Illinois Department of Transportation
Office of Highways Project Implementation / Bureau of Bridges & Structures
2300 South Dirksen Parkway / Springfield, Illinois / 62764

All IDOT Design Guides have been updated to reflect the release of the 2017 AASHTO LRFD Bridge Design Specification, 8th Edition. The following is a summary of the major changes that have been incorporated into the Bolted Splice Design Guide. The May 2018 errata incorporated additional changes to splice design. These changes have been reflected in this design guide.

- Flange Splices and web splices are now designed for 100% capacity (F_yA_e) regardless of applied loads. Web splices may have additional loads per the "Calculate Web Design Force" section of the design guide.

- As splice loads are now independent of applied loads, the sections for determining the flange/web stresses have been removed from the design guide.

- The procedure for determining the slip force requirement has been adjusted.

3.3.21 LRFD Bolted Splice Design for Composite Structures

This design guide contains a procedural outline for the design of bolted field splices in main flexural members near points of dead load contraflexure using the LRFD Code. The focus is on splices for straight bridges which are composite throughout the structure. A worked design example is also included. The design example is consistent with Design Guide 3.3.4 in beam size, span length, skew, etc.

The primary articles for bolted splice design of flexural members in the LRFD Code are:

1. General (6.13.6.1.3a)
2. Flange Splices (6.13.6.1.3b)
3. Web Splices (6.13.6.1.3c)

LRFD Splice Design Procedure, Equations, and Outline

Composite splice design is similar to non-composite splice design with the exception that composite properties are used for applicable stress calculations.

As per Section 3.3.21 of the Bridge Manual, splices should be placed near points of dead load contraflexure. These points are typically in regions of stress reversal, and as such are in positive and negative flexure depending on the load conditions. The provisions of C6.13.6.1.3b account for both positive and negative flexure in the design of the splice.

If splices are placed at points of contraflexure, the dead and live loads should be close to zero for constructability checks. While the effects of pouring sequences will induce moments at the point of dead load contraflexure, these moments are not expected to induce stresses that will control over the stresses in the final load condition. A quick stress check for constructability is typically all that is required to determine that constructability does not control.

When determining locations of splices, note that there is a penalty on the C_b term used in beam design for changes in section that are not within 20% of the unbraced length near the brace point with the smaller moment. See Article 6.10.8.2.3 and Commentary C6.10.8.2.3. This should be acknowledged during framing plan setup as it can be avoided with proper diaphragm and splice placement. Typically, if a diaphragm and splice are both placed near a point of dead load contraflexure, this penalty will not be applicable.

The assumptions for section properties and stress calculation found in other sections of Chapter 6 are applicable to splice design. Despite the fact that it is current IDOT policy to not include stud shear connectors on flange splice plates, the section shall be assumed as composite in both positive and negative flexure at splice locations.

Transformed and cracked section properties need to be calculated. As the bridge in this design guide is consistent with the bridge in Design Guide 3.3.4, to avoid repetition, calculations for some of the section properties for the given structure are not repeated in this design guide but rather may be found in Design Guide 3.3.4.

Article 6.13.1 gives guidance of the design loads for a splice. This article states that bolted splices for flexural members shall be designed at the strength limit state as specified in Article 6.13.6.1.3. Article 6.13.6.1.3 states that splices shall be designed for a load equal to the flange force (F_yA_e) for flange splices. This is stating that flange splices are now designed for 100% of the flange capacity, which is independent of the applied loads and is different than previous editions of AASHTO. The web splices are similarly designed for the web capacity, with the exception that web splices may have an additional load applied to them if the flanges of the bridge alone cannot transfer all of the applied moment. This will be explained in greater detail later in this design guide.

Determine Trial Flange Splice Plates

To begin a design, trial flange splice plates are chosen. Each flange plate shall be a minimum ½ in. thick and shall extend as near to the beam or girder width as possible. If flange widths transition at the splice location, size the flange splice plate to the smaller width. Also, commonly available plate thicknesses should be used for splice plates. See Section 3.3.21 of the Bridge Manual.

Use of inside splice plates will greatly reduce the required number of bolts, as the bolts can be considered doubly effective in shear. This, in turn, will greatly reduce the length of the splice plates, which will allow for more flange space for shear studs and reduce the probability of interference with a stiffener or diaphragm. Therefore, it is recommended that inside splice plates be used when possible.

It should be noted that there are cases where flange geometry disallows the use of inside splice plates. Inside splice plates should never be used unless there are a minimum of four rows of bolts in the flange splice plate (two rows per side of flange). If only one row is present on a side of a flange, this may cause significant bowing/cupping of the inside plate when the bolts are tightened. The geometry of Bridge Manual Figures 3.3.21-2 and 3.3.21-3 dictates that inside splice plates may not be used unless the flange is a minimum of 12 inches wide.

When choosing sizes of inside plates, Bridge Manual 3.3.21 suggests that the area of the inside plates be within 10% of the area of the outside plate. The purpose of this is so that the assumption of double shear in the bolts is valid (i.e. around 50% of the load is actually going to the outside splice plate and 50% of the load is going to the inside splice plates). This may result in inside plates being slightly thicker than outside plates. Also, if the plate thicknesses are dictated by the minimum ½ in. thickness, addition of inside plates to an already excessively thick outside plate may result in splices with much higher capacities than required. However, the addition of the inside plates will also greatly reduce the required length of the splice plates, so even though the capacity is much higher, the amount of additional steel may be negligible.

Calculate Flange Effective Area, A_e (6.13.6.1.3b)

The effective area for tension flanges is found as:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \leq A_g \qquad \text{(Eq. 6.13.6.1.3b-2)}$$

The effective area for compression flanges is found as:

$$A_e = A_g \qquad \text{(6.13.6.1.4c)}$$

Where:

ϕ_u = 0.80 (6.5.4.2)

ϕ_y = 0.95 (6.5.4.2)

F_u = specified minimum tensile strength of tension flange (ksi) (Table 6.4.1-1)

F_{yt} = specified minimum yield strength of tension flange (ksi)

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

A_n = net area of the applicable flange = $W_n t$ (in.^{2})

In which:

W_n = net width of the applicable flange, as defined in 6.8.3 as the smallest width across a chain of holes in any transverse, diagonal, or zigzag line (in.). When subtracting the hole widths, the actual hole size found in Table 6.13.2.4.2-1 should be used. Additional subtractions due to damage due to punching are not applicable, as Article 505.04(d) of the Standard Specifications does not allow punching of holes for field splices for beams.

t = thickness of the applicable flange (in.)

Calculate Flange Design Forces (6.13.6.1.4b)

The flange design forces are taken as the yield strength of the flange times the effective net area. This calculation is repeated for both top and bottom flanges.

$P_{fy} = F_{yf} A_e$ (Eq. 6.13.6.1.3b-1)

Where:

F_{yf} = specified minimum yield of the flange (ksi)

A_e = effective area of the controlling flange (in.^{2})

Check Flange Splice Plate Strength

<u>Check Tension in Splice Plates</u> (6.13.6.1.4c & 6.13.5.2)

For all splice plates in tension, the controlling resistance is the minimum resistance calculated for yielding on the gross section, fracture on the net section, and block shear. Block shear is dependent on bolt layout, and rarely controls for typical splices. A check of block shear is found after the bolt calculations in this guide.

Compression in splice plates need not be checked. See Commentary C6.13.6.1.3b.

Check yielding on the gross section:

$P_r = \phi_y P_{ny} = \phi_y F_y A_g \quad > \quad P_{cf}$ or P_{ncf} as applicable (Eq. 6.8.2.1-1)

Where:

ϕ_y = 0.95 (6.5.4.2)

P_{ny} = nominal tensile resistance for yielding in gross section (kips)

F_y = minimum specified yield strength (ksi)

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

Check fracture on the net section:

$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U \quad > \quad P_{cf}$ or P_{ncf} as applicable (Eq. 6.8.2.1-2)

Where:

ϕ_u = 0.80 (6.5.4.2)

P_{nu} = nominal tensile resistance for fracture in net section (kips)

F_u = tensile strength (ksi)

A_n = net area of the connection element (in.^{2}) $\leq 0.85A_g$ (6.13.5.2)

A_g = gross area of the connection element (in.^{2})

R_p = reduction factor for punched holes, taken as 1.0 for IDOT bridges. Section 505 of the Standard Specifications doesn't allow for punched holes in primary connections.

U = 1.0 (6.13.5.2)

Calculate Reduction Factor for Filler Plates (6.13.6.1.4)

When flange thickness transitions at splice locations, filler plates shall be used. Filler plates shall not extend beyond the flange splice plate. If individual filler plates are greater than or equal to ¼ in. thick, then a reduction factor, R, shall be applied to the Strength I bolt shear resistance. Note that this reduction is not applicable when checking slip resistance using the Service II load case. Also note that, if used, this reduction is applicable to both the outside and the inside of the splice plate and not just the side with the filler (e.g. if you only have a filler plate on the outside of the splice, the reduction is still applicable to the inside splice plate)- the reduction is due to bolt curvature and since the same bolt is used for both the outside and inside plate, the reduction is obviously applicable to both. The final splice should have a bolt layout symmetric about the centerline of splice.

This reduction factor is only applicable to plates that are considered axially loaded (i.e. flange plates only). In the odd case where a web splice requires filler plates greater than 0.25 in., this reduction factor is not applicable.

$$R = \left|\frac{(1+\gamma)}{(1+2\gamma)}\right| \qquad \text{(Eq. 6.13.6.1.4-1)}$$

Where:

$\gamma = \dfrac{A_f}{A_p}$

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

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

Determine Trial Flange Splice Plate Bolt Layout

Choose a trial flange splice bolt layout using spacing requirements detailed in Article 6.13.2.6 and Section 3.3.21 of the Bridge Manual. Splice bolts shall be ⅞ in. diameter ASTM F3125 A325 bolts with standard holes.

Check Flange Splice Bolt Shear Strength

Compare Factored Bolt Shear, P_r, with Factored Resistance, R_r

Verify $P_r \leq R_r$ for both flanges in positive and negative flexure.

Where:

P_r = factored bolt shear (kips) = $\frac{P_{fy}}{N_b}$

In which:

P_{fy} = flange forces, as calculated above

N_b = number of flange splice bolts on one side of the splice

R_r = factored bolt resistance (kips) as calculated below

Calculate Factored Shear Resistance for Bolts, R_r, (6.13.2.7)

According to Section 505 of the Standard Specifications, the thread length (the length of the threaded portion of the shaft) of a bolt is dictated by "Specification for Structural Joints Using ASTM A325 (A325M) or A490 (A490M) Bolts." These thread lengths are also found in AISC 13th Edition, Table 7-15 (Pg. 7-80). The assumption should be made that threads are present on the shear plane, even though the thread lengths dictated by the above document are only rarely long enough for this condition to occur. There have been instances where bolts have arrived on the jobsite with improper thread lengths and use of the above assumption assures that even if the thread lengths are too long the splice will still have adequate capacity.

For splices where the center-to-center distance of extreme bolts along a bolt line is greater than 50 inches on one side of a splice, calculated bolt shear strength should by multiplied by a factor of 0.8 (6.13.2.7, C6.13.2.7). This factor is independent of the resistance factor ϕ_s.

Additionally, if the grip length (the length of the unthreaded portion of the shaft) of a bolt exceeds five diameters, the nominal resistance of the bolt shall further be reduced according to 6.13.2.7. This will only affect beams with very thick flanges.

Without any of the above factors applied, the factored resistance of a bolt is taken as:

$R_r = \phi_s R_n R$

Where:

ϕ_s = 0.80 for ASTM A325 bolts in shear (6.5.4.2)

R_n = $0.45A_bF_{ub}N_s$ if threads are included on the shear plane (Eq. 6.13.2.7-1)

R = reduction factor for filler, if applicable (6.13.6.1.5)

A_b = area of bolt (in.^{2})

F_{ub} = specified minimum tensile strength of the bolt (ksi) (6.4.3)

N_s = number of slip planes, taken as one for flange splices with outside plates only, and two for splices with inside plates. Note that the addition of filler plates does not introduce another slip plane to the bolt.

Check Flange Splice Bolt Slip Resistance

$P_{slip} \leq R_r$ for top and bottom flange for both positive and negative flexure.

$R_r = R_n$ (Note: ϕ is unspecified) (Eq. 6.13.2.2-1)

Where:

R_n = $K_hK_sN_sP_t$ (kips) (Eq. 6.13.2.8-1)

K_h = hole size factor (Table 6.13.2.8-2)

K_s = 0.30. Standard Inorganic primers use a coefficient of friction of 0.50, and Organic Zinc-Rich primers require a coefficient of friction of 0.45. See AASHTO M300 and the special provision for Organic Zinc-Rich Paint System. Use of 0.30 is conservative. (Table 6.13.2.8-3)

N_s = number of slip planes. See above.

P_t = minimum required bolt tension (kips) (Table 6.13.2.8-1)

$$P_{slip} = \frac{P_{tot-slip}}{N_b}$$

Where:

$P_{tot\text{-}slip}$ = The factored Service II slip force, calculated as the Service II moment divided by the length of the moment arm A

$$= \left|\frac{M_{SERVICE\ II}}{A_f}\right| \text{ (kips)}$$

A_f = Moment arm, taken as the distance from the centroid of the deck to the centroid of the bottom flange for positive moments, and the distance from the centroid of the top flange to the centroid of the bottom flange for negative moments

$A_{f,pos}$ = $0.5t_{slab} + t_{fillet} + t_{tf} + D + 0.5t_{bf}$ (positive moment check)

$A_{f,neg}$ = $0.5t_{tf} + D + 0.5t_{bf}$ (negative moment check)

N_b = number of flange splice bolts on one side of the splice

Check Flange Splice Bolt Bearing Resistance

Verify $P_{brg} \leq R_r$ for both flange splices for positive and negative flexure.

Where:

$$P_{brg} = \frac{P_{fy}}{N_b}$$

N_b = number flange splice bolts

R_r = $\phi_{bb} R_n$

Where:

ϕ_{bb} = 0.80 (6.5.4.2)

R_n = nominal resistance of interior and end bolt holes (kips)

If $x_{clear} \geq 2.0d$ and $x_{end} \geq 2.0d$:

R_n = $2.4dtF_u$ (Eq. 6.13.2.9-1)

If $x_{clear} < 2.0d$ or $x_{end} < 2.0d$:

R_n = $1.2L_c tF_u$ (Eq. 6.13.2.9-2)

Where:

x_{clear} = clear distance between bolt holes (in.)

x_{end} = bolt clear end distance (in.)

d = nominal diameter of the bolt (in.)

t = minimum thickness of the connected material, either of the flange itself or the flange splice plates (in.)

F_u = tensile strength of the connected material (ksi) (Table 6.4.1-1)

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.)

Check Flange Splice Bolt Spacing

See Figures 3.3.21-1 to 3.3.21-3 of the Bridge Manual and LRFD Article 6.13.2.6 for guidance.

Check Flange Block Shear

Block shear does not control the flange design for typical splices, where the number of bolts per row is much larger than the number of rows of bolts. The only times block shear should be anticipated to control the design of a flange splice is if the flange is very wide (allowing for many rows of bolts), but does not require many bolts per row. If the number of bolts per row is less than the number of rows of bolts, block shear should be checked in flange splices. Otherwise, it need not be checked.

Determine Trial Web Splice Plate

To begin a design, trial web splice plates are chosen. Each plate shall be a minimum ⅜ in. thick and shall extend as near to the beam or girder web depth as possible, leaving room for the girder web welds or rolled beam fillets. See also Section 3.3.21 of the Bridge Manual.

Determine Trial Web Splice Bolt Layout

Choose a trial web splice bolt layout using spacing requirements detailed in LRFD Article 6.13.2.6 and Section 3.3.21 of the Bridge Manual. Splice bolts shall be ⅞ in. diameter ASTM A3125 A325 bolts with standard holes. A minimum of two vertical bolt rows shall be used on each side of the splice connection element (6.13.6.1.3a), with 3 in. spacing between the vertical rows. See Fig. 3.3.21-1 of the Bridge Manual. Vertical spacing of bolts within a row is 3 in. minimum, with the maximum bolt spacing dictated by the sealing requirements Eq. 6.13.2.6.2-1:

$$s < (4.0 + 4.0t) \leq 7.0 \text{ in.} \qquad \text{(Eq. 6.13.2.6.2-1)}$$

Where t is the thickness of the web splice plate.

Bolt interference between the web splice and the flange splice shall be taken into consideration when determining the bolt layout. AISC 9th Edition, Pgs. 4-137 to 4-139, and AISC 13th Edition, Tables 7-16 and 7-17 (Pgs. 7-81 and 7-82) give information on required clearances for erection tools.

When choosing a trial web splice plate and bolt layout, note that the extreme bolt bearing may control the design if the edge distance is not large enough to resist the force from the extreme bolt. Use of larger-than-minimum edge distances is often necessary, especially for larger splices.

Calculate Web Design Force

Web splice plates and bolts are designed for the sum of the web shear capacity (V_r) and a horizontal force (H_w) that simulates the flexural moment assumed to be resisted by the web at the point of the splice (Article 6.13.6.1.3c). Note that both of these terms are based off of the capacities of members (V_r is based off of the capacity of the web and H_w is based off of the capacities and eccentricities of the flanges), and the actual applied shear is never used to design the plates. The horizontal force H_w may be zero, depending upon the sizes of the flanges. The two components are orthogonal, and should be added as follows to generate the web design force:

$$P_{web} = \sqrt{V_r^2 + H_w^2}$$

<u>Calculate Web Shear Resistance, V_r</u>

$$V_r = \phi_v V_n \qquad \text{(Eq. 6.10.9.1-1)}$$

(Eq. 6.10.9.1-1)

Where:

ϕ_v = resistance factor for shear, equal to 1.00 (6.5.4.2)

V_n = nominal shear resistance (kips)

= $V_{cr} = CV_p$ for unstiffened webs and end panels of stiffened webs (Eq. 6.10.9.2-1)

$$= V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right]$$ for interior panels of stiffened webs that satisfy

$$\frac{2Dt_w}{(b_{fc}t_{fc} + b_{ft}t_{ft})} \le 2.5$$ (Eqs. 6.10.9.3.2-1,2)

$$= V_p\left[C + \frac{0.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2} + \frac{d_o}{D}}\right]$$ for interior panels of stiffened webs that do not satisfy

the preceding requirement (Eq. 6.10.9.3.2-8)

Where:

d_o = transverse stiffener spacing (in.)

D = web depth (in.)

C = ratio of shear buckling resistance to shear yield strength

For $\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_{yw}}}$:

$$C = 1.0$$ (Eq. 6.10.9.3.2-4)

For $1.12\sqrt{\frac{Ek}{F_{yw}}} < \frac{D}{t_w} \le 1.40\sqrt{\frac{Ek}{F_{yw}}}$:

$$C = \frac{1.12}{(D/t_w)}\sqrt{\frac{Ek}{F_{yw}}}$$ (Eq. 6.10.9.3.2-5)

For $\frac{D}{t_w} > 1.40\sqrt{\frac{Ek}{F_{yw}}}$:

$$C = \frac{1.57}{(D/t_w)^2}\left(\frac{Ek}{F_{yw}}\right) \qquad \text{(Eq. 6.10.9.3.2-6)}$$

Where k = 5 for unstiffened webs and $5+\frac{5}{\left(\frac{d_o}{D}\right)^2}$ for stiffened webs

(Eq. 6.10.9.3.2-7)

$$V_p = 0.58F_{yw}Dt_w \qquad \text{(Eq. 6.10.9.2-2)}$$

<u>Calculate H_w, Additional Force to Account for Moment Resisted by Web</u>

If the flanges alone cannot resist all of the factored Strength I moment, then an additional force must be applied to the web to account for the moment resisted by the web. This force is calculated as follows:

For positive moments:

$$H_w = \frac{|M_{STRENGTH\ I}| - M_{flanges}}{A_w}$$

For negative moments:

$$H_w = \frac{|M_{STRENGTH\ I}| - M_{flanges}}{A_w/2}$$

Where:

$M_{flanges}$ = moment resisted by the flanges (k-in.), taken as:

- $\min(P_{deck}, P_{fy,bottom\ flange})(A_{f,pos})$ for positive moment
- $\min(P_{fy,top\ flange}, P_{fy,bottom\ flange})(A_{f,neg})$ for negative moment

P_{deck} = $0.85f'_c t_{slab}$(Effective Flange Width)

$P_{fy,\ bottom}$ = $F_y A_{e,bf}$

$P_{fy,\ top}$ = $F_y A_{e,bf}$

$M_{STRENGTH\ I}$ = factored Strength I moment at point of splice

A_w = moment arm, taken as the distance from the centroid of the deck to the mid-depth of the web, and the distance from the top of the web to the mid-depth of the web for negative moments

$A_{w,pos}$ = $0.5t_{slab} + t_{fillet} + t_{tf} + 0.5D$ (positive moment check)

$A_{w,neg}$ = $0.5D$ (negative moment check)

Check Web Splice Plate Shear Capacity *(6.13.5.3)*

$P_{web} \leq R_r$

Where R_r is the capacity of the web splice plates, taken as the lesser of the capacity for web splice plate shear yielding and web splice plate shear rupture.

Calculate Factored Shear Resistance for Yielding of Gross Web Splice Section

$$R_r = \phi_v 0.58 F_y A_{vg} \quad \text{(Eq. 6.13.5.3-1)}$$

Where:

A_{vg} = gross area of web splice plates (in.^{2})

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

ϕ_v = 1.0 (6.5.4.2)

Calculate Factored Shear Resistance for Fracture of Net Web Splice Section

$$R_r = \phi_{vu} 0.58 R_p F_u A_{vn} \quad \text{(Eq. 6.13.5.3-2)}$$

Where:

R_p = 1.0 for holes drilled or subpunched and reamed to size. This is typical for IDOT splices.

A_{vn} = net area of web splice plates (in.^{2})

F_u = specified ultimate strength of the connection element (ksi)

ϕ_{vu} = 0.8 (6.5.4.2)

Check Web Splice Bolt Strength

Shear strength of web splice bolts shall be checked for both positive and negative flexure.

The assumption should be made that threads are present on the shear plane, even though the thread lengths dictated by "Specification for Structural Joints Using ASTM A325 (A325M) or A490 (A490M) Bolts" are only rarely long enough for this condition to occur. There have been instances where bolts have arrived on the jobsite with improper thread lengths and use of the above assumption assures that even if the thread lengths are too long the splice will still have adequate capacity.

For splices where the center-to-center distance of extreme bolts along a bolt line is greater than 50 inches on one side of a splice, calculated bolt shear strength should by multiplied by a factor of 0.8 (6.13.2.7). This factor is independent of the resistance factor ϕ_s.

Additionally, if the grip length of a bolt exceeds five diameters, the nominal resistance of the bolt shall further be reduced according to 6.13.2.7. This reduction should rarely, if ever, apply to webs.

The shear capacity per bolt, R_r, is calculated as follows:

$$R_r = \phi_s R_n R$$

Where:

ϕ_s = 0.80 (6.5.4.2)

R = reduction factor for filler, if applicable (6.13.6.1.4)

R_n = $0.45A_bF_{ub}N_s$ (kips) (Eq. 6.13.2.7-2)

Where:

A_b = area of bolt $= \frac{\pi(0.875 \text{ in.})^2}{4} = 0.6013 \text{ in.}^2$

F_{ub} = specified minimum tensile strength of the bolt (ksi) (6.4.3)

N_s = number of slip planes, taken as two for bridge web splices.

Check Web Splice Bolt Slip

The factored slip resistance of the bolts, R_n, must exceed the Service II design shear at the point of the splice:

$R_n > V_{SERVICE\ II}$

<u>Calculate Service II Shears for Web Splice Plates</u>

Controlling positive and negative live loads shall be investigated in shear force calculations. If a more conservative result is obtained, V_{DW} should be excluded.

$V_{SERVICE\ II} = 1.0(V_{DC1}) + 1.0(V_{DC2}) + 1.0(V_{DW}) + 1.3(V_{LL+IM})$

<u>Calculate Factored Slip Resistance for Bolts, R_r</u> (6.13.2.2)

$R_r = \phi R_n$ (Eq. 6.13.2.2-2)

Where:

$\phi = f_v = 1.00$ (6.5.4.2)

$R_n = K_h K_s N_s P_t$ (kips) (Eq. 6.13.2.8-1)

K_h = hole size factor (Table 6.13.2.8-2)

$K_s = 0.30$ (Table 6.13.2.8-3)

N_s = number of slip planes, taken as two for webs.

P_t = minimum required bolt tension (kips) (Table 6.13.2.8-1)

<u>Compare Service II Resultant Design Force, P_{or}, with Factored Slip Resistance, R_r</u>

Verify $V_{SERVICE\ II} \leq R_r$ for both positive and negative flexure.

Check Web Splice Plate Block Shear

$V_{uw} \leq R_r$

Where:

$R_r = \phi_{bs}(0.58F_uA_{vn})$ (Modified Eq. 6.13.4-2)

Where:

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

F_u = specified minimum tensile strength of the web specified in Table 6.4.1-1 (ksi)

ϕ_{bs} = 0.80 (6.5.4.2)

The Department only requires checking block shear on the web splice plates along the vertical path which has the least net area in pure shear. The check is analogous to that for fracture on the net section in pure tension for flange splice plates. This is a conservative simplification of Eq. 6.13.4-2. Block shear should not be anticipated to control the design of regular web splice plates. If the plates are found to fail in block shear using this equation, the full equation in 6.13.4 may be used.

Check Bolt Bearing

$P_{web} \leq R_r$

Where:

$R_r = \phi_{bb}R_nN_b$

Where:

ϕ_{bb} = 0.80 (6.5.4.2)

N_b = number of bolts on one side of connection

R_n = nominal resistance of interior and end bolt holes (kips)

If $x_{clear} \geq 2.0d$ and $x_{end} \geq 2.0d$:

$R_n = 2.4dtF_u$ (Eq. 6.13.2.9-1)

If $x_{clear} < 2.0d$ or $x_{end} < 2.0d$:

$R_n = 1.2L_ctF_u$ (Eq. 6.13.2.9-2)

Where:

x_{clear} = clear distance between bolt holes (in.)

x_{end} = bolt clear end distance (in.)

d	=	nominal diameter of the bolt (in.)
t	=	thickness of the connected material (in.)
F_u	=	tensile strength of the connected material (ksi) (Table 6.4.1-1)
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.)

Check Web Splice Bolt Spacing

See Figures 3.3.21-1 to 3.3.21-3 in the Bridge Manual and LRFD Article 6.13.2.6 for guidance.

LRFD Bolted Splice Design Example

Materials

Flanges: AASHTO M270 Grade 50
Webs: AASHTO M270 Grade 50
Flange Splice Plates: AASHTO M270 Grade 50
Web Splice Plates: AASHTO M270 Grade 50

Design Stresses

F_y = $F_{yw} = F_{yt} = F_{yf} = 50$ ksi
F_u = 65 ksi
F_{ub} = 120 ksi

Bridge Data

The bridge data here is the same as the data used to design the plate girders in Design Guide 3.3.4. The following is copied directly from that guide.

Span Length: Two spans, symmetric, 98.75 ft. each
Bridge Roadway Width: 40 ft., stage construction, no pedestrian traffic

Slab Thickness t_s: 8 in.

Fillet Thickness: Assume 0.75 in. for weight, do not use this area in the calculation of section properties

Future Wearing Surface: 50 psf

Number of Girders: 6

Girder Spacing: 7.25 ft., non-flared, all beam spacings equal

Overhang Length: 3.542 ft.

Splice Locations: 67 ft. into Span 1, 31.25 ft. into Span 2

Skew: 20°

Diaphragm Placement:

	Span 1	Span 2
Location 1:	3.33 ft.	4.5 ft.
Location 2:	22.96 ft.	25.65 ft.
Location 3:	37.0 ft.	35.42 ft.
Location 4:	51.5 ft.	48.5 ft.
Location 5:	70.67 ft.	61.75 ft.
Location 6:	91.58 ft.	76.78 ft.
Location 7:	97.42 ft.	92.94 ft.

Top of Slab Longitudinal Reinforcement: #5 bars @ 12 in. centers in positive moment regions, #5 bars @ 12 in. centers and #6 bars @ 12 in. centers in negative moment regions

Bottom of Slab Longitudinal Reinforcement: 7- #5 bars between each beam

Plate Girder Dimensions

As calculated in Design Guide 3.3.4, the plate girder has the following section properties:

Section 1

$D = 42$ in.

$t_w = 0.4375$ in.

$b_{tf} = b_{bf} = 12$ in.

t_{bf} = 0.875 in.

t_{tf} = 0.75 in.

<u>Section 2:</u>

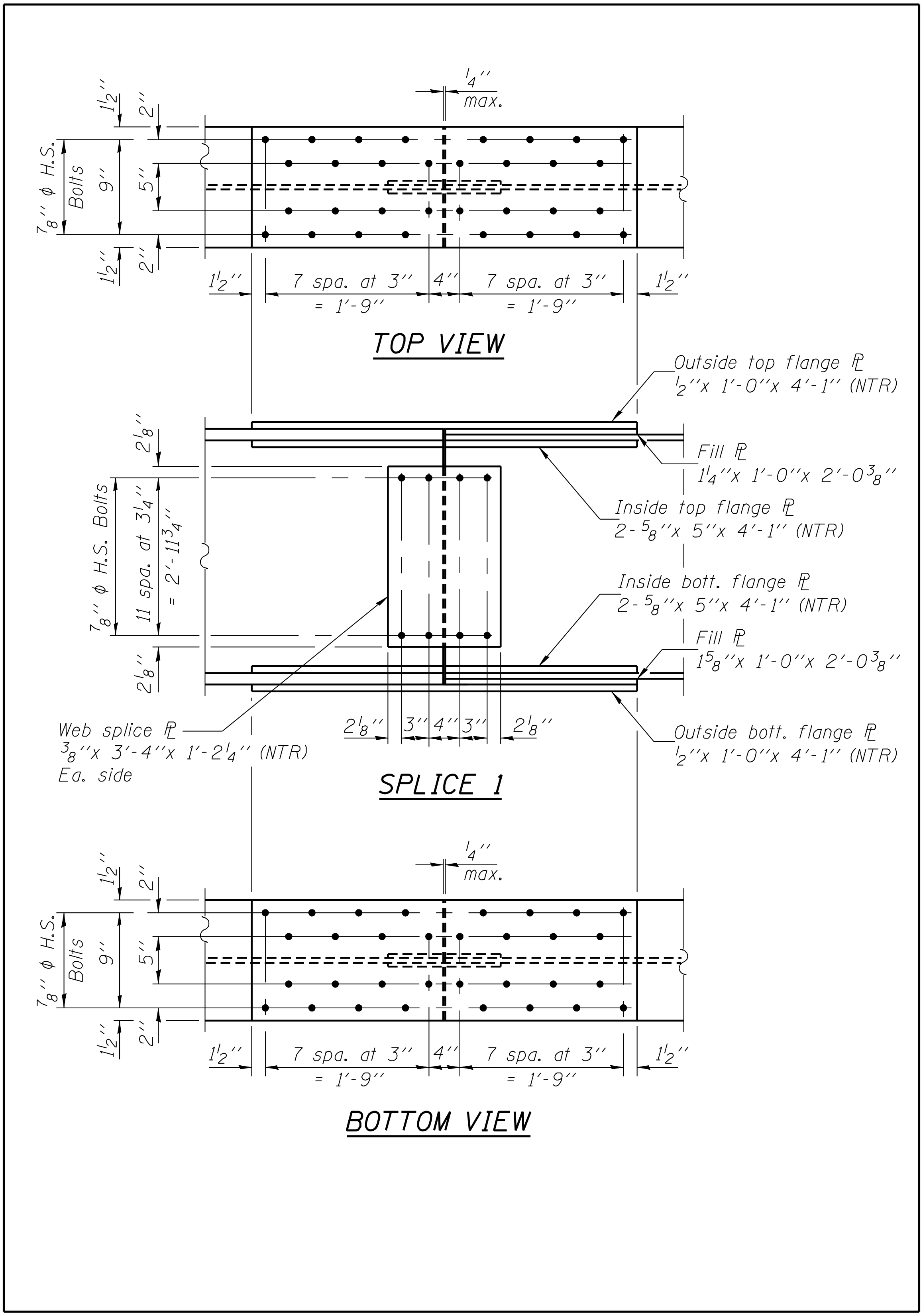
D = 42 in.

t_w = 0.5 in.

$b_{bf} = b_{tf}$ = 12 in.

t_{bf} = 2.5 in.

t_{tf} = 2.0 in.



TOP VIEW
¼'' max.
⅞'' ϕ H.S. Bolts
9''
5''
1½''
2''
7 spa. at 3'' = 1'-9''
4''
Outside top flange PL ½''x 1'-0''x 4'-1'' (NTR)
Fill PL 1¼''x 1'-0''x 2'-0⅜''
Inside top flange PL 2-⅝''x 5''x 4'-1'' (NTR)
Inside bott. flange PL 2-⅝''x 5''x 4'-1'' (NTR)
Fill PL 1⅝''x 1'-0''x 2'-0⅜''
Outside bott. flange PL ½''x 1'-0''x 4'-1'' (NTR)
Web splice PL ⅜''x 3'-4''x 1'-2¼'' (NTR) Ea. side
⅞'' ϕ H.S. Bolts
11 spa. at 3¼'' = 2'-11¾''
2⅛''
3''
SPLICE 1
BOTTOM VIEW

Figure 1

Unfactored Distributed Forces at Splice

	Moment (k-ft.)	Shear (k)
DC1	-6.8	-29.6
DC2	6.2	-4.9
DW	15.0	-11.9
Truck (+Trk+Ln)	832.0	12.0
Truck (-Trk+Ln)	-631.1	-68.4
Tandem (+Tan+Ln)	730.7	11.4
Tandem (-Tan+Ln)	-509.4	-56.5
2 Trucks -.9(2Trk+Ln)	-568.0	N/A
Construction DC1	-7.5	-31.3
Construction LL+IM	-2.2	-4.9

Determine Trial Flange Splice Plates

Bottom Flange Splice Plate

Try 12 in. x 0.5 in. plate for outside plate, 5 in. x 0.625 in. plates for inside plates

Top Flange Splice Plate

Try 12 in. x 0.5 in. plate for outside plate, 5 in. x 0.625 in. plates for inside plates

Note that these plates have a capacity well above the required capacity. However, 0.5 in. is the thinnest plate allowed for a flange, and the inside plates must be slightly thicker in order to make the areas of the outside plate and inside plates within 10% of each other.

Also note that although the top flange and bottom flange of the girders are slightly different in size, this splice is symmetric (i.e. the top flange plates and bolts are congruent with the bottom flange plates and bolts). As such, some of the calculations have been omitted for brevity.

Determine Trial Flange Splice Plate Bolt Layout

Bottom Flange Splice Plate

Try four staggered rows of bolts spaced as shown in Figure 1.

Top Flange Splice Plate

Try four staggered rows of bolts spaced as shown in Figure 1.

Calculate Flange Effective and Gross Areas, A_e and A_g (6.13.6.1.3b)

Top Flange:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \le A_g$$ (Eq. 6.13.6.1.3b-2)

Where:

ϕ_u = 0.80 (6.5.4.2)

F_u = 65 ksi

ϕ_y = 0.95 (6.5.4.2)

F_{yt} = 50 ksi

A_n = $W_n t$. As the bolt layout is staggered, there are two possible failure planes that could define W_n. The first is a straight-line failure across two of the bolt holes, which would have a shorter, more direct plate area but less hole subtractions. The second is a staggered failure across four of the bolt holes, which would have a larger indirect plate area but more hole subtractions. By inspection, the first failure plane will control the design.

= $b_f - d_{hole}$(# of holes)

Where:

b_f = 12 in.

d_{hole} = 0.9375 in. (Table 6.13.2.4.2-1)

A_n = [12 in. – 2(0.9375 in.)](0.75 in.) = 7.59 in.^{2}

$$A_e = \left(\frac{(0.8)(65\text{ ksi})}{(0.95)(50\text{ ksi})}\right)(7.59\text{ in.}^2) = 8.31\text{ in.}^2$$

A_g = (12 in.)(0.75 in.) = 9 in.^{2}

Bottom Flange:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \le A_g \qquad \text{(Eq. 6.13.6.1.4c-2)}$$

Where:

ϕ_u = 0.80 (6.5.4.2)

F_u = 65 ksi

ϕ_y = 0.95 (6.5.4.2)

F_{yt} = 50 ksi

A_n = [12 in. – 2(0.9375 in.)](0.875 in.) = 8.86 in.^{2}

$$A_e = \left(\frac{(0.8)(65\text{ ksi})}{(0.95)(50\text{ ksi})}\right)(8.86\text{ in.}^2) = 9.70\text{ in.}^2$$

A_g = (12 in.)(0.875 in.) = 10.5 in.^{2}

Calculate Strength I Flange Design Forces (6.13.6.1.3b)

Top Flange

$P_{fy} = F_{yf}A_e$ (Eq. 6.13.6.1.3b-1)

Where:

F_{yf} = 50 ksi

A_e = 8.31 in.^{2}

$P_{fy,\ top}$ = (50 ksi)(8.31 in.^{2}) = 415.5 k

Bottom Flange

$P_{fy} = F_{yf}A_e$ (Eq. 6.13.6.1.3b-1)

Where:

F_{yf} = 50 ksi

A_e = 9.70 in.^{2}

$P_{fy,\ bottom}$ = (50 ksi)(9.70 in.^{2}) = 485.0 k

Check Flange Splice Plate Strength

The top and bottom flange splice plates are identical. Therefore, the larger of the two flange forces (bottom flange) will be used in design.

Bottom Flange

Check yielding on the gross section:

$P_r = \phi_y P_{ny} = \phi_y F_y A_g > P_{cf}$ or P_{ncf} as applicable (Eq. 6.8.2.1-1)

Where:

$\phi_y = 0.95$ (6.5.4.2)

$F_y = 50$ ksi

$A_g = (12 \text{ in.})(0.5 \text{ in.}) + 2(5 \text{ in.})(0.625 \text{ in}) = 12.25 \text{ in.}^2$

$P_r = (0.95)(50 \text{ ksi})(12.25 \text{ in.}^2) = 581.88 \text{ k} > 485.0 \text{ k}$ O.K.

Check fracture on the net section:

$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U > P_{cf}$ or P_{ncf} as applicable (Eq. 6.8.2.1-2)

Where:

$\phi_u = 0.80$ (6.5.4.2)

$F_u = 65$ ksi

$A_n = [12 \text{ in.} - 2(0.9375 \text{ in.})](0.5 \text{ in.}) + 2(5 \text{ in.} - 0.9375 \text{ in.})(0.625 \text{ in.})$

$= 10.14 \text{ in.}^2$

$0.85A_g = 0.85(12.25 \text{ in.}^2) = 10.41 \text{ in.}^2$. 10.14 in.^2 controls

$R_p = 1.0$

$U = 1.0$ (6.13.5.2)

$P_r = (0.80)(65 \text{ ksi})(10.14 \text{ in.}^2)(1.0)(1.0) = 527.28 \text{ k} > 485.0 \text{ k}$ O.K.

Calculate Reduction Factor for Fillers (6.13.6.1.5)

Filler Plate Reduction, R

The required filler plate thickness is 1.25 in. for top flange splice and 1.625 for bottom flange splice. A filler plate reduction is required because both of these values exceed 0.25 in.

Top Filler Plate Reduction Factor:

A_{fill} = (12 in.)(1.25 in.) = 15.0 in.^{2}

A_p = 12.25 in.^{2} (see above) > A_{tf} = (12 in.)(0.75 in.) = 9.0 in.^{2}

$\therefore$ A_p = 9.0 in.^{2}

$$\gamma = \left|\frac{15.0\text{ in.}^2}{9.0\text{ in.}^2}\right| = 1.67$$

$$R = \left[\frac{(1+1.67)}{(1+2(1.67))}\right] = 0.62$$

Bottom Filler Plate Reduction Factor:

A_{fill} = (12 in.)(1.625 in.) = 19.5 in.^{2}

A_p = 12.25 in.^{2} > (12 in.)(0.875 in.) = 10.5 in.^{2}

$\therefore$ A_p = 10.5 in.^{2}

$$\gamma = \left|\frac{19.5\text{ in.}^2}{10.5\text{ in.}^2}\right| = 1.86$$

$$R = \left[\frac{(1+1.86)}{(1+2(1.86))}\right] = 0.61$$

Check Flange Splice Bolt Shear Strength

Calculate Factored Shear Resistance for Bolts, R_r (6.13.2.7)

Bottom Flange:

$R_r = \phi_s R_n R$

Where:

ϕ_s = 0.80 (6.5.4.2)

R = 0.62 for top plates, 0.61 for bottom plates. Use 0.61 for simplicity.

R_n = $0.45A_bF_{ub}N_s$(assumes threads are on the shear plane) (Eq. 6.13.2.7-2)

Where:

A_b = $0.25(\pi)(0.875\text{ in.})^2 = 0.60\text{ in.}^2$

F_{ub} = 120 ksi (6.4.3)

N_s = 2

R_n = $0.45(0.60\text{ in.}^2)(120\text{ ksi})(2) = 64.8$ k / bolt

$R_r = (0.80)(64.8$ k / bolt$)(0.61) = 31.6$ k / bolt

$$P_r = \left[\frac{485.0\text{k}}{16\text{bolts}}\right] = 30.3 \text{ kips per bolt} < 31.6 \text{ kips per bolt} \qquad \text{O.K.}$$

The top flange bolts have the same capacity, but a lower applied load and are OK by inspection.

Check Flange Splice Bolt Slip Resistance

Bottom Flange:

$R_r > P_{slip}$

Where:

$R_r = R_n$ (Eq. 6.13.2.2-1)

Where:

R_n = $K_hK_sN_sP_t$ (Eq. 6.13.2.8-1)

K_h = 1 (Table 6.13.2.8-2)

K_s = 0.30 (Table 6.13.2.8-3)

N_s = 2

P_t = 39 k / bolt (Table 6.13.2.8-1)

$R_r = R_n = (1)(0.30)(2)(39\text{ k / bolt}) = 23.4\text{ k / bolt}$

$$P_{slip} = \frac{P_{tot-slip}}{N_b}$$

Where:

$$P_{tot\text{-}slip} = \left|\frac{M_{SERVICE\ II}}{A_f}\right|$$

In which:

$M_{SERVICE\ ii}^{+}$ = 1.0(-6.8 k-ft.) + 1.0(6.2 k-ft.) + 1.0(15 k) + 1.3(832.0 k)

= 1096.0 k-ft. maximum positive moment

$M_{SERVICE\ II}^{-}$ = 1.0(-6.8 k-ft.) + 1.0(6.2 k-ft.) + 1.0(0 k) + 1.3(-631.1 k)

= -821.0 k-ft. maximum negative moment. Note that the future wearing surface was neglected due to it reducing the maximum load effect.

$A_{f,pos}$ = 0.5(8 in.) + 0.75 in. + 0.75 in. + 42 in. + 0.5(0.875)

= 47.94 in. (positive moment check)

$A_{f,neg}$ = 0.5(0.75 in.) + 42 in. + 0.5(0.875)

= 42.81 in. (negative moment check)

$$P_{tot\text{-}slip} = \left|\frac{(1096\ \text{k} - \text{ft.})\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{47.94\ \text{in.}}\right|$$

= 274.3 k (positive moment check)

$$P_{tot\text{-}slip} = \left| \frac{(-821.0\ k - ft.)\left(\frac{12\ in.}{ft.}\right)}{42.81\ in.} \right|$$

$$= 230.1\ k \quad \text{(negative moment check)}$$

274.3 k controls.

$N_b = 16$ bolts

$$P_{slip} = \frac{274.3\ k}{16\ bolts} = 17.1\ k / bolt < 23.4\ k / bolt \qquad \text{O.K.}$$

The top flange has the same bolt slip capacity but lower loads, and is O.K. by inspection.

Check Flange Splice Bearing Resistance

Verify $P_{brg} \le R_r$ for both flange splices for positive and negative flexure.

Where:

$$P_{brg} = \frac{P_{yf}}{N_b} = \frac{485.0\ k}{16\ bolts} = 30.3\ k / bolt$$

$R_r = \phi_{bb} R_n$

Where:

$\phi_{bb} = 0.80$ (6.5.4.2)

R_n = nominal resistance of interior and end bolt holes (kips)

If $x_{clear} \ge 2.0d$ and $x_{end} \ge 2.0d$:

$R_n = 2.4dtF_u$ (Eq. 6.13.2.9-1)

If $x_{clear} < 2.0d$ or $x_{end} < 2.0d$:

$R_n = 1.2L_ctF_u$ (Eq. 6.13.2.9-2)

Where:

x_{clear} = 3 in. - 0.9375 in. = 2.06 in.

x_{end} = 1.5 in. – 0.5(0.9375 in.) = 1.03 in.

2.0d = 2.0(0.875 in.) = 1.75 in.

$x_{end} < 2.0d$, $\therefore$ $R_n = 1.2L_ctF_u$

L_c = 1.03 in.

t = either 0.5 in. +0.625 in. = 1.125 in. for splice plates, or 0.75 in. for flange. Use 0.75 in. for simplicity.

F_u = 65 ksi (Table 6.4.1-1)

R_n = 1.2(1.03 in.)(0.75 in.)(65 ksi) = 60.26 k / bolt

R_r = 0.80(60.26 k / bolt) = 48.2 k / bolt> 30.3 k / bolt O.K.

Check Flange Splice Bolt Spacing

Using Bridge Manual Figures 3.3.21-1 and 3.3.21-3, spacing O.K.

Determine Trial Web Splice Plate

Try two ⅜ in. x 40 in. web splice plates, one on each side of the web.

Determine Trial Web Splice Bolt Layout

<u>Vertical Bolt Spacing</u>

Try 12 bolt lines spaced as shown in Figure 1.

<u>Horizontal Bolt Spacing</u>

Try two bolt lines spaced as shown in Figure 1.

Check Web Splice Plate Strength

Determine Web Design Force

$P_{web} = \sqrt{V_r^2 + H_w^2}$

<u>Calculate Factored Web Shear Resistance, V_r</u>

$V_r = \phi_v V_n$ (Eq. 6.10.9.1-1)

Where:

ϕ_v = 1.00 (6.5.4.2)

V_u = 180.68 k

V_n = CV_p for unstiffened webs. Assume web is unstiffened. (Eq. 6.10.9.2-1)

Where:

$$\frac{D}{t_w} = 96$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{(29000\text{ ksi})(5)}{(50\text{ ksi})}} = 60.3$$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{(29000\text{ ksi})(5)}{(50\text{ ksi})}} = 75.4$$

$$\therefore C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) \quad \text{(Eq. 6.10.9.3.2-6)}$$

$$= \frac{1.57}{\left(\frac{42\text{ in.}}{0.4375\text{ in.}}\right)^2}\left(\frac{(29000\text{ ksi})(5)}{50\text{ ksi}}\right)$$

$$= 0.49$$

V_p = $0.58F_{yw}Dt_w$ (Eq. 6.10.9.2-2)

= 0.58(50 ksi)(42 in.)(0.4375 in.)

= 532.9 k

CV_p = 0.49(532.9 k) = 261.1 k

V_r = 261.1 k

Calculate H_w, Additional Force to Account for Moment Resisted by Web

For positive moments:

$$H_w = \frac{|M_{STRENGTH\ I}| - M_{flanges}}{A_w}$$

For negative moments:

$$H_w = \frac{|M_{STRENGTH\ I}| - M_{flanges}}{A_w / 2}$$

Where:

$M_{flanges}$ = $\min(P_{deck}, P_{fy,bottom\ flange})(A_{f,pos})$ for positive moment

P_{deck} = 0.85(4 ksi)(8 in.)(87 in.)

= 2366.4 k

$P_{fy,\ bottom}$ = (50 ksi)(9.70 in.^{2})

= 485 k, controlling force for positive moment

$A_{f,pos}$ = 0.5(8 in.) + 0.75 in. + 0.75 In. + 42 in. + 0.5(0.875)

= 47.94 in. for positive moment

$M_{flanges}$ = (485 k)(47.94 in.)(1 ft. / 12 in.)

= 1937.5 k-ft. for positive moment

$M_{STRENGTH\ I}$ = 1.25(-6.8 k-ft.) + 1.25(6.2 k-ft.) + 1.5(15 k-ft.) + 1.75(832 k-ft.)

= 1478 k-ft. for positive moment

By inspection, the moment capacity of the flanges exceeds the Strength I moment, and no additional force is required to be added for the positive moment check.

H_w = 0 k

$M_{flanges}$ = $\min(P_{fy,top\ flange}, P_{fy,bottom\ flange})(A_{f,neg})$ for negative moment

$P_{fy,\ bottom}$ = (50 ksi)(9.70 in.^{2})

= 485 k

$P_{fy,\ top}$ = (50 ksi)(8.31 in.^{2})

= 415.5 k, controlling force for negative moment

$A_{f,neg}$ = 0.5(0.75 in.) + 42 in. + 0.5(0.875)

= 42.81 in. for negative moment

$M_{flanges}$ = (415.5 k)(42.91 in.)(1 ft. / 12 in.)

= 1485.8 k-ft. for positive moment

$M_{STRENGTH\ I}$ = 1.25(-6.8 k-ft.) + 1.25(6.2 k-ft.) + 1.5(0 k-ft.) + 1.75(-631.1 k-ft.)

= -1082.7 k-ft. for negative moment. Note that DW is neglected.

By inspection, the moment capacity of the flanges exceeds the Strength I moment, and no additional force is required to be added for the negative moment check.

H_w = 0 k

$$P_{web} = \sqrt{(261.1\,k)^2 + (0\,k)^2}$$

= 261.1 k

Check Web Splice Plate Shear Capacity *(6.13.5.3)*

$P_{web} \leq R_r$

Calculate Factored Shear Resistance for Yielding of Gross Web Splice Section

$R_r = \phi_v 0.58 F_y A_{vg}$ (Eq. 6.13.5.3-1)

Where:

A_{vg} = 2(40 in.)(0.375 in.) = 30 in.^{2}

F_y = 50 ksi

ϕ_v = 1.0 (6.5.4.2)

R_r = (1.00)(0.58)(50 ksi)(30 in.^{2})

$= 870$ k

261.1 k $<$ 870 k O.K.

<u>Calculate Factored Shear Resistance for Fracture of Net Web Splice Section</u>

$R_r = \phi_{vu} 0.58 R_p F_u A_{vn}$ (Eq. 6.13.5.3-2)

Where:

$R_p = 1.0$

$A_{vn} = 2(0.375 \text{ in.})(40 \text{ in.} - (0.9375 \text{ in.} / \text{hole})(12 \text{ holes}))$

$= 21.56 \text{ in.}^2$

$F_u = 65$ ksi

$\phi_{vu} = 0.8$ (6.5.4.2)

$R_r = (0.8)(0.58)(1.0)(65 \text{ ksi})(21.56 \text{ in.})$

$= 650.25$ k

261.1 k $<$ 650.25 k O.K.

Check Web Splice Bolt Strength

$R_r = \phi_s R_n R > P_r$

Where:

$\phi_s = 0.80$ (6.5.4.2)

$R = 1.0$ (since the webs are only 1/16 in. difference in thickness, no filler plates are used)

$R_n = 0.45 A_b F_{ub} N_s$ (kips) (Eq. 6.13.2.7-2)

Where:

$$A_b = \frac{\pi(0.875 \text{ in.})^2}{4} = 0.60 \text{ in.}^2$$

$F_{ub} = 120$ ksi (6.4.3)

$N_s = 2$

R_n = 0.45(0.60 in.^{2})(120 ksi)(2)
= 64.8 k

R_r = 0.80(64.8 k)(1.0)(24 bolts) = 1244.2 k > 261.1 k

Check Web Splice Bolt Slip

Calculate Service II Shears for Web Splice Plates

$V_{SERVICE\ ii}$ = 1.0(-29.6 k-ft.) + 1.0(-4.9 k-ft.) + 1.0(-11.9 k) + 1.3(-68.4 k)
= -135.3 k

Shear per bolt = 135.3 k / 24 bolts
= 5.6 k

Calculate Factored Slip Resistance for Bolts, R_r (6.13.2.2)

$R_n = \phi R_n$ (Eq. 6.13.2.2-2)

Where:

ϕ = 1.00 (6.5.4.2)

$R_n = K_h K_s N_s P_t$ (kips)

K_h = 1.00 (Table 6.13.2.8-2)

K_s = 0.30 (Table 6.13.2.8-3)

N_s = 2

P_t = 39 k (Table 6.13.2.8-1)

R_n = (1.00)(1.00)(0.30)(2)(39 k)
= 23.4 k > 5.6 k O.K.

Check Web Splice Plate Block Shear

$P_{web} \le R_r$

Where:

$R_r = \phi_{bs}(0.58 F_u A_{vn})$ (Modified Eq. 6.13.4-2)

Where:

A_{vn} = (40 in. – 12 holes * 0.9375 in./hole)(0.375 in.)

= 10.78 in.^{2}

F_u = 65 ksi

ϕ_{bs} = 0.80 (6.5.4.2)

$R_r = \phi_{bs}(0.58F_uA_{vn})$

= 0.8(0.58)(65 ksi)(10.78 in.^{2})

= 325.1 k

261.1 k < 325.1 k O.K.

Check Bolt Bearing

The extreme corner bolt has a 2.125 in. distance from the center of the bolt to the edge of the plate, and can utilize the thickness of both splice plates in determining the bearing capacity. Due to the 0.25 in. gap between beam segments, the top or bottom bolt adjacent to the end of beam only has a distance from the center of the bolt to the end of the beam segment equal to 2.125 in. – 0.5(0.25 in.) = 2.0 in., and can only use the smaller web thickness in determining the capacity. The top or bottom bolt adjacent to the end of the beam controls by inspection.

$P_r \leq R_r$

Where:

$P_r = P_{web}$ / # of bolts in web on one side of splice

= 261.1 k / 24 bolts

= 10.9 k

$R_r = \phi_{bb}R_n$

Where:

ϕ_{bb} = 0.80 (6.5.4.2)

If $x_{clear} \geq 2.0d$ and $x_{end} \geq 2.0d$:

$R_n = 2.4dtF_u$ (Eq. 6.13.2.9-1)

If $x_{clear} < 2.0d$ or $x_{end} < 2.0d$:

$R_n = 1.2L_ctF_u$ (Eq. 6.13.2.9-2)

Where:

$x_{clear} = 3$ in. – 0.9375 in. = 2.06 in.

$x_{end} = 2.125$ in. – 0.5(0.25 in.) = 2.0 in.

$d = 0.875$ in.

$x_{clear} \geq 2.0d$ and $x_{end} \geq 2.0d$, $\therefore R_n = 2.4L_ctF_u$

Where:

$t = 0.4375$ in. web thickness

$F_u = 65$ ksi (Table 6.4.1-1)

$L_c = 2.0$ in.

$R_n = 2.4(2.0 \text{ in.})(0.4375 \text{ in.})(65 \text{ ksi})$

$= 136.5$ k

$R_r = (0.80)(136.5 \text{ k}) = 109.2 \text{ k} > 10.9 \text{ k}$ O.K.

Check Web Splice Bolt Spacing

Using Bridge Manual Figures 3.3.21-1 and 3.3.21-3, spacing O.K.