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. Skew effects are included in the design of the composite splice and are calculated using the simplification in Chapter 6 of the LRFD Code.

The differences in the provisions between the LRFD and LFD Codes are minor and are not accounted for in this design guide.

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

1. General (6.13.6.1.4a)
2. Flange Splices (6.13.6.1.4c)
3. Web Splices (6.13.6.1.4b)

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 according to Article C6.13.6.1.4a they shall be checked for both positive and negative flexure to determine the controlling case. For the purpose of the included example design, compression stresses are positive and tension stresses are negative.

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. 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.

According to Article 6.13.1, splices should be designed for the factored forces at the location of the splice, but shall not be designed for less than 75 percent of the factored resistance of the member.

Determine Flange Stresses

Stresses shall be calculated at the mid-thickness of each flange (C6.13.6.1.4c).

Stresses shall be determined using the gross section properties (6.13.6.1.4a).

$$f = \left(\frac{Mc}{I}\right)$$

Where:

f = flange stress (ksi)

M = moment from the load (k-in.)

c = distance from the neutral axis to the mid-thickness of the flange for which the stress is calculated (in.)

I = moment of inertia of the applicable section of the beam or girder (in.^{4}). Different moments of inertia are used for different checks. The following is a summary of what moments of inertia should be used:

For DC_1 loading, the steel section alone is used.

For DC_2 and DW loading in the positive moment region, the long-term composite transformed section is used.

For LL+IM loading in the positive moment region, the short-term composite transformed section is used.

For Service II negative moment checks, an uncracked transformed section may be used if the total tension in the deck $\frac{1}{3n}\left(\frac{M_{DC2}+M_{DW}}{S_{n=27}}\right)+\frac{1}{n}\left(\frac{1.3\left(M_{LL+IM}\right)}{S_{n=9}}\right)$ does not exceed twice the modulus of rupture for the deck ($2f_r$). See 6.10.4.2.1.

For Fatigue negative moment checks, an uncracked transformed section may always be used. See 6.6.1.2.1.

For all other negative moment checks, a cracked section should always be used.

<u>Strength I Stresses</u>

Use the dead load and controlling live load plus impact stresses to calculate the Strength I load case flange stresses. Controlling positive and negative live loads, as defined in Article 3.6.1.2, shall be investigated in stress calculations. Stresses shall be factored according to Article 3.4.1, using the maximum and minimum factors. To obtain critical stresses, use the appropriate factors and exclude f_{DW} if a more conservative result is obtained.

$f_u = \gamma_{DC1}(f_{DC1}) + \gamma_{DC2}(f_{DC2}) + \gamma_{DW}(f_{DW}) + 1.75(f_{LL+IM}) + f_l$

Where:

γ_{DC1} = 1.25 or 0.90

γ_{DC2} = 1.25 or 0.90

γ_{DW} = 1.50 or 0.65

f_{DC1} = unfactored stress calculated from DC1 moment (ksi)

f_{DC2} = unfactored stress calculated from DC2 moment (ksi)

f_{DW} = unfactored stress calculated from DW moment (ksi)

f_{LL+IM} = unfactored stress calculated from LL+IM moment (ksi)

f_l = lateral flange bending stress. For top splice plates this should always be taken as zero. For bottom splice plates, the effects of skew currently may be ignored. The effects of curvature should be accounted for.

Service II Stresses

Use the dead load and controlling live load plus impact stresses to calculate Service II flange stresses. Controlling positive and negative live loads, as defined in Article 3.6.1.2, shall be investigated in stress calculations. Stresses shall be factored according to Article 3.4.1. To obtain critical Service II stresses, f_{DW} may be excluded if a more conservative result is obtained.

$f_o = 1.0(f_{DC1}) + 1.0(f_{DC2}) + 1.0(f_{DW}) + 1.3(f_{LL+IM}) + f_l$

Fatigue Stresses

Use the fatigue truck plus impact stresses to calculate Fatigue flange stresses (3.6.1.4). Stresses shall be factored according to Article 3.4.1. Positive and negative fatigue forces shall be investigated.

The fatigue and fracture limit state uses the fatigue load combinations found in Table 3.4.1-1:

$\gamma(f_r) = \gamma(f_{LL+IM}) + f_l$

Where:

γ = 1.5 for Fatigue I loading and 0.75 for Fatigue II loading

Whether Fatigue I or Fatigue II limit state is used depends upon the amount of fatigue cycles the member is expected to experience throughout its lifetime. For smaller amounts of cycles, Fatigue II, or the finite life limit state, is used. As the amounts of cycles increase, there comes a point where use of finite life limit state equations becomes excessively conservative, and use of the Fatigue I, or infinite life limit state, becomes more accurate.

To determine whether Fatigue I or Fatigue II limit state is used, the single-lane average daily truck traffic $(ADTT)_{SL}$ at 75 years must first be calculated. $(ADTT)_{SL}$ is the amount of truck traffic in a single lane in one direction, taken as a reduced percentage of the Average Daily Truck Traffic (ADTT) for multiple lanes of travel in one direction.

$$(ADTT)_{75,\,SL} = p \times ADTT_{75} \qquad \text{(Eq. 3.6.1.4.2-1)}$$

Where:

p = percentage of truck traffic in a single lane in one direction, taken from Table 3.6.1.4.2-1.

$ADTT_{75}$ is the amount of truck traffic in one direction of the bridge at 75 years. Type, Size, and Location reports usually give ADTT in terms of present day and 20 years into the future. The ADTT at 75 years can be extrapolated from this data by assuming that the ADTT will grow at the same rate i.e. follow a straight-line extrapolation using the following formula:

$$ADTT_{75} = \left(\left(ADTT_{20} - ADTT_0\right)\left(\frac{75\text{ years}}{20\text{ years}}\right) + ADTT_0\right)(DD)$$

Where:

$ADTT_{20}$ = ADTT at 20 years in the future, given on TSL

$ADTT_0$ = present-day ADTT, given on TSL

DD = directional distribution, given on TSL

The designer should use the larger number given in the directional distribution. For example, if the directional distribution of traffic was 70% / 30%, the ADTT for design should be the total volume times 0.7 in order to design for the beam experiencing the higher ADTT. If a bridge has a directional distribution of 50% / 50%, the ADTT for design should be the total volume times 0.5. If a bridge is one-directional, the ADTT for design is the full value, as the directional distribution is 100% / 0% i.e. one.

When $(ADTT)_{75,\ SL}$ is calculated, it is compared to the infinite life equivalent found in Table 6.6.1.2.3-2. If the calculated value of $(ADTT)_{75,\ SL}$ exceeds the value found in this table, then the infinite life (Fatigue I) limit state is used. If not, the finite life (Fatigue II) limit state is used.

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 through each shear plane). 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 (6.13.6.1.4c)

The effective area for tension flanges is found as:

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

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.)

Determine Strength I Controlling and Non-controlling Flange Stresses

Compare Strength I flange stresses. The largest ratio of flange stress to flange capacity will determine which flange has the controlling stress, f_{cf}. The corresponding stress on the opposite flange of the same side of the splice is the non-controlling flange stress, f_{ncf}. For example, if the bottom flange of member one has the controlling flange stress, then the non-controlling flange stress is found on the top flange of member one.

This should be repeated for both positive and negative loading. Typically for composite splices the bottom flange will be further from the neutral axis and will have a higher stress/capacity ratio and will be the controlling flange. However, this is not a given and depends greatly on flange sizes.

When calculating the flange stress to flange capacity ratio, note that there are cases where the stress capacity may be considerably less than F_y, and may not be allowed to be taken as such. A common example of this is for bottom flanges in negative moment regions, where lateral-torsional buckling or local flange buckling controls the design.

It should be noted that the LRFD code favors the sign convention that compression is positive and tension is negative. That preference is used in this design guide. However, many other sources use the opposite sign convention. Obviously, the only design concern when designing splices is that the convention chosen is consistently applied throughout the design.

Calculate Strength I Flange Design Forces (6.13.6.1.4c)

Determine the controlling and non-controlling Strength I flange design forces for both positive and negative flexure.

Controlling Flange Design Force, P_{cf}

$P_{cf} = F_{cf}A_e$

Where:

$$F_{cf} = \left(\frac{\left|\frac{f_{cf}}{R_h}\right| + \alpha\phi_f F_{yf}}{2} \right) \geq 0.75\alpha\phi_f F_{yf} \quad \text{(ksi)} \qquad \text{(Eq. 6.13.6.1.4c-1)}$$

f_{cf} = controlling flange stress (ksi)

R_h = 1.0 for non-hybrid girders and hybrid girders where the flanges yield before the web at the splice location

α = 1.0 for all cases where the full value of F_y (i.e. not a reduced value to account for buckling) was used as the capacity for the stress/capacity ratio calculation when determining the controlling flange stress. Note that, even if the full value of F_y was not used, it is still always simpler and more conservative to use α = 1.0 in this calculation, as it results in a higher flange force to be used in design. If the full value of F_y was not used in the calculation of the stress/capacity ratio, then α may be taken as the ratio of the reduced capacity to the yield stress of the flange. Use of this ratio for α essentially replaces F_{yf} with the actual flange capacity used.

ϕ_f = 1.0 (6.5.4.2)

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

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

Non-controlling Flange Design Force, P_{ncf}

$P_{ncf} = F_{ncf}A_e$

Where:

$$F_{ncf} = R_{cf}\left|\frac{f_{ncf}}{R_h}\right| \geq 0.75\alpha\phi_f F_{yf} \quad \text{(ksi)} \qquad \text{(Eq. 6.13.6.1.4c-3)}$$

$R_{cf} = \left|\frac{F_{cf}}{f_{cf}}\right|$

f_{ncf} = non-controlling flange stress (ksi)

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

Check Flange Splice Plate Strength

Check Tension in Splice Plates (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.

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)

Check Compression in Splice Plates

The factored resistance in compression, R_r, is taken as

$R_r = \phi_c F_y A_s > P_{cf}$ or P_{ncf} as applicable (Eq. 6.13.6.1.4c-4)

Where:

ϕ_c = 0.90 (6.5.4.2)

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

A_s = gross area of the splice plate (in.^{2})

Check Flange Splice Plate Fatigue

Check Fatigue Detail Design Criteria (6.6.1.2.2)

Bolted splices for most bridges are designed using fatigue detail category B (See Table 6.6.1.2.3-1, Condition 2.2). If the bridge has been hot-dip galvanized, use category D (See Table 6.6.1.2.3-1, Condition 2.3). As Section 505 of the Standard Specifications does not allow for holes to be punched full size, the fatigue category downgrade for full-sized punched holes given in Condition 2.3 should not be used.

The following design criteria shall be met:

$\gamma(\Delta f) \leq (\Delta F)_n$ (Eq. 6.6.1.2.2-1)

Where:

γ = load factor, taken as 1.5 for Fatigue I loading and 0.75 for Fatigue II loading. Determination of controlling load case (Fatigue I or Fatigue II) is explained above, in "Determine Flange Stresses."

(Δf_r) = fatigue live load stress range on flange splice plate (ksi)

$$= \frac{(f_{fat}^{+})A_e - (f_{fat}^{-})A_e}{A_{PL}}$$

Note that this method determines the total force on the effective flange area and uses that force to determine a stress on the gross section of the flange splice plate at the centerline of splice.

A_e = effective flange area corresponding to the fatigue stress (in.^{2})

A_{PL} = gross area of the flange splice plate (in.^{2})

f_{fat} = stress due to fatigue loading (ksi)

$(\Delta F)_n$ = nominal fatigue resistance (ksi)

= $(\Delta F)_{TH}$ for Fatigue I load combination (Eq. 6.6.1.2.5-1)

= $\left(\frac{A}{N}\right)^{\frac{1}{3}}$ for Fatigue II load combination (Eq. 6.6.1.2.5-2)

N = $(365)(75)n(ADTT)_{SL}$ (Eq. 6.6.1.2.5-3)

A = 120.0×10^8 ksi^3 for fatigue category B (Table 6.6.1.2.5-1)

$(\Delta F)_{TH}$ = 16.0 ksi (Table 6.6.1.2.5-3)

n = no. of stress range cycles per truck passage (Table 6.6.1.2.5-2)

$(ADTT)_{SL}$ = single-lane ADTT at 37.5 years (see above for calculation)

Calculate Reduction Factor for Fillers (6.13.6.1.5)

When flange thickness transitions at splice locations, fillers shall be used. Fill 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.

$$R = \left|\frac{(1+\gamma)}{(1+2\gamma)}\right| \qquad \text{(Eq. 6.13.6.1.5-1)}$$

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 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 High Strength (H.S.) A325 bolts with standard holes.

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).

Check Flange Splice Bolt Shear Strength

<u>Compare Factored Bolt Shear, P_r, with Factored Resistance, R_r</u>

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

Where:

P_r = factored bolt shear (kips) = $\dfrac{P_{cf} \text{ or } P_{ncf}}{N_b}$

In which:

P_{cf} and P_{ncf} are the controlling and non-controlling flange forces, respectively, as calculated above. The value used should be the one that is applicable for the flange in question (i.e. use the controlling force for the controlling flange and the non-controlling force for the non-controlling flange)

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 on one side of the splice is greater than 50 inches, calculated bolt shear strength should by multiplied by a factor of 0.8. 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 = $0.38A_bF_{ub}N_s$ 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 = 0.33. Standard Inorganic and Organic Zinc-Rich primers both require a coefficient of friction of 0.50. See AASHTO M300 and the special provision for Organic Zinc-Rich Paint System. Use of 0.33 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:

$P_{tot\text{-}slip}$ = F_sA_g (kips)

F_s = $\left|\frac{f_s}{R_h}\right|$ (ksi) (Eq. 6.13.6.1.4c-5)

f_s = maximum Service II flexural stress (ksi)

R_h = hybrid factor (6.10.1.10.1)

A_g = gross area of the flange corresponding to f_s (in.^{2})

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

Check Flange Splice Bearing Resistance:

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

Where:

$$P_{brg} = \frac{P_{cf} \text{ or } P_{ncf}}{N_b}$$

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 in 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 High Strength (H.S.) A325 bolts with standard holes. A minimum of two vertical bolt rows shall be used on each side of the splice connection element (6.13.6.1.4a). 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.

Check Web Splice Plate Strength

Web splice plates and bolts shall be designed for shear, the moment due to the eccentricity of the shear at the point of the splice, and the portion of the flexural moment assumed to be resisted by the web at the point of the splice (Article 6.13.6.1.4b). To determine the applied shear for the shear eccentricity portion of the load, the applied shear and the shear capacity must first be calculated. The splice plates are then designed for the lesser of 150% the web shear capacity (each splice plate is designed for 75% of the capacity), or the average of the applied shear and the web shear capacity.

Calculate Strength I Shear Forces,V_u

Use dead loads and the controlling live load plus impact to calculate shear forces at the splice. Controlling positive and negative live loads shall be investigated in shear force calculations. Forces shall be factored according to Article 3.4.1, using the maximum and minimum factors. To obtain the critical shears, use the appropriate factors and exclude V_{DW} if a more conservative result is obtained.

Additional vertical shears due to lateral skew effects are marginal.

$$V_u = \gamma_{DC1}(V_{DC1}) + \gamma_{DC2}(V_{DC2}) + \gamma_{DW}(V_{DW}) + 1.75(V_{LL+IM})$$

Where:

γ_{DC1} = 1.25 or 0.90

γ_{DC2} = 1.25 or 0.90

γ_{DW} = 1.50 or 0.65

Calculate Web Shear Resistance

$\phi_v V_n$ = web shear resistance (k) (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)

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

= $V_p\left[C + \dfrac{0.87(1-C)}{\sqrt{1+\left(\dfrac{d_o}{D}\right)^2} + \dfrac{d_o}{D}}\right]$ 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 $\dfrac{D}{t_w} \le 1.12\sqrt{\dfrac{Ek}{F_{yw}}}$:

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

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

$$C = \frac{1.12}{(D/t_w)}\sqrt{\frac{Ek}{F_{yw}}}$$ (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 Strength I Flexural Stress for Web Splice Plates</u>

The total flexural stress in the web splice plate may be determined by reducing the stress into two different components: stress due to shear eccentricity in the connection (M_{uv}) and stress due to the portion of the moment resisted by the web (M_{uw}). The stress due to the portion of the moment resisted by the web may further be simplified into two components: stress assuming a symmetric stress diagram (M_{uw}) plus a stress due to the eccentricity due to the actual non-symmetry of the stress diagram (H_{uw}). Note that, if the stress diagram is truly symmetric, H_{uw} equals zero.

$$f_{STRENGTH\ I} = \frac{M_{uv} + M_{uw}}{S_{PL}} + \frac{|H_{uw}|}{A_{PL}}$$

Where:

$S_{PL} = \frac{2(t_{PL}h_{PL}{}^2)}{6}$ (in.^{3})

$A_{PL} = 2(h_{PL}t_{PL})$ (in.^{2})

t_{PL} = thickness of web splice plate (in.)

h_{PL} = height of web splice plate (in.)

M_{uv}, M_{uw}, and H_{uw} are as calculated below:

Calculate M_{uv}, Moment Due to Eccentricity of Shear in Connection:

M_{uv} is the moment in the splice plate due to the shear transferring through the plate.

$M_{uv} = V_{uw}e$ (k-in.)

Where:

e = design shear eccentricity, taken as the distance from the centerline of splice to the centroid of the bolt group in the horizontal direction (in.)

V_{uw} = shear due to eccentricity of connection (kips), determined as follows:

= $1.5V_u$ if $V_u < 0.5\phi_v V_n$ (Eq. 6.13.6.1.4b-1)

= $\dfrac{(V_u + \phi_v V_n)}{2}$ otherwise (Eq. 6.13.6.1.4b-2)

Where:

V_u = factored Strength I shear loads (kips)

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

V_n = nominal shear resistance as calculated above (kips)

Calculate M_{uw}, Portion of Moment Resisted by Web (Assuming Symmetric Stress Diagram):

M_{uw} = portion of moment resisted by the web, based on a theoretic symmetric stress diagram (k-in.)

$$= \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| \quad \text{(C6.13.6.1.4b-1)}$$

Where:

t_w = web thickness (in.)

D = web depth (in.)

R_h = hybrid factor (6.10.1.10.1)

F_{cf} = design stress for the controlling flange at the point of splice specified in Article 6.13.6.1.4c; positive for tension, negative for compression (ksi)

$R_{cf} = \left|\dfrac{F_{cf}}{f_{cf}}\right|$

f_{ncf} = Strength I flexural stress at mid-thickness of the non-controlling flange at the splice location (ksi)

f_{cf} = Strength I flexural stress at mid-thickness of the controlling flange at the splice location (ksi)

Calculate H_{uw}, Additional Force to Account for Non-Symmetry of Stress Diagram:

H_{uw} = additional force due to actual non-symmetry of stress diagram (kips). Note that this term may be zero if $f_{cf} = -f_{ncf}$, as this implies symmetry of the section and therefore this term need not be considered. For splices designed noncompositely this was common for all wide-flange beams and some plate girders where the top and bottom flanges were the same size. For splices designed compositely this will be less common.

$$= \frac{t_w D}{2}(R_h F_{cf} + R_{cf} f_{ncf}) \qquad \text{(C6.13.6.1.4b-2)}$$

Where all variables are as calculated above.

<u>Compare Strength I Flexural Stress with $\phi_f F_y$</u> (6.13.6.1.4b)

$f_{STRENGTH\ I} \leq \phi_f F_y$

Where:

ϕ_f = 1.0 (6.5.4.2)

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

Check Web Splice Plate Shear Capacity *(6.13.5.3)*

$V_{uw} \leq R_r$

Where V_{uw} is the ultimate applied shear and 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.

<u>Calculate Factored Shear Resistance for Yielding of Gross Web Splice Section</u>

$R_r = \phi_v 0.58 F_y A_{vg}$ (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

$R_r = \phi_{vu} 0.58 R_p F_u A_{vn}$ (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 Plate Fatigue

The fatigue forces on a web splice are calculated similarly to the Strength I forces: there is a moment due to shear eccentricity and a moment due to the applied moment in the web. The stress due to the moment applied to the web is similarly broken into components, as it is in the Strength I stress determination.

Calculate Fatigue Shear Forces, V_{rw}

Use the fatigue truck plus impact to calculate shear forces at the splice. Positive and negative fatigue forces shall be investigated.

$V_{rw} = 0.75V_{(LL+IM)}$

Calculate Fatigue Moment Due to Shear Eccentricity, M_{rv} (C6.13.6.1.4b)

$M_{rv} = [(V_{rw}^{+})-(V_{rw}^{-})]e$

Calculate Fatigue Flexural Moment, M_{rw}

A portion of the flexural moment is assumed to be resisted by the web at the point of the splice. This flexural moment portion, M_{rw}, shall be calculated for both positive and negative flexure. The absolute values are eliminated from the equation in order to keep track of the signs.

$$M_{rw} = \frac{t_w D^2}{12}[f_{tw} - f_{bw}] \qquad \text{(Modified Eq. C6.13.6.1.4b-1)}$$

Where:

f_{tw} = flexural stress due to Fatigue loads at the bottom of the top flange (ksi)

f_{bw} = flexural stress due to Fatigue loads at the top of the bottom flange (ksi)

To avoid recalculating section properties to the insides of flanges, the stress at the midthickness of the flange may conservatively be used in lieu of the stress at the inside of the flange when calculating f_{tw} and f_{bw}.

Calculate Fatigue Total Moment Range, $M_{r\text{-total range}}$

$$M_{r\text{-total range}} = M_{rv} + (M_{rw}^{+} - M_{rw}^{-})$$

Calculate Fatigue Design Horizontal Force Resultant

The horizontal force resultant, H_{rw}, shall be calculated for both positive and negative flexure.

$$H_{rw} = \frac{t_w D}{2}(f_{tw} + f_{bw}) \qquad \text{(Modified Eq. C6.13.6.1.4b-2)}$$

Calculate Horizontal Force Resultant Range, $H_{rw\text{-range}}$

$$H_{rw\text{-range}} = H_{rw}^{+} - H_{rw}^{-}$$

Calculate Factored Fatigue Stress Range, $\gamma(\Delta f)$, for Web Splice Plates

$$\gamma(\Delta f) = \frac{M_{r-totalrange}}{S_{PL}} + \frac{H_{rw-range}}{A_{PL}}$$

Where:

$S_{PL} = \dfrac{2(t_{PL}h_{PL}^{\ 2})}{6}$ (in.^{3})

$A_{PL} = 2(h_{PL}t_{PL})$ (in.^{2})

t_{PL} = thickness of web splice plate (in.)

h_{PL} = height of web splice plate (in.)

Check Fatigue Detail Design Criteria (6.6.1.2.2)

Bolted splices are detail category B, unless they are hot-dip galvanized, in which case they are Category D. (Table 6.6.1.2.3-1)

The following design criteria shall be met:

$\gamma(\Delta f_r) \le (\Delta F)_n$ (Eq. 6.6.1.2.2-1)

Where:

$\gamma(\Delta f_r)$ = factored fatigue live load stress range on web splice plate (ksi)

$(\Delta F)_n$ = nominal fatigue resistance (ksi)

= $(\Delta F)_{TH}$ for Fatigue I load combination (Eq. 6.6.1.2.5-1)

= $\left(\dfrac{A}{N}\right)^{\frac{1}{3}}$ for Fatigue II load combination (Eq. 6.6.1.2.5-2)

N = $(365)(75)n(ADTT)_{SL}$ (Eq. 6.6.1.2.5-3)

A = 120.0×10^8 ksi^3 for fatigue category B (Table 6.6.1.2.5-1)

$(\Delta F)_{TH}$ = 16.0 ksi (Table 6.6.1.2.5-3)

n = no. of stress range cycles per truck passage (Table 6.6.1.2.5-2)

$(ADTT)_{SL}$ = single-lane ADTT at 37.5 years (see above for calculation)

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:

$$R_r = \phi_s R_n R > P_r$$

Where:

ϕ_s = 0.80 (6.5.4.2)

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

R_n = $0.38 A_b F_{ub} N_s$ (kips) (Eq. 6.13.2.7-2)

Where:

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

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.

P_r = applied shear at the extreme bolt (kips), determined as shown below.

Calculate the Polar Moment of Inertia

The polar moment of inertia of the bolts, I_p, shall be calculated with respect to the centroid of the splice bolt group. For the purpose of this design guide, the x-axis and y-axis are located at the center of the web splice bolt group.

$$I_p = \sum x^2 + \sum y^2$$

Where:

x = distance from centroid of bolt group to bolt in x-dirention (in.)

y = distance from centroid of bolt group to bolt in y-direction (in.)

Alternatively, the LRFD code allows for use of the following equation:

$$I_p = \frac{nm}{12}\left[s^2\left(n^2 - 1\right) + g^2\left(m^2 - 1\right)\right] \quad \text{(Eq. C6.13.6.1.4b-3)}$$

Where:

n = number of bolts in one vertical row

m = number of vertical rows of bolts

s = vertical pitch (in.)

g = horizontal pitch (in.)

Calculate Strength I Resultant Shear for Bolts, P_r

$$P_r = \sqrt{\left(P_S + P_{MV}\right)^2 + \left(P_H + P_{MH}\right)^2}$$

Where:

P_S = shear per bolt in the vertical direction, due to applied vertical shear (kips)

$$= \frac{V_{uw}}{N_b}$$

P_H = shear due per bolt in the horizontal direction, due to non-symmetry of section (kips)

$$= \left|\frac{H_{uw}}{N_b}\right|$$

P_{MH}= shear due to portion of moment resisted by web, in the horizontal direction (kips)

$$= \frac{(M_{uv} + M_{uw})y}{I_p}$$

P_{MV}= shear due to portion of moment resisted by web, in the vertical direction (kips)

$$= \frac{(M_{uv} + M_{uw})x}{I_p}$$

Where:

N_b = number of web splice bolts

y = distance from x-axis to the extreme bolt (in.)

x = distance from y-axis to the extreme bolt (in.)

Check Web Splice Bolt Slip

Calculate Service II Shears for Web Splice Plates

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.

$$V_{ow} = 1.0(V_{DC1}) + 1.0(V_{DC2}) + 1.0(V_{DW}) + 1.3(V_{LL+IM})$$

Calculate Service II Flexural Moment (C6.13.6.1.4b)

The Service II flexural moment is calculated similarly to the Strength I flexural moment, with slight modifications. For brevity, only the main three equations are listed here.

$$M_{ov} = V_{ow}e$$

$$M_{ow} = \frac{t_w D^2}{12}\left|f_s - f_{os}\right| \quad \text{(Modified Eq. C6.13.6.1.4b-1)}$$

Where:

f_s = maximum flexural stress due to Service II loads at mid-thickness of the flange (ksi)

f_{os} = Service II stress at mid-thickness of the other flange concurrent with f_s (ksi)

$$H_{ow} = \frac{t_w D}{2}(f_s + f_{os}) \qquad \text{(Modified Eq. C6.13.6.1.4b-2)}$$

<u>Calculate Service II Resultant Shear for Bolts</u>

The Service II resultant shear is calculated similarly to the Strength I resultant shear. For brevity, only the main equations are listed here.

$$P_{or} = \sqrt{(P_{os} + P_{oMV})^2 + (P_{oH} + P_{oMH})^2}$$

Where:

$$P_{os} = \frac{V_{ow}}{N_b}$$

$$P_{oH} = \left|\frac{H_{ow}}{N_b}\right|$$

$$P_{oMH} = \frac{(M_{ov} + M_{ow})y}{I_p}$$

$$P_{oMV} = \frac{(M_{ov} + M_{ow})x}{I_p}$$

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

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

Where:

ϕ = 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 = 0.33. (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)

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

Verify $P_{or} \leq R_r$ for both positive and negative flexure.

Check Web Splice Plate Block Shear

$V_{uw} \leq \quad R_r$

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 Extreme Bolt Bearing

There are two possible failure mechanisms involving a bolt in a web splice breaking through the edge of a plate. The first is the extreme bolt breaking through the splice plate at the extreme corner of the web splice. The other is the bolt closest to the web at the top or bottom of the splice breaking through the web itself. Both cases should be checked, but typically a controlling case can be determined by inspection.

$P_r \leq \quad R_r$

Where:

$R_r = \phi_{bb}R_n$

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 $x_{end} < 2.0d$:

$R_n = 1.2L_ctF_u$ (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
$ADTT_0$:	300 trucks
$ADTT_{20}$:	600 trucks
DD:	Two-Way Traffic (50% / 50%). Assume one lane each direction for fatigue loading
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.

Section 2:

$D = 42$ in.

$t_w = 0.5$ in.

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

$t_{bf} = 2.5$ in.

$t_{tf} = 2.0$ in.

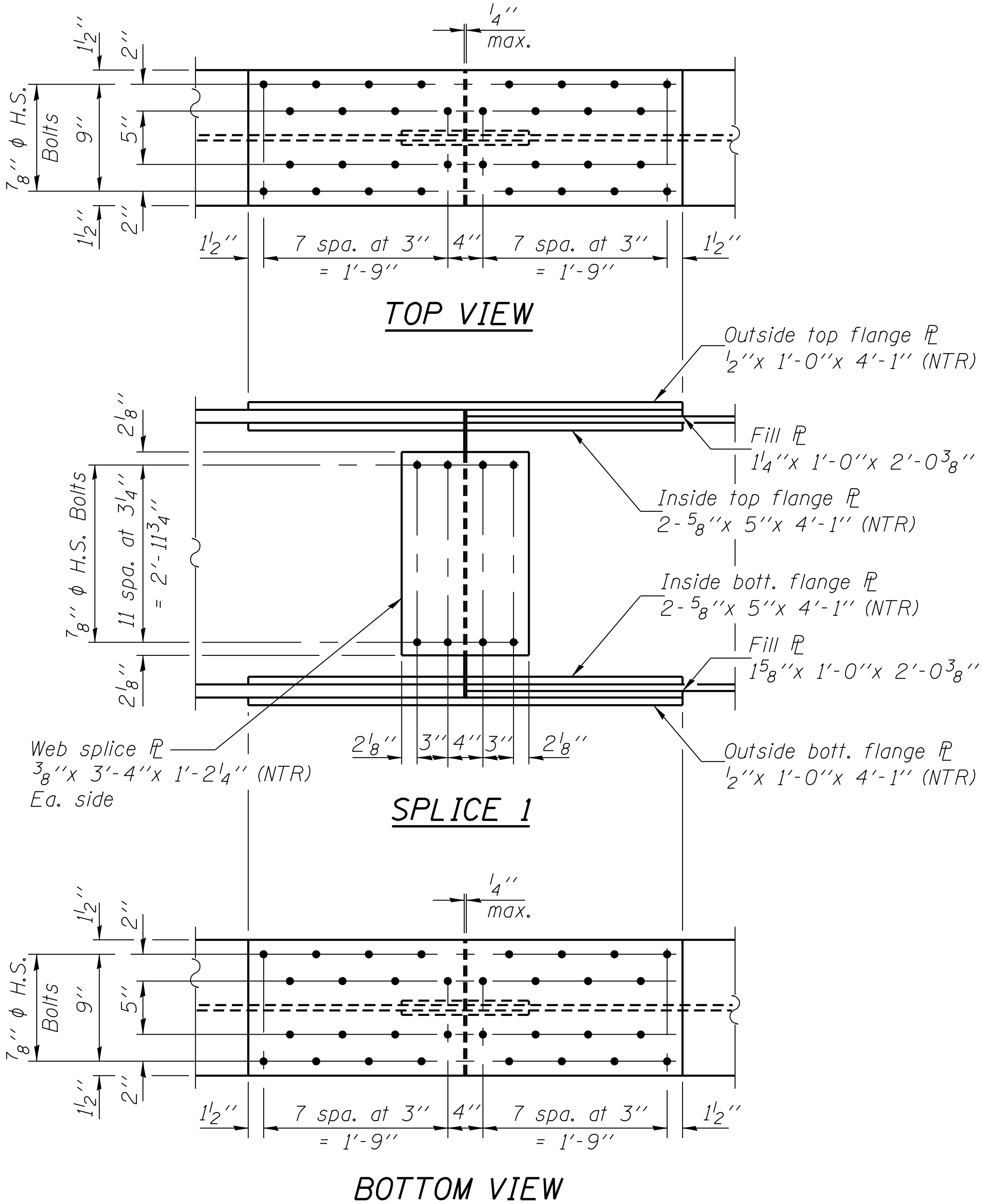
Section Properties

The section properties are consistent with those in Design Guide 3.3.4, but with the section moduli being calculated to the mid-depth of each flange instead of to the extreme fibers of the flange. For brevity, these section properties are listed below but full calculations are not shown:

Section 1 has been found to control the design. For brevity, only the section properties for this section are shown.

Section 1:

	Non-composite	Composite, n	Composite, 3n	Composite, Cracked
S_b (in.^{3})	564.48	797.79	734.09	648.50
S_t (in.^{3})	522.10	8027.40	2081.89	920.87



1/4" max.
1 1/2"
2"
7/8" φ H.S. Bolts
9"
5"
1 1/2"
7 spa. at 3" = 1'-9"
4"
7 spa. at 3" = 1'-9"
1 1/2"
TOP VIEW
Outside top flange PL 1/2"x 1'-0"x 4'-1" (NTR)
Fill PL 1 1/4"x 1'-0"x 2'-0 3/8"
Inside top flange PL 2-5/8"x 5"x 4'-1" (NTR)
Inside bott. flange PL 2-5/8"x 5"x 4'-1" (NTR)
Fill PL 1 5/8"x 1'-0"x 2'-0 3/8"
Outside bott. flange PL 1/2"x 1'-0"x 4'-1" (NTR)
2 1/8"
7/8" φ H.S. Bolts
11 spa. at 3 1/4" = 2'-11 3/4"
2 1/8"
2 1/8"
3"
4"
3"
2 1/8"
Web splice PL 3/8"x 3'-4"x 1'-2 1/4" (NTR) Ea. side
SPLICE 1
1/4" max.
1 1/2"
2"
7/8" φ H.S. Bolts
9"
5"
1 1/2"
7 spa. at 3" = 1'-9"
4"
7 spa. at 3" = 1'-9"
1 1/2"
BOTTOM VIEW

Figure 1

Unfactored Distributed Forces at Splice

	Moment (k-ft.)	Shear (k)
DC1	-7.5	-29.7
DC2	6.5	-4.9
DW	15.7	-11.9
Truck (+Trk+Ln)	831.9	12.0
Truck (-Trk+Ln)	-625.1	-68.4
Tandem (+Tan+Ln)	731.5	11.5
Tandem (-Tan+Ln)	-506.3	-56.5
2 Trucks -.9(2Trk+Ln)	-562.9	N/A
Fatigue (+FATLL+IM)	267.3	4.2
Fatigue (-FATLL+IM)	-201.1	-26.8
Construction DC1	-8.5	-31.3
Construction LL+IM	-3.9	-4.9

Calculate Unfactored Stresses

For positive flexure, the bottom flange is in tension and the top flange is in compression.
For negative flexure, the bottom flange is in compression and the top flange is in tension.
Use (+) for compression and (-) for tension.

Section 1 has been found to control the design. For brevity, section two stress calculations are omitted from this design guide.

			Stress (comp + /tens -)	
DC1 on bottom flange, non-composite section =	M_{DC1} / $S_{b(nc)}$ =	-7.5 k-ft. * (12 in. / ft.) / 564.48 in.^3 =	0.16	ksi
DC1 on top flange, non-composite section =	M_{DC1} / $S_{t(nc)}$ =	-7.5 k-ft. * (12 in. / ft.) / 522.1 in.^3 =	-0.17	ksi
$DC1_{const}$ on bottom flange, non-composite section =	$M_{DC1const}$ / $S_{b(nc)}$ =	-8.5 k-ft. * (12 in. / ft.) / 564.48 in.^3 =	0.18	ksi
$DC1_{const}$ on top flange, non-composite section =	$M_{DC1const}$ / $S_{t(nc)}$ =	-8.5 k-ft. * (12 in. / ft.) / 522.1 in.^3 =	-0.2	ksi
DC2 on bottom flange, composite section, n=27 =	M_{DC2} / $S_{b(c,3n)}$ =	6.5 k-ft. * (12 in. / ft.) / 734.09 in.^3 =	-0.11	ksi
DC2 on bottom flange, composite section, cracked =	M_{DC2} / $S_{b(c,cr)}$ =	6.5 k-ft. * (12 in. / ft.) / 648.5 in.^3 =	-0.12	ksi
DC2 on top flange, composite section, n=27 =	M_{DC2} / $S_{t(c,3n)}$ =	6.5 k-ft. * (12 in. / ft.) / 2081.89 in.^3 =	0.04	ksi
DC2 on top flange, composite section, cracked =	M_{DC2} / $S_{t(c,cr)}$ =	6.5 k-ft. * (12 in. / ft.) / 920.87 in.^3 =	0.08	ksi
DW on bottom flange, composite section, n=27 =	M_{DW} / $S_{b(c,3n)}$ =	15.7 k-ft. * (12 in. / ft.) / 734.09 in.^3 =	-0.26	ksi
DW on bottom flange, composite section, cracked =	M_{DW} / $S_{b(c,cr)}$ =	15.7 k-ft. * (12 in. / ft.) / 648.5 in.^3 =	-0.29	ksi
DW on top flange, composite section, n=27 =	M_{DW} / $S_{t(c,3n)}$ =	15.7 k-ft. * (12 in. / ft.) / 2081.89 in.^3 =	0.09	ksi
DW on top flange, composite section, cracked =	M_{DW} / $S_{t(c,cr)}$ =	15.7 k-ft. * (12 in. / ft.) / 920.87 in.^3 =	0.2	ksi

LL+IM+ on bottom flange, composite section, n=9 =	$M_{LL+IM+} / S_{b(c,n)}$ =	831.9 k-ft. * (12 in. / ft.) / 797.79 in.^3 =	-12.51	ksi
LL+IM+ on bottom flange, composite section, cracked =	$M_{LL+IM+} / S_{b(c,cr)}$ =	831.9 k-ft. * (12 in. / ft.) / 648.5 in.^3 =	-15.39	ksi
LL+IM+ on top flange, composite section, n=9 =	$M_{LL+IM+} / S_{t(c,n)}$ =	831.9 k-ft. * (12 in. / ft.) / 8027.4 in.^3 =	1.24	ksi
LL+IM+ on top flange, composite section, cracked =	$M_{LL+IM+} / S_{t(c,cr)}$ =	831.9 k-ft. * (12 in. / ft.) / 920.87 in.^3 =	10.84	ksi
LL+IM- on bottom flange, composite section, n=9 =	$M_{LL+IM-} / S_{b(c,n)}$ =	-625.1 k-ft. * (12 in. / ft.) / 797.79 in.^3 =	9.4	ksi
LL+IM- on bottom flange, composite section, cracked =	$M_{LL+IM-} / S_{b(c,cr)}$ =	-625.1 k-ft. * (12 in. / ft.) / 648.5 in.^3 =	11.57	ksi
LL+IM- on top flange, composite section, n=9 =	$M_{LL+IM-} / S_{t(c,n)}$ =	-625.1 k-ft. * (12 in. / ft.) / 8027.4 in.^3 =	-0.93	ksi
LL+IM- on top flange, composite section, cracked =	$M_{LL+IM-} / S_{t(c,cr)}$ =	-625.1 k-ft. * (12 in. / ft.) / 920.87 in.^3 =	-8.15	ksi
$LL+IM_{const}$ on bottom flange, non-composite section =	$M_{LL+IMconst} / S_{b(nc)}$ =	-3.9 k-ft. * (12 in. / ft.) / 564.48 in.^3 =	0.08	ksi
$LL+IM_{const}$ on top flange, non-composite section =	$M_{LL+IMconst} / S_{t(nc)}$ =	-3.9 k-ft. * (12 in. / ft.) / 522.1 in.^3 =	-0.09	ksi
Fatigue+ on bottom flange, composite section, n=9 =	$M_{FAT+} / S_{b(c,n)}$ =	267.3 k-ft. * (12 in. / ft.) / 797.79 in.^3 =	-4.02	ksi
Fatigue+ on top flange, composite section, n=9 =	$M_{FAT+} / S_{t(c,n)}$ =	267.3 k-ft. * (12 in. / ft.) / 8027.4 in.^3 =	0.4	ksi
Fatigue- on bottom flange, composite section, n=9 =	$M_{FAT-} / S_{b(c,n)}$ =	-201.1 k-ft. * (12 in. / ft.) / 797.79 in.^3 =	3.02	ksi
Fatigue- on top flange, composite section, n=9 =	$M_{FAT-} / S_{t(c,n)}$ =	-201.1 k-ft. * (12 in. / ft.) / 8027.4 in.^3 =	-0.3	ksi

Calculate Factored Stresses

Section 1 Top Flange: Determine Factored Stresses

Strength I Stresses:

For Strength I loading, the following sections are used:

	positive moment	negative moment
DC1	non-composite	non-composite
DC2	composite, n=27	composite, cracked
DW	composite, n=27	composite, cracked
LL+IM	composite, n=9	composite, cracked

Positive LL+IM, DW included

1.25DC1 + 1.25DC2 + 1.5DW + 1.75(LL+IM+)	1.25(-0.17 ksi) + 1.25(0.04 ksi) + 1.5(0.09 ksi) + 1.75(1.24 ksi) =	2.14	ksi
0.9DC1 + 0.9DC2 + 1.5DW + 1.75(LL+IM+)	0.9(-0.17 ksi) + 0.9(0.04 ksi) + 1.5(0.09 ksi) + 1.75(1.24 ksi) =	2.19	ksi
1.25DC1 + 1.25DC2 + 0.65DW + 1.75(LL+IM+)	1.25(-0.17 ksi) + 1.25(0.04 ksi) + 0.65(0.09 ksi) + 1.75(1.24 ksi) =	2.07	ksi
0.9DC1 + 0.9DC2 + 0.65DW + 1.75(LL+IM+)	0.9(-0.17 ksi) + 0.9(0.04 ksi) + 0.65(0.09 ksi) + 1.75(1.24 ksi) =	2.11	ksi

Positive LL+IM, DW neglected

1.25DC1 + 1.25DC2 + 1.75(LL+IM+)	1.25(-0.17 ksi) + 1.25(0.04 ksi) + 1.75(1.24 ksi) =	2.01	ksi
0.9DC1 + 0.9DC2 + 1.75(LL+IM+)	0.9(-0.17 ksi) + 0.9(0.04 ksi) + 1.75(1.24 ksi) =	2.05	ksi

Negative LL+IM, DW included

1.25DC1 + 1.25DC2 + 1.5DW + 1.75(LL+IM-)	1.25(-0.17 ksi) + 1.25(0.04 ksi) + 1.5(0.09 ksi) + 1.75(-8.15 ksi) =	-14.29	ksi
0.9DC1 + 0.9DC2 + 1.5DW + 1.75(LL+IM-)	0.9(-0.17 ksi) + 0.9(0.04 ksi) + 1.5(0.09 ksi) + 1.75(-8.15 ksi) =	-14.24	ksi
1.25DC1 + 1.25DC2 + 0.65DW + 1.75(LL+IM-)	1.25(-0.17 ksi) + 1.25(0.04 ksi) + 0.65(0.09 ksi) + 1.75(-8.15 ksi) =	-14.37	ksi
0.9DC1 + 0.9DC2 + 0.65DW + 1.75(LL+IM-)	0.9(-0.17 ksi) + 0.9(0.04 ksi) + 0.65(0.09 ksi) + 1.75(-8.15 ksi) =	-14.32	ksi

Negative LL+IM, DW neglected

1.25DC1 + 1.25DC2 + 1.75(LL+IM-)	1.25(-0.17 ksi) + 1.25(0.04 ksi) + 1.75(-8.15 ksi) =	-14.43	ksi
0.9DC1 + 0.9DC2 + 1.75(LL+IM-)	0.9(-0.17 ksi) + 0.9(0.04 ksi) + 1.75(-8.15 ksi) =	-14.38	ksi

Constructability

1.25DC1 + 1.75(LL+IMConst)	1.25(-0.2 ksi) + 1.75(-0.09 ksi) =	-0.41	ksi

Service II Stresses:

For Service II loading, if the amount of stress in the deck does not exceed $2f_r$ then an uncracked section may be used in the negative moment region. Note that for calculation of deck stresses, the section modulus used should be that for the composite section transformed to concrete, not the composite section transformed to steel. As the composite section transformed to concrete = n * $S_{tslab(c,n)}$, it is written that way in the equation below. The stresses for DC2 and DW act in the opposite direction to the LL+IM stresses and have been conservatively neglected.

$f_r = 0.24(f'_c)^{0.5}$ (5.4.2.6)

= 0.24 * (3.5 ksi)^0.5

= 0.45 ksi

$2f_r$ = 0.9 ksi

$1.3M_{LL+IM-} / (9 * S_{tslab(c,n)})$ = 1.3(-625.1 k-ft. * (12 in. / ft.)) / (9 * 2312.32 in.^3) = -0.47 ksi

Uncracked sections should be used.

	positive moment	negative moment
DC1	non-composite	non-composite
DC2	composite, n=27	composite, n=27
DW	composite, n=27	composite, n=27
LL+IM	composite, n=9	composite, n=9

Positive LL+IM, DW included

1.00DC1 + 1.00DC2 + 1.00DW + 1.30(LL+IM+) 1.00(-0.17 ksi) + 1.00(0.08 ksi) + 1.00(0.09 ksi) + 1.30(1.24 ksi) = 1.57 ksi

Positive LL+IM, DW neglected

1.00DC1 + 1.00DC2 + 1.30(LL+IM+) 1.00(-0.17 ksi) + 1.00(0.08 ksi) + 1.30(1.24 ksi) = 1.48 ksi

Negative LL+IM, DW included

1.00DC1 + 1.00DC2 + 1.00DW + 1.30(LL+IM-) 1.00(-0.17 ksi) + 1.00(0.08 ksi) + 1.00(0.09 ksi) + 1.30(-0.93 ksi) = -1.25 ksi

Negative LL+IM, DW neglected

1.00DC1 + 1.00DC2 + 1.30(LL+IM-) 1.00(-0.17 ksi) + 1.00(0.08 ksi) + 1.30(-0.93 ksi) = -1.34 ksi

Fatigue I Stresses:

Uncracked sections may be used for Fatigue I and II loading. See 6.6.1.2.1

	positive moment	negative moment
LL+IM	composite, n=9	composite, n=9

Positive LL+IM

1.50(LL+IM+) 1.50(0.4 ksi) = 0.6 ksi

Negative LL+IM

1.50(LL+IM-) 1.50(-0.3 ksi) = -0.45 ksi

Fatigue II Stresses:

Positive LL+IM

0.75(LL+IM+) 0.75(0.4 ksi) = 0.3 ksi

Negative LL+IM

0.75(LL+IM-)	0.75(-0.3 ksi) =	-0.23	ksi

Section 1 Bottom Flange: Determine Factored Stresses

Strength I Stresses:

For Strength I loading, the following sections are used:

	positive moment	negative moment
DC1	non-composite	non-composite
DC2	composite, n=27	composite, cracked
DW	composite, n=27	composite, cracked
LL+IM	composite, n=9	composite, cracked

Positive LL+IM, DW included

1.25DC1 + 1.25DC2 + 1.5DW + 1.75(LL+IM+)	1.25(0.16 ksi) + 1.25(-0.11 ksi) + 1.5(-0.26 ksi) + 1.75(-12.51 ksi) =	-22.22	ksi
0.9DC1 + 0.9DC2 + 1.5DW + 1.75(LL+IM+)	0.9(0.16 ksi) + 0.9(-0.11 ksi) + 1.5(-0.26 ksi) + 1.75(-12.51 ksi) =	-22.24	ksi
1.25DC1 + 1.25DC2 + 0.65DW + 1.75(LL+IM+)	1.25(0.16 ksi) + 1.25(-0.11 ksi) + 0.65(-0.26 ksi) + 1.75(-12.51 ksi) =	-22	ksi
0.9DC1 + 0.9DC2 + 0.65DW + 1.75(LL+IM+)	0.9(0.16 ksi) + 0.9(-0.11 ksi) + 0.65(-0.26 ksi) + 1.75(-12.51 ksi) =	-22.02	ksi

Positive LL+IM, DW neglected

1.25DC1 + 1.25DC2 + 1.75(LL+IM+)	1.25(0.16 ksi) + 1.25(-0.11 ksi) + 1.75(-12.51 ksi) =	-21.83	ksi
0.9DC1 + 0.9DC2 + 1.75(LL+IM+)	0.9(0.16 ksi) + 0.9(-0.11 ksi) + 1.75(-12.51 ksi) =	-21.85	ksi

Negative LL+IM, DW included

1.25DC1 + 1.25DC2 + 1.5DW + 1.75(LL+IM-)	1.25(0.16 ksi) + 1.25(-0.11 ksi) + 1.5(-0.26 ksi) + 1.75(11.57 ksi) =	19.92	ksi
0.9DC1 + 0.9DC2 + 1.5DW + 1.75(LL+IM-)	0.9(0.16 ksi) + 0.9(-0.11 ksi) + 1.5(-0.26 ksi) + 1.75(11.57 ksi) =	19.9	ksi
1.25DC1 + 1.25DC2 + 0.65DW + 1.75(LL+IM-)	1.25(0.16 ksi) + 1.25(-0.11 ksi) + 0.65(-0.26 ksi) + 1.75(11.57 ksi) =	20.14	ksi

0.9DC1 + 0.9DC2 + 0.65DW + 1.75(LL+IM-)	0.9(0.16 ksi) + 0.9(-0.11 ksi) + 0.65(-0.26 ksi) + 1.75(11.57 ksi) =	20.12	ksi

Negative LL+IM, DW neglected

1.25DC1 + 1.25DC2 + 1.75(LL+IM-)	1.25(0.16 ksi) + 1.25(-0.11 ksi) + 1.75(11.57 ksi) =	20.31	ksi
0.9DC1 + 0.9DC2 + 1.75(LL+IM-)	0.9(0.16 ksi) + 0.9(-0.11 ksi) + 1.75(11.57 ksi) =	20.29	ksi

Constructability

1.25DC1 + 1.75(LL+IMConst)	1.25(0.18 ksi) + 1.75(0.08 ksi) =	0.37	ksi

Service II Stresses:

	positive moment	negative moment
DC1	non-composite	non-composite
DC2	composite, n=27	composite, n=27
DW	composite, n=27	composite, n=27
LL+IM	composite, n=9	composite, n=9

Positive LL+IM, DW included

1.00DC1 + 1.00DC2 + 1.00DW + 1.30(LL+IM+)	1.00(0.16 ksi) + 1.00(-0.12 ksi) + 1.00(-0.26 ksi) + 1.30(-12.51 ksi) =	-16.47	ksi

Positive LL+IM, DW neglected

1.00DC1 + 1.00DC2 + 1.30(LL+IM+)	1.00(0.16 ksi) + 1.00(-0.12 ksi) + 1.30(-12.51 ksi) =	-16.21	ksi

Negative LL+IM, DW included

1.00DC1 + 1.00DC2 + 1.00DW + 1.30(LL+IM-)	1.00(0.16 ksi) + 1.00(-0.12 ksi) + 1.00(-0.26 ksi) + 1.30(9.4 ksi) =	12.01	ksi

Negative LL+IM, DW neglected

1.00DC1 + 1.00DC2 + 1.30(LL+IM-)	1.00(0.16 ksi) + 1.00(-0.12 ksi) + 1.30(9.4 ksi) =	12.27	ksi

Fatigue I Stresses:

Uncracked sections may be used for Fatigue I and II loading. See 6.6.1.2.1

	positive moment	negative moment
LL+IM	composite, n=9	composite, n=9

Positive LL+IM

1.50(LL+IM+)	1.50(-4.02 ksi) =	-6.03	ksi

Negative LL+IM

1.50(LL+IM-)	1.50(3.02 ksi) =	4.53	ksi

Fatigue II Stresses:

Positive LL+IM

0.75(LL+IM+)	0.75(-4.02 ksi) =	-3.02	ksi

Negative LL+IM

0.75(LL+IM-)	0.75(3.02 ksi) =	2.27	ksi

RESULTS:

	Section 1 Top Flange max	Section 2 Top Flange max	Section 1 Top Flange min	Section 2 Top Flange min	Section 1 Bottom Flange max	Section 2 Bottom Flange max	Section 1 Bottom Flange min	Section 2 Bottom Flange min	
STRENGTH I	2.19	2.5	-14.43	-7.14	20.31	7.54	-22.24	-8.82	ksi
SERVICE II	1.57	1.85	-1.34	-1.41	12.27	5.48	-16.47	-7.38	ksi
FATIGUE I	0.6	0.68	-0.45	-0.51	4.53	1.8	-6.03	-2.39	ksi
FATIGUE II	0.3	0.34	-0.23	-0.26	2.27	0.9	-3.02	-1.19	ksi
CONSTRUCTION	-0.41	0.2			0.37	-0.18			ksi

Note that construction loading clearly does not control the design of the splice.

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 Area, A_e (6.13.6.1.4c)

As this splice is in an area of stress reversal, both flanges can be in either tension or compression.

<u>Assuming top flange in tension:</u>

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

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 = $W_n t$. 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}

8.31 in.^{2} controls.

<u>Assuming top flange in compression:</u>

A_e = A_g = 9 in.^{2}

<u>Assuming bottom flange in tension:</u>

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

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}

9.70 in.^{2} controls.

Assuming bottom flange in compression:

A_e = A_g = 10.5 in.^{2}

Determine Strength I Controlling and Non-controlling Flange Stresses

	Applied		Capacity		abs(Applied/Capacity)
Max Tension Top Flange Section 1	-14.43	ksi	50	ksi	0.2886
Max Compression Top Flange Section 1	2.19	ksi	50	ksi	0.0438
Max Tension Bottom Flange Section 1	-22.24	ksi	50	ksi	0.4448
Max Compression Bottom Flange Section 1	20.31	ksi	50	ksi	0.4062

Therefore, for positive flexure, the controlling flange stress is -22.24 ksi (bottom flange in tension) and the non-controlling flange stress is 2.19 ksi (top flange in compression). These stresses result from Strength I loading with a DC factor of 0.9, a DW factor of 1.5, and positive live loading.

For negative flexure, the controlling flange stress is 20.31 ksi (bottom flange in compression) and the non-controlling flange stress is -14.43 ksi (top flange in tension). These stresses result from Strength I loading with a DC factor of 1.25, DW neglected, and negative live loading.

Calculate Strength I Flange Design Forces (6.13.6.1.4c)

Positive Flexure

Controlling Flange Design Force, P_{cf}

$$F_{cf} = \left(\frac{\left| \frac{-22.24\ \text{ksi}}{1.0} \right| + (1.00)(1.00)(50\ \text{ksi})}{2} \right) = 36.12\ \text{ksi} < 0.75(1.00)(1.00)(50\ \text{ksi})$$

$= 37.5$ ksi $\therefore F_{cf} = 37.5$ ksi (-) (Eq. 6.13.6.1.4c-1)

$P_{cf} =$ (37.5 ksi)(9.70 in.^{2}) = 363.75 k (-)

Non-controlling Flange Design Force, P_{ncf}

$$R_{cf} = \left| \frac{37.5\ \text{ksi}}{-22.24\ \text{ksi}} \right| = 1.69$$

$$F_{ncf} = (1.69)\left| \frac{2.19\ \text{ksi}}{1.0} \right| = 3.70\ \text{ksi} < 0.75(1.00)(1.00)(50\ \text{ksi}) = 37.5\ \text{ksi}$$

(Eq. 6.13.6.1.4c-3)

$\therefore F_{ncf} =$ 37.5 ksi (+)

$P_{ncf} =$ (37.5 ksi)(9.0 in.^{2}) = 337.5 k (+)

Negative Flexure

Controlling Flange Design Force, P_{cf}

$$F_{cf} = \left(\frac{\left| \frac{20.31\ \text{ksi}}{1.0} \right| + (1.00)(1.00)(50\ \text{ksi})}{2} \right) = 35.16\ \text{ksi} < 0.75(1.00)(1.00)(50\ \text{ksi})$$

$= 37.5$ ksi $\therefore F_{cf} = 37.5$ ksi (+) (Eq. 6.13.6.1.4c-1)

$P_{cf} =$ (37.5 ksi)(10.5 in.^{2}) = 393.75 k (+)

Non-controlling Flange Design Force, P_{ncf}

$$R_{cf} = \left|\frac{-37.5\,\text{ksi}}{20.31\,\text{ksi}}\right| = 1.85$$

$$F_{ncf} = (1.85)\left|\frac{-14.43\,\text{ksi}}{1.0}\right| = 26.7\text{ ksi} < 0.75(1.00)(1.00)(50\text{ ksi}) = 37.5\text{ ksi}$$

(Eq. 6.13.6.1.4c-3)

$\therefore F_{cf} = 37.5$ ksi (-)

$P_{ncf} = (37.5\text{ ksi})(8.31\text{ in.}^2) = 311.63$ k (-)

Check Flange Splice Plate Strength

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

Positive Flexure

The bottom plate is in tension in positive flexure.

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

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

$P_r = (0.95)(50\text{ ksi})(12.25\text{ in.}^2) = 581.88\text{ k} > 363.75\text{ k}$ 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 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)

$P_r = (0.80)(65\text{ ksi})(10.14\text{ in.}^2)(1.0)(1.0) = 527.28\text{ k} > 363.75\text{ k}$ O.K.

Negative Flexure

The top plate is in tension in negative flexure. The top splice plates are the same size and have the same bolt layout as the bottom splice plates and therefore will have the same capacity, 527.28 k. The applied load is 311.63 k.

527.28 k > 311.63 k O.K.

<u>Check Compression in Splice Plates</u>

Positive Flexure

The top plate is in compression in positive flexure.

<u>Check Compression in Splice Plates</u>

$P_r = \phi_c F_y A_s > P_{cf}$ or P_{ncf} as applicable (Eq. 6.13.6.1.4c-4)

Where:

ϕ_c = 0.90 (6.5.4.2)

F_y = 50 ksi

A_s = 12.25 in.^{2} (see above)

$P_r = (0.90)(50\text{ ksi})(12.25\text{ in.}^2) = 551.25\text{ k} > 337.5\text{ k}$ O.K.

Negative Flexure

The bottom plate is in compression in negative flexure. The bottom splice plates have the same capacity as the top plates, 551.25 k. The applied load to the bottom plates is P_{cf}, or 393.75 k.

551.25 k > 393.75 k O.K.

Check Flange Splice Plate Fatigue *(6.6.1.2.2)*

The bottom flange has higher fatigue stresses. Check bottom flange for fatigue, and if OK, assume top flange also OK.

Use Fatigue Category B for bolted splices. (Table 6.6.1.2.3-1)

The following design criteria shall be met:

$\gamma(\Delta f\ ) \le (\Delta F)_n$ (Eq. 6.6.1.2.2-1)

Where:

γ = load factor, taken as 1.5 for Fatigue I loading and 0.75 for Fatigue II loading. Determine whether Fatigue I or Fatigue II load factors are appropriate:

$$ADTT = \left(\left(600\,\frac{\text{trucks}}{\text{day}} - 300\,\frac{\text{trucks}}{\text{day}}\right)\left(\frac{75\text{ years}}{20\text{ years}}\right) + 300\,\frac{\text{trucks}}{\text{day}}\right)(0.5)$$

$= 713$ trucks/day

p = 1.0 for one lane (not counting shoulders) (Table 3.6.1.4.2-1)

$(ADTT)_{SL}$ = 1.0(713 trucks/day) = 713 trucks/day

The limit for infinite life for Fatigue Category B is 860 trucks/day (see Table 6.6.1.2.3-2). Therefore, use finite life (Fatigue II) loading.

$$(\Delta f_r) = \frac{(f_{fat}^{+})A_e - (f_{fat}^{-})A_e}{A_{PL}}$$

Where:

γf_{fat}^{+} = -3.02 ksi

γf_{fat}^{-} = 2.27 ksi

A_e = As these loads occur on the bottom flange, use the bottom flange effective areas

= 9.70 in.^{2} for tension loads and 10.5 in.^{2} for compression loads

A_{PL} = (12 in.)(0.5 in.) + 2(5 in.)(0.625 in.) = 12.25 in.^{2}

$$\gamma(\Delta f_r) = \frac{(-3.02\text{ ksi})(9.70\text{ in.}^2) - (2.27\text{ ksi})(10.5\text{ in.}^2)}{12.25\text{ in.}^2}$$

= -4.34 ksi, or a range of 4.34 ksi

$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}}$ for Fatigue II load combination (Eq. 6.6.1.2.5-2)

Where:

N = $(365)(75)n(ADTT)_{SL}$ (Eq. 6.6.1.2.5-3)

n = 1 cycle/truck (Table 6.6.1.2.5-2)

$$(ADTT)_{37.5,\ SL} = \left(\left(600\frac{\text{trucks}}{\text{day}} - 300\frac{\text{trucks}}{\text{day}}\right)\left(\frac{37.5\text{ years}}{20\text{ years}}\right) + 300\frac{\text{trucks}}{\text{day}}\right)(0.5)$$

= 431 trucks/day

$$N = \left(\frac{365\text{ days}}{\text{year}}\right)(75\text{ years})\left(\frac{1\text{ cycle}}{\text{truck}}\right)\left(\frac{431\text{ trucks}}{\text{day}}\right)$$

= 11.8×10^6 cycles

A = 120.0×10^8 ksi^3 for fatigue category B (Table 6.6.1.2.5-1)

$$(\Delta F)_n = \left(\frac{120 \times 10^8\text{ cycles}}{11.8 \times 10^6\text{ cycles}}\right)^{\frac{1}{3}} = 10.06\text{ ksi}$$

$\gamma(\Delta f_r)$ = 4.34 ksi < 10.06 ksi O.K.

Calculate Reduction Factor for Fillers (6.13.6.1.5)

<u>Filler Plate Reduction, R</u>

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_f = (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_f = (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

<u>Calculate Factored Shear Resistance for Bolts, R_r</u> (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 = $0.38A_bF_{ub}N_s$(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 = 0.38(0.60 in.^{2})(120 ksi)(2) = 54.7 k / bolt

R_r = (0.80)(54.7 k / bolt)(0.61) = 26.7 k / bolt

$$P_r = \left[\frac{393.75\text{ k}}{16\text{ bolts}}\right] = 24.6 \text{ kips per bolt} < 26.7 \text{ kips per bolt}$$ 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:

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

K_h = 1 (Table 6.13.2.8-2)

K_s = 0.33 (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)(0.33)(2)(39 k / bolt) = 25.7 k / bolt

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

Where:

$P_{tot\text{-}slip} = F_s A_g$

$$F_s = \left|\frac{f_s}{R_h}\right|$$ (Eq. 6.13.6.1.4c-5)

Where:

f_s = -16.47 ksi

R_h = 1.0 (6.10.1.10.1)

$$F_s = \left|\frac{-16.47\text{ ksi}}{1.0}\right| = 16.47\text{ ksi}$$

A_g = 10.5 in.^{2}

$P_{tot\text{-}slip}$ = (16.47 ksi)(10.5 in.^{2}) = 172.94 k

N_b = 16 bolts

$$P_{slip} = \frac{172.94\text{ k}}{16\text{ bolts}} = 10.8\text{ k / bolt} < 25.7\text{ k / bolt}$$ 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 $P_{brg} \leq R_r$ for both flange splices for positive and negative flexure.

Where:

$$P_{brg} = \frac{P_{cf}\text{ or }P_{ncf}}{N_b} = \frac{393.75\text{ k}}{16\text{ bolts}} = 24.6\text{ k / bolt}$$

$R_r = \phi_{bb}R_n$

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 $x_{end} < 2.0d$:

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

Where:

x_{clear} = $\sqrt{(3\text{ in.})^2 + (2\text{ in.})^2}$ - 0.9375 in. = 2.67 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> 24.6 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

Vertical Bolt Spacing

Try 12 bolt lines spaced as shown in Figure 1.

Horizontal Bolt Spacing

Try two bolt lines spaced as shown in Figure 1.

Determine Factored Shears

Strength I Shears:

Positive LL+IM, DW included			
1.25DC1 + 1.25DC2 + 1.5DW + 1.75(LL+IM+)	1.25(-29.7 k) + 1.25(-4.9 k) + 1.5(-11.9 k) + 1.75(12 k) =	-40.1	k
0.9DC1 + 0.9DC2 + 1.5DW + 1.75(LL+IM+)	0.9(-29.7 k) + 0.9(-4.9 k) + 1.5(-11.9 k) + 1.75(12 k) =	-27.99	k
1.25DC1 + 1.25DC2 + 0.65DW + 1.75(LL+IM+)	1.25(-29.7 k) + 1.25(-4.9 k) + 0.65(-11.9 k) + 1.75(12 k) =	-29.99	k
0.9DC1 + 0.9DC2 + 0.65DW + 1.75(LL+IM+)	0.9(-29.7 k) + 0.9(-4.9 k) + 0.65(-11.9 k) + 1.75(12 k) =	-17.88	k
Positive LL+IM, DW neglected			
1.25DC1 + 1.25DC2 + 1.75(LL+IM+)	1.25(-29.7 k) + 1.25(-4.9 k) + 1.75(12 k) =	-22.25	k
0.9DC1 + 0.9DC2 + 1.75(LL+IM+)	0.9(-29.7 k) + 0.9(-4.9 k) + 1.75(12 k) =	-10.14	k
Negative LL+IM, DW included			
1.25DC1 + 1.25DC2 + 1.5DW + 1.75(LL+IM-)	1.25(-29.7 k) + 1.25(-4.9 k) + 1.5(-11.9 k) + 1.75(-68.4 k) =	-180.8	k
0.9DC1 + 0.9DC2 + 1.5DW + 1.75(LL+IM-)	0.9(-29.7 k) + 0.9(-4.9 k) + 1.5(-11.9 k) + 1.75(-68.4 k) =	-168.69	k
1.25DC1 + 1.25DC2 + 0.65DW + 1.75(LL+IM-)	1.25(-29.7 k) + 1.25(-4.9 k) + 0.65(-11.9 k) + 1.75(-68.4 k) =	-170.69	k
0.9DC1 + 0.9DC2 + 0.65DW + 1.75(LL+IM-)	0.9(-29.7 k) + 0.9(-4.9 k) + 0.65(-11.9 k) + 1.75(-68.4 k) =	-158.58	k
Negative LL+IM, DW neglected			
1.25DC1 + 1.25DC2 + 1.75(LL+IM-)	1.25(-29.7 k) + 1.25(-4.9 k) + 1.75(-68.4 k) =	-162.95	k
0.9DC1 + 0.9DC2 + 1.75(LL+IM-)	0.9(-29.7 k) + 0.9(-4.9 k) + 1.75(-68.4 k) =	-150.84	k
Constructability			
1.25DC1const + 1.75(LL+IMConst)	1.25(-31.3 k) + 1.75(6.5 k) =	-27.75	k

Service II Shears:

Positive LL+IM, DW included			
1.00DC1 + 1.00DC2 + 1.00DW + 1.30(LL+IM+)	1.00(-29.7 k) + 1.00(-4.9 k) + 1.00(-11.9 k) + 1.30(12 k) =	-30.9	k
Positive LL+IM, DW neglected			
1.00DC1 + 1.00DC2 + 1.30(LL+IM+)	1.00(-29.7 k) + 1.00(-4.9 k) + 1.30(12 k) =	-19	k
Negative LL+IM, DW included			
1.00DC1 + 1.00DC2 + 1.00DW + 1.30(LL+IM-)	1.00(-29.7 k) + 1.00(-4.9 k) + 1.00(-11.9 k) + 1.30(-68.4 k) =	-135.42	k
Negative LL+IM, DW neglected			
1.00DC1 + 1.00DC2 + 1.30(LL+IM-)	1.00(-29.7 k) + 1.00(-4.9 k) + 1.30(-68.4 k) =	-123.52	k

Fatigue I Shears:

Positive LL+IM			
1.50(LL+IM+)	1.50(4.2 k) =	6.3	k
Negative LL+IM			
1.50(LL+IM-)	1.50(-26.8 k) =	-40.2	k

Fatigue II Shears:

Positive LL+IM			
0.75(LL+IM+)	0.75(4.2 k) =	3.15	k
Negative LL+IM			
0.75(LL+IM-)	0.75(-26.8 k) =	-20.1	k

Check Web Splice Plate Strength

<u>Calculate Nominal Web Shear Resistance</u>

$V_u \leq \phi_v V_n$ (Eq. 6.10.9.1-1)

Where:

$\phi_v = 1.00$ (6.5.4.2)

$V_u = 180.89 \text{ k}$

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

Where:

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

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

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

$$\therefore C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right)$$ (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)

$= 0.58(50 \text{ ksi})(42 \text{ in.})(0.4375 \text{ in.})$

$= 532.9 \text{ k}$

$CV_p = 0.49(532.9 \text{ k}) = 261.1 \text{ k}$

Calculate Strength I Flexural Stress for Web Splice Plates

$$f_{STRENGTH\ I} = \frac{M_{uv} + M_{uw}}{S_{PL}} + \frac{|H_{uw}|}{A_{PL}}$$

Where:

$S_{PL} = \dfrac{2(0.375\text{ in.})(40\text{ in.})^2}{6} = 200\text{ in.}^3$

$A_{PL} = 2(0.375\text{ in.})(40\text{ in.}) = 30\text{ in.}^2$

Calculate M_{uv}, the Moment due to Shear Eccentricity

$M_{uv} = V_{uw}e$

Where:

$e = 3.5$ in.

$V_{uw} = 1.5V_u$ if $V_u < 0.5\phi_v V_n$ (Eq. 6.13.6.1.4b-1)

$= \dfrac{(V_u + \phi_v V_n)}{2}$ otherwise (Eq. 6.13.6.1.4b-2)

Where:

$V_u = 180.8$ k

$\phi_v = 1.00$ (6.5.4.2)

$0.5\phi_v V_n = 0.5(1.00)(261.1\text{ k}) = 130.55\text{ k} < 180.8\text{ k}$

$\therefore V_{uw} = \dfrac{(180.8\text{ k} + 1.00(261.1\text{ k}))}{2} = 221.0\text{ k}$

$M_{uv} = (221.0\text{ k})(3.5\text{ in.}) = 773.5\text{ k-in.}$

Calculate M_{uw}, the Moment Resisted by the Web (Assuming a Symmetric Stress Diagram)

M_{uw} must be calculated for both positive and negative flexure to determine which case controls.

$$M_{uw} = \frac{t_w D^2}{12}\left|R_h F_{cf} - R_{cf} f_{ncf}\right| \qquad \text{(C6.13.6.1.4b-1)}$$

Where for positive flexure:

t_w = 0.4375 in.

D = 42 in.

R_h = 1.0 (6.10.1.10.1)

F_{cf} = -37.5 ksi

f_{ncf} = 2.19 ksi

f_{cf} = -22.24 ksi

$$R_{cf} = \left|\frac{37.5\text{ ksi}}{-22.24\text{ ksi}}\right| = 1.69$$

$$M_{uw} = \frac{(0.4375\text{ in.})(42\text{ in.})^2}{12}\left|(1.0)(-37.5\text{ ksi}) - (1.69)(2.19\text{ ksi})\right|$$

$$= 2649.75\text{ k-in.}$$

For negative flexure:

t_w = 0.4375 in.

D = 42 in.

R_h = 1.0 (6.10.1.10.1)

F_{cf} = 37.5 ksi

f_{ncf} = -14.43 ksi

f_{cf} = 20.31 ksi

$$R_{cf} = \left|\frac{37.5\text{ ksi}}{20.31\text{ ksi}}\right| = 1.85$$

$$M_{uw} = \frac{(0.4375\text{ in.})(42\text{ in.})^2}{12}\left|(1.0)(37.5\text{ ksi}) - (1.85)(-14.43\text{ ksi})\right|$$

$$= 4128.57\text{ k-in.}$$

Calculate H_{uw}, the Additional Force Due to Non-Symmetry of Stress Diagram

To be consistent with M_{uw}, H_{uw} also must be calculated to determine the controlling load condition.

$$H_{uw} = \frac{t_w D}{2}(R_h F_{cf} + R_{cf} f_{ncf}) \quad \text{(C6.13.6.1.4b-2)}$$

Where for positive flexure:

t_w = 0.4375 in.

D = 42 in.

R_h = 1.0 (6.10.1.10.1)

F_{cf} = -37.5 ksi

f_{ncf} = 2.19 ksi

f_{cf} = -22.24 ksi

$$R_{cf} = \left|\frac{37.5\text{ ksi}}{-22.24\text{ ksi}}\right| = 1.69$$

$$H_{uw} = \frac{(0.4375\text{ in.})(42\text{ in.})}{2}((1.0)(-37.5\text{ ksi}) + (1.69)(2.19\text{ ksi}))$$

$$= -310.5\text{ k}$$

For negative flexure:

t_w = 0.4375 in.

D = 42 in.

R_h = 1.0 (6.10.1.10.1)

F_{cf} = 37.5 ksi

f_{ncf} = -14.43 ksi

f_{cf} = 20.31 ksi

$$R_{cf} = \left|\frac{37.5\text{ ksi}}{20.31\text{ ksi}}\right| = 1.85$$

$$H_{uw} = \frac{(0.4375\text{ in.})(42\text{ in.})}{2}((1.0)(37.5\text{ ksi}) + (1.85)(-14.43\text{ ksi}))$$

$$= 99.3\text{ k}$$

For positive flexure:

$$f_{STRENGTH\ I} = \frac{773.5\text{ k}-\text{in.} + 2649.75\text{ k}-\text{in.}}{200\text{ in.}^3} + \frac{|-310.5\text{ k}|}{30\text{ in.}^2}$$

$$= 27.5\text{ ksi}$$

For negative flexure:

$$f_{STRENGTH\ I} = \frac{773.5\ \text{k} - \text{in.} + 4128.57\ \text{k} - \text{in.}}{200\ \text{in.}^3} + \frac{|99.3\ \text{k}|}{30\ \text{in.}^2}$$

$$= 27.8\ \text{ksi}$$

27.8 ksi controls.

<u>Compare Strength I Flexural Stress with $\phi_f F_y$</u> (6.13.6.1.4b)

$f_{STRENGTH\ I} \le \phi_f F_y$

Where:

$f_{STRENGTH\ I}$ = 27.8 ksi

ϕ_f = 1.0 (6.5.4.2)

F_y = 50 ksi

27.8 ksi < 50 ksi O.K.

Check Web Splice Plate Shear Capacity *(6.13.5.3)*

$V_{uw} \le R_r$

<u>Calculate Factored Shear Resistance for Yielding of Gross Web Splice Section</u>

$R_r = \phi_v 0.58 F_y A_{vg}$ (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 > 221.0 k O.K.

Calculate Factored Shear Resistance for Fracture of Net Web Splice Section

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

Where:

$R_p = 1.0$

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

$=$ 21.56 in.^{2}

$F_u =$ 65 ksi

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

$R_r =$ (0.8)(0.58)(1.0)(65 ksi)(21.56 in.)

$=$ 650.25 k > 221.0 k O.K.

Check Web Splice Plate Fatigue

Calculate Fatigue Shear Forces, V_{rw}

From flange design, Fatigue II load case controls.

$V_{rw}^{+} =$ 3.15 k

$V_{rw}^{-} =$ -20.1 k

Calculate Fatigue Moment Due to Shear Eccentricity, M_{rv} (C6.13.6.1.4b)

$M_{rv} = [(V_{rw}^{+})-(V_{rw}^{-})]e$, where all variables are as calculated above

$=$ [(3.15 k – (-20.1 k)](3.5 in.)

$=$ 81.4 k-in.

Calculate Fatigue Flexural Moment, M_{rw}

$$M_{rw} = \frac{t_w D^2}{12}[f_{tw} - f_{bw}]$$ (Modified Eq. C6.13.6.1.4b-1)

For simplicity, use the stresses at mid-thickness of flange instead of the stresses at the inside faces of the flanges. Again, Fatigue II load case controls.

For positive flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = 0.3 ksi

f_{bw} = -3.02 ksi

$$M_{rw}^{+} = \frac{(0.4375\text{ in.})(42\text{ in.})^2}{12}[0.3\text{ ksi} - (-3.02\text{ ksi})]$$

$$= 213.5\text{ k-in.}$$

For negative flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = -0.23 ksi

f_{bw} = 2.27 ksi

$$M_{rw}^{-} = \frac{(0.4375\text{ in.})(42\text{ in.})^2}{12}[-0.23\text{ ksi} - (2.27\text{ ksi})]$$

$$= -160.8\text{ k-in.}$$

<u>Calculate Fatigue Total Moment Range, $M_{r\text{-total range}}$</u>

$$M_{r\text{-total range}} = M_{rv} + (M_{rw}^{+} - M_{rw}^{-})$$

$$= 81.4\text{ k-in.} + (213.5\text{ k-in.} - (-160.8\text{ k-in.}))$$

$$= 455.7\text{ k-in.}$$

<u>Calculate Fatigue Design Horizontal Force Resultant</u>

The horizontal force resultant, H_{rw}, shall be calculated for both positive and negative flexure.

$$H_{rw} = \frac{t_w D}{2}(f_{tw} + f_{bw})$$ (Modified Eq. C6.13.6.1.4b-2)

For positive flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = 0.3 ksi

f_{bw} = -3.02 ksi

$$H_{rw}^{+} = \frac{(0.4375\text{ in.})(42\text{ in.})}{2}[0.3\text{ ksi} + (-3.02\text{ ksi})]$$

= -24.99 k

For negative flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = -0.23 ksi

f_{bw} = 2.27 ksi

$$H_{rw}^{-} = \frac{(0.4375\text{ in.})(42\text{ in.})}{2}[-0.23\text{ ksi} + (2.27\text{ ksi})]$$

= 18.74 k

Calculate Horizontal Force Resultant Range, $H_{rw\text{-}range}$

$H_{rw\text{-}range} = H_{rw}^{+} - H_{rw}^{-}$

= -24.99 k – 18.74 k

= -43.73 k

Calculate Factored Fatigue Stress Range, $\gamma(\Delta f)$, for Web Splice Plates

$$\gamma(\Delta f) = \frac{M_{r-totalrange}}{S_{PL}} + \frac{H_{rw-range}}{A_{PL}}$$

Where:

$$S_{PL} = \frac{2(0.375\text{ in.})(40\text{ in.})^2}{6} = 200\text{ in.}^3$$

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

$$\gamma(\Delta f) = \frac{455.7\text{ k} - \text{in.}}{200\text{ in.}^3} + \frac{-43.73\text{ k}}{30\text{ in.}^2}$$

= 0.82 ksi

Check Fatigue Detail Design Criteria (6.6.1.2.2)

From flange design, the allowable stress range for fatigue is 10.06 ksi.

0.82 ksi < 10.06 ksi 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)

$R_n = 0.38 A_b F_{ub} N_s$ (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)

$N_s = 2$

$R_n = 0.38(0.60 \text{ in.}^2)(120 \text{ ksi})(2)$

$= 54.72$ k

Calculate the Polar Moment of Inertia

$$\sum y^2 = 2(2)(17.875 \text{ in.})^2+2(2)(14.625 \text{ in.})^2+2(2)(11.375 \text{ in.})^2+2(2)(8.125 \text{ in.})^2 +2(2)(4.875 \text{ in.})^2+2(2)(1.625 \text{ in.})^2$$

$$= 3020.88 \text{ in.}^2$$

$$\sum x^2 = 12(2)(1.5 \text{ in.})^2 = 54.00 \text{ in.}^2$$

$$I_p = 3020.88 \text{ in.}^2+54.00 \text{ in.}^2 = 3074.88 \text{ in.}^2$$

Calculate Strength I Resultant Shear for Bolts, P_r

From plate check, positive moments control the Strength I design.

$$P_r = \sqrt{(P_S + P_{MV})^2 + (P_H + P_{MH})^2}$$

Where:

$$P_S = \frac{V_{uw}}{N_b} = \frac{221.0\text{ k}}{24} = 9.21\text{ kips per bolt}$$

$$P_H = \left|\frac{H_{uw}}{N_b}\right| = \left|\frac{-310.5\text{ k}}{24}\right| = 12.94\text{ kips per bolt}$$

$$P_{MH} = \frac{(M_{uv} + M_{uw})y}{I_p} = \frac{(773.5\text{ k}-\text{in.} + 2649.75\text{ k}-\text{in.})(17.875\text{ in.})}{3074.88\text{ in.}^2} = 19.90\text{ k}$$

$$P_{MV} = \frac{(M_{uv} + M_{uw})x}{I_p} = \frac{(773.5\text{ k}-\text{in.} + 2649.75\text{ k}-\text{in.})(1.5\text{ in.})}{3074.88\text{ in.}^2} = 1.67\text{ k}$$

$$P_r = \sqrt{(9.21\text{k} + 1.67\text{ k})^2 + (12.94\text{ k} + 19.90\text{ k})^2} = 34.60\text{ k}$$

34.60 k < 54.72 k O.K.

Check Web Splice Bolt Slip

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

V_{ow} = 135.42 k (see above)

<u>Calculate Service II Flexural Moment</u> (C6.13.6.1.4b)

M_{ov} = $V_{ow}e$ = (135.42 k)(3.5 in.) = 473.97 k-in.

$$M_{ow} = \frac{t_w D^2}{12}[f_{tw} - f_{bw}]$$

For simplicity, use the stresses at mid-thickness of flange instead of the stresses at the inside faces of the flanges.

For positive flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = 1.57 ksi

f_{bw} = -16.47 ksi

$$M^{+}_{ow} = \frac{(0.4375\text{ in.})(42\text{ in.})^2}{12}[1.57\text{ ksi} - (-16.47\text{ ksi})]$$

$$= 1160.2\text{ k-in.}$$

For negative flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = -1.34 ksi

f_{bw} = 12.27 ksi

$$M^{-}_{ow} = \frac{(0.4375\text{ in.})(42\text{ in.})^2}{12}[-1.34\text{ ksi} - (12.27\text{ ksi})]$$

$$= -875.3\text{ k-in.}$$

<u>Calculate Service II Design Horizontal Force Resultant</u>

$$H_{ow} = \frac{t_w D}{2}(f_{tw} + f_{bw})$$ (Modified Eq. C6.13.6.1.4b-2)

For positive flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = 1.57 ksi

f_{bw} = -16.47 ksi

$$H^{+}_{ow} = \frac{(0.4375\text{ in.})(42\text{ in.})}{2}[1.57\text{ ksi} + (-16.47\text{ ksi})]$$

$$= -136.89\text{ k}$$

For negative flexure:

t_w = 0.4375 in.

D = 42 in.

f_{tw} = -1.34 ksi

f_{bw} = 12.27 ksi

$$H_{ow}^{-} = \frac{(0.4375\text{ in.})(42\text{ in.})}{2}[-1.34\text{ ksi} + (12.27\text{ ksi})]$$

$$= 100.42\text{ k}$$

<u>Calculate Service II Resultant Shear for Bolts</u>

$$P_{or} = \sqrt{(P_{os} + P_{oMV})^2 + (P_{oH} + P_{oMH})^2}$$

For positive flexure:

$$P_{oS} = \frac{V_{ow}}{N_b} = \frac{135.42\text{ k}}{24} = 5.64 \text{ kips per bolt}$$

$$P_{oH} = \left|\frac{H_{ow}}{N_b}\right| = \left|\frac{-136.89\text{ k}}{24}\right| = 5.7 \text{ kips per bolt}$$

$$P_{MH} = \frac{(M_{ov} + M_{ow})y}{I_p} = \frac{(473.97\text{ k}-\text{in.} + 1160.2\text{ k}-\text{in.})(17.875\text{ in.})}{3074.88\text{ in.}^2} = 9.50\text{ k}$$

$$P_{MV} = \frac{(M_{ov} + M_{ow})x}{I_p} = \frac{(473.97\text{ k}-\text{in.} + 1160.2\text{ k}-\text{in.})(1.5\text{ in.})}{3074.88\text{ in.}^2} = 0.80\text{ k}$$

$$P_r = \sqrt{(5.64\text{ k} + 0.80\text{ k})^2 + (5.70\text{ k} + 9.50\text{ k})^2} = 16.51\text{ k}$$

For negative flexure:

$$P_{oS} = \frac{V_{ow}}{N_b} = \frac{135.42\text{ k}}{24} = 5.64 \text{ kips per bolt}$$

$$P_{oH} = \left|\frac{H_{ow}}{N_b}\right| = \left|\frac{100.42\text{ k}}{24}\right| = 4.18 \text{ kips per bolt}$$

$$P_{MH} = \frac{(M_{ov} + M_{ow})y}{I_p} = \frac{(473.97\text{ k}-\text{in.} + (-875.3\text{ k}-\text{in.}))(17.875\text{ in.})}{3074.88\text{ in.}^2} = -2.33\text{ k}$$

$$P_{MV} = \frac{(M_{ov} + M_{ow})x}{I_p} = \frac{(473.97\text{ k}-\text{in.} + (-875.3\text{ k}-\text{in.}))(1.5\text{ in.})}{3074.88\text{ in.}^2} = -0.20\text{ k}$$

$$P_r = \sqrt{(5.64\text{ k} + (-0.20\text{ k}))^2 + (4.18\text{ k} + (-2.33\text{ k}))^2} = 6.13\text{ k}$$

<u>Calculate Factored Slip Resistance for Bolts, R_r</u> (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 = $K_hK_sN_sP_t$ (kips)

K_h = 1.00 (Table 6.13.2.8-2)

K_s = 0.33 (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)(0.33)(2)(39 k)

= 25.74 k > 16.51 k O.K.

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} = (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 > 221.0 k O.K.

Check Extreme Bolt Bearing

The extreme corner of the splice plate 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. The top or bottom bolt adjacent to the end of beam only has a 1.875 in. distance from the center of the bolt to the end of the beam segment, 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.

$P_r \le R_r$

Where:

P_r = 34.6 k (See Strength I design calculations)

R_r = $\phi_{bb}R_n$

Where:

ϕ_{bb} = 0.80 (6.5.4.2)

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} = 2.31 in. or 2.06 in.

x_{end} = 1.875 in. – 0.5(0.9375 in.) = 1.41 in.

d = 0.875 in.

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

Where:

t = 0.4375 in. web thickness

F_u = 65 ksi (Table 6.4.1-1)

L_c = 1.41 in.

R_n = 1.2(1.41 in.)(0.4375 in.)(65 ksi)

= 48.11 k

R_r = (0.80)(48.11 k) = 38.5 k > 34.6 k O.K.

Check Web Splice Bolt Spacing

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