3.3.4 LRFD Composite Steel Beam Design for Straight Bridges

This design guide focuses on the LRFD design of steel beams made composite with the deck. A design procedure is presented first with an example given afterward for a two-span bridge. The procedure will explain determination of moments and capacities for one positive moment region and one negative moment region of the bridge.

For the purpose of this design guide and as general statement of IDOT policy, bridges with steel superstructures are designed to behave compositely in both positive and negative moment regions.

Article 6.10, the starting point for I-section flexural members, outlines the necessary limit state checks and their respective code references as follows:

1) Cross-Section Proportion Limits (6.10.2)
2) Constructibility (6.10.3)
3) Service Limit State (6.10.4)
4) Fatigue and Fracture Limit State (6.10.5)
5) Strength Limit State (6.10.6)

There are also several slab reinforcement requirements for negative moment regions, which will be outlined in this design guide. Composite wide flange and plate girder sections also require the design of diaphragms, stud shear connectors and splices (for typical continuous structures). Departmental diaphragm details and policies for bridges designed according to the LRFD and LFD Specifications can be found in Sections 3.3.22 and 3.3.23 of the Bridge Manual. Design and detailing policies for stud shear connectors for LRFD and LFD are covered in Section 3.3.9 of the Bridge Manual. Design Guide 3.3.9 addresses LRFD only. Splice design for LRFD and LFD is addressed in Section 3.3.21 of the Bridge Manual. Design Guide 3.3.21 covers LRFD only.

LRFD Composite Design Procedure, Equations, and Outline

Determine Trial Sections

Three important considerations (among others) for selecting trial beam sections are:

Section depth is normally dictated by the Type, Size, and Location Plan. As section depth affects roadway clearance, it should not be changed during the design process of a bridge unless there are extenuating circumstances.

Minimum dimensions for webs and flanges are given in Section 3.3.27 of the Bridge Manual. These minima are based on fabrication concerns. The cross-section proportion limits given in 6.10.2 of the LRFD Code should also be referenced when choosing a trial section.

When choosing between a wider, less thick flange and a narrower, thicker flange, note that the wider, less thick flange will have a higher lateral moment of inertia and will be more stable during construction. Also, if the flange thickness does not change by a large amount, flange bolt strength reduction factors will be reduced. Therefore, it is usually the better option to use wider, less thick flanges.

Determine Section Properties

<u>Determine Effective Flange Width</u> <u>(4.6.2.6)</u>

The effective flange width (in.) is taken as the tributary width perpendicular to the axis of the member. For non-flared bridges with equal beam spacings where the overhang width is less than half the beam spacing, the tributary width is equal to the beam spacing. For the atypical situation when the overhang is greater than half the beam spacing, the tributary width is equal to half the beam spacing plus the overhang width. For bridges with typical overhangs and beam spacings, interior beams will control the design of all beams interior and exterior. However, bridges with very large overhangs may have controlling exterior beams. Should the exterior beam control the design of the structure, all interior beams should be detailed to match the exterior beam. Do not design the exterior beams to be different from the interior beams, as this will affect future bridge widening projects.

Section Properties

For the calculation of moments, shears, and reactions, section properties for a composite section transformed by a value of n are used. Transformed sections are used for the calculation of all moments, shears, and reactions across the entire bridge, including those in the negative moment region. Do not use a cracked section or a noncomposite section to calculate moments in the negative moment region. This is advocated by the LRFD Code (6.10.1.5 and C6.10.1.5). For normal-weight concrete with a minimum 28-day compressive strength (f'_c) between 2.8 ksi and 3.6 ksi, the concrete section is transformed using a modular ratio of 9 (C6.10.1.1.1b).

Composite section properties for the calculation of member stresses are calculated using a variety of transformations and sections. Each limit state in the LRFD Code allows for different simplifications. These simplifications will be explained along with their applicable limit state later in this design guide.

Section weight is calculated using the full concrete section width. A 0.75 in. concrete fillet is typically included in the weight of dead load for plate girders (0.5 in. may be used for rolled sections) but not considered effective when calculating composite section properties. Typically, the weight of steel in the beam is multiplied by a detail factor between 1.1 and 1.2 to account for the weight of diaphragms, splice plates, and other attachments.

Calculate Moments and Shears

Using the dead loads, live loads, and load and load distribution factors, calculate moments and shears for the bridge. See Section 3.3.1 of the Bridge Manual for more information. Moments and shears have different distribution factors and differing load factors based upon the limit state being checked. This is explained in greater detail later in this design guide.

Distribution Factors *(4.6.2.2.2)*

Distribution factors should be checked for multi-lane loading in the final condition (g_m) and single lane loading to account for stage construction, if applicable (g_1). The skew correction factors for moment found in Table 4.6.2.2.2e-1 should not be applied. Unless overhangs exceed half the beam spacing or 3 ft. 8 in., the interior beam distribution should control the beam design and the exterior beam distribution factor need not be checked. For stage construction with single-lane loading, exterior beam distribution factors can become quite large and will appear to control the design of all of the beams for the structure. However, this condition is temporary and design of the entire structure for it is considered to be excessively conservative by the Department. See Bridge Manual section 3.3.1 for more details.

Distribution factors for shear and reactions should be adjusted by the skew correction factors found in Table 4.6.2.2.3c-1. These factors should be applied to the end beam shears and reactions at abutments or open joints.

The LRFD Code and Bridge Manual Chapter 3.3.1 provide a simplification for the final term in the distribution factor equations. These simplifications are found in Table 4.6.2.2.1-2, and may be applied. Note that these simplifications may be very conservative, especially for shallow girders, and may cause live loads up to 10% higher in some cases. They also may be slightly nonconservative for deep girders, especially if there is a deep girder on a shorter span. It is not IDOT policy that these simplifications be used, but they may be used to simplify calculations.

Moment

The distribution factor for moments for a single lane of traffic is calculated as:

$$g_1 = 0.06 + \left(\frac{S}{14.0}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \quad \text{(Table 4.6.2.2.2b-1)}$$

The distribution factor for multiple-lanes loaded, g_m, is calculated as:

$$g_m = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} \quad \text{(Table 4.6.2.2.2b-1)}$$

Where:

S = beam spacing (ft.)

L = span length (ft.)

t_s = slab thickness (in.)

K_g = longitudinal stiffness parameter, taken as $n\left(I + Ae_g^2\right)$ (4.6.2.2.1-1)

Where:

n = modular ratio = 9 for f'_c = 3.5 ksi (C6.10.1.1.1b)

I = moment of inertia of noncomposite beam (in.^{4}). For bridges that contain changes in section over the length of a span, this will be variable, as will A and e_g (see following). In these cases, K_g should be calculated separately for each section of the span and a weighted average of K_g should be used to calculate the final distribution factor for the span.

A = area of noncomposite beam (in.^{2})

e_g = distance between centers of gravity of noncomposite beam and slab (in.)

A simplification factor of 1.02 may be substituted for the final term in these equations, as shown in Table 4.6.2.2.1-2.

Moment (fatigue loading)

The distribution factor for calculation of moments for fatigue truck loading should be taken as the single-lane distribution factor with the multiple presence factor removed (C3.6.1.1.2).

$$g_1 \text{ (fatigue)} = \frac{g_1}{m}$$

Shear and Reaction

The distribution factors for shear and reaction are taken as:

$$g_1 = 0.36 + \frac{S}{25.0} \text{ for a single lane loaded} \quad \text{(Table 4.6.2.2.3a-1)}$$

$$g_m = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0} \text{ for multiple lanes loaded} \quad \text{(Table 4.6.2.2.3a-1)}$$

Skew correction $= 1+0.2\left(\frac{12Lt_s^3}{K_g}\right)^{0.3}\tan(\theta)$ (Table 4.6.2.2.3c-1)

A simplification factor of 0.97 may be substituted for the term using K_g in the skew correction factor equation. See Table 4.6.2.2.1-2.

Deflection

For deflection calculation, the beams are assumed to deflect equally (2.5.2.6.2). The corresponding distribution factor for this assumption is then the number of lanes (3.6.1.1.1) times the multiple presence factor (Table 3.6.1.1.2-1), divided by the number of beams:

g (deflection) $= m\left(\frac{N_L}{N_b}\right)$

Check Cross-Section Proportion Limits *(6.10.2)*

Although wide-flange sections are typically "stocky" or "stout" enough that all cross-section proportional limits are met, many of the computations in this section are useful in later aspects of design. Proportional limits typically carry more significance with plate girders and are indicators of section stability.

Check Web Proportions (6.10.2.1)

Webs without longitudinal stiffeners (wide-flange sections typically do not have longitudinal stiffeners and plate girders built in Illinois only occasionally have them – See Section 3.3.20 of the Bridge Manual) shall be proportioned such that:

$$\frac{D}{t_w} \le 150$$ (Eq. 6.10.2.1.1-1)

Where:

t_w = web thickness (in.)

D = web depth (in.). For wide-flanges, this is the clear distance between top and bottom flanges ($d – 2t_f$; where: d = section depth (in.) and t_f = flange thickness (in.))

Check Flange Proportions (6.10.2.2)

Compression and tension flanges of the noncomposite section shall meet the following requirements:

i) $\frac{b_f}{2t_f} \leq 12.0$ (Eq. 6.10.2.2-1)

ii) $b_f \geq \frac{D}{6}$ (Eq. 6.10.2.2-2)

iii) $t_f \geq 1.1\,t_w$ (Eq. 6.10.2.2-3)

iv) $0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$ (Eq. 6.10.2.2-4)

Where:

b_f = flange width of the steel section (in.)

t_f = flange thickness of the steel section (in.)

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.^{4})

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.^{4})

Note: Due to symmetry, I_{yc} and I_{yt} are identical for wide-flange sections. Therefore, requirement iv) is always satisfied for rolled shapes.

Check Constructibility *(6.10.3)*

Constructibility requirements are necessary to ensure that the main load-carrying members are adequate to withstand construction loadings. The noncomposite section must support the wet concrete deck and other live loads during pouring.

Construction loads use Strength I load factors. As there is no future wearing surface during construction, the load condition simplifies to:

$M^{CONST}_{STRENGTH I} = \gamma_p(DC_{CONST})+1.75(LL+IM)_{CONST}$ (Table 3.4.1-1)

Where:

γ_p = 1.25 for construction loading (3.4.2.1)

DC_{CONST} = dead load of beam, unhardened slab concrete, reinforcement, and formwork. According to Article 3.2.2.1 of the AASHTO LRFD Bridge Construction Specifications, the combined weight of unhardened slab concrete, reinforcement, and formwork shall not be taken as less than 0.160 kcf for normal-weight concrete. The slab and reinforcement are normally assumed to weigh 0.150 kcf. As such, the formwork is assumed to add 10 pcf.

LL_{CONST} = live load of construction equipment and workers. The Department requires that a minimum live load of 20 psf should be considered, as also stated in Article 3.3.26 of the Bridge Manual. This minimum live load accounts for the weight of the finishing machine and other live loads during construction. An impact factor of 1.33 should be applied to this load.

There are two different sets of provisions in the LRFD Code for the design of noncomposite sections, one found in 6.10.3 and a second found in Appendix A6. The provisions in Appendix A6 make use of web plastification factors that allow for increased resistances of sections, and are encouraged to be used when applicable. This design guide will outline the use of both sets of equations.

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Check Discretely Braced Compression Flanges (Chapter 6) (6.10.3.2.1)

IDOT standard diaphragm details serve as points of discrete bracing for the top and bottom flanges during construction loading. In order for a flange to be considered continuously braced, it must be either encased in hardened concrete or attached to hardened concrete using shear connectors (C6.10.1.6). Therefore, neither flange should be considered continuously braced during construction loading.

There are three checks to ensure that the capacity of the beam is not exceeded by the loading. The first (Eq. 6.10.3.2.1-1) checks beam yielding by comparing the applied stress to the yield strength of the beam. The second (Eq. 6.10.3.2.1-2) checks flange buckling by comparing the applied stress to the lateral-torsional and local flange buckling strengths of the beam. The final (Eq. 6.10.3.2.1-3) checks web yielding by comparing the applied stress to the web bend-buckling strength. Note that for non-slender sections (sections meeting the requirement of Eq. 6.10.6.2.3-1) web bend-buckling is not a concern and Eq. 6.10.3.2.1-3 need not be checked. These equations are as follows:

i) $f_{bu} + f_l \leq \phi_f R_h F_{yc}$ (Eq. 6.10.3.2.1-1)

ii) $f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$ (Eq. 6.10.3.2.1-2)

iii) $f_{bu} \leq \phi_f F_{crw}$ (Eq. 6.10.3.2.1-3)

For Equation (i):

f_{bu} = factored flange stress due to vertical loads (ksi)

= $\frac{M_{STRENGTHI}^{CONST}}{S_{nc}}$, where S_{nc} is the noncomposite section modulus

f_l = lateral flange bending stress (ksi) due to cantilever forming brackets and skew effects. Note that this term should typically be taken as zero for constructibility loading. The additional torsion due to cantilever forming brackets is partially alleviated by the addition of blocking between the first and second exterior beams during construction, and may be ignored. Constructibility skew effects due to discontinuous diaphragms and large skews will also be diminished when General Note 33 is added to the plans. See also Sections 3.3.5 and 3.3.26 of the Bridge Manual.

ϕ_f = resistance factor for flexure, equal to 1.00 according to LRFD Article 6.5.4.2

R_h = hybrid factor specified in Article 6.10.1.10.1, equal to 1.0 for non-hybrid sections (sections with the same grade of steel in the webs and flanges).

F_{yc} = yield strength of the compression flange (ksi)

For Equation (ii):

f_{bu}, f_l, and ϕ_f are as above

F_{nc} = nominal flexural resistance of the flange (ksi), taken as the lesser $F_{nc(FLB)}$ (6.10.8.2.2) and $F_{nc(LTB)}$ (6.10.8.2.3).

Where:

$F_{nc(FLB)}$ = Flange Local Buckling resistance (ksi), specified in Article 6.10.8.2.2, defined as follows:

For $\lambda_f \le \lambda_{pf}$:

$$F_{nc(FLB)} = R_b R_h F_{yc} \quad \text{(Eq. 6.10.8.2.2-1)}$$

Otherwise:

$$F_{nc(FLB)} = \left[1 - \left(1 - \frac{F_{yr}}{R_h\, F_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] R_b\, R_h\, F_{yc} \quad \text{(Eq. 6.10.8.2.2-2)}$$

Where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, taken as the smaller of $0.7F_{yc}$ and F_{yw} (yield strength of the web), but not smaller than $0.5F_{yc}$.

λ_f = slenderness ratio of compression flange

$$= \frac{b_{fc}}{2\, t_{fc}} \quad \text{(Eq. 6.10.8.2.2-3)}$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yc}}} \quad \text{(Eq. 6.10.8.2.2-4)}$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.56\sqrt{\frac{E}{F_{yr}}} \quad \text{(Eq. 6.10.8.2.2-5)}$$

R_b = web load-shedding factor, specified in Article 6.10.1.10.2. Equal to 1.0 when checking constructibility. See LRFD commentary C6.10.1.10.2.

$F_{nc(LTB)}$ = Lateral-Torsional Buckling resistance (ksi), specified in 6.10.8.2.3. For the purposes of calculating $F_{nc(LTB)}$, variables L_b, L_p, and L_r, are defined as follows:

L_b = unbraced length (in.)

L_p = limiting unbraced length to achieve nominal flexural resistance under uniform bending (in.)

$$= 1.0\, r_t \sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. 6.10.8.2.3-4)}$$

L_r = limiting unbraced length for inelastic buckling behavior under uniform bending (in.)

$$= \pi\, r_t \sqrt{\frac{E}{F_{yr}}} \qquad \text{(Eq. 6.10.8.2.3-5)}$$

Where:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c}{b_{fc}}\frac{t_w}{t_{fc}}\right)}} \qquad \text{(Eq. 6.10.8.2.3-9)}$$

D_c = depth of web in compression in the elastic range, calculated according to Article D6.3.1.

$F_{nc(LTB)}$ is then calculated as follows:

For $L_b \leq L_p$:

$$F_{nc(LTB)} = R_b R_h F_{yc} \qquad \text{(Eq. 6.10.8.2.3-1)}$$

For $L_p < L_b \leq L_r$:

$$F_{nc(LTB)} = C_b \left[1 - \left(1 - \frac{F_{yr}}{R_h\ F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b\ R_h\ F_{yc} \leq R_b\ R_h\ F_{yc}$$

(Eq. 6.10.8.2.3-2)

For $L_b > L_r$:

$$F_{nc} = F_{cr} \leq R_b R_h F_{yc}$$ (Eq. 6.10.8.2.3-3)

Where:

F_{yr} = as defined above

F_{cr} = elastic lateral-torsional buckling stress (ksi)

$$= \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}$$ (Eq. 6.10.8.2.3-8)

Where:

C_b = moment gradient modifier, defined as follows:

For $\frac{f_{mid}}{f_2} > 1$, or $f_2 = 0$:

$$C_b = 1.0$$ (Eq. 6.10.8.2.3-6)

Otherwise:

$$C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \leq 2.3$$ (Eq. 6.10.8.2.3-7)

Where:

f_{mid} = stress due to factored vertical loads at the middle of the unbraced length of the compression flange (ksi). This is found using the moment that produces the largest compression at the point under consideration. If this point is never in compression, the smallest tension value is used.

f_{mid} is positive for compression values, and negative for tension values.

f_2 = largest compressive stress due to factored vertical loads at either end of the unbraced length of the compression flange (ksi). This is found using the moment that produces the largest compression and taken as positive at the point under consideration. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, f_2 is taken as zero.

f_0 = stress due to factored vertical loads at the brace point opposite the point corresponding to f_2 (ksi), calculated similarly to f_2 except that the value is the largest value in compression taken as positive or the smallest value in tension taken as negative if the point under consideration is never in compression.

f_1 = stress at the brace point opposite to the one corresponding to f_2, calculated as the intercept of the most critical assumed linear stress variation passing through f_2 and either f_{mid} or f_0, whichever produces the smallest value of C_b (ksi). f_1 may be determined as follows:

When the variation of the moment along the entire length between the brace points is concave:

$$f_1 = f_0 \qquad \text{(Eq. 6.10.8.2.3-10)}$$

Otherwise:

$$f_1 = 2f_{mid} - f_2 \geq f_0 \qquad \text{(Eq. 6.10.8.2.3-11)}$$

For Equation (iii):

f_{bu} and ϕ_f are as above

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} < R_h F_{yc} \text{ and } F_{yw} / 0.7 \qquad \text{(Eq. 6.10.1.9.1-1)}$$

Where:

E = modulus of elasticity of steel (ksi)

D = total depth of web (in.)

t_w = web thickness (in.)

R_h = hybrid factor as stated above

F_{yc} = yield strength of compression flange (ksi)

F_{yw} = yield strength of web (ksi)

$k = \dfrac{9}{\left(\frac{D_c}{D}\right)^2}$ where D_c is the depth of web in compression in the elastic range (in.) (Eq. 6.10.1.9.1-2)

Note that if $\dfrac{2D_c}{t_w} < 5.7\sqrt{\dfrac{E}{F_{yc}}}$, Eq. (iii) should not be checked. (Eq. 6.10.6.2.3-1)

<u>Check Discretely Braced Tension Flanges (Chapter 6)</u> <u>(6.10.3.2.2)</u>

$$f_{bu} + f_l \le \phi_f R_h F_{yt} \qquad \text{(Eq. 6.10.3.2.2-1)}$$

Where all variables and assumptions are identical to those used for checking the compression flange.

<u>Check Discretely Braced Compression Flanges (Appendix A6)</u> <u>(A6.1.1)</u>

The provisions of Appendix A6 may only be used if the following requirements are satisfied:

- The bridge must be straight or equivalently straight according to the provisions of (4.6.1.2.4b)
- The specified minimum yield strengths of the flanges and web do not exceed 70.0 ksi

- $\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$, where D_c is the depth of web in compression in the elastic range (in.), t_w is the web thickness (in.), E is the modulus of elasticity of the steel (ksi), and F_{yc} is the yield strength of the compression flange (ksi) (Eq. A6.1-1). This is a web slenderness check.
- $\frac{I_{yc}}{I_{yt}} \geq 0.3$, where I_{yc} and I_{yt} are the moments of inertia of the compression and tension flanges about the vertical axis (in.^{4}) (Eq. A6.1-2)

The Fifth Edition (2010) of the AASHTO LRFD Code imposes another requirement that the bridge must have a skew of twenty degrees in order for Appendix A to be used. Currently IDOT is not enforcing this new requirement.

If these requirements are satisfied, the following equation may be used to check the compression flange:

$$M_u + \frac{1}{3} f_l S_{xc} \leq \phi_f M_{nc} \qquad \text{(Eq. A6.1.1-1)}$$

Where:

M_u = largest major-axis bending moment throughout the unbraced length (k-in.)

f_l = lateral flange bending stress as calculated using Article 6.10.1.6 (ksi), taken as zero for constructability loading.

S_{xc} = elastic section modulus of the section about the major axis of the section to the compression flange (in.^{3}).

= $\frac{M_{yc}}{F_{yc}}$. For noncomposite sections, this simplifies to S_{xc} of the noncomposite section.

ϕ_f = 1.0 for flexure

M_{nc} = smaller of $M_{nc(FLB)}$ and $M_{nc(LTB)}$ as calculated in Appendix A6.3

Where:

$M_{nc(FLB)}$ = flexural resistance based on compression flange local buckling (ksi)

$$= R_{pc}M_{yc} \text{ if } \lambda_f \leq \lambda_{pf} \qquad \text{(Eq. A6.3.2-1)}$$

$$= \left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}\ M_{yc}}\right)\left(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right]R_{pc}\ M_{yc}\ \text{otherwise}$$

(Eq. A6.3.2-2)

Where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, taken as the smaller of $0.7F_{yc}$, $R_hF_{yt}S_{xt}$ / S_{xc} and F_{yw} (yield strength of the web), but not smaller than $0.5F_{yc}$.

F_{yc} = yield strength of the compression flange (ksi)

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

F_{yw} = yield strength of the web (ksi)

S_{xc} = elastic section modulus of the section about the major axis of the section to the compression flange (in.^{3}).

= $\frac{M_{yc}}{F_{yc}}$. For noncomposite sections, this simplifies to S_{xc} of the noncomposite section.

M_{yc} = yield moment with respect to the compression flange. For noncomposite sections, this is taken as $S_{xc}F_{yc}$. (k-in.)

λ_f = slenderness ratio of compression flange

$$= \frac{b_{fc}}{2\ t_{fc}} \qquad \text{(Eq. A6.3.2-3)}$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. A6.3.2-4)}$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.95\sqrt{\frac{Ek_c}{F_{yr}}} \qquad \text{(Eq. A6.3.2-5)}$$

$$k_c = \frac{4}{\sqrt{\frac{D}{t_w}}} \quad 0.35 \le k_c \le 0.76 \text{ for built-up sections} \qquad \text{(Eq. A6.3.2-6)}$$

= 0.76 for rolled shapes

R_{pc} = web plastification factor

$$= \frac{M_p}{M_{yc}} \text{ if } \frac{2D_{cp}}{t_w} \le \lambda_{pw(Dcp)} \qquad \text{(Eq. A6.2.1-1)}$$

$$= \left[1-\left(1-\frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(Dc)}}{\lambda_{rw} - \lambda_{pw(Dc)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}} \text{ otherwise}$$

(Eq. A6.2.2-4)

Where:

M_p = plastic moment determined as specified in Article D6.1 (k-in.)

M_{yc} = $F_{yc}S_{xc}$ for noncomposite sections (k-in.)

D_{cp} = depth of web in compression at the plastic moment as specified in Article D6.3.2 (k-in.)

$\lambda_{pw(Dcp)}$ = limiting slenderness ratio for a compact web corresponding to $2D_{cp}$ / t_w

$$= \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \qquad \text{(Eq. A6.2.1-2)}$$

$\lambda_{pw(Dc)}$ = limiting slenderness ratio for a compact web corresponding to $2D_c$ / t_w

$$= \lambda_{pw(Dcp)}\left(\frac{D_c}{D_{cp}}\right) \le \lambda_{rw} \qquad \text{(Eq. A6.2.2-6)}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. A6.2.1-3)}$$

$M_{nc(LTB)}$ = flexural resistance based on lateral-torsional buckling (k-in.)

For $L_b \le L_p$:

$$M_{nc(LTB)} = R_{pc}M_{yc} \qquad \text{(Eq. A6.3.3-1)}$$

For $L_p < L_b \le L_r$:

$$M_{nc(LTB)} = C_b\left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}\,M_{yc}}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_{pc}\,M_{yc} \le R_{pc}M_{yc}$$

(Eq. A6.3.3-2)

For $L_b > L_r$:

$$M_{nc(LTB)} = F_{cr}S_{xc} \le R_{pc}M_{yc}$$ (Eq. A6.3.3-3)

Where:

L_b = unbraced length (in.)

$$L_p = 1.0\,r_t\sqrt{\frac{E}{F_{yc}}}$$ (Eq. A6.3.3-4)

$$L_r = 1.95\,r_t\frac{E}{F_{yr}}\sqrt{\frac{J}{S_{xc}h}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr}}{E}\frac{S_{xc}h}{J}\right)^2}}$$ (Eq. A6.3.3-5)

Where:

r_t = effective radius of gyration for lateral-torsional buckling (in.)

$$= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c\,t_w}{b_{fc}\,t_{fc}}\right)}}$$ (Eq. A6.3.3-10)

E = modulus of elasticity of steel (ksi)

F_{yr} = compression flange stress at onset of yielding, as calculated in $M_{nc(FLB)}$ calculations (ksi)

J = St. Venant's torsional constant, taken as the sum of the moments of inertia of all contributing members about their major axes at the end of the member, with corrections

$$= \frac{Dt_w^3}{3}+\frac{b_{fc}t_{fc}^3}{3}\left(1-0.63\frac{t_{fc}}{b_{fc}}\right)+\frac{b_{ft}t_{ft}^3}{3}\left(1-0.63\frac{t_{ft}}{b_{ft}}\right)$$

(Eq. A6.3.3-9)

S_{xc} = elastic section modulus about the major axis of the section to the compression flange. For noncomposite beams this is S_{xc} of the noncomposite section. (in.^{3})

h = section depth (in.)

C_b = moment gradient modifier, defined as follows:

For $\frac{M_{mid}}{M_2} > 1$, or $M_2 = 0$:

C_b = 1.0 (Eq. A6.3.3-6)

Otherwise:

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \quad \text{(Eq. A6.3.3-7)}$$

Where:

M_{mid} = moment due to factored vertical loads at the middle of the unbraced length (k-in.). This is found using the moment that produces the largest compression at the point under consideration. If this point is never in compression, the smallest tension value is used. M_{mid} is positive for compression values, and negative for tension values.

M_2 = largest bending moment due to factored vertical loads at either end of the unbraced length of the compression flange (k-in.). This is found using the moment that produces the largest compression and taken as positive at the point under consideration. If the moment is zero or causes tension in the flange under consideration at both ends of the unbraced length, M_2 is taken as zero.

M_0 = moment due to factored vertical loads at the brace point opposite the point corresponding to M_2 (k-in.), calculated similarly to M_2 except that the value is the largest value causing compression taken as positive or the smallest value causing tension taken as negative if the point under consideration is never in compression.

M_1 = moment at the brace point opposite to the one corresponding to M_2, calculated as the intercept of the most critical assumed linear moment variation passing through M_2 and either M_{mid} or M_0, whichever produces the smallest value of C_b (k-in.). M_1 may be determined as follows:

When the variation of the moment along the entire length between the brace points is concave:

$M_1 = M_0$ (Eq. A6.3.3-11)

Otherwise:

$M_1 = 2M_{mid} - M_2 \geq M_0$ (Eq. A6.3.3-12)

F_{cr} = elastic lateral-torsional buckling stress (ksi)

$$= \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h} \left(\frac{L_b}{r_t}\right)^2} \quad \text{(Eq. A6.3.3-8)}$$

<u>Check Discretely Braced Tension Flanges (Appendix A)</u> <u>(A6.4)</u>

The nominal flexural resistance based on tension flange yielding is taken as:

$M_{nt} = R_{pt} M_{yt}$ (Eq. A6.4-1)

Where R_{pt} and M_{yt} are calculcated similar to R_{pc} and M_{yc} above.

<u>Additional Constructibility Checks</u>

Articles 6.10.3.2.4, 6.10.3.4, and 6.10.3.5 involve consideration of issues arising from bridges with pouring sequences, staged construction, loading sequences during construction, etc. For a particular typical bridge, these articles may or may not apply. The commentaries,

however, provide useful guidance to the engineer. Article 6.10.3.3 examines shear forces in webs during construction loading. As the final Strength I shears are usually much greater than construction loading shears, this generally does not control the design.

Check Service Limit State *(6.10.4)*

The purpose of the service limit state requirements are to prevent excessive deflections (possibly from yielding or slip in connections) due to severe traffic loading in a real-life situation. Service II load factors are used for these checks: the actual dead loads are applied (load factor of 1.00), along with 1.3 times the projected live load plus impact.

$$M_{SERVICE\ II} = 1.00(DC_1 + DC_2)+1.00(DW)+1.30(LL+IM) \qquad \text{(Table 3.4.1-1)}$$

For this load case, the following flange requirements shall be met:

The top steel flange of composite sections shall satisfy:

$$f_f \leq 0.95\left(R_h\right)\left(F_{yf}\right) \qquad \text{(Eq. 6.10.4.2.2-1)}$$

Note that "top steel flange" in this case is referring to the flange adjacent to the deck and not necessarily the compression flange. This equation does not include the lateral flange stress term because for flanges attached to decks with studs f_l is assumed to be zero.

This equation need not be checked for composite sections in negative flexure if 6.10.8 is used to calculate the ultimate capacity of the section. It also need not be checked for noncompact composite sections in positive flexure (C6.10.4.2.2).

The bottom steel flange of composite sections shall satisfy:

$$f_f + \frac{f_l}{2} \leq 0.95\left(R_h\right)\left(F_{yf}\right) \qquad \text{(Eq. 6.10.4.2.2-2)}$$

Where:

f_f = flange stress due to factored vertical Service II loads (ksi), calculated as follows:

For positive moment regions:

$$f_f = \frac{M_{DC1}}{S_{nc}} + \frac{M_{DC2} + M_{DW}}{S_{n=27}} + \frac{1.3\left(M_{LL+IM}\right)}{S_{n=9}} \qquad \text{(Eq. D6.2.2-1)}$$

For negative moment regions:

Article 6.10.4.2.1 allows for an uncracked section if the stresses in the slab are less than twice the modulus of rupture of the concrete. Therefore:

$$f_f = \frac{M_{DC1}}{S_{nc}} + \frac{M_{DC2} + M_{DW}}{S_{n=27}} + \frac{1.3\left(M_{LL+IM}\right)}{S_{n=9}} \text{ if } f_{slab} < 2f_r \qquad \text{(Eq. D6.2.2-1)}$$

$$f_f = \frac{M_{DC1}}{S_{nc}} + \frac{M_{DC2} + M_{DW} + 1.3\left(M_{LL+IM}\right)}{S_{c,cr}} \text{ if } f_{slab} > 2f_r \qquad \text{(Eq. D6.2.2-1)}$$

Where:

M_{DC1} = moment due to dead loads on noncomposite section. Typically this consists of the dead loads due to the steel beam and the weight of the concrete deck and fillet (k-in.)

M_{DC2} = moment due to dead loads on composite section. Typically this consists of the dead loads due to parapets, sidewalks, railings, and other appurtenances (k-in.)

M_{DW} = moment due to the future wearing surface (k-in.)

M_{LL+IM} = moment due to live load plus impact (k-in.)

S_{nc} = section modulus of noncomposite section about the major axis of bending to the flange being checked (in.^{3})

$S_{n=27}$ = section modulus of long-term composite section about the major axis of bending to the flange being checked (deck transposed by a factor of 3n = 27) (in.^{3})

$S_{n=9}$ = section modulus of short-term composite section about the major axis of bending to the flange being checked (deck transposed by a factor of n = 9) (in.^{3})

$S_{c,cr}$ = section modulus of steel section plus longitudinal deck reinforcement about the major axis of bending to the flange being checked (cracked composite section) (in.^{3})

f_{slab} = stress at top of deck during Service II loading assuming an uncracked section (ksi). Note that when checking the concrete deck stresses, a composite section transformed to concrete should be used (as opposed to a composite section transformed to steel). This is done by multiplying the section moduli by their respective transformation ratios. Adding these transformation ratios to stress equations, the deck stress is found as follows:

$$= \frac{1}{3n}\left(\frac{M_{DC2} + M_{DW}}{S_{n=27}}\right) + \frac{1}{n}\left(\frac{1.3\left(M_{LL+IM}\right)}{S_{n=9}}\right)$$

f_r = modulus of rupture of concrete (ksi)

$$= 0.24\sqrt{f'_c}$$

Again, for straight interior beams with skews less than or equal to 45°, f_l can be assumed to be zero.

Sections in positive flexure with D / t_w > 150, and all sections in negative flexure designed using Appendix A for Strength I design shall also satisfy the following requirement:

$$f_c < F_{crw} \qquad \text{(Eq. 6.10.4.2.2-4)}$$

Where:

f_c = stress in the bottom flange in the negative moment region, calculated as f_f above (ksi)

F_{crw}= nominal web bend-buckling resistance

$$= \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} < R_hF_{yc} \text{ and } F_{yw} / 0.7 \qquad \text{(Eq. 6.10.1.9.1-1)}$$

Where:

E = modulus of elasticity of steel (ksi)

D = total depth of web (in.)

t_w = web thickness (in.)

R_h = hybrid factor as stated above

F_{yc} = yield strength of compression flange (ksi)

F_{yw} = yield strength of web (ksi)

$$k = \frac{9}{\left(\frac{D_c}{D}\right)^2} \qquad \text{(Eq. 6.10.1.9.1-2)}$$

Where:

D_c = depth of web in compression, calculated in accordance with Appendix D6.3.1.

Equation 6.10.4.2.2-3 is only necessary for sections noncomposite in final loading. As it is IDOT practice is to provide shear stud connectors along the entire length of bridges, this equation is not applicable. As stated in Section 3.3.2 of the Bridge Manual, the live load deflection and span-to-depth ratio provisions of Article 2.5.2.6.2, referenced by Article 6.10.4.1, are only applicable to bridges with pedestrian traffic.

The code provisions in 6.10.4.2.2 reference Appendix B6, which allows for moment redistribution. This is not allowed by the Department.

Check Fatigue and Fracture Limit State *(6.10.5)*

Fatigue and fracture limit state requirements are necessary to prevent beams and connections from failing due to repeated loadings.

For the fatigue load combination, all moments are calculated using the fatigue truck specified in Article 3.6.1.4. The fatigue truck is similar to the HL-93 truck, but with a constant 30 ft. rear axle spacing. Lane loading is not applied. The fatigue truck loading uses a different Dynamic Load Allowance of 15% (Table 3.6.2.1-1).

Note: The fatigue truck loading does not use multiple presence factors. The value of the live load distribution factor must be divided by 1.2 when finding moments from the fatigue truck (3.6.1.1.2).

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

$M_{FATIGUE\ I}$ = 1.5(LL+IM+CE) (Table 3.4.1-1)

$M_{FATIGUE\ II}$ = 0.75(LL+IM+CE) (Table 3.4.1-1)

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.

Regardless of limit state, the full section shall satisfy the following equation:

$$\gamma\left(\Delta f\right) \le \left(\Delta F\right)_n \qquad \text{(Eq. 6.6.1.2.2-1)}$$

Where:

γ = Fatigue load factor, specified in Table 3.4.1-1

$\left(\Delta f\right)$ = Fatigue load combination stress range (ksi)

$$= \frac{M^{+}_{FATIGUE} - M^{-}_{FATIGUE}}{S_{n=9}}$$

Currently, the effects of lateral flange bending stress need not be considered in the check of fatigue.

$\left(\Delta F\right)_n$ = nominal fatigue resistance (ksi), found as follows:

For Fatigue I limit state:

$$\left(\Delta F\right)_n = \left(\Delta F\right)_{TH} \qquad \text{(Eq. 6.6.1.2.5-1)}$$

Where $(\Delta F)_{TH}$ is the threshold stress range found in Table 6.6.1.2.3-1 or Table 6.6.1.2.5-3 (ksi)

For Fatigue II limit state:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{(Eq. 6.6.1.2.5-2)}$$

Where:

A = constant from Table 6.6.1.2.3-1 or Table 6.6.1.2.5-1 (ksi^3). For typical painted rolled sections and painted plate girders:

- Fatigue category A should be used in positive moment regions
- Fatigue Category C should be used in negative moment regions to account for the effect of the studs (See Descriptions 1.1 and 8.1 in Table 6.6.1.2.3-1).

For typical rolled sections and plate girders made of weathering steel:

- Fatigue Category B should be used in positive moment regions
- Fatigue Category C should be used in negative moment regions (See Descriptions 1.2 and 8.1 in Table 6.6.1.2.3-1).

For plate girders, brace points should also be checked for category C' (See Description 4.1).

$$N = \left(\frac{365\text{ days}}{\text{year}}\right)(75\text{ years})\left(\frac{n\text{ cycles}}{\text{truck}}\right)\left(\frac{(ADTT)_{37.5,\,SL}\text{ trucks}}{\text{day}}\right)$$

(Eq. 6.6.1.2.5-3)

Where:

n = number of stress cycles per truck passage, taken from Table 6.6.1.2.5-2

$(ADTT)_{37.5,\,SL}$ = single lane ADTT at 37.5 years. This is calculated in a similar fashion as the calculation of $(ADTT)_{75,\,SL}$ above except that the multiplier 37.5/20 is used in place of the multiplier 75/20 when extrapolating.

Additionally, all rolled sections shall satisfy the Temperature Zone 2 Charpy V-notch fracture toughness requirements of Article 6.6.2. See also Section 3.3.7 of the Bridge Manual.

Interior panels of stiffened webs in negative moment regions shall also satisfy the shear requirements of 6.10.9.3.3. See 6.10.5.3. This section is similar to that discussed in the Check Strength Limit State (Shear) portion of this design guide.

Many typical bridges made with rolled steel sections need not satisfy the requirements of distortion-induced fatigue given in Article 6.6.1.3. This can become a consideration for bridges with high skews, curved members, etc.

Check Strength Limit State (Moment) (6.10.6)

Strength limit state compares the ultimate shear and moment capacities of the sections to the factored Strength I loads applied to the bridge. Strength I factors for this limit state are as follows:

$M_{STRENGTH\ I} = \gamma_p(DC+DW)+1.75(LL+IM+CE)$

Where:

γ_p = For DC: 1.25 max., 0.9 min. (Table 3.4.1-2)

For DW: 1.50 max., 0.65 min..

For single-span bridges, use the maximum values of γ_p in all locations.

<u>Positive Moment Regions</u>

Composite positive moment regions are checked using Chapter 6 only. Appendix A6 does not apply to composite positive moment regions.

The strength limit state requirements for compact sections differ from those for noncompact sections. Compact rolled sections are defined in Article 6.10.6.2.2 as sections that satisfy the following:

i) The bridge is straight or equivalently straight as per 4.6.1.2.4b

ii) $F_y \leq 70$ ksi

iii) The cross-section proportional limit web requirements are satisfied (6.10.2.1.1)

iv) $$\frac{2(D_{cp})}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$$ (Eq. 6.10.6.2.2-1)

Where:

D_{cp} = depth of web in compression at the plastic moment, as calculated using the procedure in Article D6.3.2 (in.)

If these requirements are met, the section shall satisfy the following strength limit state requirement (6.10.7.1):

$$M_u + \frac{1}{3}(f_l)(S_{xt}) \leq \phi_f (M_n)$$ (Eq. 6.10.7.1.1-1)

Again, f_l = 0 for straight interior beams on bridges with skews less than or equal to 45°.

M_n is the nominal flexural resistance of the section, determined as follows:

For $D_p \leq 0.1D_t$:

$$M_n = M_p$$ (Eq. 6.10.7.1.2-1)

Otherwise:

$$M_n = M_p\left[1.07 - 0.7\left(\frac{D_p}{D_t}\right)\right]$$ (Eq. 6.10.7.1.2-2)

Where:

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.)

D_t = total depth of the composite section (in.)

M_p = plastic moment capacity (k-in.) of the composite section calculated using the procedure in Article D6.1

For continuous spans, the following requirement also applies:

$$M_n \leq 1.3R_hM_y \qquad \text{(Eq. 6.10.7.1.2-3)}$$

Where:

R_h = hybrid factor, taken as 1.0 for non-hybrid sections (sections with the same grade of steel in the webs and flanges)

M_y = controlling yield moment of section (k-in.). In composite positive moment regions, the controlling yield moment will likely be governed by the bottom flange.

$$= 1.25M_{DC1} + 1.25M_{DC2} + 1.5M_{DW} + \left(F_{yf} - \frac{M_{DC1}}{S_{nc}} - \frac{M_{DC2} + M_{DW}}{S_{c,n=27}}\right)S_{c,n=9}$$

(Eq. D6.2.2-1 and 2)

The purpose of this check is to ensure that the plastic moment capacity does not exceed the yield strength by too large a margin. If this were to happen, significant changes in stiffness may occur in the beam during yielding. These changes in stiffness can result in redistribution of moments, resulting in non-conservative designs in different parts of the bridge.

If the beam does not meet the compactness criteria listed above, it is noncompact and shall satisfy the following equations:

$$f_{bu} \leq \phi_f F_{nc} \qquad \text{(compression flanges)} \qquad \text{(Eq. 6.10.7.2.1-1)}$$

$$f_{bu} + \frac{1}{3}f_l \leq \phi_f F_{nt} \qquad \text{(tension flanges)} \qquad \text{(Eq. 6.10.7.2.1-2)}$$

Where:

f_{bu} = flange stress calculated without consideration of lateral bending stresses (ksi)

ϕ_f = 1.0 for flexure

f_l = lateral flange bending stress (ksi)

$F_{nc} = R_b R_h F_{yc}$ (Eq. 6.10.7.2.2-1)

$F_{nt} = R_h F_{yt}$ (Eq. 6.10.7.2.2-2)

Where:

R_b = web load-shedding factor. In most cases this is assumed to be 1.0 (6.10.1.10.2). However, there are cases involving slender webs (not satisfying the cross-section proportion limits of 6.10.2.1.1) that also have a shallow depth of web in compression where R_b need be calculated. In these cases, the provisions of 6.10.1.10.2 shall be followed. However, this is not typical in IDOT bridges and will not be outlined in this design guide.

R_h = hybrid factor, taken as 1.0 for non-hybrid sections (sections with the same grade of steel in the web and flanges)

F_{yc} = yield strength of compression flange (ksi)

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

Regardless of whether or not the section is compact, it shall satisfy the following ductility requirement:

$D_p \leq 0.42D_t$ (Eq. 6.10.7.3-1)

This check is analogous to checking a reinforced concrete beam for c/d < 0.42 (see C5.7.2.1). The purpose is to prevent the concrete slab eccentricity from becoming too large, resulting in a section where concrete will crush in order to maintain compatibility with the yielding steel.

Negative Moment Regions

Composite negative moment regions may be checked using either the provisions of Chapter 6 or, if applicable, the provisions of Appendix A6. Use of provisions of Appendix A6 is encouraged when applicable. This design guide will outline the use of both sets of equations.

Check Discretely Braced Compression Flanges (Chapter 6) (6.10.8.1.1)

In this check, the applied compression flange stress is checked against the local flange buckling stress and the lateral-torsional buckling stress. Both of these stresses are based upon F_y of the compression flange, so a comparison to the yield strength of the compression flange is inherent in this check and is not a separate check, as it is when checking constructibility.

$$f_{bu} + \frac{1}{3} f_l \le \phi_f F_{nc} \qquad \text{(Eq. 6.10.8.1.1-1)}$$

Where:

f_{bu} = factored flange stress due to vertical loads (ksi)

$$= \frac{1.25M_{DC1}}{S_{nc}} + \frac{1.25M_{DC2} + 1.5M_{DW}}{S_{c,n=27}} + \frac{1.75M_{LL+IM}}{S_{c,n=9}}$$

f_l = lateral flange bending stress (ksi) due to cantilever forming brackets and skew effects. Note that this term should typically be taken as zero for a majority of typical bridges if the requirements of Article 503.06 of the IDOT Standard Specifications are met and the bridge skew is less than or equal to 45°. See also Sections 3.3.5 and 3.3.26 of the Bridge Manual.

ϕ_f = resistance factor for flexure, equal to 1.00 according to LRFD Article 6.5.4.2

F_{nc} = nominal flexural resistance of the flange (ksi), taken as the lesser $F_{nc(FLB)}$ (6.10.8.2.2) and $F_{nc(LTB)}$ (6.10.8.2.3).

Where:

$F_{nc(FLB)}$ = Flange Local Buckling resistance (ksi), specified in Article 6.10.8.2.2, defined as follows:

For $\lambda_f \le \lambda_{pf}$:

$$F_{nc(FLB)} = R_b R_h F_{yc} \qquad \text{(Eq. 6.10.8.2.2-1)}$$

Otherwise:

$$F_{nc(FLB)} = \left[1-\left(1-\frac{F_{yr}}{R_h\,F_{yc}}\right)\left(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right]R_b\,R_h\,F_{yc} \quad \text{(Eq. 6.10.8.2.2-2)}$$

Where:

F_{yc} = compression flange stress at the onset of nominal yielding within the cross section, taken as the smaller of $0.7F_{yc}$ and F_{yw} (yield strength of the web), but not smaller than $0.5F_{yc}$.

λ_f = slenderness ratio of compression flange

$$= \frac{b_{fc}}{2\,t_{fc}} \quad \text{(Eq. 6.10.8.2.2-3)}$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yc}}} \quad \text{(Eq. 6.10.8.2.2-4)}$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.56\sqrt{\frac{E}{F_{yr}}} \quad \text{(Eq. 6.10.8.2.2-5)}$$

R_b = web load-shedding factor, specified in Article 6.10.1.10.2, typically assumed to be 1.0 for cases that meet the cross-section proportion limits of 6.10.2. See above for further discussion.

$F_{nc(LTB)}$ = Lateral-Torsional Buckling Resistance (ksi), specified in Article 6.10.8.2.3. For the purposes of calculating $F_{nc(LTB)}$, variables L_b, L_p, and L_r, are defined as follows:

L_b = unbraced length (in.)

L_p = limiting unbraced length to achieve nominal flexural resistance under uniform bending (in.)

$$= 1.0\,r_t\sqrt{\frac{E}{F_{yc}}} \quad \text{(Eq. 6.10.8.2.3-4)}$$

L_r = limiting unbraced length for inelastic buckling behavior under uniform bending (in.)

$$= \quad \pi r_t \sqrt{\frac{E}{F_{yr}}} \qquad \text{(Eq. 6.10.8.2.3-5)}$$

Where:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \qquad \text{(Eq. 6.10.8.2.3-9)}$$

D_c = depth of web in compression, calculated according to Article D6.3.1.

$F_{nc(LTB)}$ is then calculated as follows:

For $L_b \leq L_p$:

$$F_{nc(LTB)} = R_b R_h F_{yc} \qquad \text{(Eq. 6.10.8.2.3-1)}$$

For $L_p < L_b \leq L_r$:

$$F_{nc(LTB)} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] R_b R_h F_{yc} \leq R_b R_h F_{yc}$$

(Eq. 6.10.8.2.3-2)

For $L_b > L_r$:

$$F_{nc} = F_{cr} \leq R_b R_h F_{yc} \qquad \text{(Eq. 6.10.8.2.3-3)}$$

Where:

F_{yr} = as defined above

F_{cr} = elastic lateral-torsional buckling stress (ksi)

$$= \frac{C_b R_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \qquad \text{(Eq. 6.10.8.2.3-8)}$$

Where:

C_b = moment gradient modifier, defined as follows:

For $\frac{f_{mid}}{f_2} > 1$, or $f_2 = 0$:

$$C_b = 1.0 \quad \text{(Eq. 6.10.8.2.3-6)}$$

Otherwise:

$$C_b = 1.75 - 1.05\left(\frac{f_1}{f_2}\right) + 0.3\left(\frac{f_1}{f_2}\right)^2 \leq 2.3 \quad \text{(Eq. 6.10.8.2.3-7)}$$

Where:

f_{mid} = stress due to factored vertical loads at the middle of the unbraced length of the compression flange (ksi). This is found using the moment that produces the largest compression at the point under consideration. If this point is never in compression, the smallest tension value is used. f_{mid} is positive for compression values, and negative for tension values.

f_2 = largest compressive stress due to factored vertical loads at either end of the unbraced length of the compression flange (ksi). This is found using the moment that produces the largest compression and taken as positive at the point under consideration. If the stress is zero or tensile in the flange under consideration at both ends of the unbraced length, f_2 is taken as zero.

f_0 = stress due to factored vertical loads at the brace point opposite the point corresponding to f_2 (ksi), calculated similarly to f_2 except that the value is the largest value in compression taken as positive or the smallest value in tension taken as negative if the point under consideration is never in compression.

f_1 = stress at the brace point opposite to the one corresponding to f_2, calculated as the intercept of the most critical assumed linear stress variation passing through f_2 and either f_{mid} or f_0, whichever produces the smallest value of C_b (ksi). f_1 may be determined as follows:

When the variation of the moment along the entire length between the brace points is concave:

$$f_1 = f_0 \qquad \text{(Eq. 6.10.8.2.3-10)}$$

Otherwise:

$$f_1 = 2f_{mid} - f_2 \geq f_0 \qquad \text{(Eq. 6.10.8.2.3-11)}$$

Check Continuously Braced Tension Flanges (Chapter 6) (6.10.8.1.3)

$$f_{bu} \leq \phi_f R_h F_{yf} \qquad \text{(Eq. 6.10.8.1.3-1)}$$

Where all variables and assumptions are identical to those used for checking the compression flange.

Check Discretely Braced Compression Flanges (Appendix A6) (A6.1.1)

The provisions of Appendix A6 may only be used if the following requirements are satisfied:

- The bridge must be straight or equivalently straight according to the provisions of (4.6.1.2.4b)
- The specified minimum yield strengths of the flanges and web do not exceed 70.0 ksi

- $\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$, where D_c is the depth of web in compression in the elastic range (in.), t_w is the web thickness (in.), E is the modulus of elasticity of the steel (ksi), and F_{yc} is the yield strength of the compression flange (ksi) (Eq. A6.1-1). This is a web slenderness check.
- $\frac{I_{yc}}{I_{yt}} \geq 0.3$, where I_{yc} and I_{yt} are the moments of inertia of the compression and tension flanges about the vertical axis (in.^{4}) (Eq. A6.1-2)

The Fifth Edition (2010) of the AASHTO LRFD Code imposes another requirement that the bridge must have a skew of twenty degrees in order for Appendix A to be used. Currently IDOT is not enforcing this new requirement.

If these requirements are satisfied, the following equation may be used to check the compression flange:

$$M_u + \frac{1}{3} f_l S_{xc} \leq \phi_f M_{nc} \qquad \text{(Eq. A6.1.1-1)}$$

Where:

M_u = largest major-axis bending moment throughout the unbraced length (k-in.)

f_l = lateral flange bending stress as calculated using Article 6.10.1.6 (ksi).

S_{xc} = elastic section modulus of the section about the major axis of the section to the compression flange (in.^{3}).

= $\frac{M_{yc}}{F_{yc}}$. For noncomposite sections, this simplifies to S_{xc} of the noncomposite section.

ϕ_f = 1.0 for flexure

M_{nc} = smaller of $M_{nc(FLB)}$ and $M_{nc(LTB)}$ as calculated in Appendix A6.3

Where:

$M_{nc(FLB)}$ = flexural resistance based on compression flange local buckling (ksi)

= $R_{pc}M_{yc}$ if $\lambda_f \leq \lambda_{pf}$ (Eq. A6.3.2-1)

$$= \left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}\ M_{yc}}\right)\left(\frac{\lambda_f-\lambda_{pf}}{\lambda_{rf}-\lambda_{pf}}\right)\right]R_{pc}\ M_{yc}\ \text{otherwise}$$

(Eq. A6.3.2-2)

Where:

F_{yr} = compression flange stress at the onset of nominal yielding within the cross section, taken as the smaller of $0.7F_{yc}$, $R_hF_{yt}S_{xt}$ / S_{xc} and F_{yw} (yield strength of the web), but not smaller than $0.5F_{yc}$.

F_{yc} = yield strength of the compression flange (ksi)

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

F_{yw} = yield strength of the web (ksi)

S_{xt} = elastic section modulus of the section about the major axis of the section to the tension flange (in.^{3}).

$$= \frac{M_{yt}}{F_{yt}}.$$

S_{xc} = elastic section modulus of the section about the major axis of the section to the compression flange (in.^{3}).

$$= \frac{M_{yc}}{F_{yc}}.$$

M_{yc} = yield moment with respect to the compression flange, calculated in accordance with Appendix D6.2.

λ_f = slenderness ratio of compression flange

$$= \frac{b_{fc}}{2\ t_{fc}} \qquad \text{(Eq. A6.3.2-3)}$$

λ_{pf} = limiting slenderness ratio for a compact flange

$$= 0.38\sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. A6.3.2-4)}$$

λ_{rf} = limiting slenderness ratio for a noncompact flange

$$= 0.95\sqrt{\frac{Ek_c}{F_{yr}}} \qquad \text{(Eq. A6.3.2-5)}$$

$$k_c = \frac{4}{\sqrt{\frac{D}{t_w}}} \quad 0.35 \le k_c \le 0.76 \text{ for built-up sections} \qquad \text{(Eq. A6.3.2-6)}$$

= 0.76 for rolled shapes

R_{pc} = web plastification factor

$$= \frac{M_p}{M_{yc}} \text{ if } \frac{2D_{cp}}{t_w} \le \lambda_{pw(Dcp)} \qquad \text{(Eq. A6.2.1-1)}$$

$$= \left[1-\left(1-\frac{R_hM_{yc}}{M_p}\right)\left(\frac{\lambda_w - \lambda_{pw(Dc)}}{\lambda_{rw} - \lambda_{pw(Dc)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}} \text{ otherwise}$$

(Eq. A6.2.2-4)

Where:

M_p = plastic moment determined as specified in Article D6.1 (k-in.)

M_{yc} = $F_{yc}S_{xc}$ for noncomposite sections (k-in.)

D_{cp} = depth of web in compression at the plastic moment as specified in Article D6.3.2 (k-in.)

$\lambda_{pw(Dcp)}$ = limiting slenderness ratio for a compact web corresponding to $2D_{cp} / t_w$

$$= \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_hM_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \qquad \text{(Eq. A6.2.1-2)}$$

$\lambda_{pw(Dc)}$ = limiting slenderness ratio for a compact web corresponding to $2D_c / t_w$

$$= \lambda_{pw(Dcp)}\left(\frac{D_c}{D_{cp}}\right) \le \lambda_{rw} \qquad \text{(Eq. A6.2.2-6)}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. A6.2.1-3)}$$

$M_{nc(LTB)}$ = flexural resistance based on lateral-torsional buckling (k-in.)

For $L_b \le L_p$:

$$M_{nc(LTB)} = R_{pc}M_{yc} \qquad \text{(Eq. A6.3.3-1)}$$

For $L_p < L_b \le L_r$:

$$M_{nc(LTB)} = \quad C_b\left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}\ M_{yc}}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_{pc}\ M_{yc} \le R_{pc}M_{yc}$$

(Eq. A6.3.3-2)

For $L_b > L_r$:

$$M_{nc(LTB)} = \quad F_{cr}S_{xc} \le R_{pc}M_{yc} \qquad \text{(Eq. A6.3.3-3)}$$

Where:

L_b = unbraced length (in.)

$$L_p = 1.0\, r_t\sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. A6.3.3-4)}$$

$$L_r = 1.95\, r_t\frac{E}{F_{yr}}\sqrt{\frac{J}{S_{xc}h}}\sqrt{1+\sqrt{1+6.76\left(\frac{F_{yr}}{E}\frac{S_{xc}h}{J}\right)^2}} \qquad \text{(Eq. A6.3.3-5)}$$

Where:

r_t = effective radius of gyration for lateral-torsional buckling (in.)

$$= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c\ t_w}{b_{fc}\ t_{fc}}\right)}} \qquad \text{(Eq. A6.3.3-10)}$$

E = modulus of elasticity of steel (ksi)

F_{yr} = compression flange stress at onset of yielding, as calculated in $M_{nc(FLB)}$ calculations (ksi)

J = St. Venant's torsional constant, taken as the sum of the moments of inertia of all contributing members about their major axes at the end of the member, with corrections

$$= \frac{Dt_w^3}{3}+\frac{b_{fc}t_{fc}^3}{3}\left(1-0.63\frac{t_{fc}}{b_{fc}}\right)+\frac{b_{ft}t_{ft}^3}{3}\left(1-0.63\frac{t_{ft}}{b_{ft}}\right)$$

(Eq. A6.3.3-9)

S_{xc} = elastic section modulus about the major axis of the section to the compression flange. For noncomposite beams this is S_{xc} of the noncomposite section. (in.^{3})

h = depth between centerlines of flanges (in.)

C_b = moment gradient modifier, defined as follows:

For $\frac{M_{mid}}{M_2} > 1$, or $M_2 = 0$:

$$C_b = 1.0 \qquad \text{(Eq. A6.3.3-6)}$$

Otherwise:

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \qquad \text{(Eq. A6.3.3-7)}$$

Where:

M_{mid} = moment due to factored vertical loads at the middle of the unbraced length (k-in.). This is found using the moment that produces the largest compression at the point under consideration. If this point is never in compression, the smallest tension value is used. M_{mid} is positive for compression values, and negative for tension values.

M_2 = largest bending moment due to factored vertical loads at either end of the unbraced length of the compression flange (k-in.). This is found using the moment that produces the largest compression and taken as positive at the point under consideration. If the moment is zero or causes tension in the flange under consideration at both ends of the unbraced length, M_2 is taken as zero.

M_0 = moment due to factored vertical loads at the brace point opposite the point corresponding to M_2 (k-in.), calculated similarly to M_2 except that the value is the largest value causing compression taken as positive or the smallest value causing tension taken as negative if the point under consideration is never in compression.

M_1 = moment at the brace point opposite to the one corresponding to M_2, calculated as the intercept of the most critical assumed linear moment variation passing through M_2 and either M_{mid} or M_0, whichever produces the smallest value of C_b (k-in.). M_1 may be determined as follows:

When the variation of the moment along the entire length between the brace points is concave:

$M_1 = M_0$ (Eq. A6.3.3-11)

Otherwise:

$M_1 = 2M_{mid} - M_2 \geq M_0$ (Eq. A6.3.3-12)

F_{cr} = elastic lateral-torsional buckling stress (ksi)

$$= \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h} \left(\frac{L_b}{r_t}\right)^2} \qquad \text{(Eq. A6.3.3-8)}$$

<u>Check Continuously Braced Tension Flanges (Appendix A)</u> <u>(A6.1.4)</u>

At the strength limit state, the following requirement shall be satisfied:

$M_u = \phi_f R_{pt} M_{yt}$ (Eq. A6.1.4-1)

Where all variables are calculated similar to above.

Check Strength Limit State (Shear) (6.10.9)

The web of the steel section is assumed to resist all shear for the composite section. Webs shall satisfy the following shear requirement:

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

Where:

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

V_u = factored Strength I shear loads (kips)

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

$$\frac{2Dt_w}{(b_{fc}t_{fc} + b_{ft}t_{ft})} \leq 2.5$$ (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

Where:

C = ratio of shear buckling resistance to shear yield strength

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

C = 1.0 (Eq. 6.10.9.3.2-4)

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

$$C = \frac{1.12}{(D/t_w)}\sqrt{\frac{Ek}{F_{yw}}}$$ (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)

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

Slab Reinforcement Checks:

Slab reinforcement shall be checked for stresses at Fatigue I limit state. If the standard IDOT longitudinal reinforcement over piers does not meet these requirements, additional capacity shall be added by increasing the size of the steel section, not by adding reinforcement. Crack control need not be checked if the provisions of 6.10.1.7 are met. The provisions of 6.10.1.7 require a longitudinal reinforcement area equal to 1% of the deck cross-sectional area in negative moment regions; standard IDOT longitudinal reinforcement over piers meets these requirements. Reinforcement shall extend one development length beyond the point of dead load contraflexure. Additional shear studs at points of dead load contraflexure are not required.

<u>Check Fatigue I Limit State</u> <u>(5.5.3)</u>

For fatigue considerations, concrete members shall satisfy:

$\gamma(\Delta f) \leq (\Delta F)_{TH}$ (Eq. 5.5.3.1-1)

Where:

γ = load factor specified in Table 3.4.1-1 for the Fatigue I load combination

= 1.5

(Δf) = live load stress range due to fatigue truck (ksi)

$$= \frac{\left| M^{+}_{FATIGUE\,I} - M^{-}_{FATIGUE\,I} \right|}{S_{c,cr}}$$

$$(\Delta F)_{TH} = 24 - 0.33 f_{min} \quad \text{(Eq. 5.5.3.2-1)}$$

Where:

f_{min} = algebraic minimum stress level, tension positive, compression negative (ksi). The minimum stress shall be taken as that from Service I factored dead loads (DC2 with the inclusion of DW at the discretion of the designer), combined with that produced by $M^{+}_{FATIGUE\,I}$.

LRFD Composite Design Example: Two-Span Plate Girder

Design Stresses

f'_c	=	3.5 ksi
f_y	=	60 ksi (reinforcement)
$F_y = F_{yw} = F_{yc} = F_{yt}$	=	50 ksi (structural steel)

Bridge Data

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.

Skew: 20°

Diaphragm Placement:

	Span 1	Span 2
Location 1:	3.33 ft.	4.5 ft.
Location 2:	22.96 ft.	15.85 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

Determine Trial Sections

Try the following flange and web sections for the positive moment region:

$D = 42$ in.

$t_w = 0.4375$ in.

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

$t_{bf} = 0.875$ in.

$t_{tf} = 0.75$ in.

Note that the minimum web thickness has been chosen and the minimum flange size has been chosen for the top flange.

Try the following flange and web sections for the negative moment region:

D	=	42 in.
t_w	=	0.5 in.
$b_{bf} = b_{tf}$	=	12 in.
t_{bf}	=	2.5 in.
t_{tf}	=	2.0 in.

The points of dead load contraflexure has been determined to be approximately 67 ft. into span one and 31.75 ft. into span two. Section changes will occur at these points.

Determine Section Properties

Simplifying values for terms involving K_g from Table 4.6.2.2.1-2 will be used in lieu of the exact values. Therefore, K_g will not be calculated.

Positive Moment Region Noncomposite Section Properties:

Calculate $c_{b(nc)}$ and $c_{t(nc)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Top Flange	9.0	43.25	389.25
Web	18.38	21.875	402.06
Bottom Flange	10.5	0.4375	4.59
Total	37.88		795.91

$c_{b(nc)} = Y_{b(nc)} =$ distance from extreme bottom fiber of steel beam to neutral axis (in.)

$$= \frac{\sum(A * Y_b)}{\sum A} = \frac{795.91\text{ in.}^3}{37.88\text{ in.}^2} = 21.01\text{ in.}$$

$c_{t(nc)} =$ distance from extreme top fiber of steel beam to neutral axis (in.)

$$= h_{(nc)} - Y_{b(nc)}$$

$h_{(nc)}$ = depth of noncomposite section (in.)

= $t_{tf} + D + t_{bf}$

= 0.75 in. + 42 in. + 0.875 in.

= 43.625 in.

$c_{t(nc)}$ = 43.625 in. – 21.01 in.

= 22.615 in.

Calculate $I_{(nc)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Top Flange	0.42	9.0	22.24	4451.98
Web	2701.13	18.38	0.87	2714.88
Bottom Flange	0.67	10.5	-20.57	4444.56
Total				11611.42

$$S_{b(nc)} = \frac{I_{(nc)}}{c_{b(nc)}} = \frac{11611.42 \text{ in.}^4}{21.01 \text{ in.}} = 552.66 \text{ in.}^3$$

$$S_{t(nc)} = \frac{I_{(nc)}}{c_{t(nc)}} = \frac{11611.42 \text{ in.}^4}{22.615 \text{ in.}} = 513.44 \text{ in.}^3$$

<u>Positive Moment Region Composite Section Properties (n):</u>

The effective flange width is equal to the beam spacing of 87 in. (4.6.2.6.1)

For f'_c = 3.5, a value of n = 9 may be used for the modular ratio. (C6.10.1.1.1b)

$$\text{Transformed Effective Flange Width (n)} = \frac{87 \text{ in.}}{9} = 9.67 \text{ in.}$$

Calculate $c_{b(c,n)}$ and $c_{t(c,n)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Slab	77.36	48.38	3742.68
Top Flange	9.0	43.25	389.25
Web	18.38	21.88	402.15

Bottom Flange	10.5	0.44	4.62
Total	115.24		4538.70

$c_{b(c,n)}$ = $Y_{b(c,n)}$ = distance from extreme bottom fiber of steel beam to composite (n) neutral axis (in.)

$$= \frac{\sum(A * Y_b)}{\sum A} = \frac{4538.70 \text{ in.}^3}{115.24 \text{ in.}^2} = 39.38 \text{ in.}$$

$c_{tslab(c,n)}$ = distance from extreme top fiber of slab to composite (n) neutral axis (in.)

$= h_{(c)} - Y_{b(c,n)}$

$h_{(c)}$ = depth of composite section (in.)

$= t_{slab} + t_{fillet} + t_{tf} + D + t_{bf}$

$= 8 \text{ in.} + 0.75 \text{ in.} + 0.75 \text{ in.} + 42 \text{ in.} + 0.875 \text{ in.}$

$= 52.375 \text{ in.}$

$c_{tslab(c,n)} = 52.375 \text{ in.} - 39.38 \text{ in.}$

$= 13.00 \text{ in.}$

$c_{tbm(c,n)}$ = distance from extreme top fiber of steel beam to composite (n) neutral axis (in.)

$$= \left| c_{tslab(c,n)} - t_{slab} - t_{fillet} \right|$$

$$= \left| 13.00 \text{ in.} - 8 \text{ in.} - 0.75 \text{ in.} \right|$$

$= 4.25 \text{ in.}$

Calculate $I_{(c,n)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Slab	412.59	77.36	9.00	6678.75
Top Flange	0.42	9.0	3.87	135.91
Web	2701.13	18.38	-17.51	8330.00
Bottom Flange	0.67	10.5	-38.94	15922.07
Total				31066.73

$$S_{b(c,n)} = \frac{I_{(c,n)}}{c_{b(c,n)}} = \frac{31066.73 \text{ in.}^4}{39.38 \text{ in.}} = 788.90 \text{ in.}^3$$

$$S_{tslab(c,n)} = \frac{I_{(c,n)}}{c_{tslab(c,n)}} = \frac{31066.73 \text{ in.}^4}{13.00 \text{ in.}} = 2389.75 \text{ in.}^3$$

$$S_{tbm(c,n)} = \frac{I_{(c,n)}}{c_{tbm(c,n)}} = \frac{31066.73 \text{ in.}^4}{4.25 \text{ in.}} = 7309.82 \text{ in.}^3$$

Positive Moment Region Composite Section Properties (3n):

Transformed Effective Flange Width (3n) = $\frac{87 \text{ in.}}{3(9)}$ = 3.22 in.

Calculate $c_{b(c,n)}$ and $c_{t(c,n)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Slab	25.76	48.38	1246.27
Top Flange	9.0	43.25	389.25
Web	18.38	21.88	402.15
Bottom Flange	10.5	0.44	4.62
Total	63.64		2042.29

$c_{b(c,3n)}$ = $Y_{b(c,3n)}$ = distance from extreme bottom fiber of steel beam to composite (3n) neutral axis (in.)

$$= \frac{\sum(A * Y_b)}{\sum A} = \frac{2042.29 \text{ in.}^3}{63.64 \text{ in.}^2} = 32.09 \text{ in.}$$

$c_{tslab(c,3n)}$ = distance from extreme top fiber of slab to composite (3n) neutral axis (in.)

= $h_{(c)} - Y_{b(c,3n)}$

= 52.375 in. – 32.09 in.

= 20.29 in.

$c_{tbm(c,n)}$ = distance from extreme top fiber of steel beam to composite (n) neutral axis (in.)

$$= \left| c_{tslab(c,3n)} - t_{slab} - t_{fillet} \right|$$

$$= \left| 20.29 \text{ in.} - 8 \text{ in.} - 0.75 \text{ in.} \right|$$

= 11.54 in.

Calculate $I_{(c,3n)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Slab	137.39	25.76	16.29	6973.17
Top Flange	0.42	9.0	11.17	1123.34
Web	2701.13	18.38	-10.21	4617.14
Bottom Flange	0.67	10.5	-31.65	10518.76
Total				23232.41

$$S_{b(c,3n)} = \frac{I_{(c,3n)}}{c_{b(c,3n)}} = \frac{23232.41 \text{ in.}^4}{32.09 \text{ in.}} = 723.98 \text{ in.}^3$$

$$S_{tslab(c,3n)} = \frac{I_{(c,3n)}}{c_{tslab(c,3n)}} = \frac{23232.41 \text{ in.}^4}{20.29 \text{ in.}} = 1145.02 \text{ in.}^3$$

$$S_{tbm(c,3n)} = \frac{I_{(c,3n)}}{c_{tbm(c,3n)}} = \frac{23232.41 \text{ in.}^4}{11.54 \text{ in.}} = 2013.21 \text{ in.}^3$$

<u>Negative Moment Region Noncomposite Section Properties:</u>

Calculate $c_{b(nc)}$ and $c_{t(nc)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Top Flange	24.0	45.5	1092.0
Web	21.0	23.5	493.5
Bottom Flange	30.0	1.25	37.5
Total	75.0		1623.0

$c_{b(nc)} = Y_{b(nc)} =$ distance from extreme bottom fiber of steel beam to neutral axis (in.)

$$= \frac{\sum(A * Y_b)}{\sum A} = \frac{1623.0 \text{ in.}^3}{75.0 \text{ in.}^2} = 21.64 \text{ in.}$$

$c_{t(nc)} =$ distance from extreme top fiber of steel beam to neutral axis (in.)

$= h_{(nc)} - Y_{b(nc)}$

$h_{(nc)} =$ depth of noncomposite section (in.)

$= t_{tf} + D + t_{bf}$

= 2.0 in. + 42 in. + 2.5 in.

= 46.5 in.

$c_{t(nc)}$ = 46.5 in. – 21.64 in.

= 24.86 in.

Calculate $I_{(nc)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Top Flange	8.0	24.0	23.86	13671.19
Web	3087.0	21.0	1.86	3159.65
Bottom Flange	15.63	30.0	-20.39	12488.19
Total				29319.03

$$S_{b(nc)} = \frac{I_{(nc)}}{c_{b(nc)}} = \frac{29319.03 \text{ in.}^4}{21.64 \text{ in.}} = 1354.85 \text{ in.}^3$$

$$S_{t(nc)} = \frac{I_{(nc)}}{c_{t(nc)}} = \frac{29319.03 \text{ in.}^4}{24.86 \text{ in.}} = 1179.37 \text{ in.}^3$$

Negative Moment Region Composite Section Properties (n):

The effective flange width is equal to the beam spacing of 87 in. (4.6.2.6.1)

$$\text{Transformed Effective Flange Width (n)} = \frac{87 \text{ in.}}{9} = 9.67 \text{ in.}$$

Calculate $c_{b(c,n)}$ and $c_{t(c,n)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Slab	77.36	51.25	3964.70
Top Flange	24.0	45.50	1092.0
Web	21.0	23.50	493.5
Bottom Flange	30.0	1.25	37.5
Total	152.36		5587.7

$c_{b(c,n)}$ = $Y_{b(c,n)}$ = distance from extreme bottom fiber of steel beam to composite (n) neutral axis (in.)

$$= \frac{\sum (A * Y_b)}{\sum A} = \frac{5587.7 \text{ in.}^3}{152.36 \text{ in.}^2} = 36.67 \text{ in.}$$

$c_{tslab(c,n)}$ = distance from extreme top fiber of slab to composite (n) neutral axis (in.)

= $h_{(c)} - Y_{b(c,n)}$

$h_{(c)}$ = depth of composite section (in.)

= $t_{slab} + t_{fillet} + t_{tf} + D + t_{bf}$

= 8 in. + 0.75 in. + 2.0 in. + 42 in. + 2.5 in.

= 55.25 in.

$c_{tslab(c,n)}$ = 55.25 in. – 36.67 in.

= 18.58 in.

$c_{tbm(c,n)}$ = distance from extreme top fiber of steel beam to composite (n) neutral axis (in.)

$$= \left| c_{tslab(c,n)} - t_{slab} - t_{fillet} \right|$$

$$= \left| 18.58 \text{ in.} - 8 \text{ in.} - 0.75 \text{ in.} \right|$$

= 9.83 in.

Calculate $I_{(c,n)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Slab	412.59	77.36	14.58	16857.50
Top Flange	8.0	24.0	8.83	1879.25
Web	3087.0	21.0	-13.17	6729.43
Bottom Flange	15.63	30.0	-35.42	37652.92
Total				63119.10

$$S_{b(c,n)} = \frac{I_{(c,n)}}{c_{b(c,n)}} = \frac{63119.10 \text{ in.}^4}{36.67 \text{ in.}} = 1721.27 \text{ in.}^3$$

$$S_{tslab(c,n)} = \frac{I_{(c,n)}}{c_{tslab(c,n)}} = \frac{63119.10 \text{ in.}^4}{18.58 \text{ in.}} = 3397.15 \text{ in.}^3$$

$$S_{tbm(c,n)} = \frac{I_{(c,n)}}{c_{tbm(c,n)}} = \frac{63119.10 \text{ in.}^4}{9.83 \text{ in.}} = 6421.07 \text{ in.}^3$$

<u>Negative Moment Region Composite Section Properties (3n):</u>

The effective flange width is equal to the beam spacing of 87 in. (4.6.2.6.1)

$$\text{Transformed Effective Flange Width (n)} = \frac{87 \text{ in.}}{3(9)} = 3.22 \text{ in.}$$

Calculate $c_{b(c,n)}$ and $c_{t(c,n)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Slab	25.76	51.25	1320.20
Top Flange	24.0	45.5	1092.0
Web	21.0	23.50	493.5
Bottom Flange	30.0	1.25	37.5
Total	100.76		2943.2

$c_{b(c,3n)}$ = $Y_{b(c,n)}$ = distance from extreme bottom fiber of steel beam to composite (3n) neutral axis (in.)

$$= \frac{\sum(A * Y_b)}{\sum A} = \frac{2943.2 \text{ in.}^3}{100.76 \text{ in.}^2} = 29.21 \text{ in.}$$

$c_{tslab(c,3n)}$ = distance from extreme top fiber of slab to composite (n) neutral axis (in.)

= $h_{(c)} - Y_{b(c,n)}$

$h_{(c)}$ = depth of composite section (in.)

= $t_{slab} + t_{fillet} + t_{tf} + D + t_{bf}$

= 8 in. + 0.75 in. + 2.0 in. + 42 in. + 2.5 in.

= 55.25 in.

$c_{tslab(c,3n)}$ = 55.25 in. – 29.21 in.

= 26.04 in.

$c_{tbm(c,n)}$ = distance from extreme top fiber of steel beam to composite (3n) neutral axis (in.)

$= \left| c_{tslab(c,3n)} - t_{slab} - t_{fillet} \right|$

$= \left| 26.04 \text{ in.} - 8 \text{ in.} - 0.75 \text{ in.} \right|$

= 17.29 in.

Calculate $I_{(c,3n)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Slab	137.39	25.76	22.04	12650.6
Top Flange	8.0	24.0	16.29	6376.7
Web	3087.0	21.0	-5.71	3771.7
Bottom Flange	15.63	30.0	-27.96	23468.5
Total				46267.5

$$S_{b(c,3n)} = \frac{I_{(c,3n)}}{c_{b(c,3n)}} = \frac{46267.5 \text{ in.}^4}{29.21 \text{ in.}} = 1583.96 \text{ in.}^3$$

$$S_{tslab(c,3n)} = \frac{I_{(c,3n)}}{c_{tslab(c,3n)}} = \frac{46267.5 \text{ in.}^4}{26.04 \text{ in.}} = 1776.8 \text{ in.}^3$$

$$S_{tbm(c,3n)} = \frac{I_{(c,3n)}}{c_{tbm(c,3n)}} = \frac{46267.5 \text{ in.}^4}{17.29 \text{ in.}} = 2676.0 \text{ in.}^3$$

<u>Negative Moment Region Composite Section Properties (Cracked Section):</u>

Calculate $c_{b(c,cr)}$ and $c_{t(c,cr)}$:

	A (in.^{2})	Y_b (in.)	A * Y_b (in.^{3})
Top Slab Reinforcement	5.44	51.78	281.68
Bottom Slab Reinforcement	2.17	49.19	106.74
Top Flange	24.0	45.5	1092.0
Web	21.0	23.5	493.5
Bottom Flange	30.0	1.25	37.5
Total	82.61		2011.42

$c_{b(c,cr)}$ = $Y_{b(c,cr)}$ = distance from extreme bottom fiber of steel beam to composite (cracked section) neutral axis (in.)

$$= \frac{\sum(A * Y_b)}{\sum A} = \frac{2011.42 \text{ in.}^3}{82.61 \text{ in.}^2} = 24.35 \text{ in.}$$

$c_{trt(c,cr)}$ = $h_{(c)}$ – top of slab clear cover – transverse bar diameter - $c_{b(c,cr)}$

= 55.25 in. – 2 in. (conservative) – 0.625 in. - 24.35 in.

= 28.28 in.

$$c_{tbm(c,cr)} = \left|\left(h_{(c)} - t_{slab} - t_{fillet}\right) - c_{b(c,cr)}\right|$$

$$= \left|(55.25 \text{ in.} - 8 \text{ in.} - 0.75 \text{ in.}) - 24.35 \text{ in.}\right|$$

$$= 22.15 \text{ in.}$$

Calculate $I_{(c,cr)}$:

	I_o (in.^{4})	A (in.^{2})	d (in.)	$I_o + Ad^2$ (in.^{4})
Top Slab Reinforcement	0.17	5.44	27.41	4085.4
Bottom Slab Reinforcement	0.05	2.17	24.84	1339.00
Top Flange	8.0	24.0	21.15	10743.74
Web	3087.0	21.0	-0.85	3102.17
Bottom Flange	15.63	30.0	-23.10	16023.93
Total				35290.99

$$S_{b(c,cr)} = \frac{I_{(c,cr)}}{c_{b(c,cr)}} = \frac{35290.99 \text{ in.}^4}{24.35 \text{ in.}} = 1449.32 \text{ in.}^3$$

$$S_{tslab(c,cr)} = \frac{I_{(c,cr)}}{c_{tslab(c,cr)}} = \frac{35290.99 \text{ in.}^4}{28.28 \text{ in.}} = 1247.91 \text{ in.}^3$$

$$S_{tbm(c,cr)} = \frac{I_{(c,cr)}}{c_{tbm(c,cr)}} = \frac{35290.99 \text{ in.}^4}{22.15 \text{ in.}} = 1593.27 \text{ in.}^3$$

Distribution Factors *(4.6.2.2.2)*

Moment

$$g_1 = 0.06 + \left(\frac{S}{14.0}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$ (Table 4.6.2.2.2b-1)

Where:

S = 7.25 ft.

L = 98.75 ft.

$$\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1} = 1.02$$ (Table 4.6.2.2.1-2)

$$g_1 = 0.06 + \left(\frac{7.25\text{ ft.}}{14.0}\right)^{0.4}\left(\frac{7.25\text{ ft.}}{98.75\text{ ft.}}\right)^{0.3}(1.02)$$

$$= 0.418$$

The distribution factor for multiple-lanes loaded, g_m, is calculated as:

$$g_m = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$ (Table 4.6.2.2.2b-1)

$$= 0.075 + \left(\frac{7.25\text{ ft.}}{9.5}\right)^{0.6}\left(\frac{7.25\text{ ft.}}{98.75\text{ ft.}}\right)^{0.2}(1.02)$$

$$= 0.589$$

The distribution factor for multiple-lanes loaded controls.

Moment (fatigue loading)

$$g_1\text{ (fatigue)} = \frac{g_1}{m} = \frac{0.418}{1.2} = 0.348$$

Shear and Reaction

$$g_1 = 0.36 + \frac{S}{25.0} = 0.36 + \frac{7.25\text{ ft.}}{25.0}$$ (Table 4.6.2.2.3a-1)

$$= 0.650$$

$$g_m = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0} = 0.2 + \left(\frac{7.25\text{ ft.}}{12}\right) - \left(\frac{7.25\text{ ft.}}{35}\right)^{2.0}$$ (Table 4.6.2.2.3a-1)

$$= 0.761$$

The distribution factor for multiple-lanes loaded controls.

$$\text{Skew correction} = 1+0.2\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan(\theta)$$

$$= 1+0.2(0.97)\tan\left(20^o\right)$$

$$= 1.07$$

Note that the term $\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}$ is taken as 0.97. This is found in Table 4.6.2.2.1-2.

<u>Deflection</u>

$$\text{g (deflection)} = m\left(\frac{N_L}{N_b}\right) = 0.85\left(\frac{3}{6}\right) = 0.425$$

Dead Loads

<u>Positive Moment Region</u>

DC1:

Beam:

$$\left(0.49\ \frac{k}{ft^3}\right)\left[(12\ \text{in.})(0.75\ \text{in.})+(42\ \text{in.})(0.4375\ \text{in.})+(12\ \text{in.})(0.875\ \text{in.})\right]\left(\frac{1\ \text{ft.}^2}{144\ \text{in.}^2}\right)(1.15)$$

$$= 0.148\ \text{k/ft.}$$

Slab: $$\left(0.15\ \text{k/ft.}^3\right)\left(\frac{8\ \text{in}}{12\ \text{in./ft.}}\right)(7.25\ \text{ft.}) = 0.725\ \text{k/ft.}$$

Fillet: $$\left(0.15\ \text{k/ft.}^3\right)\left(\frac{0.75\ \text{in.}}{12\ \text{in./ft.}}\right)\left(\frac{12\ \text{in.}}{12\ \text{in./ft.}}\right) = 0.009\ \text{k/ft.}$$

Total: $= 0.882$ k/ft.

<u>Composite</u>

DC2:

Parapets: $$\frac{(0.45\ \text{k/ft.})(2\ \text{parapets})}{6\ \text{beams}} = 0.150\ \text{k/ft.}$$

DW:

FWS: $\left(0.050 \text{ k/ft.}^2\right)\left(7.25 \text{ ft.}\right)$ = 0.363 k/ft.

Negative Moment Region

DC1:

Beam: $\left(0.49 \dfrac{\text{k}}{\text{ft}^3}\right)\left[\left(12 \text{ in.}\right)\left(2.0 \text{ in.}\right)+\left(42 \text{ in.}\right)\left(0.5 \text{ in.}\right)+\left(12 \text{ in.}\right)\left(2.5 \text{ in.}\right)\right]\left(\dfrac{1 \text{ ft.}^2}{144 \text{ in.}^2}\right)\left(1.15\right)$

= 0.293 k/ft.

Slab: $\left(0.15 \text{ k/ft.}^3\right)\left(\dfrac{8 \text{ in}}{12 \text{ in./ft.}}\right)\left(7.25 \text{ ft.}\right)$ = 0.725 k/ft.

Fillet: $\left(0.15 \text{ k/ft.}^3\right)\left(\dfrac{0.75 \text{ in.}}{12 \text{ in./ft.}}\right)\left(\dfrac{12 \text{ in.}}{12 \text{ in./ft.}}\right)$ = 0.009 k/ft.

Total: = 1.027 k/ft.

Composite

DC2:

Parapets: $\dfrac{\left(0.45 \text{ k/ft.}\right)\left(2 \text{ parapets}\right)}{6 \text{ beams}}$ = 0.150 k/ft.

DW:

FWS: $\left(0.050 \text{ k/ft.}^2\right)\left(7.25 \text{ ft.}\right)$ = 0.363 k/ft.

Positive Moment Region: Calculate Moments and Shears

Using the bridge and section data shown above, the following moments and shears have been calculated:

At point 0.375L in span 1 (near the point of maximum positive moment) and 0.625L in span 2, the unfactored distributed moments are:

M_{DC1} = 485.3 k-ft.
M_{DC2} = 86.8 k-ft.
M_{DW} = 209.8 k-ft.

M_{LL+IM} = 1213.9 k-ft.

The maximum unfactored construction loading occurs near point 0.35L in span 1 and 0.65L in span 2:

M_{DC}^{CONST} = 517.0 k-ft.

M_{LL+IM}^{CONST} = 106.1 k-ft.

As per Bridge Manual 3.3.22, a diaphragm (diaphragm location 3) has been placed at the point of maximum Strength I moment. The maximum construction loading occurs between this diaphragm and diaphragm that precedes it. The corresponding construction loading factored moments for the diaphragm locations (brace points) and midpoint for these diaphragms are:

$M_{STRENGTH\,I}^{BRACE\,POINT\,1}$ = 752.4 k-ft., corresponding to diaphragm location 2

$M_{STRENGTH\,I}^{MIDPOINT}$ = 824.7 k-ft., corresponding to the midpoint of diaphragm locations 2 and 3

$M_{STRENGTH\,I}^{BRACE\,POINT\,2}$ = 823.0 k-ft., corresponding to diaphragm location 3

For this bridge, the maximum factored Fatigue loading range occurs near point 0.53L in span 1 and 0.47L in span 2. For simplicity, assume that a diaphragm connection point and shear stud weldment are present at this location. This is conservative and will allow us to check all detail categories using one fatigue range, as opposed to calculating multiple fatigue ranges for different aspects of design.

$M_{FATIGUE\,I}^{+}$ = 535.5 k-ft. $M_{FATIGUE\,I}^{-}$ = -235.2 k-ft.

$M_{FATIGUE\,II}^{+}$ = 267.8 k-ft. $M_{FATIGUE\,II}^{-}$ = -117.6 k-ft.

These values contain impact and include γ of 1.5 for Fatigue I and 0.75 for Fatigue II.

The factored, distributed moments for Strength Limit State, Service Limit State, and Constructibility Limit State are found to be:

$M_{STRENGTH\ I}$ = 3154.2 k-ft.

$M_{SERVICE\ II}$ = 2360.0 k-ft.

$M_{STRENGTH\ I}^{CONST}$ = 831.9 k-ft.

The maximum factored shears for the positive moment section have been found to be:

V_{CONST} = 49.9 kips (@ abutment)

$V_{STRENGTH\ I}$ = 219.8 kips (@ abutment)

Positive Moment Region: Check Cross-Section Proportion Limits (6.10.2)

<u>Check Web Proportions</u>

$$\frac{D}{t_w} \leq 150 \qquad \text{(Eq. 6.10.2.1.1-1)}$$

$$\frac{D}{t_w} = \frac{42\text{ in.}}{0.4375\text{ in.}} = 96 \leq 150 \qquad \text{O.K.}$$

<u>Check Flange Proportions</u>

i) $$\frac{b_f}{2t_f} \leq 12.0 \qquad \text{(Eq. 6.10.2.2-1)}$$

$$\frac{b_{tf}}{2t_{tf}} = \frac{12\text{ in.}}{2(0.75\text{ in.})} = 8 \qquad \frac{b_{bf}}{2t_{bf}} = \frac{12\text{ in.}}{2(0.875\text{ in.})} = 6.86$$

8 and 6.86 < 12.0 O.K.

ii) $$b_f \geq \frac{D}{6} \qquad \text{(Eq. 6.10.2.2-1)}$$

b_f = 12 in.

$$\frac{D}{6} = \frac{42\text{ in.}}{6} = 7\text{ in.}$$

12 in. > 7 in. O.K.

iii) $t_f \geq 1.1\, t_w$ (Eq. 6.10.2.2-3)

t_f = 0.75 in. or 0.875 in.

$1.1t_w$ = 1.1(0.4375 in.) = 0.48 in.

0.75 in. and 0.875 in. > 0.48 in. O.K.

iv) $0.1 \leq \dfrac{I_{yc}}{I_{yt}} \leq 10$ (Eq. 6.10.2.2-4)

$$I_{yc} = \frac{1}{12} t_{tf} b_{tf}{}^3 = \frac{1}{12}(0.75\ \text{in.})(12\ \text{in.})^3 = 108\ \text{in.}^4$$

$$I_{yt} = \frac{1}{12} t_{bf} b_{bf}{}^3 = \frac{1}{12}(0.875\ \text{in.})(12\ \text{in.})^3 = 126\ \text{in.}^4$$

$$\frac{I_{yc}}{I_{yt}} = \frac{108\ \text{in.}^4}{126\ \text{in.}^4} = 0.86$$

$0.1 \leq 0.86 \leq 10$ O.K.

Positive Moment Region: Check Constructibility *(6.10.3)*

As the section is noncomposite, this section could be checked using the provisions of Appendix A. For explanatory purposes, Chapter 6 will be used in this check. Examples of usage of Appendix A will be found in the negative moment design section of this design guide.

<u>Check Discretely Braced Flanges in Compression</u> <u>(6.10.3.2.1)</u>

i) $f_{bu} + f_l \leq \phi_f R_h F_{yc}$ (Eq. 6.10.3.2.1-1)

$$f_{bu} = \frac{M^{CONST}_{STRENGTHI}}{S_{nc}}$$

$$= \frac{831.9\ \text{k} - \text{ft.}\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{513.44\ \text{in.}^3}$$

$$= 19.44\ \text{ksi}$$

f_l = assume 0 ksi

ϕ_f = 1.00 (6.5.4.2)

R_h = 1.0 (6.10.1.10.1)

F_{yc} = 50 ksi

$f_{bu} + f_l$ = 19.44 ksi < (1.00)(1.0)(50 ksi) = 50 ksi O.K.

ii) $f_{bu} + \frac{1}{3} f_l \leq \phi_f (F_{nc})$ (Eq. 6.10.3.2.1-2)

F_{nc} is taken as the lesser $F_{nc(FLB)}$ and $F_{nc(LTB)}$ (6.10.8.2.1)

Determine $F_{nc(FLB)}$: (6.10.8.2.2)

$$\lambda_f = \frac{b_{fc}}{2t_{fc}} = \frac{12.0 \text{ in.}}{2(0.75 \text{ in.})} = 8$$ (Eq. 6.10.8.2.2-3)

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}} = 0.38\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 9.2$$ (Eq. 6.10.8.2.2-5)

$\lambda_f \leq \lambda_{pf}$, therefore:

$F_{nc(FLB)} = R_b R_h F_{yc}$ (Eq. 6.10.8.2.2-1)

Where:

R_b = 1.0 (Eq. 6.10.1.10.2)

R_h = 1.0 (Eq. 6.10.1.10.1)

F_{yc} = 50 ksi

$F_{nc(FLB)}$ = (1.0)(1.0)(50 ksi) = 50 ksi

Determine $F_{nc(LTB)}$:

$$L_b = (37 \text{ ft.} - 22.96 \text{ ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right) = 168.5 \text{ in.}$$ (6.10.8.2.3)

$$L_p = 1.0\, r_t \sqrt{\frac{E}{F_{yc}}}$$ (Eq. 6.10.8.2.3-4)

Where:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c\ t_w}{b_{fc}\ t_{fc}}\right)}}$$ (Eq. 6.10.8.2.3-9)

Where:

D_c = $c_{t(nc)} - t_{tf}$

= 22.62 in. – 0.75 in.

= 21.87 in.

$$r_t = \frac{12\text{ in.}}{\sqrt{12\left(1+\frac{1}{3}\frac{(21.87\text{ in.})(0.4375\text{ in.})}{(12.0\text{ in.})(0.75\text{ in.})}\right)}}$$

= 2.98 in.

$$L_p = 1.0\,(2.98\text{ in.})\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 71.77\text{ in.}$$

$$L_r = \pi\, r_t \sqrt{\frac{E}{F_{yr}}}$$ (Eq. 6.10.8.2.3-5)

Where:

F_{yr} = lesser of F_{yw} and $0.7F_{yc}$ but not less than $0.5F_{yc}$

= 0.7(50 ksi)

= 35 ksi

$$L_r = \pi\,(2.98\text{ in.})\sqrt{\frac{29000\text{ ksi}}{35\text{ ksi}}}$$

= 269.5 in.

$L_p < L_b < L_r$, therefore:

$$F_{nc(LTB)} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h\ F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_b\ R_h\ F_{yc} \le R_b\ R_h\ F_{yc}$$

(Eq. 6.10.8.2.3-2)

Where:

F_{yr} = 35 ksi

R_h = 1.0

F_{yc} = 50 ksi

L_b = 168.5 in.

L_p = 71.77 in.

L_r = 269.5 in.

For $\frac{f_{mid}}{f_2} > 1$, or $f_2 = 0$, C_b = 1.0 (Eq. 6.10.8.2.3-6)

As there are no composite loads and no change in section over the braced length, $\frac{f_{mid}}{f_2} = \frac{M_{mid}}{M_2}$. As M_{mid} is greater than both moments at bracepoints (824.7 k-ft. vs. 823.0 k-ft. and 752.4 k-ft.), $\frac{M_{mid}}{M_2} > 1$ and $C_b = 1.0$.

R_b = 1.0

$$F_{nc(LTB)} = (1.0)\left[1-\left(1-\frac{35\text{ ksi}}{(1.0)(50\text{ ksi})}\right)\left(\frac{168.5\text{ in.}-71.77\text{ in.}}{269.5\text{ in.}-71.77\text{ in.}}\right)\right](1.0)(1.0)(50\text{ ksi})$$

= 42.66 ksi

$R_bR_hF_{yc}$ = (1.0)(1.0)(50 ksi) = 50 ksi

$F_{nc(LTB)}$ = 42.66 ksi

F_{nc} = 42.66 ksi

$$f_{bu} + \frac{1}{3} f_l = 19.44\text{ ksi} + \frac{1}{3}(0\text{ ksi}) = 19.44\text{ ksi}$$

$\phi_f F_{nc}$ = 1.00(42.66 ksi) = 42.66 ksi

19.44 ksi < 42.66 ksi O.K.

iii) $f_{bu} \leq \phi_f F_{crw}$ (Eq. 6.10.3.2.1-3)

Equation 6.10.3.2.1-3 need not be checked for non-slender webs. Check slenderness:

$$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$$ (Eq. 6.10.6.2.3-1)

$$\frac{2D_c}{t_w} = \frac{2(21.87\text{ in.})}{0.4375\text{ in.}} = 99.98$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 137.27$$

99.98 < 137.27, the section is not slender and Eq. 6.10.3.2.1-3 need not be checked.

<u>Check Discretely Braced Flanges in Tension</u> (6.10.3.2.2)

$f_{bu} + f_l \leq \phi_f R_h F_{yt}$ (Eq. 6.10.3.2.2-1)

$$f_{bu} = \frac{M_{STRENGTHI}^{CONST}}{S_{nc}} = \frac{831.9\text{ k} - \text{ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{552.66\text{ in.}^3} = 18.06\text{ ksi}$$

18.06 ksi + 0 ksi = 18.06 ksi < (1.00)(1.0)(50 ksi) = 50 ksi O.K.

<u>Check Shear</u> (6.10.3.3)

As V_{CONST} = 49.9 k < $V_{STRENGTH\ I}$ = 219.8 k, and both loads are applied to the same web section, construction shear loading will not control the design by inspection.

Positive Moment Region: Check Service Limit State *(6.10.4)*

<u>Check Permanent Deformations</u> <u>(6.10.4.2)</u>

The top steel flange shall satisfy:

i) $f_f \leq 0.95(R_h)(F_{yf})$ (Eq. 6.10.4.2.2-1)

Where:

$$f_f = \frac{M_{DC1}}{S_{nc}} + \frac{M_{DC2} + M_{DW}}{S_{n=27}} + \frac{1.3\left(M_{LL+IM}\right)}{S_{n=9}} \qquad \text{(Eq. D6.2.2-1)}$$

Where:

M_{DC1} = 485.3 k-ft.

M_{DC2} = 86.8 k-ft.

M_{DW} = 209.8 k-ft.

M_{LL+IM} = 1213.9 k-ft.

$S_{n=9}$ = 7309.82 in.^{3}

$S_{n=27}$ = 2013.21 in.^{3}

S_{nc} = 513.44 in.^{3}

$$f_f = \frac{485.3\ k-ft.\left(\frac{12\ in.}{ft.}\right)}{513.44\ in.^3} + \frac{\left(86.8 + 209.8\right)k-ft.\left(\frac{12\ in.}{ft.}\right)}{2013.21\ in.^3} + \frac{1.3\ \left(1213.9\ k-ft.\right)\left(\frac{12\ in.}{ft.}\right)}{7309.82\ in.^3}$$

$= 15.7\ ksi < (0.95)(1.0)(50.0\ ksi) = 47.5\ ksi$ O.K.

The bottom flange shall satisfy:

ii) $$f_f + \frac{f_l}{2} \leq 0.95\left(R_h\right)\left(F_{yf}\right) \qquad \text{(Eq. 6.10.4.2.2-2)}$$

Where:

f_l = 0 ksi for skews less than 45 degrees

$$f_f = \frac{M_{DC1}}{S_{nc}} + \frac{M_{DC2} + M_{DW}}{S_{n=27}} + \frac{1.3\left(M_{LL+IM}\right)}{S_{n=9}} \qquad \text{(Eq. D6.2.2-1)}$$

Where:

M_{DC1} = 485.3 k-ft.

M_{DC2} = 86.8 k-ft.

M_{DW} = 209.8 k-ft.

M_{LL+IM} = 1213.9 k-ft.

$S_{n=9}$ = 788.90 in.^{3}

$S_{n=27} = 723.98 \text{ in.}^3$

$S_{nc} = 552.66 \text{ in.}^3$

$$f_f = \frac{485.3 \text{ k}-\text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{552.66 \text{ in.}^3} + \frac{(86.8+209.8)\text{ k}-\text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{723.98 \text{ in.}^3} + \frac{1.3\,(1213.9 \text{ k}-\text{ft.})\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{788.90 \text{ in.}^3}$$

$= 39.46 \text{ ksi}$

$$f_f + \frac{f_l}{2} = 39.46 \text{ ksi} + 0 \text{ ksi} = 39.46 \text{ ksi}$$

$(0.95)(1.0)(50.0 \text{ ksi}) = 47.5 \text{ ksi} > 39.46 \text{ ksi}$ O.K.

As this is a composite section in positive flexure and $D / t_w < 150$, Eq. 6.10.4.2.2-4 need not be checked.

Positive Moment Region: Check Fatigue and Fracture Limit State (6.10.5)

<u>Determine $(ADTT)_{SL}$ and Appropriate Limit State</u>

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

Where:

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

$= 713 \text{ trucks/day}$

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

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

From Table 6.6.1.2.3-1, use Detail Category A for base metal checks in bottom flange at maximum fatigue range, use Detail Category C for base metal checks in top flange at maximum fatigue range, and use Detail Category C' for base metal checks at brace points. From Table 6.6.1.2.3-2, the 75-year infinite life equivalent $(ADTT)_{SL}$ is 530 trucks/day for Category A, 1290 trucks/day for Category C, and 745 trucks/day for

Category C'. Therefore, use Fatigue I (infinite life) limit state for Category A checks, and Fatigue II (finite life) limit state for Category C and C' checks.

Determine Stress Range and Compare to Allowable Stress Range

$$\gamma\,(\Delta f) \le (\Delta F)_n \qquad \text{(Eq. 6.6.1.2.2-1)}$$

$$\gamma(\Delta f) = \frac{M^{+}_{FATIGUE} - M^{-}_{FATIGUE}}{S_{n=9}} + f_l$$

Where:

$M^{+}_{FATIGUE\,I}$ = 535.5 k-ft. $M^{-}_{FATIGUE\,I}$ = -235.2 k-ft.

$M^{+}_{FATIGUE\,II}$ = 267.8 k-ft. $M^{-}_{FATIGUE\,II}$ = -117.6 k-ft.

$S_{n=9}$ = 788.90 in.^{3} for outside of bottom flange

= 7309.82 in.^{3} for outside of top flange

= 806.82 in.^{3} for inside of bottom flange

γf_l = = 0 ksi

$$\gamma(\Delta f) = \frac{(535.5 - (-235.2))\,k - ft.\left(\frac{12\ in.}{ft.}\right)}{788.90\ in.^3}$$

= 11.72 ksi for outside of bottom flange

$$\gamma(\Delta f) = \frac{(267.8 - (-117.6))\,k - ft.\left(\frac{12\ in.}{ft.}\right)}{806.82\ in.^3}$$

= 5.73 ksi for inside of bottom flange

$$\gamma(\Delta f) = \frac{(267.8 - (-117.6))\,k - ft.\left(\frac{12\ in.}{ft.}\right)}{7309.82\ in.^3}$$

= 0.63 ksi for top of top flange

For Fatigue I Limit State:

$$(\Delta F)_n = (\Delta F)_{TH} \qquad \text{(Eq. 6.6.1.2.5-1)}$$

$(\Delta F)_{TH}$ = 24.0 ksi for Category A.

$\gamma(\Delta f)$ = 11.72 ksi < 24.0 ksi O.K.

For Fatigue II Limit State:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{(Eq. 6.6.1.2.5-2)}$$

Where:

A = 44×10^8 ksi for Detail Category C and C'

$$N = \left(\frac{365 \text{ days}}{\text{year}}\right)(75 \text{ years})\left(\frac{n \text{ cycles}}{\text{truck}}\right)\left(\frac{(ADTT)_{37.5,\,SL} \text{ trucks}}{\text{day}}\right)$$

(Eq. 6.6.1.2.5-3)

n = 1.0 (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

$$(\Delta F)_n = \left(\frac{44 \times 10^8 \text{ cycles}}{11.8 \times 10^6 \text{ cycles}}\right)^{\frac{1}{3}} = 7.20 \text{ ksi}$$

$\gamma(\Delta f)$ = 5.73 ksi and 0.63 ksi < 7.20 ksi O.K.

Positive Moment Region: Check Strength Limit State (Moment) *(6.10.6)*

<u>Check Composite Sections in Positive Flexure</u> (6.10.6.2.2)

Check Compactness

i) The bridge is straight or equivalently straight as per 4.6.1.2.4b

ii) $F_y = 50 \text{ ksi} \leq 70 \text{ ksi}$ O.K.

iii) Cross-section proportion limit web requirements are satisfied (see above)

iv) $\frac{2(D_{cp})}{t_w} \leq 3.76\sqrt{\frac{E}{F_{yc}}}$ (Eq. 6.10.6.2.2-1)

Find D_p, the depth to the plastic neutral axis from the top of the slab, to determine D_{cp}, the depth of the web in compression at the plastic moment capacity.

Per Table D6.1-1:

P_{rt} = force in top reinforcement in fully plastic section (k)

= $F_{yrt}A_{rt}$ = 60 ksi(2.25 in.^{2}) = 135 k

P_s = force in slab in fully plastic section (k)

= $0.85f'_c$(eff. flange width)t_{slab} = 0.85(3.5 ksi)(87 in.)(8 in.) = 2070.6 k

P_{rb} = force in bottom reinforcement in fully plastic section (k)

= $F_{yrb}A_{rt}$ = 60 ksi(2.17 in.^{2}) = 130.2 k

P_c = force in top flange in fully plastic section (k)

= $F_{yc}b_{fc}t_{fc}$ = (50 ksi)(12 in.)(0.75 in.) = 450 k

P_w = force in web in fully plastic section (k)

= $F_{yw}Dt_w$ = (50 ksi)(42 in.)(0.4375 in.) = 918.75 k

P_t = force in bottom flange in fully plastic section (k)

= $F_{yt}b_{ft}t_{ft}$ = (50 ksi)(12 in.)(0.875 in.) = 525 kips

The fully plastic force in the steel beam is:

$P_t + P_w + P_c$ = 450 k + 918.75 k + 525.0 k = 1893.75 k

The fully plastic (ultimate) force in the slab is:

$P_s + P_{rb} + P_{rt}$ = 2070.6 k + 130.2 k + 135.0 k = 2335.8 kips

$P_t + P_w + P_c < P_s + P_{rb} + P_{rt}$

$\therefore$ The plastic neutral axis is in the slab, take D_{cp} as 0. However, as D_p and $\overline{Y}$ are useful in later calculations, they should still be calculated.

Check Case III:

$$\left(\frac{c_{rb}}{t_s}\right)P_s + P_{rb} + P_{rt} \quad =$$

$$\left(\frac{8\text{ in.} - 1\text{ in.} - 1.5(0.625\text{ in.})}{8\text{ in.}}\right)(2070.6\text{ k}) + 130.2\text{ k} + 135.0\text{ k}$$

$$= \quad 1834.33\text{ k}$$

$$P_t + P_w + P_c \quad = \quad 525\text{ k} + 918.75\text{ k} + 450\text{ k}$$

$$= \quad 1893.75\text{ k}$$

$$P_t + P_w + P_c > \left(\frac{c_{rb}}{t_s}\right)P_s + P_{rb} + P_{rt}, \therefore \bar{Y} \quad = t_s\left(\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right)$$

$$\bar{Y} \quad = \quad (8\text{ in.})\left(\frac{450\text{ k} + 918.75\text{ k} + 525\text{ k} - 135\text{ k} - 130.2\text{ k}}{2070.6\text{ k}}\right)$$

= 6.29 in., measured from top of slab. Therefore $D_p = \bar{Y} = 6.29$ in.

$$\frac{2\left(D_{cp}\right)}{t_w} = \frac{2\left(0\right)}{0.4375\text{ in.}} = 0.0$$

$$3.76\sqrt{\frac{E}{F_{yc}}} = 3.76\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 90.55$$

$0 < 90.55$ O.K.

The section is compact.

Check Flexural Resistance ***(6.10.7)***

$$M_u + \frac{1}{3}\left(f_l\right)\left(S_{xt}\right) \le \phi_f\left(M_n\right) \qquad \text{(Eq. 6.10.7.1.1-1)}$$

$M_{STRENGTH\ I}$ = 1.25(485.3 k-ft.+86.8 k-ft.)+1.5(209.8 k-ft.) + 1.75(1213.9 k-ft.)

= 3154.2 k-ft.

f_l = 0 ksi

$$S_{xt} = \frac{M_{yt}}{F_{yt}}$$

$$M_{yt} = S_{b(c,n)}\left(F_{yf} - \frac{1.25M_{DC1}}{S_{b(nc)}} - \frac{1.25M_{DC2}}{S_{b(c,3n)}} - \frac{1.5M_{DW}}{S_{b(c,3n)}}\right) + 1.25M_{DC1} + 1.5M_{DC2} + 1.75M_{DW}$$

(Eqs. D6.2.2-1,2)

$$= \left(788.9\text{ in.}^3\right)\left(50\text{ ksi} - \left(\frac{12\text{ in.}}{\text{ft.}}\right)\left(\frac{1.25(485.3\text{ k}-\text{ft.})}{552.66\text{ in.}^3} + \frac{1.25(86.8\text{ k}-\text{ft.})}{723.98\text{ in.}^3} + \frac{1.5(209.8\text{ k}-\text{ft.})}{723.98\text{ in.}^3}\right)\right)$$

$$+ \left(\frac{12\text{ in.}}{\text{ft.}}\right)\left(1.25(485.3\text{ k - ft.}) + 1.25(86.8\text{ k - ft.}) + 1.5(209.8\text{ k - ft.})\right)$$

= 35878 k-in.

$$S_{xt} = \frac{35878\text{ k}-\text{in.}}{50\text{ ksi}} = 717.6\text{ in.}^3$$

For $D_p \leq 0.1D_t$, $M_n = M_p$ (Eq. 6.10.7.1.2-1)

Otherwise, $M_n = M_p\left[1.07 - 0.7\left(\frac{D_p}{D_t}\right)\right]$ (Eq. 6.10.7.1.2-2)

D_p = 6.29 in.

$0.1D_t$ = 0.1(0.875 in. + 42 in. + 0.75 in. + 0.75 in. + 8 in.) = 5.24 in. < 6.29 in.

$\therefore M_n = M_p\left[1.07 - 0.7\left(\frac{D_p}{D_t}\right)\right]$ (Eq. 6.10.7.1.2-2)

Where:

As per Table D6.1-1, M_p for Case III is calculated as follows:

$$M_p = \left(\frac{\overline{Y}^2 P_s}{2t_s}\right) + \left[P_{rt}d_{rt} + P_{rb}d_{rb} + P_c d_c + P_w d_w + P_t d_t\right]$$

Where:

$\overline{Y}$ = 6.29 in.

d_{rt} = distance from plastic neutral axis to centroid of top reinforcement (in.)

= 6.29 in. – 2.5 in. – 1.5(0.625 in.) = 2.85 in.

d_{rb} = distance from plastic neutral axis to centroid of bottom reinforcement (in.)

= 6.29 in. – (8 in. – 1 in. – 1.5(0.625 in.) = 0.23 in.

d_c = distance from plastic neutral axis to centroid of top flange (in.)

= (8 in. – 6.29 in.) + 0.75 + 0.5(0.75 in.) = 2.84 in.

d_w = distance from plastic neutral axis to centroid of web (in.)

= (8 in. – 6.29 in.) + 0.75 in. + 0.75 in. + 0.5(42 in.) = 24.21 in.

d_t = distance from plastic neutral axis to centroid of tension flange (in.)

= (8 in. – 6.29 in.) + 0.75 in. + 0.75 in. + 42 in. + 0.5(0.875 in.) = 45.65 in.

$$M_p = \frac{(6.29\text{ in.})^2(2070.6\text{ k})}{2(8\text{ in.})} + (135\text{ k})(2.85\text{ in.}) + (130.2\text{ k})(0.23\text{ in.}) +$$

$$(450\text{ k})(2.84\text{ in.}) + (918.75\text{ k})(24.21\text{ in.}) + (525\text{ k})(45.65\text{ in.})$$

$$= 53021.97\text{ k-in.}$$

$$M_n = (53021.97\text{ k} - \text{in.})\left[1.07 - 0.7\left(\frac{6.29\text{ in.}}{52.38\text{ in.}}\right)\right]$$

$$= 52276.54\text{ k-in.}$$

$$\phi_f M_n = 1.00(52276.54\text{ k-in.}) = 52276.54\text{ k-in.}$$

$$M_u + \frac{1}{3}(f_l)(S_{xt}) = 3154.2\text{ k-ft.}\left(\frac{12\text{ in.}}{\text{ft.}}\right) + \frac{1}{3}(0\text{ ksi})(717.6\text{ in.}^3)$$

$$= 37850.4\text{ k-in.} < 52276.54\text{ k-in.}$$ O.K.

Check Eq. 6.10.7.1.2-3:

$M_n \leq 1.3 R_h M_y$ (Eq. 6.10.7.1.2-3)

Where:

R_h = 1.0

M_y = 35878 k-in.

$1.3R_hM_y$ = 46641 k-in. < 52276.54 k-in.

Recalculate M_n using 46641 k-in.

$$M_n = (46641\,k - in.)\left[1.07 - 0.7\left(\frac{6.29\text{ in.}}{52.38\text{ in.}}\right)\right]$$

= 45985 k-in.

$M_u + \frac{1}{3}(f_l)(S_{xt})$ = 37850.4 k-in. < 45985 k-in. O.K.

Check Ductility Requirement (6.10.7.3)

$D_p \leq 0.42D_t$ (Eq. 6.10.7.3-1)

Where:

D_p = 6.29 in.

$0.42D_t$ = 0.42(52.38 in.) = 22.0 in.

6.29 in. < 22.0 in. O.K.

Positive Moment Region: Check Strength Limit State (Shear) *(6.10.9)*

The maximum shear for the positive moment section occurs at the abutments. The end section of the beam technically could be considered stiffened because the first diaphragm is close enough to the bearing stiffener to warrant it. However, this is irrelevant because end sections of stiffened webs and unstiffened webs use the same equations for shear capacity of their respective webs:

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

Where:

ϕ_v = 1.00 (6.5.4.2)

V_u = 219.2 kips

$V_n = V_{cr} = CV_p$ (Eq. 6.10.9.2-1)

Where:

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

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

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

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

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

$$= 0.49$$

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

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

$$= 532.9\ \text{k}$$

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

219.2 kips < 261.1 kips O.K.

The proposed positive moment section is adequate.

Negative Moment Region: Calculate Moments and Shears

The regions adjacent to the pier control the negative moment design of the beams. The first unstiffened web section controls the shear design of the beams.

Since an unstiffened web design is already shown in the positive moment region, in order to illustrate a stiffened web design the regions closest to the pier will be shown for shear design even though they do not control the final design of the beam.

At the pier, the unfactored distributed moments are:

M_{DC1} = -1466.9 k-ft.

M_{DC2} = -225.6 k-ft.

M_{DW} = -545.2 k-ft.

M_{LL+IM} = -1669.0 k-ft.

The maximum construction loading moments are found at the pier:

M_{DC}^{CONST} = -1543.5 k-ft.

M_{LL+IM}^{CONST} = -307.9 k-ft.

The corresponding construction loading factored moments for the diaphragm locations (brace points) and midpoint for these diaphragms are:

$M_{STRENGTH\,I}^{BRACE\,POINT\,1}$ = -2468.2 k-ft., corresponding to the pier

$M_{STRENGTH\,I}^{MIDPOINT}$ = -2238.8 k-ft., corresponding to the first midpoint in the second span

$M_{STRENGTH\,I}^{BRACE\,POINT\,2}$ = -2017.8 k-ft., corresponding to the first diaphragm in the second span

Note that the last diaphragm section in the first span has a larger unbraced length, but smaller moments. In a full design this bay would also need to be checked. However, for brevity, only the pier section will be checked in this design guide.

The moments resulting in the maximum fatigue stress range occur near the point of splice:

$M_{FATIGUE\,I}^{+}$ = 400.9 k-ft. $M_{FATIGUE\,I}^{-}$ = -301.6 k-ft.

$M_{FATIGUE\,II}^{+}$ = 200.5 k-ft. $M_{FATIGUE\,II}^{-}$ = -150.8 k-ft.

These values contain impact and include γ of 1.5 for Fatigue I and 0.75 for Fatigue II.

The factored, distributed moments for Strength Limit State, Service Limit State, and Constructibility Limit State are found to be:

$M_{STRENGTH\ I}$ = -5854.1 k-ft.

$M_{SERVICE\ II}$ = -4407.3 k-ft.

$M^{CONST}_{STRENGTH\ I}$ = -2468.2 k-ft.

The factored shears at the pier have been found to be:

V_{CONST} = 103.9 kips (@ pier)

$V_{STRENGTH\ I}$ = 300.5 kips (@ pier)

Note, again, that the pier section is being used even though it does not control the shear design. This is to illustrate a stiffened web design.

Negative Moment Region: Check Cross-Section Proportion Limits (6.10.2)

Check Web Proportions

$$\frac{D}{t_w} \le 150 \qquad \text{(Eq. 6.10.2.1.1-1)}$$

$$\frac{D}{t_w} = \frac{42\text{ in.}}{0.5\text{ in.}} = 84 \le 150 \qquad \text{O.K.}$$

Check Flange Proportions

i) $$\frac{b_f}{2t_f} \le 12.0 \qquad \text{(Eq. 6.10.2.2-1)}$$

$$\frac{b_{tf}}{2t_{tf}} = \frac{12\text{ in.}}{2(2\text{ in.})} = 3 \qquad \frac{b_{bf}}{2t_{bf}} = \frac{12\text{ in.}}{2(2.5)} = 2.4$$

3 and 2.4 < 12.0 O.K.

ii) $b_f \geq \frac{D}{6}$ (Eq. 6.10.2.2-1)

$b_f = 12 \text{ in.}$

$\frac{D}{6} = \frac{42 \text{ in.}}{6} = 7 \text{ in.}$

12 in. > 7 in. O.K.

iii) $t_f \geq 1.1\,t_w$ (Eq. 6.10.2.2-3)

$t_f = 2.0 \text{ in. or } 2.5 \text{ in.}$

$1.1t_w = 1.1(0.5 \text{ in.}) = 0.55 \text{ in.}$

2.0 in. and 2.5 in. > 0.55 in. O.K.

iv) $0.1 \leq \frac{I_{yc}}{I_{yt}} \leq 10$ (Eq. 6.10.2.2-4)

$$I_{yc} = \frac{1}{12} t_{bf} b_{bf}^3 = \frac{1}{12}(2.5 \text{ in.})(12 \text{ in.})^3 = 360 \text{ in.}^4$$

$$I_{yt} = \frac{1}{12} t_{tf} b_{tf}^3 = \frac{1}{12}(2.0 \text{ in.})(12 \text{ in.})^3 = 288 \text{ in.}^4$$

$$\frac{I_{yc}}{I_{yt}} = \frac{360 \text{ in.}^4}{288 \text{ in.}^4} = 1.25$$

$0.1 \leq 1.25 \leq 10$ O.K.

Negative Moment Region: Check Constructibility *(6.10.3, App. A6)*

Check Applicability of Appendix A6 (A6.1.1)

The bridge is straight, F_y for all portions of the girder is 70 ksi or less, and $\frac{I_{yc}}{I_{yt}} > 0.3$. The skew is 20 degrees and there are no curvature effects.

Verify that beam is non-slender. $\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$:

For noncomposite sections in negative flexure:

$D_c = Y_{b(nc)} - t_{bf} = 21.64 \text{ in.} - 2.5 \text{ in.} = 19.14 \text{ in.}$

$$\frac{2D_c}{t_w} = \frac{2(19.14 \text{ in.})}{0.5 \text{ in.}} = 76.6$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 137.3$$

76.6 < 137.3. Therefore, Appendix A may be used.

Determine Web Plastification Factors (App A6.2)

Check Web Compactness. If $\frac{2D_{cp}}{t_w} \le \lambda_{pw(Dcp)}$, the web is compact (Eq. A6.2.1-1).

$D_{cp} = Y_{PNA} - t_{bf}$

Determine Y_{PNA} for noncomposite section:

$$(50 \text{ ksi})(12 \text{ in.})(2 \text{ in.}) + (50 \text{ ksi})(0.5 \text{ in.})(46.5 \text{ in.} - Y_{PNA} \text{ in.} - 2 \text{ in.}) =$$
$$(50 \text{ ksi})(12 \text{ in.})(2.5 \text{ in.}) + (50 \text{ ksi})(0.5 \text{ in.})(Y_{PNA} \text{ in.} - 2.5 \text{ in.})$$

$Y_{PNA} = 17.5$ in.

$D_{cp} = 17.5 \text{ in.} - 2.5 \text{ in.}$

$= 15.0$ in.

$$\frac{2D_{cp}}{t_w} = \frac{2(15.0 \text{ in.})}{0.5 \text{ in.}} = 60 \text{ in.}$$

$$\lambda_{pw(Dcp)} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \qquad \text{(Eq. A6.2.1-2)}$$

Where:

$R_h = 1$

M_p = (50 ksi)(12 in.)(2 in.)[42 in. – 15 in. + 0.5(2 in.)] +
(50 ksi)(0.5 in.)(0.5)(42 in. – 15.0 in.)^2 +
(50 ksi)(0.5 in.)(0.5)(15.0 in.)^2 +
(50 ksi)(12 in.)(2.5 in.)[15.0 in. + 0.5(2.5 in.)]
= 69900 k-in.

M_y = smaller of M_{yc} and M_{yt}. For noncomposite sections, $M_{yc} = F_yS_{xc}$ and $M_{yt} = F_yS_{xt}$. By inspection, F_yS_{xt} is smaller.
= (50 ksi)(1179.37 in.^{3})
= 58968.50 k-in.

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 137.3\text{, as calculated above.}$$

$$\lambda_{pw(Dcp)} = \frac{\sqrt{\dfrac{29000\text{ ksi}}{50\text{ ksi}}}}{\left(0.54\dfrac{69900\text{ k}-\text{in.}}{(1.0)(58968.50\text{ k}-\text{in.})}-0.09\right)^2}$$

= 79.58

$$\lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) = (137.3)\left(\frac{15.0\text{ in.}}{19.14\text{ in.}}\right) = 107.6 > 79.58\text{, 79.58 controls.}$$

60 < 79.58, therefore $R_{pc} = \dfrac{M_p}{M_{yc}}$ and $R_{pt} = \dfrac{M_p}{M_{yt}}$

$$R_{pc} = \frac{69900\text{ k}-\text{in.}}{(50\text{ ksi})(1354.85\text{ in.}^3)} = 1.03 \qquad \text{(Eq. A6.2.1-4)}$$

$$R_{pt} = \frac{69900\text{ k}-\text{in.}}{(50\text{ ksi})(1179.37\text{ ksi})} = 1.19 \qquad \text{(Eq. A6.2.1-5)}$$

Check Compression Flange (A6.1.1)

$$M_u + \frac{1}{3}f_lS_{xc} \le \phi_fM_{nc} \qquad \text{(Eq. A6.1.1-1)}$$

Where:

M_u = 2468.2 k-ft.
f_l = Assume zero for Constructibility.
S_{xc} = 1354.85 in.^{3}

ϕ_f = 1.00 for flexure (6.5.4.2)

M_{nc} = smaller of $M_{nc(FLB)}$ and $M_{nc(LTB)}$ as calculated in Appendix A6.3

Determine $M_{nc(FLB)}$:

$$\lambda_f = \frac{12 \text{ in.}}{2(2.5 \text{ in.})} = 2.4 \qquad \text{(Eq. A6.3.2-3)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 \qquad \text{(Eq. A6.3.2-4)}$$

$2.4 < 9.15$, $\therefore M_{nc(FLB)} = R_{pc}M_{yc} = M_p$ (Eq. A6.3.2-1)

$M_{nc(FLB)}$ = 69900 k-in.

Determine $M_{nc(LTB)}$:

L_b = 54 in. (Eq. A6.3.2-3)

$$L_p = 1.0\, r_t \sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eq. A6.3.3-4)}$$

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c}{b_{fc}}\frac{t_w}{t_{fc}}\right)}} \qquad \text{(Eq. A6.3.3-10)}$$

D_c = Y_b - t_{bf}

= 21.64 in. – 2.5 in. = 19.14 in.

$$r_t = \frac{12 \text{ in.}}{\sqrt{12\left(1+\frac{1}{3}\frac{(19.14 \text{ in.})(0.5 \text{ in.})}{(12 \text{ in.})(2.5 \text{ in.})}\right)}}$$

= 3.29 in.

$$L_p = 1.0(3.29 \text{ in.})\sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}} = 79.23 \text{ in.}$$

54 in. < 79.23 in.,

$\therefore M_{nc(LTB)} = R_{pc}M_{yc}$

Where:

R_{pc} = 1.03 (See FLB computations)

M_{yc} = F_yS_{xc} = (50 ksi)(1354.85 in.^{3}) = 67742.5 k-in.

$M_{nc(LTB)}$ = 1.03(67742.5 k-in.)

= 69900 k-in. < 69900 k-in., $M_{nc(LTB)} = M_{nc(FLB)} = M_p$.

M_{nc} = 69900 k-in.

$$M_u + \frac{1}{3} f_l S_{xc} = (2468.2\ \text{k} - \text{ft.})\left(\frac{12\ \text{in.}}{\text{ft.}}\right) + \frac{1}{3}(0\ \text{ksi})(1354.9\ \text{in.}^3)$$

$$= 29618.4\ \text{k-in.}$$

$$\phi_f M_{nc} = 1.00(69900\ \text{k-in.}) = 69900\ \text{k-in.}$$

29618.4 k-in. < 69900 k-in. O.K.

<u>Check Tension Flange</u> <u>(A6.4)</u>

$M_{nt} = R_{pt}M_{yt} = M_p$ = 69900 k-in. (Eq. A6.4-1)

29618.4 k-in. < 69900 k-in. O.K.

<u>Check Shear</u> (6.10.3.3)

As V_{CONST} = 103.9 k < $V_{STRENGTH\ I}$ = 300.5 k, and the steel web capacity does not change from construction loading to full Strength I loading, construction shear loading will not control the web design by inspection.

Negative Moment Region: Check Service Limit State *(6.10.4)*

<u>Find stresses in slab and compare to $2f_r$:</u>

$$f_f = \frac{(225.6 + 545.2)\text{k} - \text{ft.}\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{27(2676.0\ \text{in.}^3)} + \frac{1.3\ (1669.0\ \text{k} - \text{ft.})\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{9(3397.15\ \text{in.}^3)}$$

$$= 0.98\ \text{ksi}$$

$$2f_r = 2(0.24)\sqrt{f'_c} = 2(0.24)\sqrt{(3.5\ \text{ksi})} = 0.90\ \text{ksi}$$ (6.10.1.7)

0.98 ksi > 0.90 ksi. The deck is assumed to be cracked.

<u>Check Permanent Deformations</u> (6.10.4.2)

i) The top flange shall satisfy $f_f \leq 0.95\,(R_h)\,(F_{yf})$ (Eq. 6.10.4.2.2-1)

Where:

$$f_f = \frac{1466.9\ k-ft.\left(\frac{12\ in.}{ft.}\right)}{1179.37\ in.^3} + \frac{\left(225.6\ k-ft. + 545.2\ k-ft. + 1.3\left(1669.0\ k-ft.\right)\right)\left(\frac{12\ in.}{ft.}\right)}{1593.27\ in.^3}$$

$= 37.07$ ksi

$R_h = 1.0$

$F_{yf} = 50$ ksi

$0.95R_hF_{yf} = 47.5$ ksi > 37.07 ksi O.K.

ii) The bottom flange shall satisfy $f_f + \frac{f_l}{2} \leq 0.95\left(R_h\right)\left(F_{yf}\right)$ (Eq. 6.10.4.2.2-2)

Where:

$$f_f = \frac{1466.9\ k-ft.\left(\frac{12\ in.}{ft.}\right)}{1354.85\ in.^3} + \frac{\left(225.6\ k-ft. + 545.2\ k-ft. + 1.3\left(1669.0\ k-ft.\right)\right)\left(\frac{12\ in.}{ft.}\right)}{1449.32\ in.^3}$$

$= 37.33$ ksi

$R_h = 1.0$

$F_{yf} = 50$ ksi

$f_l = 0$ ksi for skews less than 45 degrees

$$f_f + \frac{f_l}{2} = 37.33\ ksi + \frac{0\ ksi}{2} = 37.33\ ksi$$

$0.95R_hF_{yf} = 47.5$ ksi > 37.33 ksi O.K.

Check $f_c < F_{crw}$: (Eq. 6.10.4.2.2-4)

Where:

$f_c = 37.33$ ksi

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} < R_hF_{yc} \text{ and } F_{yw} / 0.7$$ (Eq. 6.10.1.9.1-1)

Where:

$E = 29000$ ksi

$D = 42$ in.

$t_w = 0.5$ in.

$R_h = 1.0$

$F_{yc} = 50$ ksi

$F_{yw} = 50$ ksi

$$k = \frac{9}{\left(\frac{D_c}{D}\right)^2} \qquad \text{(Eq. 6.10.1.9.1-2)}$$

$$D_c = \left[\frac{37.33 \text{ ksi}}{37.07 \text{ ksi} + 37.33 \text{ ksi}}\right](46.5 \text{ in.}) - 2.5 \text{ in.} \qquad \text{(Eq. D6.3.1-1)}$$

$$= 20.83 \text{ in.}$$

Note that, even if lateral effects were present, they would not be used in calculating the depth of web in compression. This is because at the location of the web (centerline of flange) the lateral stresses are neutral.

$$k = \frac{9}{\left(\frac{20.83 \text{ in.}}{42 \text{ in.}}\right)^2} = 36.59$$

$$F_{crw} = \frac{0.9(29000 \text{ ksi})(36.59)}{\left(\frac{42 \text{ in.}}{0.5 \text{ in.}}\right)^2} = 135.3 \text{ ksi} > R_h F_{yc} = 50 \text{ ksi}$$

37.33 ksi < 1.0(50 ksi) = 50 ksi O.K.

Negative Moment Region: Check Fatigue and Fracture Limit State (6.10.5)

The maximum fatigue range for the negative moment region occurs near the point of splice.

Determine $(ADTT)_{SL}$ and Appropriate Limit State

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

Where:

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

$$= 713\ trucks/day$$

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

From Table 6.6.1.2.3-1, use Detail Category C for base metal checks in outside of top flange at maximum fatigue range, and use Detail Category C' for base metal checks at brace points (inside of top flange). Use Detail Category A for base metal checks in bottom flange at maximum fatigue range. From Table 6.6.1.2.3-2, the 75-year infinite life equivalent $(ADTT)_{SL}$ 1290 trucks/day for Category C and 745 trucks/day for Category C'. Therefore, use Fatigue II (finite life) limit state for Category C and C' checks. The equivalent $(ADTT)_{SL}$ for Category A is 530 trucks/day. Therefore, use Fatigue I (infinite life) limit state for Category A.

Lateral flange bending effects are currently not considered for fatigue stress checks.

Determine Stress Range and Compare to Allowable Stress Range

$$\gamma(\Delta f) \le (\Delta F)_n \qquad \text{(Eq. 6.6.1.2.2-1)}$$

$$\gamma(\Delta f) = \frac{M^{+}_{FATIGUE} - M^{-}_{FATIGUE}}{S_{n=9}}$$

Where:

$M^{+}_{FATIGUE\ I}$ = 400.9 k-ft. $M^{-}_{FATIGUE\ I}$ = -301.6 k-ft.

$M^{+}_{FATIGUE\ II}$ = 200.5 k-ft. $M^{-}_{FATIGUE\ II}$ = -150.8 k-ft.

$S_{n=9}$ = 6421.07 in.^{3} for outside of top flange

= 1847.21 in.^{3} for inside of bottom flange

$= \quad 1721.27 \text{ in.}^3$ for outside of bottom flange

For top flange:

$$\gamma(\Delta f) = \frac{(200.5 - (-150.8))\,\text{k} - \text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{8061.19 \text{ in.}^3}$$

$= \quad 0.52$ ksi for inside of top flange

For Bottom Flange:

$\gamma f_l \quad = \quad 0$ ksi for both load combinations

$$\gamma(\Delta f) = \frac{(200.5 - (-150.8))\,\text{k} - \text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{1847.21 \text{ in.}^3}$$

$= \quad 2.28$ ksi for inside of bottom flange

$$\gamma(\Delta f) = \frac{(400.9 - (-301.6))\,\text{k} - \text{ft.}\left(\frac{12 \text{ in.}}{\text{ft.}}\right)}{1721.27 \text{ in.}^3}$$

$= \quad 4.90$ ksi for outside of bottom flange

For Fatigue II Limit State:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \text{(Eq. 6.6.1.2.5-2)}$$

Where:

$A \quad = \quad 44 \times 10^8$ ksi for Detail Category C and C'

$$N = \left(\frac{365 \text{ days}}{\text{year}}\right)(75 \text{ years})\left(\frac{n \text{ cycles}}{\text{truck}}\right)\left(\frac{(ADTT)_{37.5,\,SL} \text{ trucks}}{\text{day}}\right)$$

(Eq. 6.6.1.2.5-3)

$n \quad = \quad 1.5$ (Table 6.6.1.2.5-2)

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

$= \quad 431 \text{ trucks/day}$

$$N = \left(\frac{365 \text{ days}}{\text{year}}\right)(75 \text{ years})\left(\frac{1.5 \text{ cycles}}{\text{truck}}\right)\left(\frac{431 \text{ trucks}}{\text{day}}\right)$$

$$= 17.6 \times 10^6 \text{ cycles}$$

$$(\Delta F)_n = \left(\frac{44 \times 10^8 \text{ cycles}}{17.6 \times 10^6 \text{ cycles}}\right)^{\frac{1}{3}} = 6.30 \text{ ksi}$$

$\gamma(\Delta f) = 0.52 \text{ ksi and } 2.28 \text{ ksi} < 6.30 \text{ ksi}$ O.K.

For Fatigue I Limit State:

$(\Delta F)_n = (\Delta F)_{TH}$ (Eq. 6.6.1.2.5-1)

$(\Delta F)_{TH} = 24.0$ ksi for Category A.

$\gamma(\Delta f) = 4.90 \text{ ksi} < 24.0 \text{ ksi}$ O.K.

<u>Check Special Fatigue Requirement for Webs</u> <u>(6.10.5.3)</u>

$V_u \le V_{cr}$ (Eq. 6.10.5.3-1)

V_u = shear in web due to unfactored permanent load plus factored fatigue load

$= 1.0(62.3 \text{ k}) + 1.0(9.7 \text{ k}) + 1.0(23.4 \text{ k}) + 1.5(39.3 \text{ k})$

$= 154.3 \text{ k}$

$V_{cr} = CV_p$ (Eq. 6.10.9.3.3-1)

Where:

$$\frac{D}{t_w} = 84$$

$$k = 5 + \frac{5}{\left(\frac{54 \text{ in.}}{42 \text{ in.}}\right)^2} = 8.02$$ (Eq. 6.10.9.3.2-7)

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{(29000 \text{ ksi})(8.02)}{(50 \text{ ksi})}} = 76.3$$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{(29000\text{ ksi})(8.02)}{(50\text{ ksi})}} = \quad 95.5$$

$$\therefore C \quad = \frac{1.12}{\left(\frac{42\text{ in.}}{0.5\text{ in.}}\right)}\sqrt{\frac{(29000\text{ ksi})(8.02)}{50\text{ ksi}}} \qquad \text{(Eq. 6.10.9.3.2-5)}$$

$$= \quad 0.91$$

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

$$= 0.58(50\text{ ksi})(42\text{ in.})(0.5\text{ in.})$$

$$= 609\text{ k}$$

$$CV_p = 0.91(609\text{ k}) = 554.2\text{ k}$$

554.2 k > 138.8 k O.K.

Negative Moment Region Check Strength Limit State (Moment) (6.10.6, App. A6)

<u>Check Applicability of Appendix A6</u> <u>(A6.1.1)</u>

The bridge is straight, F_y for all portions of the girder is 70 ksi or less, and $\frac{I_{yc}}{I_{yt}} > 0.3$. The skew is 20 degrees and there are not significant curvature effects.

Verify that the web is not slender. $\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E}{F_{yc}}}$:

$$D_c = \left(\frac{f_c}{f_c + f_t}\right)d - t_{fc} \qquad \text{(Eq. D6.3.1-1)}$$

$$f_t = \frac{1.25(1466.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1179.37\text{ in.}^3}$$

$$+ \frac{(1.25(225.6\text{ k}-\text{ft.}) + 1.5(545.2\text{ k}-\text{ft.}) + 1.75\,(1669.0\text{ k}-\text{ft.}))\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1589.21\text{ in.}^3}$$

$$= \quad 49.02\text{ ksi}$$

$$f_c = \frac{1.25(1466.9\ \text{k}-\text{ft.})\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{1354.85\ \text{in.}^3}$$

$$+\frac{(1.25(225.6\ \text{k}-\text{ft.})+1.5(545.2\ \text{k}-\text{ft.})+1.75\ (1669.0\ \text{k}-\text{ft.}))\left(\frac{12\ \text{in.}}{\text{ft.}}\right)}{1445.63\ \text{in.}^3}$$

$= 49.61$ ksi

$d = 46.5$ in.

$t_{fc} = 2.5$ in.

$$D_c = \left(\frac{49.61\ \text{ksi}}{49.61\ \text{ksi}+49.02\ \text{ksi}}\right)(46.5\ \text{in.})-2.5\ \text{in.}$$

$= 20.89$ in.

$$\frac{2D_c}{t_w} = \frac{2(20.89\ \text{in.})}{0.5\ \text{in.}} = 83.6$$

$$5.7\sqrt{\frac{E}{F_{yc}}} = 5.7\sqrt{\frac{29000\ \text{ksi}}{50\ \text{ksi}}} = 137.3$$

83.6 < 137.3. Therefore, the web is not slender and Appendix A may be used.

Determine Web Plastification Factors (App A6.2)

Check Web Compactness. If $\frac{2D_{cp}}{t_w} \leq \lambda_{pw(Dcp)}$, the web is compact (Eq. A6.2.1-1). Use Appendix D to determine D_{cp}, the depth of web in compression of the fully plastic section:

Per Table D6.1-2:

P_{rt} = force in top reinforcement in fully plastic section (k)

$= F_{yrt}A_{rt} = (60\ \text{ksi})(5.44\ \text{in.}^2) = 326.4$ k

P_{rb} = force in bottom reinforcement in fully plastic section (k)

$= F_{yrb}A_{rt} = (60\ \text{ksi})(2.17\ \text{in.}^2) = 130.2$ k

P_c = force in bottom flange in fully plastic section (k)

$= F_{yc}b_{fc}t_{fc} = (50\ \text{ksi})(12\ \text{in.})(2.5\ \text{in.}) = 1500$ k

P_w = force in web in fully plastic section (k)

$= F_{yw}Dt_w = (50\ \text{ksi})(42\ \text{in.})(0.5\ \text{in.}) = 1050$ k

P_t = force in top flange in fully plastic section (k)

= $F_{yt}b_{ft}t_{ft}$ = (50 ksi)(12 in.)(2 in.) = 1200 k

By inspection, $P_c + P_w > P_t + P_{rb} + P_{rt}$. Therefore, the plastic neutral axis is in the web.

$$D_{cp} = \frac{D}{2A_w F_{yw}}\left[F_{yt}A_t + F_{yw}A_w + F_{yrs}A_{rs} - F_{yc}A_c\right] \qquad \text{(D6.3.2-2)}$$

$$= \frac{D}{2P_w}\left[P_t + P_w + P_{rt} + P_{rb} - P_c\right]$$

$$= \frac{42\text{ in.}}{2(1050\text{ k})}\left[1200\text{ k} + 1050\text{ k} + 326.4\text{ k} + 130.2\text{ k} - 1500\text{ k}\right]$$

$$= 24.13\text{ in.}$$

$$\frac{2\left(D_{cp}\right)}{t_w} = \frac{2\left(24.13\text{ in.}\right)}{0.5\text{ in.}} = 96.52$$

$$\lambda_{pw(Dcp)} = \frac{\sqrt{\frac{E}{F_{yc}}}}{\left(0.54\frac{M_p}{R_h M_y} - 0.09\right)^2} \le \lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) \qquad \text{(Eq. A6.2.1-2)}$$

Where:

R_h = 1

$$M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right] + \left[P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c\right] \qquad \text{(Table D6.1-2)}$$

$\bar{Y}$ = $D - D_{cp}$ = 42 in. – 24.13 in. = 17.87 in. (Table D6.1-2)

d_{rt} = $\bar{Y}$ + t_{tf} + t_{fillet} + t_{slab} – 2.5 in. – 0.625 in. – 0.5[(0.5)(0.625 in. + 0.75 in.)]

= 17.87 in. + 2 in. + 0.75 in. + 8 in. – 3.47 in. = 25.15 in.

d_{rb} = $\bar{Y}$ + t_{tf} + t_{fillet} + 1 in. + 0.625 in. + (0.5)(0.625 in.)

= 17.87 in. + 2 in. + 0.75 in. + 1.94 in. = 22.56 in.

d_t = $\bar{Y}$ + 0.5t_{tf}

= 17.87 in. + 0.5(2 in.) = 18.87 in.

d_b = D – $\bar{Y}$ + 0.5t_{bf}

= 42 in. – 17.87 in. + 0.5(2.5 in.) = 25.38 in.

$$M_p = \frac{1050\text{ k}}{2(42\text{ in.})}\left[(17.87\text{ in.})^2 + (42\text{ in.} - 17.87\text{ in.})^2\right]$$

$$+ (326.4\text{ k})(25.15\text{ in.}) + (130.2\text{ in.})(22.56\text{ in.})$$

$$+ (1200\text{ k})(18.87\text{ in.}) + (1500\text{ k})(25.38\text{ in.})$$

$$= 83130.19\text{ k-in.}$$

$$M_y = 1.25(M_{DC1} + M_{DC2}) + 1.5M_{DW} + S_{c,cr}\left(F_{yf} - \frac{1.25M_{DC1}}{S_{nc}} - \frac{1.25M_{DC2} + 1.5M_{DW}}{S_{c,cr}}\right)$$

Use smaller of M_{yc} and M_{yt}.

$$M_{yc} = 1.25(1466.9\text{ k-ft.} + 225.6\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) + 1.5(545.2\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) +$$

$$1445.63\text{ in.}^3\left(50\,\frac{\text{k}}{\text{in.}^2} - \frac{1.25(1466.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1354.82\text{ in.}^3}\right.$$

$$\left. - \frac{1.25(225.6\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) + 1.5(545.2\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1445.63\text{ in.}^3}\right)$$

$$= 70807\text{ k-in.}$$

$$M_{yt} = 1.25(1466.9\text{ k-ft.} + 225.6\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) + 1.5(545.2\text{ k-ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) +$$

$$1589.21\text{ in.}^3\left(50\,\frac{\text{k}}{\text{in.}^2} - \frac{1.25(1466.9\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1179.37\text{ in.}^3}\right.$$

$$\left. - \frac{1.25(225.6\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right) + 1.5(545.2\text{ k}-\text{ft.})\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1589.21\text{ in.}^3}\right)$$

$$= 71814\text{ k-in.}$$

$$M_y = 70807\text{ k-in.}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 137.3$$

$$\lambda_{pw(Dcp)} = \frac{\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}}}{\left(0.54\frac{83130.19\text{ k}-\text{in.}}{(1.0)(70807\text{ k}-\text{in.})}-0.09\right)^2}$$

$$= 81.39$$

$$\lambda_{rw}\left(\frac{D_{cp}}{D_c}\right) = (137.3)\left(\frac{24.13\text{ in.}}{20.89\text{ in.}}\right) = 158.59$$

The controlling value for $\lambda_{pw(Dcp)}$ is 81.39

$\frac{2(D_{cp})}{t_w}$ = 96.52 > 81.39, therefore the web is noncompact. The web has already been calculated as non-slender, therefore Eq. A6.2.2-1 need not be checked (see Check Applicability of Appendix A6). The web plastification factors therefore are:

$$R_{pc} = \left[1-\left(1-\frac{R_h M_{yc}}{M_p}\right)\left(\frac{\lambda_w-\lambda_{pw(Dc)}}{\lambda_{rw}-\lambda_{pw(Dc)}}\right)\right]\frac{M_p}{M_{yc}} \le \frac{M_p}{M_{yc}} \qquad \text{(Eq. A6.2.2-4)}$$

$$R_{pt} = \left[1-\left(1-\frac{R_h M_{yt}}{M_p}\right)\left(\frac{\lambda_w-\lambda_{pw(Dc)}}{\lambda_{rw}-\lambda_{pw(Dc)}}\right)\right]\frac{M_p}{M_{yt}} \le \frac{M_p}{M_{yt}} \qquad \text{(Eq. A6.2.2-5)}$$

Where:

$$\lambda_w = \frac{2D_c}{t_w} = 83.6 \qquad \text{(Eq. A6.2.2-2)}$$

$$\lambda_{rw} = 5.7\sqrt{\frac{E}{F_{yc}}} = 137.3 \qquad \text{(Eq. A6.2.2-3)}$$

$$\lambda_{pw(Dc)} = \lambda_{pw(Dcp)}\left(\frac{D_c}{D_{cp}}\right) \le \lambda_{rw} \qquad \text{(Eq. A6.2.2-6)}$$

$$= (81.39)\left(\frac{20.89\text{ in.}}{24.13\text{ in.}}\right) = 70.5$$

M_p = 83130 k-in.

M_{yc} = 70807 k-in.

$M_{yt} \quad = \quad 74003$ k-in.

$R_h \quad = \quad 1.0$

$$R_{pc} = \left[1 - \left(1 - \frac{(1.0)(70807\ \text{k-in.})}{(83130\ \text{k-in.})}\right)\left(\frac{83.6 - 70.5}{137.3 - 70.5}\right)\right]\frac{83130\ \text{k-in.}}{70807\ \text{k-in.}}$$

$= \quad 1.14$

$$\frac{M_p}{M_{yc}} = \frac{83130\ \text{k-in.}}{70807\ \text{k-in.}} = 1.17 > 1.14,\ 1.14\ \text{controls.}$$

$$R_{pt} = \left[1 - \left(1 - \frac{(1.0)(71814\ \text{k-in.})}{(83130\ \text{k-in.})}\right)\left(\frac{83.6 - 70.5}{137.3 - 70.5}\right)\right]\frac{83130\ \text{k-in.}}{71814\ \text{k-in.}}$$

$= \quad 1.13$

$$\frac{M_p}{M_{yt}} = \frac{83130\ \text{k-in.}}{71814\ \text{k-in.}} = 1.16 > 1.13,\ 1.13\ \text{controls.}$$

<u>Check Compression Flange</u> <u>(App. A6.1.1)</u>

$$M_u + \frac{1}{3} f_l S_{xc} \le \phi_f M_{nc} \qquad \text{(Eq. A6.1.1-1)}$$

Where:

$$M_u = (5854.1\ \text{k-ft.})\left(\frac{12\ \text{in.}}{\text{ft.}}\right) = 70249\ \text{k-in.}$$

f_l = 0 ksi for skews less than 45 degrees

$$S_{xc} = \frac{M_{yc}}{F_{yc}} = \frac{70807\ \text{k-in.}}{50\ \text{ksi}} = 1416.14\ \text{in.}^3$$

ϕ_f = 1.0 for flexure (6.5.4.2)

M_{nc} = smaller of $M_{nc(FLB)}$ and $M_{nc(LTB)}$ as calculated in Appendix A6.3

Determine $M_{nc(FLB)}$:

$$\lambda_f = \frac{12\ \text{in.}}{2\,(2.5\ \text{in.})} = 2.4 \qquad \text{(Eq. A6.3.2-3)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{29000\ \text{ksi}}{50\ \text{ksi}}} = 9.15 \qquad \text{(Eq. A6.3.2-4)}$$

$2.4 < 9.15,\ \therefore M_{nc(FLB)} = R_{pc} M_{yc} = 1.14(70807\ \text{k-in.})$ (Eq. A6.3.2-1)

$M_{nc(FLB)} = 80719$ k-in.

Determine $M_{nc(LTB)}$:

$L_b = 54$ in. (Eq. A6.3.2-3)

$$L_p = 1.0\, r_t \sqrt{\frac{E}{F_{yc}}}$$ (Eq. A6.3.3-4)

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c\, t_w}{b_{fc}\, t_{fc}}\right)}}$$ (Eq. A6.3.3-10)

$$= \frac{12\text{ in.}}{\sqrt{12\left(1 + \frac{1}{3}\frac{(20.89\text{ in.})(0.5\text{ in.})}{(12\text{ in.})(2.5\text{ in.})}\right)}}$$

$= 3.28$ in.

$$L_p = 1.0(3.28\text{ in.})\sqrt{\frac{29000\text{ ksi}}{50\text{ ksi}}} = 79.0\text{ in.}$$

54 in. < 79.0 in., $\therefore M_{nc(LTB)} = R_{pc}M_{yc} = M_{nc(FLB)}$ (Eq. A6.3.3-1)

$M_{nc(LTB)} = M_{nc(FLB)} = 80719$ k-in.

$M_{nc} = 80719$ k-in.

$$M_u + \frac{1}{3} f_l S_{xc} = 70249\text{ k-in.} + \frac{1}{3}(0\text{ ksi})(1416.14\text{ in.}^3)$$

$= 70249$ k-in.

$\phi_f M_{nc} = 1.00(80719$ k-in.$) = 80719$ k-in.

70249 k-in. < 80719 k-in. O.K.

<u>Check Tension Flange (Appendix A)</u> <u>(A6.4)</u>

$M_{nt} = \phi_f R_{pt} M_{yt} = 1.00(1.13)(71814$ k-in.$) = 81150$ k-in. (Eq. A6.4-1)

70249 k-in. < 81150 k-in. O.K.

Negative Moment Region: Check Strength Limit State (Shear) (6.10.9)

As stated previously, the controlling shear design occurs at the location of the first stiffener into the second span. However, the design shown will be that for the web section closest to the pier so as to illustrate a stiffened web design.

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

Where:

ϕ_v = 1.00 (6.5.4.2)

V_u = 300.5 kips

Verify that $\frac{2Dt_w}{(b_{fc}t_{fc} + b_{ft}t_{ft})} \leq 2.5$ (Eq. 6.10.9.3.2-1)

$$\frac{2(42\text{ in.})(0.5\text{ in.})}{(12\text{ in.})(2.5\text{ in.}) + (12\text{ in.})(2\text{ in.})} = 0.78 < 2.5\text{, therefore}$$

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

Where:

$$\frac{D}{t_w} = 84$$

$$k = 5 + \frac{5}{\left(\frac{54\text{ in.}}{42\text{ in.}}\right)^2} = 8.02$$

$$1.12\sqrt{\frac{Ek}{F_{yw}}} = 1.12\sqrt{\frac{(29000\text{ ksi})(8.02)}{(50\text{ ksi})}} = 76.4$$

$$1.40\sqrt{\frac{Ek}{F_{yw}}} = 1.40\sqrt{\frac{(29000\text{ ksi})(8.02)}{(50\text{ ksi})}} = 95.5$$

$$\therefore C = \frac{1.12}{\left(\frac{42\text{ in.}}{0.5\text{ in.}}\right)}\sqrt{\frac{(29000\text{ ksi})(8.02)}{50\text{ ksi}}}$$ (Eq. 6.10.9.3.2-5)

$$= 0.91$$

$V_p = 0.58F_{yw}Dt_w$ (Eq. 6.10.9.3.2-3)

= 0.58(50 ksi)(42 in.)(0.5 in.)

= 609 k

$$V_n = (609\text{ k})\left[0.84 + \frac{0.87(1-0.91)}{\sqrt{1+\left(\frac{54\text{ in.}}{42\text{ in.}}\right)^2}}\right] = 583.5\text{ k} \qquad \text{(Eq. 6.10.9.3.2-2)}$$

299.7 kips < 583.5 kips O.K.

The proposed negative moment section is adequate.

Slab Reinforcement Checks:

The slab reinforcement should be checked for fatigue at the location of the pier. At that location, the maximum loading is -444.5 k-ft. and the minimum loading is zero. These loads are already factored for the Fatigue I load case.

<u>Check Fatigue I Limit State</u> <u>(6.10.1.7)</u>

$\gamma(\Delta f) \leq (\Delta F)_{TH}$

Where:

γ = 1.5, already included in calculated Fatigue I moments

$$\gamma(\Delta f) = \frac{\left|444.5\text{ k}-\text{ft.} - (0\text{ k}-\text{ft.})\right|\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1244.73\text{ in.}^3} = 4.29\text{ ksi}$$

$$(\Delta F)_{TH} = 24 - 0.33 f_{min} \qquad \text{(Eq. 5.5.3.2-1)}$$

Where:

$$f_{min} = \frac{\left|1466.9\text{ k}-\text{ft.} + 225.6\text{ k}-\text{ft.} + 545.2\text{ k}-\text{ft.} + 0\text{ k}-\text{ft.}\right|\left(\frac{12\text{ in.}}{\text{ft.}}\right)}{1244.73\text{ in.}^3} = 21.57\text{ ksi}$$

$(\Delta F)_{TH} = 24 - 0.33(21.57 \text{ ksi}) = 16.88 \text{ ksi} > 4.29 \text{ ksi}$ O.K.