Appendix J

Example of Proposed Changes

J.1 Introduction

The proposed changes are illustrated with reference to a 200-ft, single span, Washington DOT WF bridge girder with debonded strands and no skew. The example illustrates the design of an interior girder with a focus on: (1) following the proposed detailing rules for debonding (Section 4.2), (2) evaluating *AASHTO LRFD* Article 5.8.3.5 (tensile capacity of longitudinal reinforcement due to bending plus shear), (3) evaluating the tensile tie capacity at and near the support using the strut and tie model proposed in Section 4.3, and (4) evaluating the principal stress in the web as described in Section 3.8.3.

J.2 Assumptions

The superstructure consists of seven WF100G girders spaced at 6'-5". The beams have been designed to act compositely with 2-in. thick haunches and 8-in. thick slab. The design live load is HL-93 plus lane loads where specified. A spreadsheet was used to perform all calculations.

J.2.1 Materials

Cast-in-place concrete slab and haunch:

- Actual thickness = 8 in.
- Structural thickness, t_s = 7.5 in. (A ½-in.-thick wearing surface is considered for dead load calculations)
- Haunch thickness = 2 in.
- Specified concrete compressive strength, f'_c = 5.0 ksi

Precast concrete beams: Washington DOT WF100G

- Specified concrete compressive strength at transfer, f'_{ci} = 7.5 ksi
- Specified concrete compressive strength for use in design, f'_c = 10 ksi
- Concrete unit weight, w_c = 0.150 kcf
- Overall beam length = 200 ft
- Design span = 199 ft

Prestressing strands: 0.7-in. diameter, seven-wire, low-relaxation

- Area of one strand = 0.293 in.^{2}
- Specified tensile strength, f_{pu} = 270 ksi
- Yield strength, $f_{py} = 0.9f_{pu}$ = 243 ksi
- Stress limits:
  - Before transfer, $f_{pi} \leq 0.75f_{pu}$ = 202.5 ksi
  - At service limit state (after all losses), $f_{pe} \leq 0.80f_{py}$ = 194.4 ksi
- Modulus of elasticity, E_p = 28,500 ksi

Reinforcing bars:

- Yield strength, f_y = 60 ksi
- Modulus of elasticity, E_s = 29,000 ksi

Future wearing surface: 2 in. additional concrete

New Jersey-type barrier: weight = 0.300 kips/ft/side

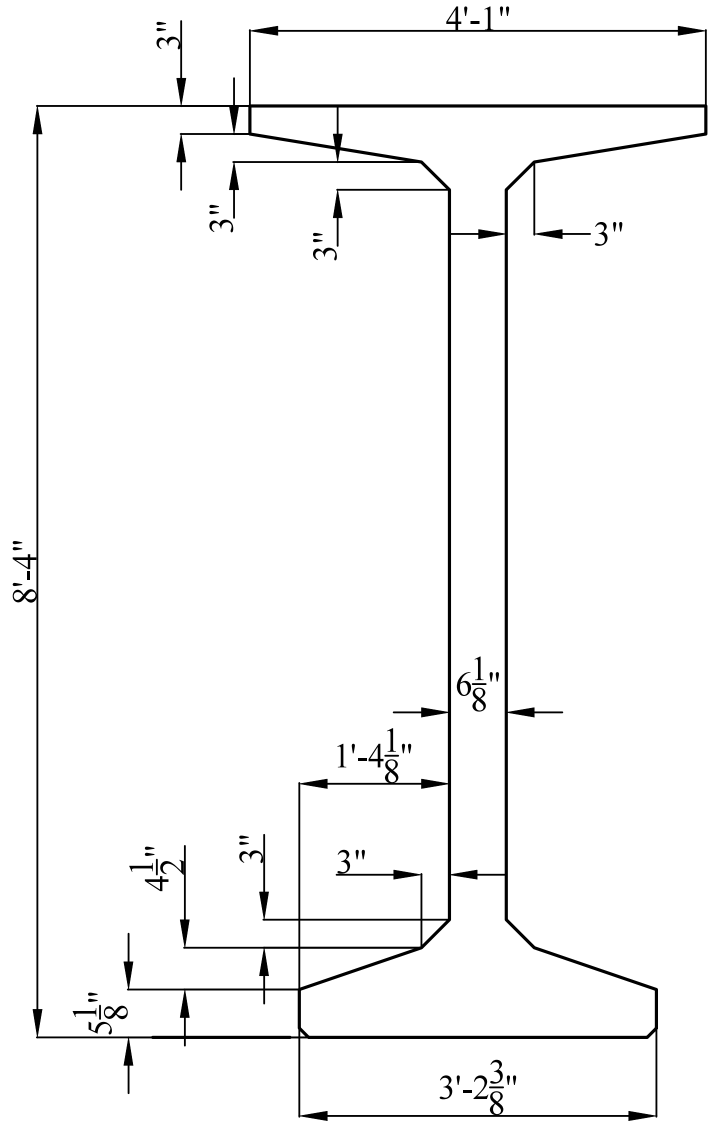

Figure J.1. Dimensions of Washington DOT WF100G girder

J.3 Number and Arrangement of Prestressing Strands

Loads and stresses for limit states Service I, Service III, Strength I, and Fatigue I were checked using a spreadsheet.

As a trial, forty-two 0.7-in. diameter strands were selected.

The controlling limit state is tension in bottom fiber at midspan from Service III loading:

$f_{t,bottom} = 0.40 \text{ ksi} \leq 0.19\sqrt{f'_c} = 0.600 \text{ ksi}$ [*LRFD* Table 5.9.4.2.2-1]

Debonding of the strands at 5 ft increments was iterated in the spreadsheet to satisfy tensile transfer stresses:

$$f_{t,} = 0.597 \text{ ksi} \le 0.24\sqrt{f'_{ci}} = 0.657 \text{ ksi} \qquad [\textit{LRFD} \text{ Table 5.9.4.1.2-1}]$$

The geometry of the debonded strands is shown in Figure J.2, and the resulting stresses from the initial loading (prestress transfer) is shown in Figure J.3. The number of debonded strands is 18 and the debonding ratio, *dr* = 18/42 = 0.428.

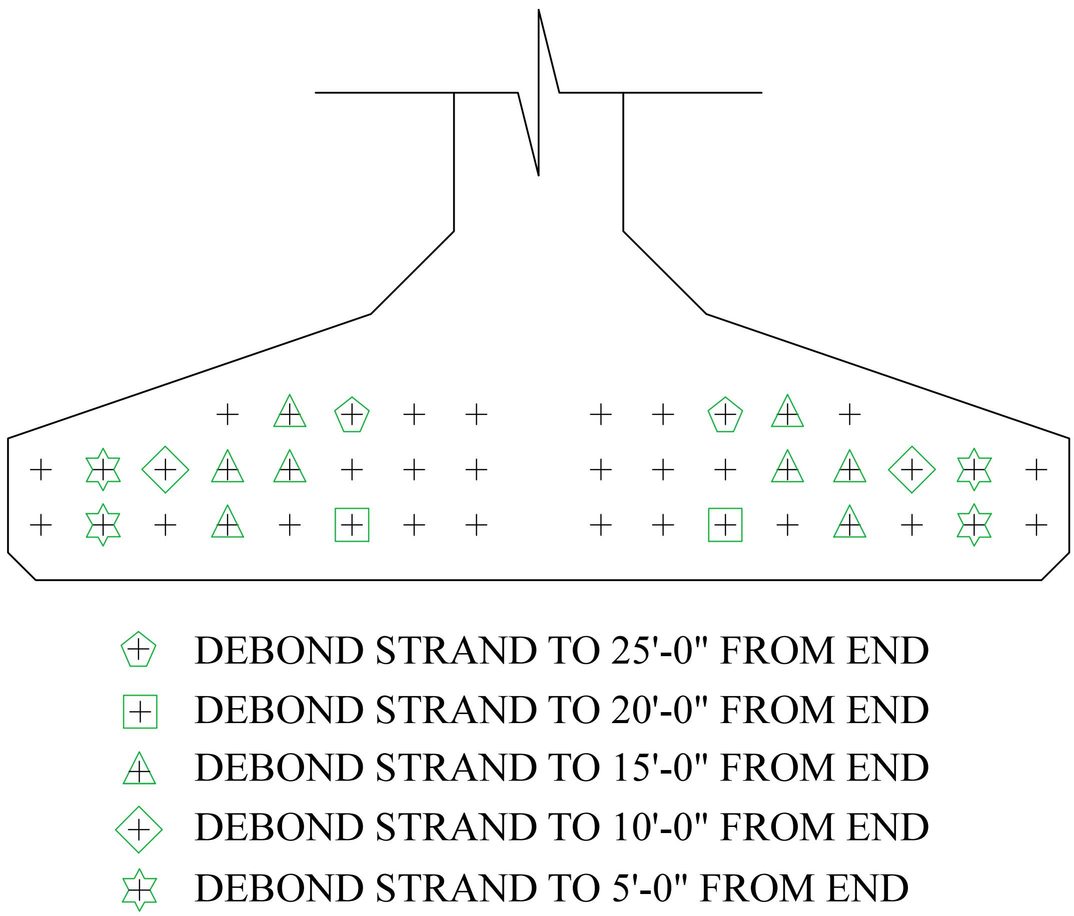

Figure J.2. Geometry of debonded strands

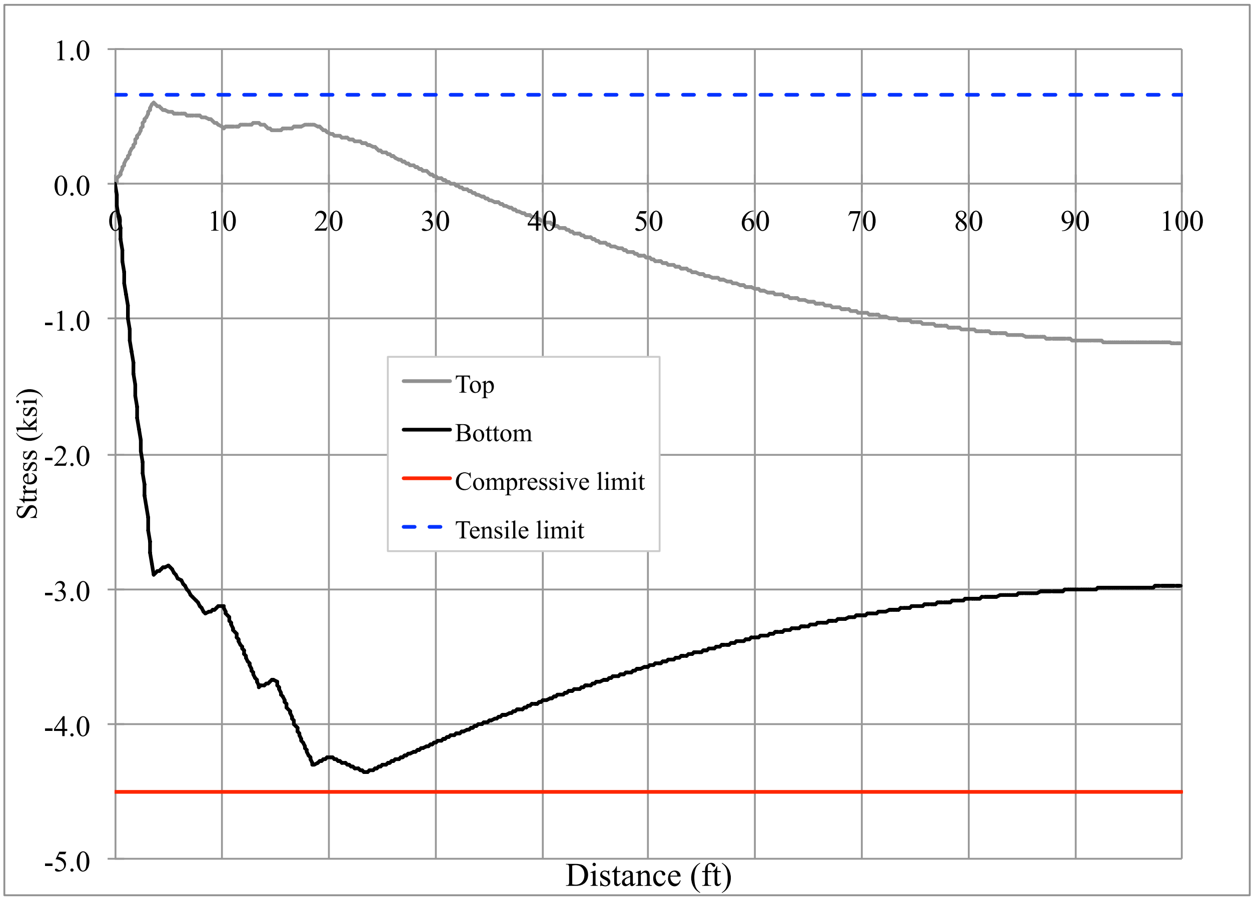

Figure J.3. Stresses due to initial loading (prestress transfer)

Check the geometry of debonded strands needs to meet the requirements of Section 4.2:

- The total number of debonded strands shall not exceed 60 percent of the total number of strands unless test results or successful past practices indicate that a larger percentage of strands may be debonded. *Satisfied, debonding ratio = 42.9%.*
- The number of debonded strands in any horizontal row within the bottom flange height other than the bottom row shall not exceed 80 percent of the number of strands in that row. *Satisfied, maximum row debonding ratio = 50%.*
- Tensile force in prestressing reinforcement ($A_{ps}f_{ps}$) shall exceed the tensile force of the nonprestressed reinforcement (A_sf_s) at all sections. Development of straight and bent-up strands as well as overhangs, if present, should be taken into account for determining the value of f_{ps}. *Satisfied per spreadsheet calculations.*
- No more than 40 percent of debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any section. [No change from current *AASHTO LRFD Article 5.11.4.3.*] *Satisfied, maximum number of strands having the debonding terminate at any section* $= 6 \leq 0.40*18 = 7$.
- Satisfy *AASHTO LRFD* Articles 5.10.10.1, 5.10.10.2, and 5.8.3.5. *Satisfied per spreadsheet calculations.*

- No more than 50 percent of the bottom row of strands shall be debonded. *Satisfied, 43% of bottom row is debonded.*
- The outer-most strands in all rows located within the full-width section of the flange shall remain bonded. *Satisfied.*
- With the exception of the outermost strands, strands further from the section vertical centerline shall be debonded prior to those nearer the centerline. *Satisfied.*
- Strands in the flange within the web width should remain bonded. *Satisfied.*
- Debonded strands shall be symmetrically distributed about the vertical centerline of the cross section of the member. *Satisfied.*
- Full flange width bearing shall be provided at supports. Full flange width bearing is not necessary if a steel sole plate is provided. The width of the sole plate shall be at least one half of the width of the bulb. *Satisfied by providing full width bearing pads.*

J.3.1 Evaluating *AASHTO LRFD* Article 5.8.3.5

AASHTO LRFD Article 5.8.3.5 requires that the tensile steel be designed to not only carry the force from the moment but also the additional tensile component caused by a shear strut. Refer to Section 1.3.1.1 of this report or *AASHTO LRFD* Article 5.8.3.5. For all sections past the critical section (d_v from face of support), *AASHTO LRFD* Eq. 5.8.3.5-1 applies:

$$T = \frac{|M_u|}{d_v \phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right) cot\theta$$

$$A_{ps}f_{ps} + A_s f_y - T \geq 0$$

This inequality cannot be met for a short portion –at the ends of the beam where the prestressing strands are not fully developed.

At a distance of $d_v cot\theta$ from the inside face of bearing, *AASHTO LRFD* Eq. 5.8.3.5-2 applies:

$$T = \left(\frac{V_u}{\phi_v} - V_p - 0.5V_s\right) cot\theta$$

$$A_{ps}f_{ps} + A_s f_y - T \geq 0$$

Where: V_u, V_p, V_s, and θ can be taken at d_v from the inside face of support.

From the spreadsheet it can be seen that the inequality $A_{ps}f_{ps} + A_s f_y - T \geq 0$ is met at $d_v \cot\theta$ from the face of support (*AASHTO LRFD* Eq. 5.8.3.5-1). The inequality is also met at and past the critical section dv from the inside face of bearing. Thus no additional longitudinal steel need be provided.

J.4 Tensile Tie Design for Girder End Region

Design the confining reinforcement at the support to resist a horizontal tie force, *t*. Refer to Section 2.5.2 of this report for a derivation of the following equations:

The tension in the horizontal tie, t, located at y_p from the soffit may be calculated:

$t = \alpha V_u/\phi = (n_f/N_w)[x_p/(h_b\text{-}y_p) + (x_p\text{-}c_b)/y_p]\ V_u/\phi$

Where:

α = tension tie coefficient = $(n_f/N_w)[x_p/(h_b\text{-}y_p) + (x_p\text{-}c_b)/y_p]$

n_f = number of bonded strands in one side of outer portion of web = 12

N_w = number of total bonded strands = 24

x_p = horizontal distance to girder centerline of centroid of n_f strands in outer portion of bulb = [3*2.25 + 3*4.25 + 6.25 + 8.25 + 10.25 + 12.25 + 16.25]/12 = 6.06 in.

y_p = vertical distance to girder soffit of centroid of n_f strands in outer portion of bulb = [5*2 + 4*4 + 3*6]/12 = 3.67 in.

h_b = 5.125 + 4.5 + 3 = 12.625 in.

$c_b = (b_b/2)(1\text{-}n_f/N_w)$

b_b = width of bearing pad = 38.375 – 2*1 = 36.375 in.

Thus, c_b = (36.375/2)(1-12/24) = 9.09

Thus, α = (12/24)[6.06/(12.625 – 3.67) + (6.06 – 9.09)/3.67] = -0.074

Because α is negative the horizontal tie experiences compression instead of tension. Therefore, provide the *AASHTO LRFD* minimum confining reinforcement of No. 3 bars at 6 in. spacing per *AASHTO LRFD* Article 5.10.10.2. Extend this reinforcement to act as confining reinforcement to a distance of 1.5*d* from the end of member per *AASHTO LRFD* Article C5.10.10.1.

Check that this spacing for the confining reinforcement meets the requirements of *AASHTO LRFD* Article 5.10.10.1 for splitting resistance.

$P_r = f_sA_s \geq 4\%P_t$

Where:

P_t = prestress force at transfer, taken at the end of the beam, but assuming strands are fully developed = $f_{pi}A_{ps}$ = (185.8)(24)(0.293) = 1304 kips

$4\%P_t$ = 0.4(2286) = 52.2 kips

f_s = stress in steel not to exceed 20 ksi

A_s = total area of vertical reinforcement within a distance of h/4 from the end of beam.

h/4 = 100/4 = 25 in. for the non-composite section

Using a cover of 1.5 in., the maximum number of bars that can fit within h/4 is given by:

1 + (25 – 1.5)/6 = 4.91 bars, round down to 4 bars.

Using two legs of a No. 5 bar at each stirrup locations: A_s = (4)(2)(0.31) = 2.48 in.^{2}

Thus P_r = (20)(2.48) = 49.6 kips < $4\%P_t$ = 52.2 kips No Good

Try No. 4 vertical bars at a spacing of 3 in.

Maximum number of bars that can fit within $h/4 = 1 + (25 - 1.5)/3 = 8.8$, round down to 8 bars.

Using two legs of a No. 4 bar at each stirrup locations: $A_s = (8)(2)(0.20) = 3.2$ in.^{2}

Thus $P_r = (20)(3.2) = 64$ kips > $4\%P_t = 52.2$ kips

Therefore use No. 4 stirrups at a spacing of 3 in. from beginning of beam to a distance of $h/4$.

J.5 Web Cracking Due to Principal Stress

It is first required to calculate the cross-section properties for a typical interior beam.

J.5.1 Non-composite Beam Section Properties

A_{nc} = area of non-composite = 1082.8 in.^{2}

e = eccentricity of prestressing strands = 44.6 in.

h = overall depth of beam = 100 in.

I_{nc} = moment of inertia about the centroid of the non-composite precast beam = 154492 in.^{4}

y_{bnc} = vertical distance from soffit to centroid of the non-composite precast beam = 49.27 in.

J 5.2 Composite Beam Section Properties

The flange and haunch widths must be transformed by the modular ratio to provide cross-sectional properties equivalent to the concrete girder:

$$n = \text{modular ratio} = \frac{E_c(slab)}{E_c(beam)} = \sqrt{\frac{f'_c(slab)}{f'_c(beam)}} = \sqrt{\frac{5}{10}} = 0.707$$

Transform slab width = n(effective flange width)(t_s) = (0.707)(77) = 54.4 in.^{2}

Transform slab area = n(effective flange width)(t_s) = (0.707)(77)(7.5) = 408 in.^{2}

Transform slab moment of inertia = $(1/12)(77)(54.5)^3$ = 1914 in.^{4}

Transform haunch width = (0.707)(47) = 30.4 in.^{2}

Transform haunch area = (30.4)(2) = 60.8 in.^{2}

Transform haunch moment of inertia = $(1/12)(30.4)(2)^3$ = 20.3

Table J.1. Composite Beam Section Properties

	Area (in.^{2})	y_b (in.)	A_{yb} (in.^{3})	I (in.^{4})	Area$(y_{bc} - y_b)^2$ in.^{4}
Beam	1082.8	48.27	52267	1524912	319975
Haunch	60.8	101	6142	76809	76809
Deck	408	105.75	43183	662865	662865
Σ	1552		101592	1526846	1059648

A_c = area of composite section = 1552 in.^{2}

I_c = moment of inertia of composite section = 1526846 + 1059648 = 2586495 in.^{4}
y_{bc} = vertical distance from the bottom of the section to the composite neutral axis = (101592)/(1552) = 65.46 in.

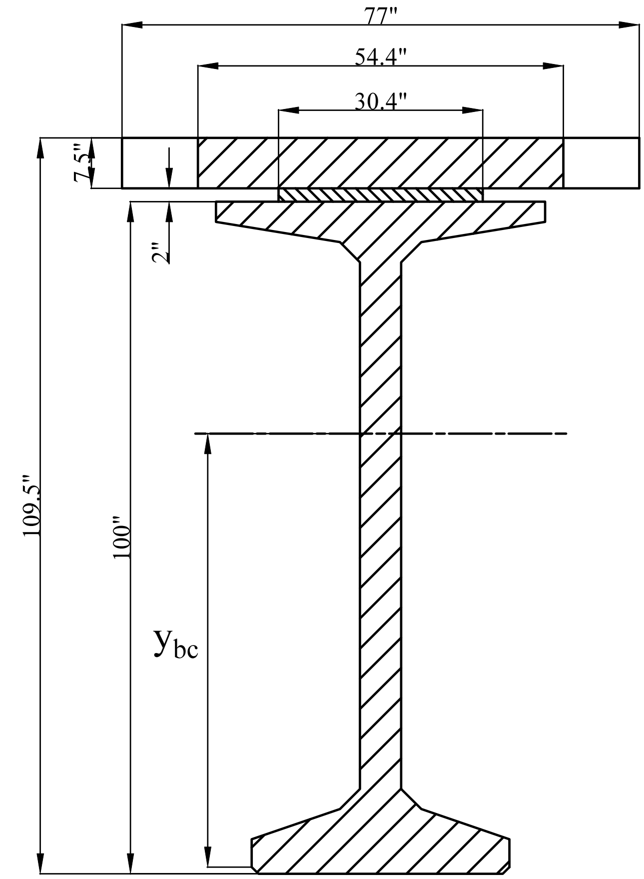

Figure J.4. Dimensions of transform section (shown hatched)

J.5.3 Calculation of principal tensile stress

$$f_{pc} = \frac{P_{pe}}{A_{nc}} - \frac{P_{pe}e}{I_{nc}}\left(y - y_{bnc}\right) + \frac{M_{dnc}\left(y - y_{bnc}\right)}{I_{nc}} + \frac{M_L\left(y - y_{bc}\right)}{I_c} \qquad \text{Eq. J.1}$$

The critical section for shear is located a distance of d_v = 7.9 ft from the face of support. At this location, due to Service III loads:

M_{dnc} = moment due to external loads applied only to the non-composite section = 1398 k-ft (taken from spreadsheet)

M_L = moment due to external loads applied only to the composite section = 493 k-ft (taken from spreadsheet)

P_{pe} = effective prestressing force at the section after losses = 933 kips (taken from spreadsheet)

Thus, the axial stress in the web, computed at the composite neutral axis is:

$$f_{pc} = \frac{933}{1083} - \frac{933(44.6)}{1524912}(65.46 - 49.27) + \frac{1398(12)(65.46 - 49.27)}{2586495} + \frac{493(12)(65.46 - 65.46)}{2586495} = 0.995 \text{ ksi}$$

J.5.4 Calculation of shear stress

The shear stress is given by:

$$v = \frac{V_{dnc}Q_{nc}}{t_w I_{nc}} + \frac{V_L Q_c}{t_w I_c} \quad \text{Eq. J.2}$$

Where,

The critical section is at a distance of *dv* from the face of the support.

Loading is due to Service III loads

Q_{nc} = First moment of inertia of the area above *y* (defined in Figure 3.27) taken about the neutral axis of the non-composite section (see Figure 3.27 for a graphical depiction) = 17644 in.^{3} from spreadsheet calculations.

Q_c = Defined in the same manner as Q_{nc} except the composite section is used = 19494 in.^{3} from spreadsheet calculations.

V_{dnc} = shear force applied to the non-composite section only = 181.6 kips (from spreadsheet)

V_L = shear force applied to the composite section = 82.3 kips

Thus:

$$v = \frac{(181.6)(17644)}{(6.125)(1524912)} + \frac{(82.3)(19494)}{(6.125)(2586495)} = 0.444 \text{ ksi}$$

The principal tensile web stress at the composite centroid is calculated from Eq. J.3:

$$-f_t = \frac{f_{pc}}{2} - \sqrt{\left(\frac{f_{pc}}{2}\right)^2 + v^2} \quad \text{Eq. J.3}$$

$$-f_v = \frac{0.995}{2} - \sqrt{\left(\frac{0.995}{2}\right)^2 + 0.444^2} = -0.169 \text{ ksi}$$

The principal stress in the web shall not exceed $0.11\sqrt{f'_c} = 0.11\sqrt{10} = 0.348$ ksi per *AASHTO LRFD* Article 5.8.5. 0.169 < 0.348, thus this stress meets code.