shear connections)

As specified in AASHTO LRFD Article 6.13.2.7, for a bolt in a lap splice tension connection greater than 38.0 in. in length, the nominal shear resistance, R_n, is taken as 0.83 times the value given by the applicable equation above. For bolted flange splices, the 38.0 in. length is measured between the extreme bolts on only one side of the splice and is normally not exceeded.

The bearing resistance of the flange splice bolt holes is also to be checked at the strength limit state as specified in AASHTO LRFD Article 6.13.2.9. For standard-size holes, the factored bearing resistance of the holes, R_r, is calculated as the resistance factor for bolts bearing on material, ϕ_{bb} = 0.80, times the nominal bearing resistance of the bolt holes, R_n, computed as follows (AASHTO LRFD Eqs. 6.13.2.9-1 and 6.13.2.9-2):

- With bolts spaced at a clear distance between not less than 2.0*d* and with a clear end distance not less than 2.0*d*:

$$R_n = 2.4 d t F_u \qquad (2.1.1.3\text{-}3)$$

- If either the clear distance between holes is less than 2.0*d*, or the clear end distance is less than 2.0*d*:

$$R_n = 1.2 L_c t F_u \qquad (2.1.1.3\text{-}4)$$

[1] In determining the factored shear resistance of the bolts, if the flange splice-plate thickness closest to the nut is greater than or equal to 0.5-in. thick, the nominal shear resistance of the bolts should be determined assuming the threads are excluded from the shear planes for bolts less than 1.0 in. in diameter. For bolts greater than or equal to 1.0 in. in diameter, the nominal shear resistance of the bolts should be determined assuming the threads are excluded from the shear planes if the flange splice-plate thickness closest to the nut is greater than 0.75 in. in thickness. Otherwise, the threads should be assumed included in the shear planes. The preceding assumes there is one washer under the nut, and that there is no stick-out beyond the nut, which represents the worst case for this determination. Web splice connections will generally have threads included in the shear plane due to the relatively thin splice plates in the web connections.

where:

d = nominal diameter of the bolt (in.)
t = thickness of the connected material (in.)
F_u = tensile strength of the connected material specified in AASHTO LRFD Table 6.4.1-1 (ksi)
L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in.)

The bearing resistance of the connection is the sum of the bearing resistances of the individual bolt holes parallel to the line of the design force[2]. Note that when the bearing resistance of the bolt holes is greater than or equal to the total factored shear resistance of the bolts within the holes under consideration, bearing does not control and need not be checked.

2.1.1.4 Flange Splice Plates

2.1.1.4.1 General

For a typical flange splice with inner and outer splice plates, an approach is needed to proportion P_{fy} to the inner and outer plates. According to AASHTO LRFD Article C6.13.6.1.3b, at the strength limit state, P_{fy} may be assumed equally divided to the inner and outer flange splice plates when the areas of the inner and outer plates do not differ by more than 10 percent. In this case, the shear resistance of the bolted connection should be checked for P_{fy} acting in double shear.

Should the areas of the inner and outer splice plates differ by more than 10 percent, the force in each plate should be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the inner and outer plates. In this case, the shear resistance of the bolted connection should be checked for the larger of the calculated splice-plate forces acting on a single shear plane.

2.1.1.4.2 Splice Plates in Tension

AASHTO LRFD Article 6.13.6.1.3b specifies that the design force in flange splice plates subject to tension at the strength limit state is not to exceed the factored resistance of the splice plates in tension specified in AASHTO LRFD Article 6.13.5.2; that is, the splice plates are to be checked for yielding on the gross section, fracture on the net section, and for block shear rupture. Block shear rupture will not typically control the design of flange splice plates of typical proportion.

According to AASHTO LRFD Article 6.13.5.2, the factored yield resistance of a connected element in tension, R_r, is computed from AASHTO LRFD Equation 6.8.2.1-1 as follows:

$$R_r = \phi_y F_y A_g \qquad (2.1.1.4.2\text{-}1)$$

where:

ϕ_y = resistance factor for yielding of tension members as specified in

[2] Assuming the sum of the flange splice-plate thicknesses exceeds the thickness of the thinner flange at the point of splice, and the splice plate areas satisfy the 10 percent rule described in Section 2.1.1.4.1 General, the bearing resistance of the connection will be governed by the flange on either side of the splice with the smaller product of the thickness and specified minimum tensile strength, F_u, of the flange. Otherwise, the bearing resistance of each individual component should be checked to determine the component governing the bearing resistance of the connection.

AASHTO LRFD Article 6.5.4.2 = 0.95

F_y = specified minimum yield strength of the connected element (ksi)

A_g = gross cross-sectional area of the connected element (in.^{2})

According to AASHTO LRFD Article 6.13.5.2, the factored net section fracture resistance of a connected element in tension, R_r, is computed from AASHTO LRFD Equation 6.8.2.1-2 as follows:

$$R_r = \phi_u F_u A_n R_p U \qquad (2.1.1.4.2\text{-}2)$$

where:

ϕ_u = resistance factor for fracture of tension members as specified in AASHTO LRFD Article 6.5.4.2 = 0.80

F_u = tensile strength of the connected element specified in AASHTO LRFD Table 6.4.1-1 (ksi)

A_n = net cross-sectional area of the connected element determined as specified in AASHTO LRFD Article 6.8.3 (in.^{2})

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size, and 1.0 for bolt holes drilled full size or subpunched and reamed to size (use 1.0 for splice plates)

U = reduction factor to account for shear lag (use 1.0 for splice plates)

Furthermore, according to AASHTO LRFD Article 6.13.5.2, for splice plates subject to tension, the design net area of the splice plates, A_n, must not exceed $0.85A_g$, where A_g is the gross area of the splice plates.

The factored block shear rupture resistance, R_r, is determined according to the provisions of AASHTO LRFD Article 6.13.4 as follows (AASHTO LRFD Eq. 6.13.4-1):

$$R_r = \phi_{bs} R_p \left(0.58 F_u A_{vn} + U_{bs} F_u A_{tn}\right) \le \phi_{bs} R_p \left(0.58 F_y A_{vg} + U_{bs} F_u A_{tn}\right) \qquad (2.1.1.4.2\text{-}3)$$

where:

ϕ_{bs} = resistance factor for block shear rupture as specified in *AASHTO LRFD* Article 6.5.4.2 = 0.80

A_{vg} = gross area along the plane resisting shear stress (in.^{2})

A_{vn} = net area along the plane resisting shear stress (in.^{2})

A_{tn} = net area along the plane resisting tension stress (in.^{2})

U_{bs} = reduction factor for block shear rupture resistance taken equal to 0.50 when the tension stress is non-uniform and 1.0 when the tension stress is uniform (use 1.0 for splice plates)

2.1.1.4.3 Splice Plates in Compression

The factored yield resistance of the splice plates in compression is the same as the factored yield resistance of the splice plates in tension given by Eq. (2.1.1.4.2-1), and therefore, need not be checked. Buckling of the splice plates in compression is not a concern since the unsupported length of the plates is limited by the maximum bolt spacing and end distance requirements.

2.1.2 Slip Resistance Check

The moment resistance provided by the nominal slip resistance of the flange splice bolts is checked against the factored moment for checking slip. The factored moments for checking slip are taken as the moment at the point of splice under Load Combination

Service II, as specified in AASHTO LRFD Table 3.4.1-1, and also the factored moment at the point of splice due to the deck casting sequence, as specified in AASHTO LRFD Article 3.4.2.1.

The nominal slip resistance of the bolts, R_n, is computed as follows (AASHTO LRFD Eq. 6.13.2.8-1):

$$R_n = K_h K_s N_s P_t \qquad (2.1.2\text{-}1)$$

where:

N_s = number of slip planes per bolt
P_t = minimum required bolt tension specified in AASHTO LRFD Table 6.13.2.8-1 (kips)
K_h = hole size factor specified in AASHTO LRFD Table 6.13.2.8-2
K_s = surface condition factor specified in AASHTO LRFD 6.13.2.8-3

As discussed in AASHTO LRFD Article C6.13.6.1.3b, when checking the slip resistance of the bolts for a typical flange splice with inner and outer splice plates, the flange slip force is assumed divided equally to the two slip planes regardless of the ratio of the splice plate areas. Unless slip occurs on both planes, slip of the connection cannot occur. Therefore, in this case, the slip resistance of the bolted connection should always be computed assuming two slip planes (i.e. $N_s = 2$ in Eq. (2.1.2-1)).

The moment resistance provided by the nominal slip resistance of the flange splice bolts is computed as described in Section 2.1.1.2 Moment Resistance Check, substituting the nominal slip resistance of the bolts for P_{fy}, and checked against the corresponding factored moment for checking slip defined above. For checking slip due to the factored deck casting moment, the moment resistance of the noncomposite section is used. Should the flange bolts not be sufficient to resist the factored moment for checking slip at the point of splice, the additional moment is to be resisted by the web, as described in Section 2.2.2 Slip Resistance Check Should the web bolts not be sufficient to resist the additional moment, only then should consideration be given to adding additional bolts to the flange splices.

A check of the flexural stresses in the flange splice plates under Load Combination Service II to control permanent deformations is not required since the combined areas of the flange splice plates must typically equal or exceed the areas of the smaller flanges to which they are attached.

For the box sections listed previously in Section 2.1.1.1 General, the St. Venant torsional shear in the bottom flange is to be considered in the design of the bottom flange splice when checking the bolts for slip. Rather than using the vector sum in this case, the shear is conservatively subtracted from the nominal slip resistance of the flange splice bolts prior to computing the moment resistance. St. Venant torsional shears and longitudinal warping stresses due to cross-section distortion are typically neglected in top flanges of tub sections once the flanges are continuously braced. Longitudinal warping stresses due to cross-section distortion are typically relatively small in the bottom flange at the service limit state and for constructibility and may be neglected when checking bottom flange splices for slip.

For straight girders and for horizontally curved girders, the effects of flange lateral bending need not be considered in checking the bolted connections of the flange splices for slip. Flange lateral bending will increase the flange slip force on one side of the splice and decrease the slip force on the other side of the splice; slip cannot occur unless it occurs on both sides of the splice.

2.1.3 Filler Plates (AASHTO LRFD Article 6.13.6.1.4)

Filler plates are typically used on bolted flange splices of flexural members (and sometimes on web splices) when the thicknesses of the adjoining plates at the point of splice are different (Figure 1-1).

At bolted flange splices, it is often advantageous to transition one or more of the flange thicknesses down adjacent to the point of splice, if possible, so as to reduce the required size of the filler plate, or possibly change the width of the flanges at the splice and keep the thicknesses constant in order to eliminate the need for a filler plate altogether.

Fillers thicker than ¼ in. must be secured by additional bolts to ensure that the fillers are an integral part of the connection; that is, to ensure that the shear planes are well-defined and that no reduction in the factored shear resistance of the bolts results.

AASHTO LRFD Article 6.13.6.1.4 provides two choices for developing fillers ¼ in. or more in thickness in girder flange splices. The choices are to either: 1) extend the fillers beyond the splice plate with the filler extension secured by enough additional bolts to distribute the total stress uniformly over the combined section of the member or filler; or 2) in lieu of extending and developing the fillers, reduce the factored shear resistance of the bolts by the following factor (AASHTO LRFD Eq. 6.13.6.1.4-1):

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)} \right] \qquad (2.1.3\text{-}1)$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})

A_p = smaller of either the connected plate area on the side of the connection with the filler or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

The reduction factor, R, accounts for the reduction in the nominal shear resistance of the bolts due to bending in the bolts and will likely result in having to provide additional bolts in the connection to develop the filler(s). Note that the reduction factor is only to be applied on the side of the splice with the filler(s). For practical reasons, consideration should be given to using the same number of bolts on either side of the splice. Note that fillers ¼ in. or more in thickness are not to consist of more than two plates, unless approved by the Engineer. Additional requirements regarding the specified minimum yield strength of fillers are given in AASHTO LRFD Article 6.13.6.1.4.

The slip resistance of the connection is not affected by the filler. Therefore, as specified in AASHTO LRFD Article 6.13.6.1.4, for slip-critical connections, the nominal slip resistance of the bolts is not to be adjusted for the effect of the fillers.

2.2 Web Splice Design (AASHTO LRFD Article 6.13.6.1.3c)

2.2.1 Strength Limit State Design

2.2.1.1 General

The basis of the design method is to design the web splice as a minimum for a design web force taken equal to the smaller factored shear resistance of the web, V_r, on either side of the splice. The factored shear resistance of the web is calculated as:

$$V_r = \phi_v V_n \qquad (2.2.1.1\text{-}1)$$

where ϕ_v is the resistance factor for shear (= 1.0), and V_n is the nominal shear resistance of the web determined as specified in AASHTO LRFD Article 6.10.9 or 6.11.9, as applicable.

Since the web splice is being designed to develop the full factored shear resistance of the web as a minimum and the eccentricity of the shear is small relative to the depth of the connection, the effect of the small moment introduced by the eccentricity of the web connection is ignored at all limit states.

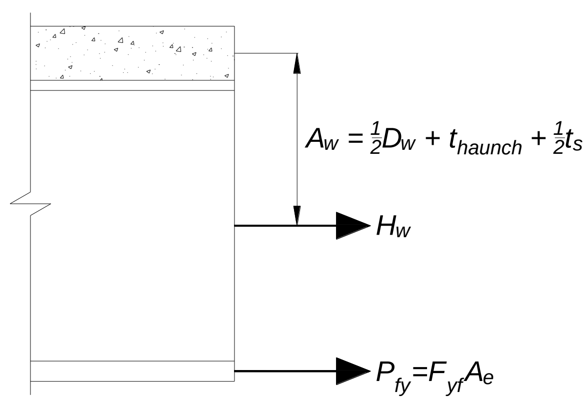
If the moment resistance provided by the flanges (refer to Section 2.1.1.2 Moment Resistance Check) is not sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice plates and their connections are to be designed for the following resultant design web force, R, taken equal to the vector sum of the smaller factored shear resistance, V_r, and a horizontal force, H_w, located at the mid-depth of the web that provides the required moment resistance in conjunction with the flange splices:

$$R = \sqrt{(V_r)^2 + (H_w)^2} \qquad (2.2.1.1\text{-}2)$$

The horizontal force in the web, H_w, is computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flanges divided by the appropriate moment arm, A_w, to the mid-depth of the web. The moment arm, A_w, is computed as follows:

<u>Composite Sections Subject to Positive Flexure</u>

For composite sections subject to positive flexure, the moment arm, A_w, is taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch (Figure 2.2.1.1-1):

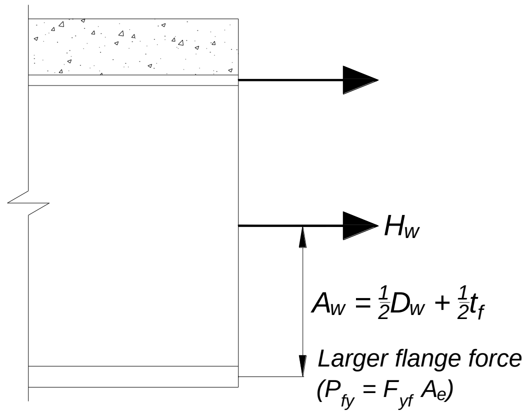


<u>Note</u>: A_w is measured from the mid-depth of the web to the mid-thickness of the deck.

Fig. 2.2.1.1-1 Calculation of the Moment Arm, A_w, for Composite Sections Subject to Positive Flexure

<u>Composite Sections Subject to Negative Flexure and Noncomposite Sections</u>

For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm, A_w, is taken as the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy} (Figure 2.2.1.1-2):



Note: A_w is measured from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever has the larger value of P_{fy}.

Fig. 2.2.1.1-2 Calculation of the Moment Arm, A_w, for Composite Sections Subject to Negative Flexure and Noncomposite Sections

H_w is conservatively assumed to act at the mid-depth of the web. Because the resultant web force is assumed divided equally to all of the bolts, the traditional vector analysis is not applied.

Lastly, for the box sections listed in Section 2.1.1.1 General, the effect of the additional St. Venant torsional shear in the web may be ignored in the design of the web splice at the strength limit state since the web splice is being designed as a minimum for the full factored shear resistance of the web.

2.2.1.2 Web Splice Bolts

The number of bolts required on one side of the web splice at the strength limit state is found by dividing the computed design web force by the factored shear resistance of the bolts. The factored shear resistance of the bolts, R_r, is calculated as the resistance factor for ASTM F3125 bolts in shear, ϕ_s = 0.80, times the nominal shear resistance of the bolts, R_n, calculated assuming the threads are included in the shear planes for most common web splices since the web splice-plate thicknesses are normally less than or equal to ½ in.

Note that the greater than 38.0 in. length reduction for the shear resistance of the bolts only applies to lap-splice tension connections (AASHTO LRFD Article 6.13.2.7), and therefore should not be applied in the design of the web splice.

As a minimum, two vertical rows of bolts spaced at the maximum spacing for sealing bolts specified in AASHTO LRFD Article 6.13.2.6.2 should be provided, with a closer spacing and/or additional rows provided only as needed.

The bearing resistance of the web splice bolt holes is also to be checked at the strength limit state as specified in AASHTO LRFD Article 6.13.2.9. For standard holes, the factored bearing resistance of the holes, R_r, is calculated as the resistance factor for bolts bearing on material, ϕ_{bb} = 0.80, times the nominal bearing resistance of the bolt holes, R_n, computed from Eq. (2.1.1.3-3) or Eq. (2.1.1.3-4), as applicable. The bearing resistance may be calculated as the sum of the bearing resistances of the individual bolt holes parallel to the line of the web design force. Note that when the bearing resistance of the bolt holes is greater than or equal to the total factored shear resistance of the bolts within the holes under consideration, bearing does not control and need not be checked.

When a moment contribution from the web is required, the resultant forces causing bearing on the web bolt holes are inclined (Figure 2.2.1.2-1). The bearing resistance of each bolt hole in the web can conservatively be calculated in this case using the clear edge distance, as shown on the left-hand side of Figure 2.2.1.2-1. This calculation is conservative since the resultant forces act in the direction of inclined distances that are larger than the clear edge distance. This calculation is also likely to be a conservative calculation for the bolt holes in the adjacent rows. Should the bearing resistance be exceeded, it is recommended that the clear edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Other options would be to calculate the bearing resistance based on the inclined distances, or to resolve the resultant forces in the direction parallel to the clear edge distance, or to refine the calculation for the bolt holes in the adjacent rows. In cases where the bearing resistance is controlled by the web splice plates, the smaller of the clear edge or end distance on the splice plates can conservatively be used to compute the bearing resistances of each hole, as shown on the right-hand side of Figure 2.2.1.2-1.

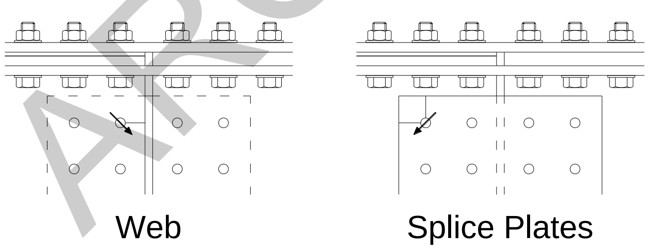


Fig. 2.2.1.2-1 Computing the Bearing Resistance of the Web Splice Bolt Holes for an Inclined Resultant Design Web Force

2.2.1.3 Web Splice Plates

Webs are to be spliced symmetrically by plates on each side. The splice plates are to extend as near as practical for the full depth between flanges without impinging on bolt assembly clearances. Required bolt assembly clearances are given in Tables 7-15 and 7-16 of AISC (2011), as applicable. For bolted web splices with thickness differences of $^{1}/_{16}$ in. or less, filler plates should not be provided. A minimum gap of ½ in. between the girder sections at the splice should be provided to provide drainage and allow for fit-up.

The factored shear resistance of the web at the strength limit state, V_r, is not to exceed the lesser of the factored shear resistances of the web splice plates determined as specified in AASHTO LRFD Article 6.13.5.3. AASHTO LRFD Article 6.13.5.3 specifies that the factored shear resistance, R_r, of a connected element is to be taken as the smaller value based on shear yielding or shear rupture.

For shear yielding, the factored shear resistance of the splice plates, R_r, is conservatively based on the shear yield stress (i.e. $F_y/\sqrt{3} = 0.58F_y$) as follows (AASHTO LRFD Eq. 6.13.5.3-1):

$$R_r = \phi_v 0.58 F_y A_{vg} \qquad (2.2.1.3\text{-}1)$$

where:

ϕ_v = resistance factor for shear specified in AASHTO LRFD Article 6.5.4.2 = 1.0

A_{vg} = gross area of the splice plates subject to shear (in.^{2})

F_y = specified minimum yield strength of the splice plates (ksi)

For shear rupture, the factored shear resistance of the splice plates, R_r, is taken as follows (AASHTO LRFD Eq. 6.13.5.3-2):

$$R_r = \phi_{vu} 0.58 R_p F_u A_{vn} \quad (2.2.1.3\text{-}2)$$

where:

ϕ_{vu} = resistance factor for shear rupture of connected elements specified in AASHTO LRFD Article 6.5.4.2 = 0.80

A_{vn} = net area of the splice plates subject to shear (in.^{2})

F_u = tensile strength of the splice plates specified in AASHTO LRFD Table 6.4.1-1 (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size (use 1.0 for splice plates)

The factored shear resistance of the web at the strength limit state, V_r, is also not to exceed the factored block shear rupture resistance of the web splice plates determined as specified in AASHTO LRFD Article 6.13.4 (see Eq. (2.1.1.4.2-3)).

2.2.2 Slip Resistance Check

As a minimum, AASHTO LRFD Article 6.13.6.1.3c specifies that bolted connections for web splices be checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice. The factored shear for checking slip is taken as the shear in the web at the point of splice under Load Combination Service II, as specified in AASHTO LRFD Table 3.4.1-1, or the factored shear at the point of splice due to the deck casting sequence, as specified in AASHTO LRFD Article 3.4.2.1, whichever governs.

Should the flange bolts not be sufficient to resist the factored moment for checking slip at the point of splice (see Section 2.1.2 Slip Resistance Check), the web splice bolts should instead be checked for slip under a web slip force taken equal to the vector sum of the factored shear and a horizontal force, H_w, located at the mid-depth of the web that provides the necessary slip resistance in conjunction with the flange splices. The horizontal force in the web, H_w, is computed as the portion of the factored moment for checking slip at the point of splice that exceeds the moment resistance provided by the nominal slip resistance of the flange splice bolts divided by the appropriate moment arm, A_w, to the mid-depth of the web. The moment arm, A_w, is computed as described in Section 2.2.1.1 General.

The computed slip force is then divided by the nominal slip resistance of the bolts, determined as specified in AASHTO LRFD Article 6.13.2.8, to determine the number of web splice bolts required on one side of the splice to resist slip. The nominal slip resistance of the bolts, R_n, is computed from Eq. (2.1.2-1).

For the box sections listed in Section 2.1.1.1 General, the shear for checking slip is taken as the sum of the flexural and St. Venant torsional shears in the web subject to additive shears since slip is a serviceability criterion. For boxes with inclined webs, the factored shear is taken as the component of the factored vertical shear in the plane of the web (AASHTO LRFD Eq. 6.11.9-1).

3 DESIGN EXAMPLES

3.1 Design Example 1

3.1.1 General

Design Example 1 is an example design of a bolted field splice for the interior girder of an I-section flexural member. The splice is located near the point of permanent load contraflexure in the end span of a three-span continuous bridge on right supports with spans of 140-175-140 ft and a girder spacing of 12.0 ft. The girder plate sizes at the point of splice are given in Table 3.1.1-1. The unfactored design moments at the point of splice are also listed below. ⅞-in.-diameter ASTM F3125 Grade 325 bolts are used for both the flange and web splices. The threads are assumed excluded from the shear planes in the flange splices, and included in the shear planes in the web splice.

The section on the left-hand side of the splice is homogeneous utilizing ASTM A709 Grade 50W steel for the flanges and web. The section on the right-hand side of the splice is a hybrid section utilizing ASTM A709 Grade HPS 70W steel for the flanges and ASTM A709 Grade 50W steel for the web. The structural thickness of the concrete deck is 9.0 in. The concrete deck haunch is ignored in this particular example.

M_{DC1} = +248 kip-ft
M_{DC2} = +50 kip-ft
M_{DW} = +52 kip-ft
M_{+LL+IM} = +2,469 kip-ft
M_{-LL+IM} = -1,754 kip-ft
$M_{deck\ casting}$ = +1,300 kip-ft

Table 3.1.1-1 Design Example 1 Plate Dimensions

Side of Splice	Top Flange Width (in.)	Top Flange Thickness (in.)	Web Depth (in.)	Web Thickness (in.)	Bottom Flange Width (in.)	Bottom Flange Thickness (in.)
Left	16 [a]	1 [a]	69	½	18 [a]	1-$^{3}/_{8}$ [a]
Right	18 [b]	1 [b]	69	$^{9}/_{16}$	20 [b]	1 [b]

[a] – Flange is ASTM A709 Grade 50W
[b] – Flange is ASTM A709 Grade HPS 70W

The factored Strength I design moments at the point of splice are computed as follows:

Positive Moment = 1.25(248+50) + 1.5(52) + 1.75(2,469) = +4,771 kip-ft

Negative Moment = 0.9(248+50) + 0.65(52) + 1.75(-1,754) = -2,768 kip-ft

3.1.2 Flange Splice Design

3.1.2.1 Strength Limit State Design

3.1.2.1.1 Bolts

<u>*Top Flange*</u>

The left side of the splice has the smaller design yield resistance (i.e., the top flange on the left side has a smaller area and lower yield strength).

Assuming 4 rows of bolts across the width of the flange (Eq. (2.1.1.1-3)

$A_e = 1.18(1)[16\text{-}4(^{15}/_{16})] = 14.4 \text{ in.}^2 < 1(16) \text{ in.}^2 = 16.0 \text{ in.}^2$

$P_{fy} = 50(14.4) = 720$ kips

The factored shear resistance of the bolts in double shear ($N_s = 2$) is computed as follows:

Bolts with threads excluded from the shear plane (Eq. (2.1.1.3-1)):

$$R_r = \phi_s R_n = 0.80 * 0.56 A_b F_{ub} N_s = 0.80(0.56)(0.601)(120)(2) = 64.6 \text{ kips}$$

Number of Bolts Required: $N = 720/64.6 = 11.1$

Use 4 rows with 3 bolts per row = 12 bolts on each side of the splice.

<u>*Bottom Flange*</u>

Check which side of the splice has the flange with the smaller design yield resistance:

<u>Left Side</u>

$A_e = 1.18(1.375)[18\text{-}4(^{15}/_{16})] = 23.1 \text{ in.}^2 < 1.375(18) \text{ in.}^2 = 24.75 \text{ in.}^2$

$P_{fy} = 50(23.1) = 1{,}152$ kips

Right Side

A_e = 1.02(1)[20-4($^{15}/_{16}$)] = 16.6 in.^{2} < 1.0(20) in.^{2}

P_{fy} = 70(16.6) = 1,162 kips > 1,152 kips; therefore, the left side controls

P_{fy} = 1,152 kips

Reduction in bolt factored shear resistance due to filler (Eq. (2.1.3-1))):

$$R = \frac{1+\frac{0.375}{1}}{1+\frac{2(0.375)}{1}} = 0.79$$

Note that the splice plate, filler plate, and flange widths will be equal in the splice. Consequently, the area ratios are only a function of the thickness of the flange and fillers.

Number of Bolts Required (threads excluded from the shear planes): N = 152/(0.79(64.6)) = 22.6

Use 4 rows with 6 bolts per row = 24 bolts on each side of the splice.

3.1.2.1.2 Moment Resistance

Positive Moment (Figure 2.1.1.2-1; t_{haunch} = 0)

Use P_{fy} for the bottom flange = 1,152 kips.

Flange Moment Arm: $A = D_w + t_{ft}/2 + t_{fc} + t_s/2$ = 69 + 1.375/2 + 1 + 9/2 = 75.2 in.

M_{flange} = 1,152 x (75.2/12) = 7,218 kip-ft > 4,771 kip-ft   ok

Negative Moment (Figure 2.1.1.2-2)

Use the smaller value of P_{fy} for the top and bottom flanges. In this case, the top flange has the smaller value of P_{fy} = 720 kips.

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2$ = 69 + (1.375+1)/2 = 70.2 in.

M_{flange} = 720 x (70.2/12) = 4,211 kip-ft >|-2,768 kip-ft| ok

Therefore, the flanges have adequate capacity to resist the Strength I moment requirements at the splice. No moment contribution from the web is required.

3.1.2.1.3 Splice Plates

The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice. The width of the inside splice plates should be such that the plates clear the flange-to-web weld on each side of the web by a minimum of ⅛ in. Therefore, for the bottom-flange splice, try a ¾ in. x 18 in. outside splice plate and two ⅞ in. x 8 in. inside splice plates. Include a ⅜ in. x 18 in. filler plate on the outside. All plates are ASTM A709 Grade 50W steel.

At the strength limit state, P_{fy} may be assumed equally divided to the inner and outer flange splice plates when the areas of the inner and outer plates do not differ by more than 10 percent. In this case, the areas of the inner and outer plates are within 4 percent. Therefore, P_{fy} is equally divided to the inner and outer plates and the shear resistance of the bolted connection is checked above for P_{fy} acting in double shear.

Check the factored yield resistance of the splice plates in tension (Eq. (2.1.1.4.2-1)):

Outside splice plate:

$$R_r = 0.95(50)(18.0)(0.75) = 641 \text{ kips} > 1{,}152/2 = 576 \text{ kips} \quad \text{ok}$$

Inside splice plates:

$$R_r = 0.95(50)(2)(8.0)(0.875) = 665 \text{ kips} > 1{,}152/2 = 576 \text{ kips} \quad \text{ok}$$

Check the net section fracture resistance of the splice plates in tension (Eq. (2.1.1.4.2-2)). As specified in AASHTO LRFD Article 6.8.3, for design calculations, the width of standard-size bolt holes is taken as the nominal diameter of the holes, or $^{15}/_{16}$ in. for a ⅞-in.-diameter bolt:

Outside plate:

$$R_r = 0.80(70)[18.0 - 4(0.9375)](0.75)(1.0)(1.0) = 599 \text{ kips} > 1{,}152/2 = 576 \text{ kips} \quad \text{ok}$$

Inside plates:

$$R_r = 0.80(70)[2(8.0) - 4(0.9375)](0.875)(1.0)(1.0) = 600 \text{ kips} > 1{,}152/2 = 576 \text{ kips} \quad \text{ok}$$

Also, according to AASHTO LRFD Article 6.13.5.2, for splice plates subject to tension, the design net area, A_n, must not exceed $0.85A_g$.

Outside plate:

$$0.85(18.0)(0.75) = 11.5 \text{ in.}^2 > A_n = [18.0 - 4(0.9375)](0.75) = 10.7 \text{ in.}^2 \quad \text{ok}$$

Inside plates:

$$0.85(2)(8.0)(0.875) = 11.9 \text{ in.}^2 > A_n = [2(8.0) - 4(0.9375)](0.875) = 10.7 \text{ in.}^2 \quad \text{ok}$$

In order to check the block shear rupture resistance of the splice plates and the flange (and later on the factored bearing resistance of the bolt holes in Section 3.1.2.1.4 Bearing Resistance Check), the bolt spacings and bolt edge and end distances must first be established and checked. Refer to the bolt pattern shown in Figure 3.1.2.1.3-1.

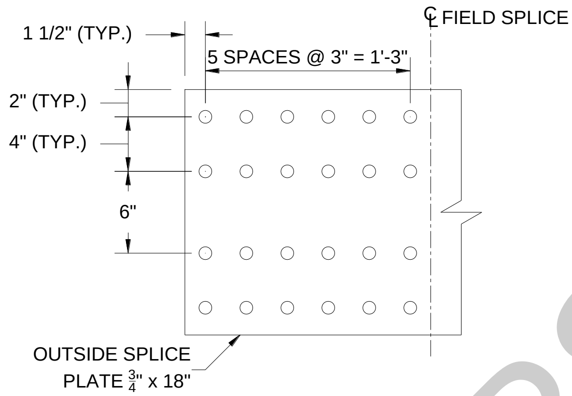


Fig. 3.1.2.1.3-1 Outside Bottom Flange Splice Plate – Plan View

As specified in AASHTO LRFD Article 6.13.2.6.1, the minimum spacing between centers of bolts in standard holes is not to be less than 3.0*d*, where *d* is the diameter of the bolt. For ⅞-in.-diameter bolts:

$$s_{min} = 3d = 3(0.875) = 2.63 \text{ in.} \quad \text{use } 3.0 \text{ in.}$$

Since the length between the extreme bolts (on one side of the splice) measured parallel to the line of action of the force is less than 38.0 in., no reduction in the factored shear resistance of the bolts is required, as originally assumed.

As specified in AASHTO LRFD Article 6.13.2.6.2, to seal against the penetration of moisture in joints, the spacing, *s*, of a single line of bolts adjacent to a free edge of an outside plate or shape (when the bolts are not staggered) must satisfy the following requirement:

$$s \leq (4.0 + 4.0t) \leq 7.0 \text{ in.}$$

where *t* is the thickness of the thinner outside plate or shape. First, check for sealing along the edges of the outer splice plate (the thinner plate) parallel to the direction of the applied force. The bolt lines closest to the edges of the flanges are assumed to be 1-½ in. from the edges of the flanges. A ¾ in. gap is assumed between the girder flanges at the splice to allow the splice to provide drainage and allow for fit-up:

$$s_{max} = 4.0 + 4.0(0.75) = 7.00 \text{ in.} > 3.75 \text{ in.} \quad \text{ok}$$

Check for sealing along the free edge at the end of the splice plate:

$s_{max} = 4.0 + 4.0(0.75) = 7.00\ \text{in.} > 6.0\ \text{in.}$ ok

Note that the maximum pitch requirements for stitch bolts specified in AASHTO LRFD Article 6.13.2.6.3 apply only to the connection of plates in mechanically fastened built-up members and are not to be applied here in the design of the splice.

The edge distance of bolts is defined as the distance perpendicular to the line of force between the center of a hole and the edge of the component. In this example, the edge distance of 2.0 in. satisfies the minimum edge distance requirement of 1-⅛ in. specified for ⅞-in.-diameter bolts in AASHTO LRFD Table 6.13.2.6.6-1. This distance also satisfies the maximum edge distance requirement of 8.0*t* (not to exceed 5.0 in.) = 8.0(0.75) = 6.0 in. specified in AASHTO LRFD Article 6.13.2.6.6.

The end distance of bolts is defined as the distance along the line of force between the center of a hole and the end of the component. In this example, the end distance of 1-½ in. satisfies the minimum end distance requirement of 1-⅛ in. specified for ⅞-in.-diameter bolts. The maximum end distance requirement of 6.0 in. is also obviously satisfied. Although not specifically required, note that the distance from the corner bolts to the corner of the splice plate, equal to $\sqrt{(1.5)^2 + (2.0)^2} = 2.5\ \text{in.}$, also satisfies the maximum end distance requirement. If desired, the corners of the plate can be clipped to meet this requirement. Fabricators generally prefer that the end distance on the all girder flanges at the point of splice be increased a minimum of ¼ in. from the design value to allow for girder trim.

Check the block shear rupture resistance of the splice plates in tension (Eq. (2.1.1.4.2-3). Assume the potential block shear failure planes on the outside and inside splice plates shown in Fig. 3.1.2.1.3-2.

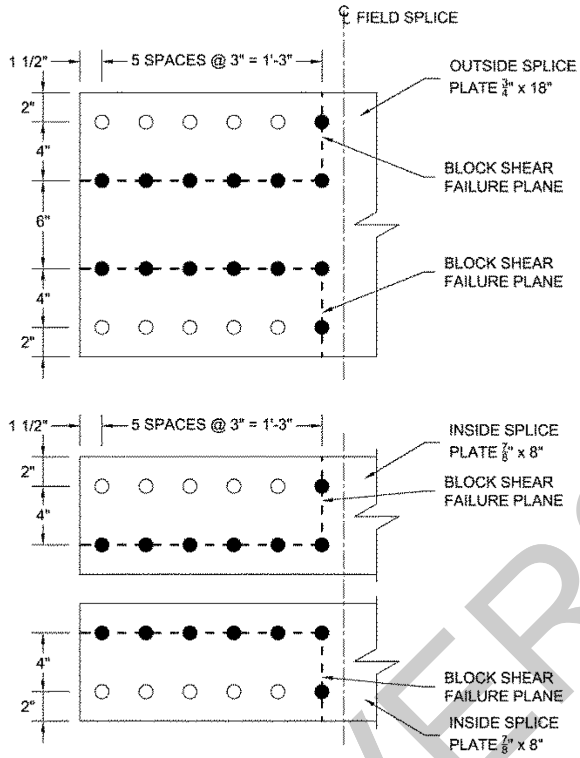


Fig. 3.1.2.1.3-2 Bottom Flange Splice – Assumed Block Shear Failure Planes in the Splice Plates

Check the outside splice plate. A_{tn} is the net area along the place resisting the tensile stress.

$$A_{tn} = 2[4.0 + 2.0 - 1.5(0.9375)](0.75) = 6.89 \text{ in.}^2$$

A_{vn} is the net area along the place resisting the shear stress.

$$A_{vn} = 2[5(3.0) + 1.5 - 5.5(0.9375)](0.75) = 17.01 \text{ in.}^2$$

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

$$A_{vg} = 2[5(3.0) + 1.5](0.75) = 24.75 \text{ in.}^2$$

Therefore:

$$R_r = 0.80(1.0)[0.58(70)(17.01) + 1.0(70)(6.89)] = 938 \text{ kips} < 0.80(1.0)[0.58(50)(24.75) + 1.0(70)(6.89)] = 960 \text{ kips}$$

$$\therefore R_r = 938 \text{ kips} > \frac{1{,}152}{2} = 576 \text{ kips} \quad \text{ok}$$

Check the inside splice plates.

$$A_{tn} = 2[4.0 + 2.0 - 1.5(0.9375)](0.875) = 8.04 \text{ in.}^2$$

$$A_{vn} = 2[5(3.0) + 1.5 - 5.5(0.9375)](0.875) = 19.85 \text{ in.}^2$$

$$A_{vg} = 2[5(3.0) + 1.5](0.875) = 28.87 \text{ in.}^2$$

$$R_r = 0.80(1.0)[0.58(70)(19.85) + 1.0(70)(8.04)] = 1{,}095 \text{ kips} < 0.80(1.0)[0.58(50)(28.87) + 1.0(70)(8.04)] = 1{,}120 \text{ kips}$$

$$\therefore R_r = 1{,}095 \text{ kips} > \frac{1{,}152}{2} = 576 \text{ kips} \quad \text{ok}$$

Check the block shear rupture resistance in tension of the critical girder bottom flange at the splice. Since the areas and yield strengths of the flanges on each side of the splice differ, both sides need to be checked. Only the calculations for the flange on the right-hand side of the splice, which is determined to be the critical flange for this check, are shown below. Two potential failure modes are investigated for the flange as shown in Figure 3.1.2.1.3-3.

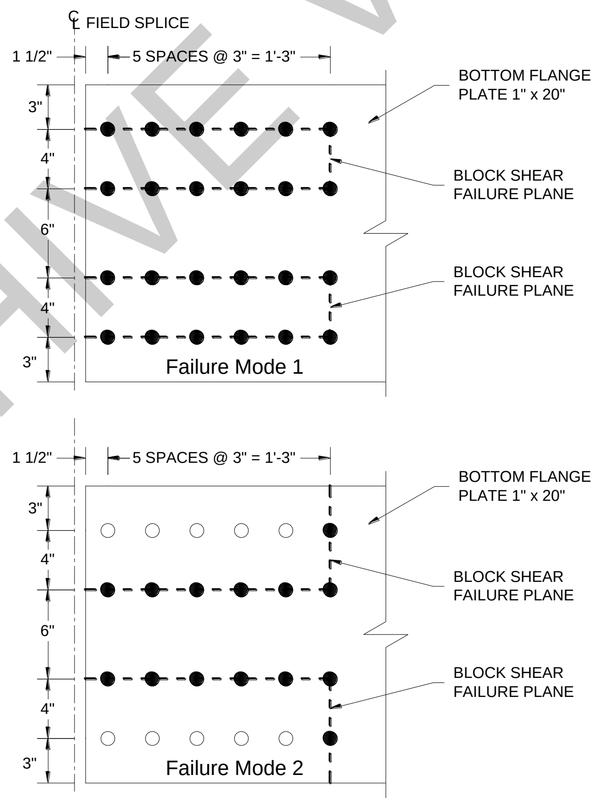


Fig. 3.1.2.1.3-3 Bottom Flange Splice – Assumed Block Shear Failure Planes in the Flange on the Right-Hand Side of the Splice

For Failure Mode 1:

$$A_{tn} = 2[4.0 - 0.9375](1.0) = 6.13 \text{ in}^2$$

$$A_{vn} = 4[5(3.0) + 1.5 - 5.5(0.9375)](1.0) = 45.37 \text{ in}^2$$

$$A_{vg} = 4[5(3.0) + 1.5](1.0) = 66.00 \text{ in}^2$$

$$R_r = 0.80(1.0)[0.58(85)(45.37) + 1.0(85)(6.13)] = 2{,}206 \text{ kips} < 0.80(1.0)[0.58(70)(66.00) + 1.0(85)(6.13)] = 2{,}560 \text{ kips}$$

$$\therefore R_r = 2{,}206 \text{ kips} > 1{,}152 \text{ kips} \quad \text{ok}$$

For Failure Mode 2:

$$A_{tn} = 2[4.0 + 3.0 - 1.5(0.9375)](1.0) = 11.19 \text{ in.}^2$$

$$A_{vn} = 2[5(3.0) + 1.5 - 5.5(0.9375)](1.0) = 22.69 \text{ in.}^2$$

$$A_{vg} = 2[5(3.0) + 1.5](1.0) = 33.00 \text{ in.}^2$$

$$R_r = 0.80(1.0)[0.58(85)(22.69) + 1.0(85)(11.19)] = 1{,}656 \text{ kips} < 0.80(1.0)[0.58(70)(33.00) + 1.0(85)(11.19)] = 1{,}833 \text{ kips}$$

$$\therefore R_r = 1{,}656 \text{ kips} > 1{,}152 \text{ kips} \quad \text{ok}$$

Because the splice is located near a point of permanent load contraflexure and the factored flexural resistance of the tension flange is less than or equal to the specified minimum yield strength of the tension flange at that point, AASHTO LRFD Eq. 6.10.1.8-1 will not control at this location and need not be checked since this equation was used to design the flange splices at the strength limit state.

Since the combined area of the inside and outside flange splice plates is greater than the area of the smaller bottom flange at the point of splice, fatigue of the base metal of the bottom flange splice plates adjacent to the slip-critical bolted connections does not need to be checked. Similarly, the flexural stresses in the splice plates at the service limit state under the Service II load combination need not be checked.

Calculations similar to the above show that a ⅝ in. x 16 in. outside splice plate with two $^{11}/_{16}$ in. x 7 in. inside splice plates are sufficient to resist the design yield resistance of the top flange, P_{fy} = 720 kips. A filler plate is not required. All plates are again ASTM A709 Grade 50W steel.

3.1.2.1.4 Bearing Resistance Check

The bearing resistance of the connected material at the strength limit state is calculated as the sum of the bearing resistances of the individual bolts (holes) parallel to the line of the applied force.

The bearing resistance of connected material in the bottom flange splice will be checked. The sum of the inner and outer splice plate thicknesses exceeds the thickness of the thinner flange at the point of splice, and the splice plate areas satisfy the 10 percent rule described in Section 2.1.1.4.1 General. Therefore, check which flange on either side of the splice has the smaller product of the thickness times the specified minimum tensile strength, $F_{u,}$ of the flange to determine which flange controls the bearing resistance of the connection.

Bottom Flange Left Side: $(1.375)(70) = 96.3 \text{ kips/in.}$
Bottom Flange Right Side: $(1.0)(85) = 85.0 \text{ kips/in.} \text{ (governs)}$

For standard-size holes, the nominal bearing resistance, R_n, parallel to the applied bearing force is given by Eq. (2.1.1.3-3) or Eq. (2.1.1.3-4), as applicable.

For the four bolt holes adjacent to the end of the flange, the end distance is 1-½ in. Therefore, the clear distance, L_c, between the edge of the hole and the end of the flange is:

$$L_c = 1.5 - \frac{0.9375}{2} = 1.03 \text{ in.} < 2.0d = 2.0(0.875) = 1.75 \text{ in.}$$

Therefore, use Eq. (2.1.1.3-4):

$$R_n = 4(1.2L_c tF_u) = 4[1.2(1.03)(1.0)(85)] = 420 \text{ kips}$$

Since:

$$R_r = \phi_{bb} R_n$$

R_r = 0.80(420) = 336 kips

The total factored shear resistance of the bolts in the four holes adjacent to the end of the flange is 4(64.6) = 258 kips < 336 kips. Therefore, the factored shear resistance of the bolts controls and bearing does not control for the four end holes.

For the other twenty bolt holes, the center-to-center distance between the bolt holes in the direction of the applied force is 3.0 in. Therefore, the clear distance, L_c, between the edges of the adjacent holes is:

$$L_c = 3.0 - 0.9375 = 2.0625 \text{ in.} > 2.0d = 1.75 \text{ in.}$$

Therefore, use Eq. (2.1.1.3-3):

$$R_n = 20(2.4dtF_u) = 20[2.4(0.875)(1.0)(85)] = 3{,}570 \text{ kips}$$

Since:

$$R_r = \phi_{bb} R_n$$

R_r = 0.80(3,570) = 2,856 kips

The total factored shear resistance of the bolts in the twenty interior bolt holes is 20(64.6) = 1,292 kips < 2,856 kips. Therefore, the factored shear resistance of the bolts controls and bearing does not control for the twenty interior bolt holes.

The total factored shear resistance of the bolts in the twenty-four holes is:

R_r = 258 + 1,292 = 1,550 kips > P_{fy} = 1,152 kips ok

Calculations similar to the above show that the bearing resistance of the connected material in the top flange splice does not control, and that the total factored shear resistance of the bolts in the twelve bolt holes in the top flange splice is sufficient.

3.1.2.2 Slip Resistance Check

The moment resistance provided by nominal slip resistance of the flange splice bolts is checked against the factored moment for checking slip. The factored moments for checking slip are taken as the moment at the point of splice under Load Combination Service II, and also the factored moment at the point of splice due to the deck casting sequence.

Service II Positive Moment (Figure 2.1.1.2-1; t_{haunch} = 0)

Service II Positive Moment = 1.0(+248+50) + 1.0(+52) + 1.3(+2,469) = +3,560 kip-ft

Use the nominal slip resistance of the bottom flange splice bolts.

The nominal slip resistance of a single bolt assuming a Class B surface condition and standard holes is Eq. (2.1.2-1):

$$R_n = K_h K_s N_s P_t = 1.0(0.50)(2)(39.0) = 39.0 \text{ kips}$$

Nominal slip resistance of the bottom flange splice with 24 bolts: P_t = 24(39.0 kips/bolt) = 936 kips

Flange Moment Arm: $A = D_w + t_{ft}/2 + t_{fc} + t_s/2$ = 69 + 1.375/2 + 1 + 9/2 = 75.2 in.

M_{flange} = 936 x (75.2/12) = 5,866 kip-ft > 3,560 kip-ft ok

Service II Negative Moment (Figure 2.1.1.2-2)

Service II Negative Moment = 1.0(+248+50) + 1.0(+52) + 1.3(-1,754) = -1,930 kip-ft

Use the nominal slip resistance of the top or bottom flange splice bolts, whichever is smaller.

Nominal slip resistance of the top flange splice with 12 bolts: P_t = 12(39.0 kips/bolt) = 468 kips < 936 kips

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2 = 69 + (1.375+1)/2 = 70.2$ in.

M_{flange} = 468 x (70.2/12) = 2,738 kip-ft > |-1,930| kip-ft   ok

Deck Casting (Figure 2.1.1.2-2)

$M_{deck\ casting}$ = 1.4(+1,300) = +1,820 kip-ft

The deck-casting moment is applied to the noncomposite section. Use the nominal slip resistance of the top or bottom flange splice bolts, whichever is smaller.

Nominal slip resistance of the top flange splice with 12 bolts: P_t = 12(39.0 kips/bolt) = 468 kips < 936 kips

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2 = 69 + (1.375+1)/2 = 70.2$ in.

M_{flange} = 468 x (70.2/12) = 2,738 kip-ft > 1,820 kip-ft   ok

Therefore, the flanges have adequate slip resistance to resist the Service II and deck casting moment requirements at the splice. No moment contribution from the web is required.

3.1.3 Web Splice Design

3.1.3.1 Strength Limit State Design

3.1.3.1.1 Bolts

In this case, since the moment resistance provided by the flanges is sufficient to resist the factored moment at the strength limit state at the point of splice, the web splice bolts are designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web on either side of the splice. The factored shear resistance, V_r, of the smaller ½ in. x 69 in. Grade 50W web on the left side of the splice with a transverse-stiffener spacing of 3 times the web depth is determined to be (AASHTO LRFD Article 6.10.9):

$V_r = \phi_v V_n$ = 468 kips

The factored shear resistance of the bolts in double shear (N_s = 2) is computed as follows:

Bolts with threads included in the shear plane (Eq. (2.1.1.3-2)):

$$R_r = \phi_s R_n = 0.80 * 0.45 A_b F_{ub} N_s = 0.80(0.45)(0.601)(120)(2) = 51.9 \text{ kips}$$

Number of Bolts Required: N = 468/51.9 = 9.02

Note that the greater than 38.0 in. length reduction for the shear resistance of the bolts only applies to lap-splice tension connections (AASHTO LRFD Article 6.13.2.7), and is not to be applied in the design of the web splice.

The AASHTO specification requires at least two rows of bolts in the web over the depth of the web (AASHTO LRFD Article 6.13.6.1.3a). The maximum permitted spacing of the bolts for sealing is $s \leq (4.0 + 4.0t) \leq 7.0$ in., where t is the thickness of the splice plate (AASHTO LRFD Article 6.13.2.6.2). Assuming the splice plate thickness will be one-half the smaller web thickness at the point of splice plus $^1/_{16}$ in. gives a splice plate thickness of:

$$t = \frac{1}{2} \text{ x } \frac{1}{2} + \frac{1}{16} = \frac{5}{16} \text{ in.}$$

which is equal to the minimum permitted thickness of structural steel (AASHTO LRFD Article 6.7.3). The maximum bolt spacing for the $^5/_{16}$ in. splice plate is:

$$4.0 + 4 \text{ x } \frac{5}{16} = 5.25 \text{ in.}$$

Using a 3.0 in. gap from the top and bottom of the web to the top and bottom web splice bolts so as to not impinge on bolt assembly clearances, the available web depth for the bolt pattern is 69 – (2 x 3) = 63.0 in. The number of bolts required to meet the maximum bolt spacing is:

Number of bolts = $1 + \frac{63}{5.25}$ = 13 bolts in two vertical rows on each side of the splice

= 26 bolts > 9.02.

3.1.3.1.2 Splice Plates

The web splice plates are $^5/_{16}$ in. x 66 in. The plates are ASTM A709 Grade 50W steel. Note that no filler is required since the difference in the web thicknesses at the point of splice is equal to $^1/_{16}$ in. (AASHTO LRFD Article 6.13.6.1.3c).

The factored shear resistance of the web at the strength limit state, V_r, is not to exceed the factored shear yielding or factored shear rupture resistance of the web splice plates (AASHTO LRFD Article 6.13.6.1.3c).

For shear yielding, the factored resistance of the web splice plates is determined as (Eq. (2.2.1.3-1)):

$$R_r = 1.0(0.58)(50)(2)(0.3125)(66.0) = 1{,}196\ \text{kips}$$

$$R_r = 1{,}196\ \text{kips} > V_r = 468\ \text{kips} \quad \text{ok}$$

For shear rupture, the factored resistance of the web splice plates is determined as (Eq. (2.2.1.3-2)):

$$R_r = 0.80(0.58)(1.0)(70)(2)[66.0 - 26(0.9375)](0.3125) = 845\ \text{kips}$$

$$R_r = 845\ \text{kips} > V_r = 468\ \text{kips} \quad \text{ok}$$

In order to check the block shear rupture resistance of the splice plates (and later on the factored bearing resistance of the bolt holes in Section 3.1.3.1.3 Bearing Resistance Check), the bolt edge and end distances must first be established and checked.

The edge distance of bolts is defined as the distance perpendicular to the line of force between the center of a hole and the edge of the component. In this example, the edge distance from the center of the vertical line of holes in the web plate to the edge of the field piece of 2.0 in. satisfies the minimum edge distance requirement of 1-⅛ in. specified for ⅞-in.-diameter bolts in AASHTO LRFD Table 6.13.2.6.6-1. This distance also satisfies the maximum edge distance requirement of 8.0*t* (not to exceed 5.0 in.) = 8.0(0.3125) = 2.5 in. specified in AASHTO LRFD Article 6.13.2.6.6. The edge distance for the outermost vertical row of holes on the web splice plates is set at 2.0 in.

The end distance of bolts is defined as the distance along the line of force between the center of a hole and the end of the component. In this example, the end distance of 1-½ in. at the top and bottom of the web splice plates satisfies the minimum end distance requirement of 1-⅛ in. specified for ⅞-in.-diameter bolts. The maximum end distance requirement of 2-½ in. is also satisfied. Although not specifically required, note that the distance from the corner bolts to the corner of the web splice plate, equal to $\sqrt{(2.0)^2 + (1.5)^2} = 2.5\ \text{in.}$, also satisfies the maximum end distance requirement.

The factored shear resistance of the web, V_r, is also not to exceed the block shear rupture resistance of the web splice plates at the strength limit state (Eq. (2.1.1.4.2-3)). Because of the overall length of the connection, the block shear rupture resistance normally does not control for web splice plates of typical proportion, but the check is illustrated here for completeness. Assume the block shear failure plane on the web splice plates shown in Fig. 3.1.3.1.2-1:

A_{tn} is the net area along the place resisting the tensile stress.

$$A_{tn} = 2[3.0 + 2.0 - 1.5(0.9375)](0.3125) = 2.25\ \text{in.}^2$$

A_{vn} is the net area along the place resisting the shear stress.

$$A_{vn} = 2[66.0 - 1.5 - 12.5(0.9375)](0.3125) = 32.99 \text{ in.}^2$$

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

$$A_{vg} = 2[66.0 - 1.5](0.3125) = 40.31 \text{ in.}^2$$

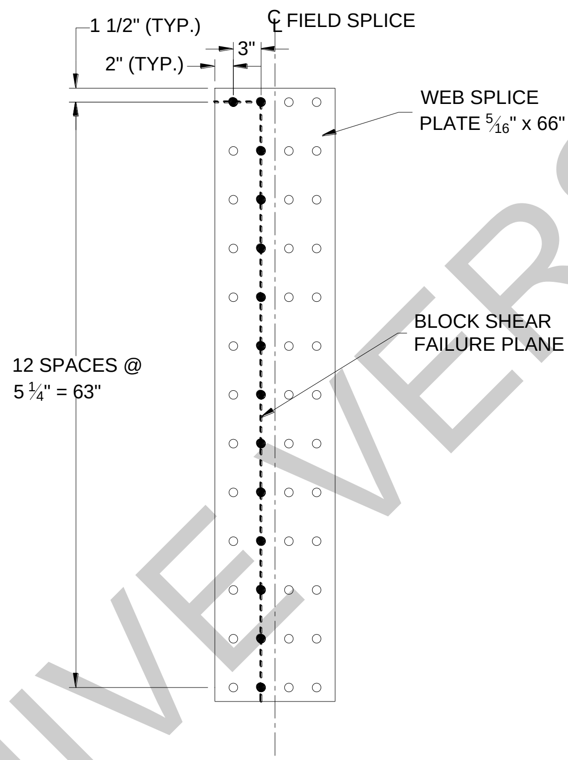


Fig. 3.1.3.1.2-1 Assumed Block Shear Failure Plane in the Web Splice Plates

Therefore:

$$R_r = 0.80(1.0)[0.58(70)(32.99) + 1.0(70)(2.25)] = 1{,}198 \text{ kips} > 0.80(1.0)[0.58(50)(40.31) + 1.0(70)(2.25)] = 1{,}061 \text{ kips}$$

$$\therefore R_r = 1{,}061 \text{ kips} > V_r = 468 \text{ kips} \quad \text{ok}$$

Since the combined area of the web splice plates is greater than the area of the smaller web at the point of splice, the fatigue stresses in the base metal of the web splice plates adjacent to the slip-critical bolted connections need not be checked. Also, the flexural stresses in the splice plates at the service limit state under the Service II load combination need not be checked.

3.1.3.1.3 Bearing Resistance Check

Check the bearing resistance of the web-splice bolt holes at the strength limit state. Since the web at the point of splice with the smaller product of its thickness times its specified minimum tensile strength, F_u (i.e., the web on the left side of the splice) is less than the sum of the web splice-plate thicknesses times the corresponding F_u of the splice plates, the web on the left side of the splice controls the bearing resistance of the connection.

For standard-size holes, the nominal bearing resistance, R_n, parallel to the applied bearing force is given by Eq. (2.1.1.3-3) or Eq. (2.1.1.3-4), as applicable.

For the two bolt holes at the bottom of the web splice, the clear distance, L_c, between the edge of the hole and the end of the web in the direction of the applied force is:

$$L_c = 3.0 - \frac{0.9375}{2} = 2.5313 \text{ in.} > 2.0d = 2.0(0.875) = 1.75 \text{ in.}$$

Therefore, use Eq. (2.1.1.3-4):

$$R_n = 2(2.4dtF_u) = 2[2.4(0.875)(0.5)(70)] = 147 \text{ kips}$$

Since:

$$R_r = \phi_{bb} R_n$$

R_r = 0.80(147) = 118 kips

The total factored shear resistance of the bolts in the two holes at the bottom of the web splice is 2(51.9) = 104 kips < 118 kips. Therefore, the factored shear resistance of the bolts controls and bearing does not control for these two holes.

The center-to-center distance between the other twenty-four bolt holes in the direction of the applied force is 5-¼ in. Therefore, the clear distance, L_c, between the edges of the adjacent holes is:

$$L_c = 5.25 - 0.9375 = 4.3125 \text{ in.} > 2.0d = 2.0(0.875) = 1.75 \text{ in.}$$

Therefore, use Eq. (2.1.1.3-3):

$$R_n = 24(2.4dtF_u) = 24[2.4(0.875)(0.5)(70)] = 1{,}764 \text{ kips}$$

Since:

$$R_r = \phi_{bb} R_n$$

R_r = 0.80(1,764) = 1,411 kips

The total factored shear resistance of the other twenty-four bolts in the web splice is 24(51.9) = 1,246 kips < 1,411 kips. Therefore, the factored shear resistance of the bolts controls and bearing does not control for other twenty-four holes.

The total factored shear resistance of the bolts in the twenty-six holes is:

R_r = 104 + 1,246 = 1,350 kips > V_r = 468 kips ok

3.1.3.2 Slip Resistance Check

Since the moment resistance provided by the nominal resistance of the flange splice bolts is sufficient to resist the factored moment for checking slip at the point of splice in this case (see Section

3.1.2.2 Slip ***Resistance Check***), the web splice bolts are simply checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice (AASHTO LRFD Article 6.13.6.1.3c). The factored shear for checking slip is taken as the shear in the web at the point of splice under Load Combination Service II, or the shear in the web at the point of splice due to the deck casting sequence, whichever governs.

The unfactored shears at the point of splice are as follows:

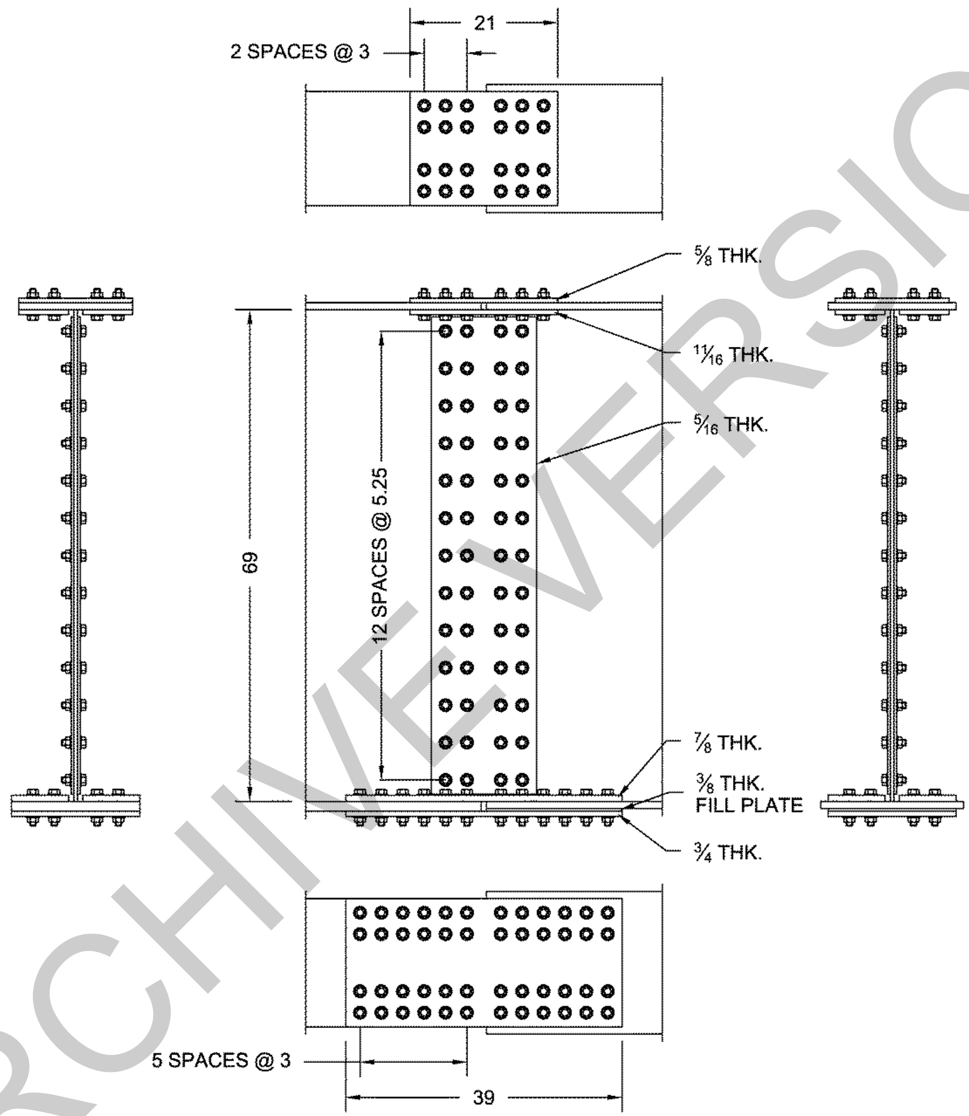
V_{DC1}	=	-82 kips
V_{DC2}	=	-12 kips
V_{DW}	=	-11 kips
V_{+LL+IM}	=	+19 kips
V_{-LL+IM}	=	-112 kips
$V_{deck\ casting}$	=	-82 kips

By inspection, the Service II negative shear controls.

Service II Negative Shear = 1.0(-82 + -12) + 1.0(-11) + 1.3(-112) = -250.6 kips > $V_{deck\ casting}$ = 1.4(-82) = -114.8 kips

Slip resistance of web splice with 26 bolts: P_t = 26(39.0 kips/bolt) = 1,014 kips > |-250.6| kips ok

A schematic of the final splice for Design Example 1 is shown below.



DESIGN EXAMPLE 1 Bolted Splice Design

3.2 Design Example 2

3.2.1 General

Design Example 2 is an example design of a bolted field splice for the exterior girder of an I-section flexural member. The splice is located near the point of permanent load contraflexure in the end span of a three-span continuous bridge on right supports with spans of 234-300-234 ft and a girder spacing of 12.0 ft. The girder plate sizes at the point of splice are given in Table 3.2.1-1. The unfactored design moments at the point of splice are also listed below. ⅞-in.-diameter ASTM F3125 Grade 325 bolts are used for both the flange and web splices. The threads are assumed excluded from the shear planes in the flange splices, and included in the shear planes in the web splice. The calculation of the factored shear resistance and the nominal slip resistance of the bolts is given in Design Example 1.

The sections on both slides of the splice are homogeneous utilizing ASTM A709 Grade 50 steel for the flanges and web. The structural thickness of the concrete deck is 8.0 in. The concrete deck haunch is ignored in this particular example.

M_{DC1} = -1,564 kip-ft
M_{DC2} = -242 kip-ft
M_{DW} = -315 kip-ft
M_{+LL+IM} = +5,627 kip-ft
M_{-LL+IM} = -7,117 kip-ft
$M_{deck\ casting}$ = +3,006 kip-ft

Table 3.2.1-1 Design Example 2 Plate Dimensions

Side of Splice	Top Flange Width (in.)	Top Flange Thickness (in.)	Web Depth (in.)	Web Thickness (in.)	Bottom Flange Width (in.)	Bottom Flange Thickness (in.)
Left	19	1	109	¾	19	1
Right	22	2	109	¾	22	2

By inspection, the left side has the smallest flanges which will control the design. Also, 1-in. fillers are required for both top and bottom flange splices.

The factored Strength I design moments at the point of splice are computed as follows:

Positive Moment = 0.9(-1,564 + -242) + 0.65(-315) + 1.75(+5,627) = +8,017 kip-ft

Negative Moment = 1.25(-1,564 + -242) + 1.5(-315) + 1.75(-7,117) = -15,185 kip-ft

3.2.2 Flange Splice Design

3.2.2.1 Strength Limit State Design

3.2.2.1.1 Bolts

Top Flange

The left side of the splice has the smaller design yield resistance (i.e., the top flange on the left side has a smaller area).

Assuming 4 rows of bolts across the width of the flange (Eq. (2.1.1.1-3)
$A_e = 1.09(1)[19\text{-}4(^{15}/_{16})] = 16.6\text{ in.}^2 < 1(19)\text{ in.}^2 = 19.0\text{ in.}^2$

$P_{fy} = 50(16.6) = 830$ kips

Reduction in bolt factored shear resistance due to filler (Eq. (2.1.3-1)):

$$R = \frac{1+\frac{1}{1}}{1+\frac{2(1)}{1}} = 0.67$$

Note that the splice plate, filler plate, and flange widths will be equal in the splice. Consequently, the area ratios are only a function of the thickness of the flange and fillers.

Number of Bolts Required (threads excluded from the shear planes): $N = 830/(0.67(64.6)) = 19.2$

Use 4 rows with 5 bolts per row = 20 bolts on each side of the splice.

Bottom Flange

The left side of the splice has the smaller design yield resistance (i.e., the bottom flange on the left side has a smaller area).

Assuming 4 rows of bolts across the width of the flange (Eq. (2.1.1.1-3)

$A_e = 1.09(1)[19\text{-}4(^{15}/_{16})] = 16.6\text{ in.}^2 < 1(19)\text{ in.}^2 = 19.0\text{ in.}^2$

P_{fy} = 50(16.6) = 830 kips

Reduction in bolt factored shear resistance due to filler (Eq. (2.1.3-1)):

$$R = \frac{1+\frac{1}{1}}{1+\frac{2(1)}{1}} = 0.67$$

Number of Bolts Required (threads excluded from the shear planes): N = 830/(0.67(64.6)) = 19.2

Use 4 rows with 5 bolts per row = 20 bolts on each side of the splice.

3.2.2.1.2 Moment Resistance

Positive Moment (Figure 2.1.1.2-1; t_{haunch} = 0)

Use P_{fy} for the bottom flange = 830 kips.

Flange Moment Arm: $A = D_w + t_{ft}/2 + t_{fc} + t_s/2$ = 109 + 1/2 + 1 + 8/2 = 114.5 in.

M_{flange} = 830 x (114.5/12) = 7,948 kip-ft < 8,017 kip-ft; therefore, the flanges do not have adequate capacity by themselves to resist the factored Strength I positive moment at the point of splice.

Required Horizontal Web Force to satisfy the Strength I moment requirement (Figure 2.2.1.1-1; t_{haunch} = 0):

$$H_w = \frac{\text{Strength I Moment-Flange Moment}}{\frac{D_w}{2} + t_c + \frac{t_s}{2}} = \frac{(8{,}017 - 7{,}948) \times 12}{54.5 + 1 + 4} = 13.9 \text{ kips}$$

Negative Moment (Figure 2.1.1.2-2)

Use the smaller value of P_{fy} for the top and bottom flanges. In this case, both flanges have the same value of P_{fy} = 830 kips.

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2$ = 109 + (1 + 1)/2 = 110 in.

M_{flange} = 830 x (110/12) = 7,608 kip-ft < |-15,185 kip-ft|; therefore, the flanges do not have adequate capacity by themselves to resist the factored Strength I negative moment at the point of splice.

Required Horizontal Web Force to satisfy the Strength I moment requirement (Figure 2.2.1.1-2):

$$H_w = \frac{\text{Strength I Moment-Flange Moment}}{\frac{D_w}{2} + \frac{t_f}{2}} = \frac{(15{,}185 - 7{,}608) \times 12}{54.5 + 0.5} = 1{,}653 \text{ kips} > 13.9 \text{ kips}$$

Therefore, negative moment controls the web connection design (see Section 3.2.3.1.1 Bolts)

3.2.2.1.3 Splice Plates

The width of the outside splice plate should be at least as wide as the width of the narrowest flange at the splice. The width of the inside splice plates should be such that the plates clear the flange-to-web weld on each side of the web by a minimum of ⅛ in. Therefore, for both the top- and bottom-flange splice, try a $^{9}/_{16}$ in. x 19 in. outside splice plate and two ⅝ in. x 8-½ in. inside splice plates. Include a 1 in. x 19 in. filler plate on the outside for both flange splices. All plates are ASTM A709 Grade 50 steel.

At the strength limit state, P_{fy} may be assumed equally divided to the inner and outer flange splice plates when the areas of the inner and outer plates do not differ by more than 10 percent. In this case, the areas of the inner and outer plates are within 1 percent. Therefore, P_{fy} is equally divided to the inner and outer plates and the shear resistance of the bolted connection is checked above for P_{fy} acting in double shear.

Check the factored yield resistance of the splice plates in tension (Eq. (2.1.1.4.2-1)):

Outside splice plate:

$$R_r = 0.95(50)(19.0)(0.5625) = 508 \text{ kips} > 830/2 = 415 \text{ kips} \quad \text{ok}$$

Inside splice plates:

$$R_r = 0.95(50)(2)(8.5)(0.625) = 505 \text{ kips} > 830/2 = 415 \text{ kips} \quad \text{ok}$$

Check the net section fracture resistance of the splice plates in tension (Eq. (2.1.1.4.2-2)). As specified in AASHTO LRFD Article 6.8.3, for design calculations, the width of standard-size bolt holes is taken as the nominal diameter of the holes, or $^{15}/_{16}$ in. for a ⅞-in.-diameter bolt:

Outside plate:

$$R_r = 0.80(65)[19.0 - 4(0.9375)](0.5625)(1.0)(1.0) = 446 \text{ kips} > 830/2 = 415 \text{ kips} \quad \text{ok}$$

Inside plates:

$$R_r = 0.80(65)[2(8.5) - 4(0.9375)](0.625)(1.0)(1.0) = 431 \text{ kips} > 830/2 = 415 \text{ kips} \quad \text{ok}$$

Also, according to AASHTO LRFD Article 6.13.5.2, for splice plates subject to tension, the design net area, A_n, must not exceed $0.85A_g$.

Outside plate:

$$0.85(19.0)(0.5625) = 9.1\text{ in.}^2 > A_n = [19.0 - 4(0.9375)](0.5625) = 8.6\text{ in.}^2 \quad \text{ok}$$

Inside plates:

$$0.85(2)(8.5)(0.625) = 9.0\text{ in.}^2 > A_n = [2(8.5) - 4(0.9375)](0.625) = 8.3\text{ in.}^2 \quad \text{ok}$$

The flange splice bolt spacings are established and checked as illustrated in Section 3.1.2.1.3 Splice Plates of Design Example 1. Separate calculations similar to those illustrated in Design Example 1 (Sections 3.1.2.1.3 Splice Plates and 3.1.2.1.4 Bearing Resistance Check) indicate that the block shear rupture resistance of the flange splice plates and the girder flanges, and the bearing resistance of the flange splice bolt holes are sufficient.

Because the splice is located near a point of permanent load contraflexure and the factored flexural resistance of the tension flange is less than or equal to the specified minimum yield strength of the tension flange at that point, AASHTO LRFD Eq. 6.10.1.8-1 will not control at this location and need not be checked since this equation was used to design the flange splices at the strength limit state.

Since the combined area of the inside and outside flange splice plates is greater than the area of the smaller flange at the point of splice, fatigue of the base metal of the splice plates adjacent to the slip-critical bolted connections does not need to be checked. Similarly, the flexural stresses in the splice plates at the service limit state under the Service II load combination need not be checked.

3.2.2.2 Slip Resistance Check

The moment resistance provided by the nominal slip resistance of the flange splice bolts is checked against the factored moment for checking slip. The factored moments for checking slip are taken as the moment at the point of splice under Load Combination Service II, and also the factored moment at the point of splice due to the deck casting sequence.

Service II Positive Moment (Figure 2.1.1.2-1; $t_{haunch} = 0$)

Service II Positive Moment = 1.0(-1,564 + -242) + 1.0(-315) + 1.3(+5,627) = +5,194 kip-ft

Use the slip resistance of the bottom-flange splice bolts.

Nominal slip resistance of the bottom-flange splice with 20 bolts: P_t = 20(39.0 kips/bolt) = 780 kips

Flange Moment Arm: $A = D_w + t_{ft}/2 + t_{fc} + t_s/2$ = 109 + 1/2 + 1 + 8/2 = 114.5 in.

M_{flange} = 780 x (114.5/12) = 7,442 kip-ft > 5,194 kip-ft   ok

Service II Negative Moment (Figure 2.1.1.2-2)

Service II Negative Moment = 1.0(-1,564 + -242) + 1.0(-315) + 1.3(-7,117) = -11,373 kip-ft

Use the smaller of the slip resistances of the top and bottom flange splice bolts. In this case, the bolts in both flange splices have the same nominal slip resistance = 780 kips.

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2$ = 109 + (1 + 1)/2 = 110 in.

M_{flange} = 780 x (110/12) = 7,150 kip-ft < |-11,373|kip-ft; therefore, the flange splice bolts do not have adequate slip resistance by themselves to prevent slip under the factored Service II negative moment at the point of splice.

Required Horizontal Web Force to satisfy the Service II moment requirement (Figure 2.2.1.1-2):

$$H_w = \frac{\text{Service II Moment-Flange Moment}}{\frac{D_w}{2} + \frac{t_f}{2}} = \frac{\left(|-11{,}373| - 7{,}150\right) \text{ x } 12}{54.5 + 0.5} = 921 \text{ kips}$$

See Section

3.2.3.2 Slip Resistance ***Check***.

Deck Casting (Figure 2.1.1.2-2)

$M_{deck\ casting}$ = 1.4(+3,006) = +4,208 kip-ft

The deck-casting moment is applied to the noncomposite section. Use the nominal slip resistance of the top or bottom flange splice bolts, whichever is smaller. In this case, the bolts in both flange splices have the same nominal slip resistance = 780 kips.

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2$ = 109 + (1 + 1)/2 = 110 in.

M_{flange} = 780 x (110/12) = 7,150 kip-ft > 4,208 kip-ft   ok

3.2.3 Web Splice Design

3.2.3.1 Strength Limit State Design

3.2.3.1.1 Bolts

Since the moment resistance provided by the flange splices is not sufficient to resist the factored moment at the strength limit state at the point of splice in this case, the web splice bolts are designed at the strength limit state for a design web force taken equal to the vector sum of the smaller factored shear resistance of the web on either side of the splice and the horizontal web force, H_w = 1,653 kips, computed in Section 3.2.2.1.2 Moment Resistance (Eq. (2.2.1.1-2)). The factored shear resistance, V_r, of the ¾ in. x 109 in. Grade 50 web with a transverse-stiffener spacing at the point of splice of 2 times the web depth is determined to be (AASHTO LRFD Article 6.10.9):

$V_r = \phi_v V_n = 1{,}312$ kips

$$R = \sqrt{(V_r)^2 + (H_w)^2} = \sqrt{(1{,}312)^2 + (1{,}653)^2} = 2{,}110 \text{ kips}$$

Number of Bolts Required (threads included in the shear plane): $N = 2{,}110/51.9 = 40.7$

Note that the greater than 38.0 in. length reduction for the shear resistance of the bolts only applies to lap-splice tension connections (AASHTO LRFD Article 6.13.2.7) and is not to be applied in the design of the web splice.

The AASHTO specification requires at least two rows of bolts in the web over the depth of the web (AASHTO LRFD Article 6.13.6.1.3a). The maximum permitted spacing of the bolts for sealing is $s \leq (4.0 + 4.0t) \leq 7.0$ in., where t is the thickness of the splice plate (AASHTO LRFD Article 6.13.2.6.2). Assuming the splice plate thickness will be one-half the smaller web thickness at the point of splice plus 1/16 in. gives a splice plate thickness of:

$$t = \frac{1}{2} \text{ x } \frac{3}{4} + \frac{1}{16} = \frac{7}{16} \text{ in.}$$

The maximum bolt spacing for the $^7/_{16}$ in. splice plate is:

$$4.0 + 4 \text{ x } \frac{7}{16} = 5.75 \text{ in.}$$

Try a 5-⅛ in. spacing. Using a 3-¼ in. gap from the top and bottom of the web to the top and bottom web splice bolts so as to not impinge on bolt assembly clearances, the available web depth for the bolt pattern is 109 – (2 x 3.25) = 102.5 in. The number of bolts required to meet the maximum bolt spacing is:

Number of bolts $= 1 + \frac{102.5}{5.125} = 21$ bolts in two vertical rows on each side of the splice

= 42 bolts > 40.7.

3.2.3.1.2 Splice Plates

The web splice plates are $^7/_{16}$ in. x 106 in. The plates are ASTM A709 Grade 50 steel. Note that no filler is required since the web thicknesses are the same on each side of the splice.

The factored shear resistance of the web at the strength limit state, V_r, is not to exceed the factored shear yielding or factored shear rupture resistance of the web splice plates (AASHTO LRFD Article 6.13.6.1.3c).

For shear yielding, the factored resistance of the web splice plates is determined as (Eq. (2.2.1.3-1)):

$$R_r = 1.0(0.58)(50)(2)(0.4375)(106.0) = 2{,}690 \text{ kips}$$

$$R_r = 2{,}690 \text{ kips} > V_r = 1{,}312 \text{ kips} \quad \text{ok}$$

For shear rupture, the factored resistance of the web splice plates is determined as (Eq. (2.2.1.3-2)):

$$R_r = 0.80(0.58)(1.0)(65)(2)[106.0 - 42(0.9375)](0.4375) = 1{,}758 \text{ kips}$$

$$R_r = 1{,}758 \text{ kips} > V_r = 1{,}312 \text{ kips} \quad \text{ok}$$

The factored shear resistance of the web, V_r, is also not to exceed the block shear rupture resistance of the web splice plates. In order to check the block shear rupture resistance of the web splice plates (and later on the factored bearing resistance of the bolt holes in Section 3.2.3.1.3 Bearing Resistance Check), the bolt edge and end distances must first be established and checked.

The edge distance of bolts is defined as the distance perpendicular to the line of force between the center of a hole and the edge of the component. In this example, the edge distance from the center of the vertical line of holes in the web plate to the edge of the field piece of 2.0 in. satisfies the minimum edge distance requirement of 1-⅛ in. specified for ⅞-in.-diameter bolts in AASHTO LRFD Table 6.13.2.6.6-1. This distance also satisfies the maximum edge distance requirement of 8.0*t* (not to exceed 5.0 in.) = 8.0(0.4375) = 3.5 in. specified in AASHTO LRFD Article 6.13.2.6.6. The edge distance for the outermost vertical row of holes on the web splice plates is set at 2.0 in.

The end distance of bolts is defined as the distance along the line of force between the center of a hole and the end of the component. In this example, the end distance of 1-¾ in. at the top and bottom of the web splice plates satisfies the minimum end distance requirement of 1-⅛ in. specified for ⅞-in.-diameter bolts. The maximum end distance requirement of 3-½ in. is also satisfied. Although not specifically required, note that the distance from the corner bolts to the corner of the web splice plate, equal to $\sqrt{(2.0)^2 + (1.75)^2} = 2.7 \text{ in.}$, also satisfies the maximum end distance requirement.

Separate calculations similar to those illustrated in Design Example 1 (Section 3.1.3.1.2 Splice Plates) indicate that the block shear rupture resistance of the web splice plates is sufficient.

Since the combined area of the web splice plates is greater than the area of the web at the point of splice, the fatigue stresses in the base metal of the web splice plates adjacent to the slip-critical bolted connections need not be checked. Also, the flexural stresses in the splice plates at the service limit state under the Service II load combination need not be checked.

3.2.3.1.3 Bearing Resistance Check

Check the bearing resistance of the web-splice bolt holes at the strength limit state. In this example, the web on each side of the splice has the same thickness and specified minimum tensile strength, F_u. Since in this case the thickness of the web at the point of splice times its specified minimum tensile strength, F_u, is less than the sum of the web splice-plate thicknesses times the corresponding F_u of the splice plates, the web controls the bearing resistance of the connection.

Since a moment contribution from the web is required in this case, the resultant forces causing bearing on the web bolt holes are inclined. To calculate the bearing resistance of each bolt hole in the web for inclined resultant forces, the clear edge distance can conservatively be used (Figure 2.2.1.2-1). Since the resultant forces act in the direction of inclined distances that are larger than the clear edge distance, the check is conservative. Other options for checking the bearing resistance were discussed previously in Section 2.2.1.2 Web Splice Bolts. Based on the edge distance from the center of the hole to the edge of the field section of 2.0 in., the clear edge distance, L_c, is computed as:

$$L_c = 2.0 - \frac{0.9375}{2} = 1.53 \text{ in.} < 2.0d = 2.0(0.875) = 1.75 \text{ in.}$$

Therefore, use Eq. (2.1.1.3-4):

$$R_n = 42\left[1.2L_c t F_u\right] = 42\left[1.2(1.53)(0.75)(65)\right] = 3{,}759 \text{ kips}$$

Since:

$$R_r = \phi_{bb} R_n$$

R_r = 0.80(3,759) = 3,007 kips

The total factored shear resistance of the forty-two bolts in the web splice is 42(51.9) = 2,180 kips < 3,007 kips. Therefore, the factored shear resistance of the bolts controls and bearing does not control.

The total factored shear resistance of the bolts in the forty-two holes is:

R_r = 2,180 kips > R = 2,110 kips   ok

Had bearing controlled and the bearing resistance been exceeded, the preferred option would be to increase the edge distance slightly in lieu of increasing the number of bolts or thickening the web.

3.2.3.2 Slip Resistance Check

Since the moment resistance provided by the nominal slip resistance of the flange splice bolts is not sufficient to resist the factored moment for checking slip at the point of splice in this case, the web splice bolts are checked for slip under a web slip force taken equal to the vector sum of the factored shear in the web at the point of splice and a horizontal force, H_w = 921 kips (computed in Section 3.2.2.2 Slip Resistance Check), located at the mid-depth of the web that provides the necessary slip resistance in conjunction with the flanges. The factored shear for checking slip is taken as the shear in the web at the point of splice under Load Combination Service II, or the shear in the web at the point of splice due to the deck casting sequence, whichever governs.

The unfactored shears at the point of splice are as follows:

V_{DC1}	=	-147 kips
V_{DC2}	=	-28 kips
V_{DW}	=	-37 kips
V_{+LL+IM}	=	+19 kips
V_{-LL+IM}	=	-126 kips
$V_{deck\ casting}$	=	-79 kips

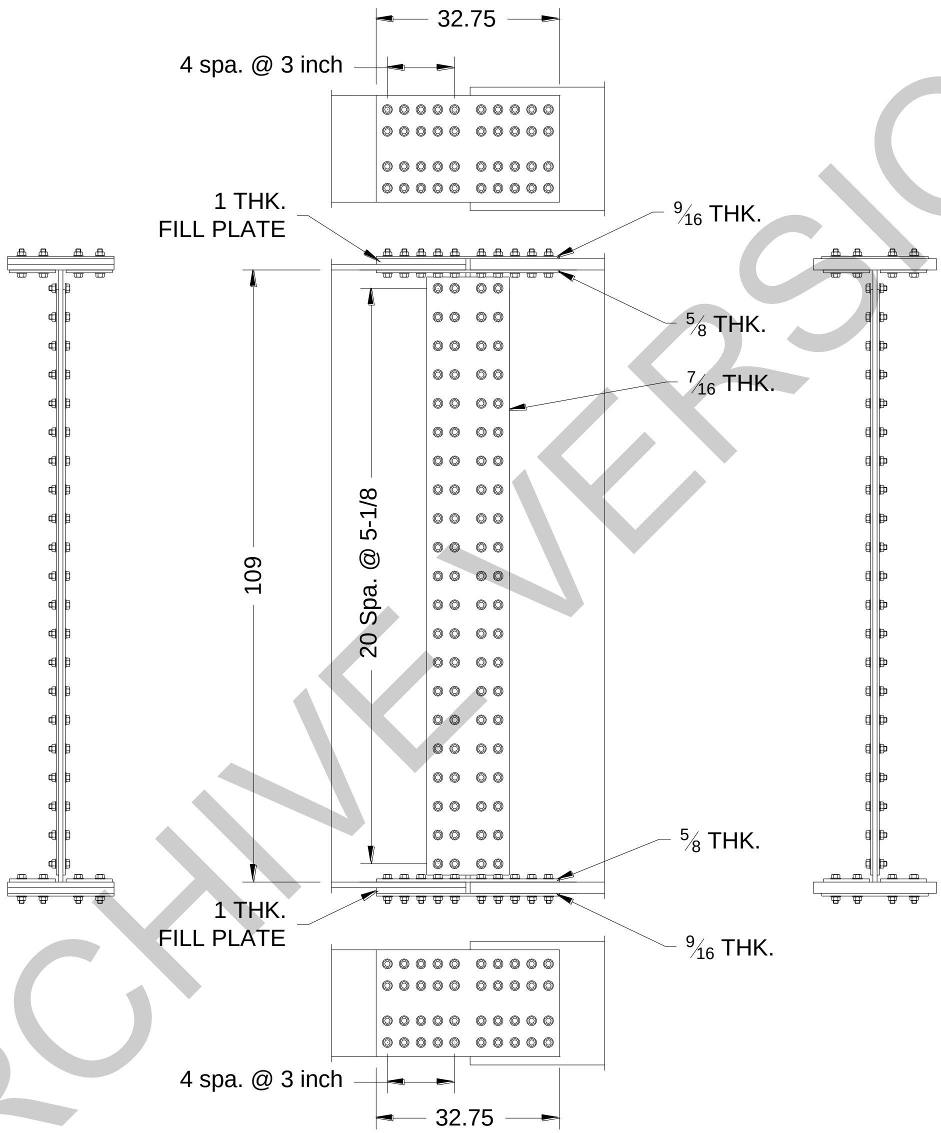
By inspection, the Service II negative shear controls.

Service II Negative Shear = 1.0(-147 + -28) + 1.0(-37) + 1.3(-126) = -376 kips > $V_{deck\ casting}$ = 1.4(-79) = -111 kips

$$R = \sqrt{(-376)^2 + (921)^2} = 995 \text{ kips}$$

Slip resistance of web splice with 42 bolts: P_t = 42(39.0 kips/bolt) = 1,638 kips > 995 kips ok

A schematic of the final splice for Design Example 2 is shown below.



DESIGN EXAMPLE 2 Bolted Splice Design

3.3 Design Example 3

3.3.1 General

Design Example 3 is an example design of a bolted field splice for a horizontally curved continuous twin trapezoidal tub girder on radial supports with spans of approximately 150-200-150 ft, a radius of curvature of 550 ft, and a girder spacing of 12.0 ft. The web slope on each tub girder is 1-to-4. The sections on both slides of the splice are homogeneous utilizing ASTM A709 Grade 50W steel for the flanges and web. The girder plate sizes at the point of splice are given in Table 3.3.1-1. The unfactored design moments and torques at the point of splice are also listed below. ⅞-in.-diameter ASTM F3125 Grade 325 bolts are used for both the flange and web splices. The threads are assumed excluded from the shear planes in the flange splices, and included in the shear planes in the web splice. The calculation of the factored shear resistance and the nominal slip resistance of the bolts is given in Design Example 1.

A 9-½ in. concrete deck with an effective width of 234 in. was used in the design. The concrete deck haunch is 5.0 in. measured from the top of the web.

Only the calculations to determine the number of bolts are presented herein to demonstrate the calculation procedure for a tub girder with torsional and flexural force resultants at the splice location. The remaining calculations for the splice that are not shown are similar to those demonstrated previously for I-girder bolted splices in Design Examples 1 and 2.

M_{DC1}	=	+2,417 k-ft	$T_{deck\ casting}$	=	-217 k-ft
M_{DC2}	=	+251 k-ft	T_{DC1}	=	-252 k-ft
M_{DW}	=	+339 k-ft	T_{DC2}	=	-51 k-ft
M_{+LL+IM}	=	+5,066 k-ft	T_{DW}	=	-39 k-ft
M_{-LL+IM}	=	-2,926 k-ft	T_{+LL+IM}	=	+309 k-ft
$M_{deck\ casting}$	=	+4,082 kip-ft	T_{-LL+IM}	=	-537 k-ft

Table 3.3.1-1 Design Example 3 Plate Dimensions

Side of Splice	Top Flange Width (in.)	Top Flange Thickness (in.)	Web Depth (in.)	Web Thickness (in.)	Bottom Flange Width (in.)	Bottom Flange Thickness (in.)
Left	18	1	80.39 [a]	⅝	76	¾
Right	20	1-¼	80.39 [a]	⅝	76	1-¼

[a] – Web depth measured along the web slope; the vertical web depth is 78.0 in.

By inspection, the left side has the smallest flanges which will control the design. Also, fillers are required for both top and bottom flange splices (¼ in. for the top flange and ½ in. for the bottom flange).

The factored Strength I design moments at the point of splice are computed as follows:

Positive Moment = 1.25(2,417 + 251) + 1.5(339) + 1.75(5,066) = +12,709 kip-ft

Negative Moment = 0.90(2,417 + 251) + 0.65(339) + 1.75(-2,926) = -2,499 kip-ft

3.3.2 Flange Splice Design

3.3.2.1 Strength Limit State Design

3.3.2.1.1 Bolts

<u>*Top Flange*</u>

The left side of the splice has the smaller design yield resistance (i.e., the top flange on the left side has a smaller area).

Assuming 4 rows of bolts across the width of the flange (Eq. (2.1.1.1-3)

$A_e = 1.18(1)[18\text{-}4(^{15}/_{16})] = 16.8 \text{ in.}^2 < 1(18) \text{ in.}^2 = 18.0 \text{ in.}^2$

$P_{fy} = 50(16.8) = 840$ kips

Reduction in bolt factored shear resistance due to filler (Eq. (2.1.3-1)):

$$R = \frac{1+\dfrac{0.25}{1}}{1+\dfrac{2(0.25)}{1}} = 0.83$$

Note that the splice plate, filler plate, and flange widths will be equal in the splice. Consequently, the area ratios are only a function of the thickness of the flange and fillers.

Number of Bolts Required (threads excluded from the shear plane): $N = 840/(0.83(64.6)) = 15.7$

Use 4 rows with 4 bolts per row = 16 bolts on each side of the splice in each top flange.

The effects of flange lateral bending due to curvature in this case need not be considered in the design of the top-flange splices since the flange splices are designed to develop the full yield capacity of the flanges, which cannot be exceeded in the design of the flanges for combined major-axis and lateral bending for constructibility and at the strength limit state, and because the top flanges are continuously braced at the strength limit state. Longitudinal warping stresses due to cross-section distortion also do not need to be considered because the top flanges are continuously braced at the strength limit state.

Bottom Flange

The left side of the splice has the smaller design yield resistance (i.e., the bottom flange on the left side has a smaller area).

Assuming 21 rows of bolts across the width of the flange (Eq. (2.1.1.1-3)

$A_e = 1.18(0.75)[76\text{-}21(^{15}/_{16})] = 49.8 \text{ in.}^2 < 0.75(76) \text{ in.}^2 = 57.0 \text{ in.}^2$

$P_{fy} = 50(49.8) = 2{,}490$ kips

For box sections in horizontally curved bridges, the vector sum of the St. Venant torsional shear and the design yield resistance is to be considered in the design of the bottom flange splice at the strength limit state (AASHTO LRFD Article 6.13.6.1.3b). Longitudinal warping stresses due to cross-section distortion may be ignored at the strength limit state since the bottom flange splices are designed to develop the full design yield capacity of the flanges. Flange lateral bending due to curvature is not a consideration for bottom flanges of box girders.

Calculate the factored St. Venant torsional shear flow, *f*, in the bottom flange at the point of splice for the Strength I load combination. The negative live load plus impact torque controls by inspection.

For the DC_1 torque, which is applied to the non-composite section, the enclosed area, A_o, is computed for the non-composite box section. The vertical depth, D_w, between the mid-thickness of the flanges, which is equal to 78.0 in., is used. The bottom-flange width between the mid-thickness of the tub-girder webs is 72.0 in. Therefore:

$$A_o = \frac{(111+72)}{2} * (78.0 + 0.375 + 0.5) * \frac{1 \text{ ft}^2}{144 \text{ in.}^2} = 50.1 \text{ ft}^2$$

From AASHTO LRFD Eq. C6.11.1.1-1, the St. Venant torsional shear flow is calculated as:

$$f = \frac{T}{2A_o}$$

$$f = \frac{1.0(1.25)(-252)}{2(50.1)} = -3.14 \text{ kips/ft}$$

For the torques applied to the composite section (i.e. the DC_2, DW and LL+IM torques), calculate A_o for the composite section from the mid-thickness of the bottom flange to the mid-thickness of the concrete deck (considering the deck haunch):

$$A_o = \frac{(111+72)}{2} * (78.0 + 0.375 + 5.0 + \frac{9.5}{2}) * \frac{1\text{ ft}^2}{144\text{ in.}^2} = 56.0\text{ ft}^2$$

$$f = \frac{1.0|1.25(-51)+1.5(-39)+1.75(-537)|}{2(56.0)} = -9.48\text{ kips/ft}$$

$$f_{total} = -3.14 + -9.48 = -12.62\text{ kips/ft}$$

The factored St. Venant torsional shear at the strength limit state, V_{SV}, at the point of splice is computed as:

$$V_{SV} = f_{total} b_f = |-12.62| \frac{72.0}{12} = 75.7\text{ kips}$$

The resultant bolt shear force is computed as:

$$R = \sqrt{(P_{fy})^2 + (V_{sv})^2} = \sqrt{(2{,}490)^2 + (75.7)^2} = 2{,}491\text{ kips}$$

Reduction in bolt factored shear resistance due to filler (Eq. (2.1.3-1)):

$$R = \frac{1 + \frac{0.5}{0.75}}{1 + \frac{2(0.5)}{0.75}} = 0.71$$

Number of Bolts Required (threads excluded from the shear planes): N = 2,491/(0.71(64.6)) = 54.3

Use 21 rows with 3 bolts per row = 63 bolts on each side of the splice.

3.3.2.1.2 Moment Resistance

Positive Moment (Figure 2.1.1.2-1)

Use P_{fy} for the bottom flange = 2,490 kips.

Flange Moment Arm: $A = D_w + t_{ft}/2 + t_{haunch} + t_s/2$ = 78 + 0.75/2 + 5 + 9.5/2 = 88.1 in.

M_{flange} = 2,490 x (88.1/12) = 18,281 kip-ft > 12,709 kip-ft ok

Negative Moment (Figure 2.1.1.2-2)

Use the smaller value of P_{fy} for the top and bottom flanges. In this case, the top flanges have the smaller value of P_{fy} = 2 x 840 = 1,680 kips.

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2 = 78 + (0.75 + 1)/2 = 78.9$ in.

$M_{flange} = 1{,}680 \times (78.9/12) = 11{,}046$ kip-ft > |-2,499 kip-ft| ok

Therefore, the flanges have adequate capacity to resist the Strength I moment requirements at the splice. No moment contribution from the web is required.

3.3.2.2 Slip Resistance Check

The moment resistance provided by the nominal slip resistance of the flange splice bolts is checked against the factored moment for checking slip. The factored moments for checking slip are taken as the moment at the point of splice under Load Combination Service II, and also the factored moment at the point of splice due to the deck casting sequence.

St. Venant torsional shears and longitudinal warping stresses due to cross-section distortion are typically neglected in top flanges of tub sections once the flanges are continuously braced. Longitudinal warping stresses due to cross-section distortion are typically relatively small in the bottom flange at the service limit state and for constructibility and may be neglected when checking bottom flange splices for slip. As discussed in Section 2.1.2 Slip Resistance Check, the effects of flange lateral bending also need not be considered in checking the bolted connections of the flange splices for slip.

<u>Service II Positive Moment (Figure 2.1.1.2-1)</u>

Service II Positive Moment = 1.0(+2,417+251) + 1.0(+339) + 1.3(+5,066) = +9,593 kip-ft

Use the nominal slip resistance of the bottom flange splice bolts. For box sections in horizontally curved bridges, for checking slip, the St. Venant torsional shear in the bottom flange is conservatively subtracted from the slip resistance provided by the bottom flange bolts (see Section 2.1.2 Slip Resistance Check).

Calculate the factored St. Venant torsional shear flow, *f*, in the bottom flange at the point of splice for the Service II load combination. The negative live load plus impact torque controls by inspection.

For the DC_1 torque, which is applied to the non-composite section, the enclosed area, A_o, for the non-composite box section was computed previously to be 50.1 ft^2 (Section 3.3.2.1.1 Bolts).

From AASHTO LRFD Eq. C6.11.1.1-1, the St. Venant torsional shear flow is calculated as:

$$f = \frac{T}{2A_o}$$

$$f = \frac{1.0(-252)}{2(50.1)} = -2.51 \text{ kips/ft}$$

For the torques applied to the composite section (i.e. the DC_2, DW and LL+IM torques), A_o for the composite section was computed previously to be 56.0 ft^2 (Section 3.3.2.1.1 Bolts). Therefore:

$$f = \frac{\left|1.0(-51) + 1.0(-39) + 1.3(-537)\right|}{2(56.0)} = -7.04 \text{ kips/ft}$$

$$f_{total} = -2.51 + -7.04 = -9.55 \text{ kips/ft}$$

The bottom-flange width between the mid-thickness of the tub-girder webs is 72.0 in. The factored St. Venant torsional shear for the Service II load combination, V_{SV}, at the point of splice is computed as:

$$V_{SV} = f_{total} b_f = \left|-9.55\right| \frac{72.0}{12} = 57.3 \text{ kips}$$

Nominal slip resistance of the bottom flange splice with 63 bolts:

P_t = 63(39.0 kips/bolt) = 2,457 kips – 57.3 kips = 2,400 kips

Flange Moment Arm: $A = D_w + t_{ft}/2 + t_{haunch} + t_s/2$ = 78 + 0.75/2 + 5 + 9.5/2 = 88.1 in.

M_{flange} = 2,400 x (88.1/12) = 17,620 kip-ft > 9,593 kip-ft ok

Service II Negative Moment (Figure 2.1.1.2-2)

Service II Negative Moment = 1.0(+2,417+251) + 1.0(+339) + 1.3(-2,926) = -797 kip-ft

Use the nominal slip resistance of the top or bottom flange splice bolts, whichever is smaller.

Slip resistance of the top flange splice with 16 bolts each: P_t = 2 x 16(39.0 kips/bolt) = 1,248 kips < 2,400 kips

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2$ = 78 + (0.75 + 1)/2 = 78.9 in.

M_{flange} = 1,248 x (78.9/12) = 8,206 kip-ft > |-797|kip-ft ok

Deck Casting (Figure 2.1.1.2-2)

$M_{deck\ casting}$ = 1.4(+4,082) = +5,715 kip-ft

The deck-casting moment is applied to the noncomposite section. Use the nominal slip resistance of the top or bottom flange splice bolts, whichever is smaller.

Slip resistance of the top flange splice with 16 bolts each: P_t = 2 x 16(39.0 kips/bolt) = 1,248 kips < 2,400 kips

Flange Moment Arm: $A = D_w + (t_{ft} + t_{fc})/2 = 78 + (0.75 + 1)/2 = 78.9$ in.

M_{flange} = 1,248 x (78.9/12) = 8,206 kip-ft > 5,715 kip-ft   ok

Therefore, the flanges have adequate slip resistance to resist the Service II and deck casting moment requirements at the splice. No moment contribution from the web is required.

3.3.3 Web Splice Design

3.3.3.1 Strength Limit State Design

3.3.3.1.1 Bolts

Since the moment resistance provided by the flanges is sufficient to resist the factored moment at the strength limit state at the point of splice in this case, the web splice bolts are designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web on either side of the splice. Since the web splice is being designed to develop the full factored shear resistance of the web at the strength limit state, the effect of the additional St. Venant torsional shear in the web may be ignored at the strength limit state. The factored shear resistance, V_r, of the unstiffened ⅝ in. x 80.39 in. Grade 50W web at the point of splice is determined to be (AASHTO LRFD Article 6.10.9):

$V_r = \phi_v V_n = 401$ kips

Number of Bolts Required (threads included in the shear plane): $N = 401/51.9 = 7.7$

Note that the greater than 38.0 in. length reduction for the shear resistance of the bolts only applies to lap-splice tension connections (AASHTO LRFD Article 6.13.2.7), and is not to be applied in the design of the web splice.

The AASHTO specification requires at least two rows of bolts in the web over the depth of the web (AASHTO LRFD Article 6.13.6.1.3a). The maximum permitted spacing of the bolts for sealing is $s \leq (4.0 + 4.0t) \leq 7.0$ in., where t is the thickness of the splice plate (AASHTO LRFD Article 6.13.2.6.2). Assuming the splice plate thickness will be one-half the smaller web thickness at the point of splice plus $^1/_{16}$ in. gives a splice plate thickness of:

$$t = \frac{5}{8} \text{ x } \frac{1}{2} + \frac{1}{16} = \frac{3}{8} \text{ in.}$$

A filler plate is not required since the webs are the same thickness on both sides of the splice. The maximum bolt spacing for the ⅜ in. splice plate is:

$$4.0 + 4 \text{ x } \frac{3}{8} = 5.5 \text{ in.}$$

Using approximately a 4-$^{7}/_{16}$ in. gap from the top and bottom of the web to the top and bottom web splice bolts so as to not impinge on bolt assembly clearances, the available web depth for the bolt pattern is 80.39 – (2 x 4.4375) = 71.515 in. The number of bolts required to meet the maximum bolt spacing is:

Number of bolts = 1 + $\frac{71.515}{5.5}$ = 14 bolts in two vertical rows each side of splice

= 28 bolts > 7.7.

3.3.3.2 Slip Resistance Check

Since the moment resistance provided by the nominal resistance of the flange splice bolts is sufficient to resist the factored moment for checking slip at the point of splice in this case (see Section 3.3.2.2 Slip Resistance Check), the web splice bolts are checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice (AASHTO LRFD Article 6.13.6.1.3c). The factored shear for checking slip is taken as the shear in the web at the point of splice under Load Combination Service II, or the shear in the web at the point of splice due to the deck casting sequence, whichever governs. For tub girders in horizontally curved bridges (and since slip is a serviceability requirement), the shear is taken as the sum of the factored flexural and St. Venant torsional shears in the web subjected to additive shears when checking slip (AASHTO LRFD Article 6.13.6.1.3c). Since the tub girder has inclined webs, the factored shear is taken as the component of the factored vertical shear in the plane of the web.

The unfactored vertical shears in the critical web (i.e. the web subject to additive flexural and St. Venant torsional shears) at the point of splice are as follows:

V_{DC1}	=	-109 kips
V_{DC2}	=	-12 kips
V_{DW}	=	-12 kips
V_{+LL+IM}	=	+35 kips
V_{-LL+IM}	=	-89 kips
$V_{deck\ casting}$	=	-89 kips

By inspection, the Service II negative shear controls.

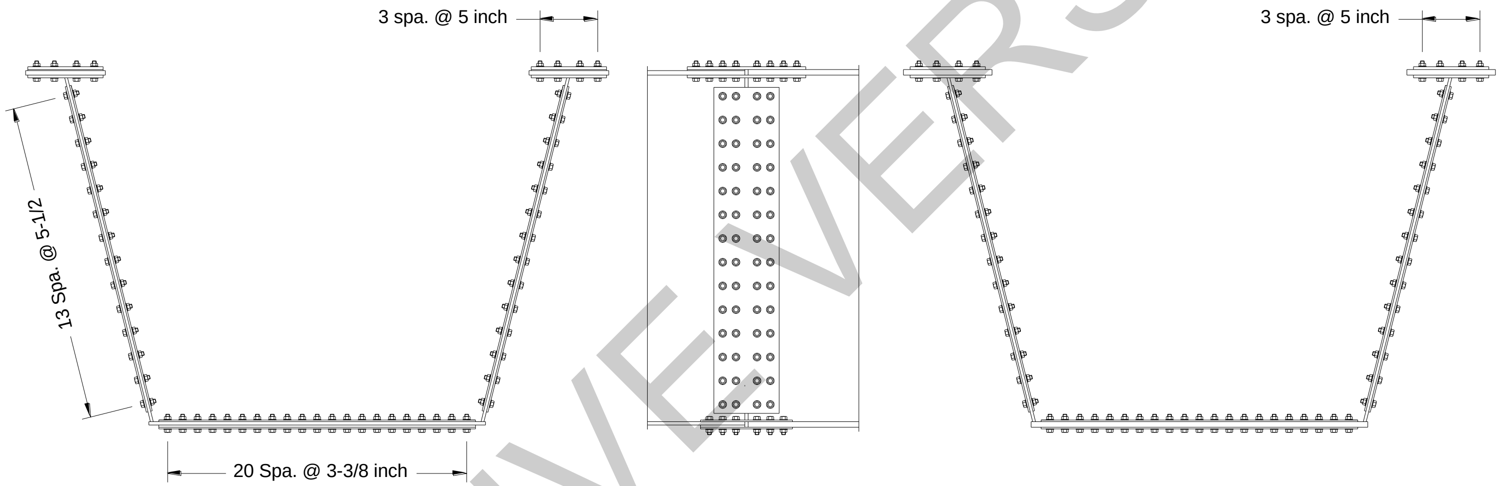
Service II Negative Shear = 1.0(-109 + -12) + 1.0(-12) + 1.3(-89) = -249 kips > $V_{deck\ casting}$ = 1.4(-89) = -125 kips

The factored shear in the plane of the inclined web is computed as:

$$V_i = \frac{V}{\cos\theta} = \frac{-249}{\cos 14^{\circ}} = -257 \text{ kips}$$

Slip resistance of web splice with 28 bolts: P_t = 28(39.0 kips/bolt) = 1,092 kips > |-257| kips ok

A schematic of the final splice for Design Example 3 is shown below (spice plate sizes are not shown).



DESIGN EXAMPLE 3 Bolted Splice Design

4 ACKNOWLEDGEMENTS

The authors would like to thank the efforts of the many people that have assisted in the effort of compiling the information contained within this document. First and foremost is the AASHTO T-14 Technical Committee for Structural Steel Design chaired by Mr. Norman McDonald of the Iowa DOT that formed an ad-hoc task group to address the broad topic of simplifying the provisions related to bolted field splice design contained in the AASHTO *LRFD Bridge Design Specifications*. The helpful comments and guidance provided by Mr. Edward Wasserman of Modjeski and Masters, Inc., Mr. Hormoz Seradj of the Oregon DOT, and Dr. Francesco Russo of Michael Baker International in the development of the improved design method are also gratefully acknowledged. The authors would also like to thank Mr. Christopher Garrell of the NSBA for his efforts in developing a design spreadsheet to accompany this document, and Mr. Matt Shergalis of the NSBA for his assistance with the formatting of this document.

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