American Association of State Highway and Transportation Officials

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ERRATA

Dear Customer:

Due to errors found after the publication had been completed, AASHTO has reprinted the pages and added two point pages listed below in order to make the following errata changes to the *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges with Design Examples for I-Girder and Box-Girder Bridges*:

Page No(s).	Affected Section	Errata Change
Horizontally Curved Steel Box Girder Example		
pp. 333–336	Appendix E.6	Add the following paragraph immediately before the last paragraph on p. 334:

In certain situations, the Engineer may wish to consider the effect of the access hole in the diaphragm on the section properties of the effective diaphragm section. From separate but similar calculations to the above, considering the effect of an assumed 36-in deep access hole centered in the middle of the example diaphragm, the maximum shear stress in the effective bottom flange due to the diaphragm shear is calculated to be 6.13 ksi. Therefore, in this particular case, the effect of the access hole on the bottom flange shear stress is not significant.

pp. 339–340	Appendix E.9	Change "H525" to "HS25" on p. 339
pp. 341–342	Appendix E.10	Change "H525" to "HS25" on p. 342
pp. 355–356	Appendix E.12	Add the following to the end of the text on p. 355:

Eq. (A10) gives the normal distortional warping stress at any point in the cross section. The value of C_w is obtained from either Figure A2, A3, or A9.

$$\sigma_{dw} = \frac{C_w y}{I\beta a}\left(m\ell \text{ or } T\right) \qquad \text{(A10)}$$

where: y = distance along the transverse vertical axis of the box from the neutral axis to the point under consideration

Eq. (A11) gives the axial (brace) force due to distortional forces applied to the box. The value of C_b is obtained from either Figure A5 or A10.

Page No(s).	Affected Section	Errata Change

Horizontally Curved Steel Box Girder Example

$$F_b = C_b \left[\frac{\sqrt{1+\left(\frac{a+b}{2h}\right)^2}}{2\left(1+\frac{a}{b}\right)} \right] \frac{(m\ell \ \text{ or } \ T)}{a} \qquad \text{(A11)}$$

Since only two loading positions for concentrated loads are considered in the charts, it is often necessary to interpolate between figures. The principle of superposition applies for more than one torque. Figure A11 shows the effect of β on the influence line for diaphragm forces when the diaphragm is rigid."

pp. 355–356	Appendix E.12	Add Figures A2 and A3 before Figure A4 on p. 356
pp. 359–360	Appendix E.13	In the equation for "f_{tran}" on p. 359, which is the third equation from the top of the page, "50 ksi" should be "20 ksi."
pp. 361–362	Appendix E.14	In the middle of p. 362, in the equation for "f_{tran}," "50 ksi" should be "20 ksi."
pp. 363–364	Appendix E.14	Change "H525" to "HS25" on p. 363
pp. 365–366	Appendix E.15	Change "H525" to "HS25" on p. 366

The following new pages have been added to accommodate omitted figures:

pp. 357–357.1	Appendix E.12	Move down Figures A5 through A9
		Add Figure A10 on p. 357.1
pp. 357.2–358	Appendix E.12	Add Figure A11 on p. 357.2

Please substitute the original pages of text with the enclosed pages. We apologize for any inconvenience this may have caused.

AASHTO Publications Staff
October 2003

R_1 shall be taken as:

$$R_1 = \frac{97\sqrt{k}}{\sqrt{\frac{1}{2}\left[\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq. (10-5)}$$

where:

k = plate buckling coefficient
k_s = shear buckling coefficient

Try: k = 4.0 and k_s = 5.34

Since the denominator of Eq (10-5) is approximately equal to 1.0 in this case, $R_1 = 97\sqrt{k} = 97 \text{ x } \sqrt{4.0} = 194$. Since 191 < 194, use Eq. (10-4) to compute the critical flange stress.

$$F_{cr} = 50 \text{ x } 1.0 = 50 \text{ ksi}$$

A very rigid longitudinal flange stiffener is required to provide k = 4.0 to ensure that a node forms at the stiffener. The Engineer should try to match the stiffener size to the required critical flange stress in order to try and obtain the most economical solution. Always assuming k equal to 4.0, which requires the largest size longitudinal stiffener(s), may not be the optimum solution for cases where the resulting higher critical flange stress is not necessary. Since a critical flange stress less than 50 ksi would be satisfactory for this case, try k = 2.0 instead of 4.0, which will result in a lower critical flange buckling stress, but which will also result in a significantly smaller longitudinal flange stiffener. Since the denominator of Eq. (10-5) is again approximately equal to 1.0 with k taken equal to 2.0:

$$R_1 = 97 \text{ x } \sqrt{2.0} = 137 < 191$$

Therefore, compute R_2.

$$R_2 = \frac{210\sqrt{k}}{\sqrt{\frac{1}{1.2}\left[\Delta - 0.4 + \sqrt{(\Delta - 0.4)^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}\right]}} \qquad \text{Eq. (10-7)}$$

$$R_2 = \frac{210\sqrt{2.0}}{\sqrt{\frac{1}{1.2}\left[1.0 - 0.4 + \sqrt{(1.0 - 0.4)^2 + 4\left(\frac{1.18}{50}\right)^2\left(\frac{2.0}{5.34}\right)^2}\right]}}$$

$$R_2 = \frac{297}{\sqrt{0.833\left[0.6 + \sqrt{(1.0 - 0.4)^2 + 4\left(\frac{1.18}{50}\right)^2\left(\frac{2.0}{5.34}\right)^2}\right]}} = 297$$

Check $R_1 < \frac{b_s}{t_f}\sqrt{F_y} \le R_2$. Since 137 < 191 < 297, use Eq. (10-6) to compute the critical flange stress.

$$F_{cr} = F_y\left(\Delta - 0.4\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{R_2 - \frac{b_s}{t_f}\sqrt{F_y}}{R_2 - R_1}\right)\right]\right\}\right) \qquad \text{Eq. (10-6)}$$

$$F_{cr} = 50\left(1.0 - 0.4\left\{1 - \sin\left[\frac{\pi}{2}\left(\frac{297 - \frac{40.5}{1.5}\sqrt{50}}{297 - 137}\right)\right]\right\}\right) = 47.26 \text{ ksi}$$

$$\frac{|-46.47|}{47.26} = 0.98 < 1.00 \qquad \text{OK}$$

The actual value of k_s will be determined in the next section on the design of the longitudinal flange stiffener. The critical stress will then be checked using the actual value of k_s to determine if there is a significant change in the stress.

The bottom flange at interior supports acting in combination with the internal diaphragm is subject to bending in two directions plus the torsional and diaphragm shear (ignoring through-thickness bending of the flange plate under its own self-weight). Therefore, Article 10.4.2.1 also requires that the combined stress in non-composite box flanges at supports, due to vertical bending in the box girder and diaphragm and due to the diaphragm plus torsional shear, caused by the factored loads, not exceed the specified minimum yield stress of the flange. As specified in Article C10.4.2.1, for such cases, the combined stress in the flange due to the factored loads can be checked using the general form of the Huber-von Mises-Hencky yield criterion as follows:

$$\sqrt{\sigma_x^2 - \sigma_x\sigma_y + \sigma_y^2 + 3f_v^2} \le F_y \qquad \text{Eq. (C10-3)}$$

where:

σ_x = average longitudinal stress in the bottom flange due to vertical bending of the girder (ksi)

σ_y = stress in the bottom flange due to vertical bending of the diaphragm over the bearing sole plate (ksi)

f_v = shear stress in the bottom flange due to the sum of the diaphragm and torsional shears (ksi)

F_y = specified minimum yield stress of the bottom flange (ksi)

For a box supported on two bearings (the case in this example), the bottom-flange bending stress due to vertical bending of the diaphragm over the bearing sole plate (i.e. σ_y) is typically relatively small and will be neglected for simplicity in this example. In this case, σ_x can simply be checked instead against the critical stress given by Eq. (10-2), which results from Eq. (C10-3) when σ_y is taken equal to zero.

From previous calculations on pages 331 and 332, the total factored St. Venant torsional shear stress in the bottom flange is equal to 1.18 ksi.

To estimate the shear stress in the bottom flange due to the diaphragm shear, assume a 1 in x 12 in top flange for the diaphragm. As permitted in Article 10.4.2.1, assume that 18 times the thickness of the bottom (box) flange (18 x 1.5 = 27 in) is effective with the diaphragm. The diaphragm is assumed to be 78 inches deep and 1 in thick. From separate calculations, the moment of inertia of the effective section is 112,375 in^4 and the neutral axis is located 31.05 in above the mid-thickness of the bottom flange. Subsequent calculations on page 339 indicate that the total factored vertical component of the diaphragm shear is 1,384 kips. The maximum shear stress in the effective bottom flange due to the diaphragm shear is therefore approximated as:

$$f_v = \frac{VQ}{It_f} = \frac{1{,}384(27/2)(1.5)(31.05)}{112{,}375(1.5)} = 5.16 \text{ ksi}$$

$$f_{v\,tot} = 1.18 + 5.16 = 6.34 \text{ ksi}$$

In certain situations, the Engineer may wish to consider the effect of the access hole in the diaphragm on the section properties of the effective diaphragm section. From separate but similar calculations to the above, considering the effect of an assumed 36-in deep access hole centered in the middle of the example diaphragm, the maximum shear stress in the effective bottom flange due to the diaphragm shear is calculated to be 6.13 ksi. Therefore, in this particular case, the effect of the access hole on the bottom flange shear stress is not significant.

The average factored longitudinal stress in the bottom flange due to vertical bending, σ_x, was computed earlier (page 331) to be –46.47 ksi. Since σ_y is assumed to equal zero in this case, σ_x may be checked against the critical stress given by Eq. (10-2), as discussed previously.

$$F_{cr} = F_y \Delta \qquad \text{Eq. (10-2)}$$

where:

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq. (10-3)}$$

For this check, f_v in Eq (10-3) is taken as the total shear stress in the flange.

$$\Delta = \sqrt{1 - 3\left(\frac{6.34}{50}\right)^2} = 0.976$$

$$F_{cr} = 50(0.976) = 48.80 \text{ ksi}$$

$$\frac{|-46.47|}{48.80} = 0.95 < 1.0 \qquad \text{OK}$$

Note that the direct use of Equation (C10-3) would effectively yield the same answer for this case:

$$\sqrt{\sigma_x^2 - \sigma_x \sigma_y + \sigma_y^2 + 3f_v^2} \le F_y \qquad \text{Eq. (C10-3)}$$

$$\sqrt{(-46.47)^2 + 3(6.34)^2} = 47.75 \text{ ksi}$$

$$\frac{47.75}{50} = 0.95 \qquad \text{OK}$$

Another equally valid approach, which would involve some additional calculation, would be to instead check the combined principal stresses in the flange according to the following form of the Huber-von Mises-Hencky yield criterion:

$$\sqrt{\sigma_1^2 - \sigma_1 \sigma_2 + \sigma_2^2} \le F_y \qquad \text{Eq. (C10-1)}$$

where: σ_1, σ_2 = maximum and minimum principal stresses in the bottom flange (ksi)

$$= \left(\frac{\sigma_x + \sigma_y}{2}\right) \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + f_v^2}$$

(Ref: Ugural, A. C. and Fenster, S. K. (1975). Advanced Strength and Applied Elasticity, Elsevier North Holland Publishing Co., Inc., New York, NY, pp. 105–107)

For this case:

$$\sigma_{1,2} = \left(\frac{-46.47 + 0}{2}\right) \pm \sqrt{\left(\frac{-46.47 - 0}{2}\right)^2 + (6.34)^2}$$

$$\sigma_{1,2} = -47.32 \text{ ksi, } 0.85 \text{ ksi}$$

$$\sqrt{(-47.32)^2 - [(-47.32)(0.85)] + (0.85)^2} = 47.75 \text{ ksi}$$

$$\frac{47.75}{50} = 0.95 \qquad \text{OK}$$

For a box supported on a single bearing, the effect of bending in the plane of the diaphragm is likely to be more significant and should be considered. The effective section specified in Article 10.4.2.1 may be used to compute σ_y due to vertical bending of the internal diaphragm over the sole plate. In this case, either Equation (C10-3) or (C10-1) should then be used to make the check.

Although not illustrated here, bend buckling of the web must also be checked at the Strength limit state according to the provisions of Section 6. The critical compressive flange stresses, computed above, should not exceed the critical compressive web stress (adjusted for the thickness of the flange).

E.9 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36
DESIGN OF INTERNAL DIAPHRAGM

Try a 1-in thick A36 diaphragm plate.

Compute the maximum factored vertical shear in the diaphragm. The dead load shears will be computed by conservatively summing the vertical shears in the critical web of G2 acting on each side of the interior pier section. The shears include the St. Venant torsional shear.

Load	Shear (kips)	Source
Steel	$\lvert -46 \rvert + 47 = 93$	3D Finite Element Analysis
Deck	$\lvert -185 \rvert + 185 = 370$	(in critical web)
SupImp	$\lvert -96 \rvert + 102 = \underline{198}$	
Total DL	661	

The live load shear in the critical web at the interior pier is governed by two lanes of lane loading plus impact. The live load shear in the diaphragm will be computed by summing the vertical shears in the critical web acting on each side of the interior-pier section and subtracting two times the concentrated load portion of the lane loading plus impact (i.e., one concentrated load per lane). For HS25 loading, the concentrated load portion of the lane loading is equal to 26.0 x 1.25 = 32.5 kips. The impact factor is conservatively taken equal to 1.4 (Table 3.5.6.2). Therefore:

Load	Shear (kips)
Live Load HS25 + Impact	$\lvert -163 \rvert + 170 = 333 - 2(32.5)(1.4) = 242$

Determine the total factored vertical shear in the diaphragm at the Strength limit state.

$$V = 1.3(93 + 370 + 198) + 1.3(5/3)(242) = 1{,}384 \text{ kips}$$

Compute the shear capacity according to **AASHTO Eq. (10-113)**.
Separate calculations indicate that C = 1.0.

$$V_u = CV_p \qquad \textbf{AASHTO Eq. (10-113)}$$

$$V_p = 0.58F_y Dt_w = 0.58(36)(78)(1.0) = 1{,}629 \text{ kips}$$

$$V_u = 1.0(1{,}629) = 1{,}629 \text{ kips}$$

$$\frac{1{,}384}{1{,}629} = 0.85 < 1.0 \qquad \text{OK}$$

The internal diaphragm is subject to vertical bending over the bearing sole plates in addition to shear. Therefore, Article 10.2.2.2 requires that the principal stresses in internal support diaphragms be computed and that the combined principal stresses not exceed the specified minimum yield stress of the diaphragm.

Compute the maximum factored shear stress in the diaphragm web. First, separate out the shears due to vertical bending (V_b) and due to St. Venant torsion (V_T).

The sum of the total Steel plus Deck shears is equal to 93 + 370 = 463 kips. Referring to the calculations on page 331, the shear flow in the non-composite box is computed as:

$$SF = \frac{T}{2A_o} = \frac{26}{2(56.0)(12)} = 0.0193 \text{ kips/in}$$

$$V_T = 0.0193(80.4) = 15.52 \text{ kips}$$

The vertical component of V_T is computed as:

$$(V_T)_v = 15.52\left(\frac{78}{80.4}\right) = 15.06 \text{ kips}$$

$$V_b = 463 - 15.06 = 447.9 \text{ kips}$$

The sum of the total Superimposed Dead Load and Live Load plus Impact shears is equal to 198 + 5/3(242) = 601 kips. Referring to the calculations on page 332, the shear flow in the composite box is computed as:

$$SF = \frac{T}{2A_o} = \frac{|-1{,}956|}{2(61.0)(12)} = 1.33 \text{ kips/in}$$

$$V_T = 1.33(80.4) = 106.9 \text{ kips}$$

The vertical component of V_T is computed as:

$$(V_T)_v = 106.9\left(\frac{78}{80.4}\right) = 103.7 \text{ kips}$$

$$V_b = 601 - 103.7 = 497.3 \text{ kips}$$

The factored shear stress due to torsion in the diaphragm web is therefore equal to:

$$(f_v)_T = 1.3(0.0193/1.0 + 1.33/1.0) = 1.75 \text{ ksi}$$

The average factored shear stress due to vertical bending in the diaphragm web is equal to:

$$(f_v)_b = \frac{1.3(447.9 + 497.3)}{78(1.0)} = 15.75 \text{ ksi}$$

Therefore, the total factored shear stress in the diaphragm web is equal to:

$$f_v = (f_v)_T + (f_v)_b = 1.75 + 15.75 = 17.50 \text{ ksi}$$

As specified in Article C10.2.2.2, the combined principal stresses in the diaphragm due to the factored loads can be checked using the general form of the Huber-von Mises-Hencky yield criterion as follows:

$$\sqrt{\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2} \le F_y \qquad \text{Eq. (C10-1)}$$

where: σ_1, σ_2 = critical maximum and minimum principal stresses in the diaphragm (ksi)

$$= \left(\frac{\sigma_y + \sigma_z}{2}\right) \pm \sqrt{\left(\frac{\sigma_y - \sigma_z}{2}\right)^2 + f_v^2} \qquad \text{Eq. (C10-2)}$$

σ_y = stress in the diaphragm due to vertical bending of the diaphragm over the bearing sole plate (ksi)

σ_z = stress in the diaphragm due to bending of the diaphragm about its longitudinal axis (ksi)

f_v = shear stress in the diaphragm (ksi)

F_y = specified minimum yield stress of the diaphragm (ksi)

For a box supported on two bearings (the case in this example), the stress in the diaphragm due to vertical bending of the diaphragm over the bearing sole plate (i.e., σ_y) is typically relatively small and will be neglected for simplicity in this example. σ_z is also typically neglected. From Equation (C10-2), if no bending is assumed, the two principal stresses are simply equal tensile and compressive stresses with a magnitude equal to the shear stress as follows:

$$\sigma_{1,2} = 0 \pm \sqrt{(0)^2 + (17.50)^2} = \pm 17.50 \text{ ksi}$$

Checking the combined principal stresses according to Equation (C10-1):

$$\sqrt{(17.50)^2 - (17.50)(-17.50) + (-17.50)^2} = 30.31 \text{ ksi}$$

$$\frac{30.31}{36} = 0.84 \quad \text{OK}$$

It should be noted that for this case with no bending assumed where $\sigma_2 = -\sigma_1$, the preceding calculation is equivalent to simply limiting the principal stress, σ_1, in the diaphragm, which is equal to the shear stress in the diaphragm due to the factored loads, to the shear yield stress, $\tau_y = F_y / \sqrt{3}$, as follows:

$$\tau_y = \frac{F_y}{\sqrt{3}} = \frac{36}{\sqrt{3}} = 20.78 \text{ ksi}$$

$$\frac{17.50}{20.78} = 0.84 \quad \text{OK}$$

In this case, this check is essentially equivalent to the shear check illustrated on page 339. However, the Engineer is asked to compute the principal stresses in the diaphragm because when the magnitude of the principal tensile stress due to the factored loads is deemed significant, the Engineer may wish to also consider the magnitude of this stress under the factored fatigue live load plus impact. A large principal tensile stress under this loading condition may preclude the use of certain fatigue-sensitive details on the diaphragm. Thus, even for cases where no bending of the diaphragm is assumed, the Engineer may still need to consider the critical principal stress range under the moving fatigue live load when highly fatigue-sensitive details are present on the diaphragm. It should be noted that the direction of the principal tensile stress may change for different positions of the live load.

For a box supported on a single bearing, the effect of the bending stress in the plane of the diaphragm is likely to be more significant and should be considered. As specified in Article 10.4.2.1, a width of the bottom (box) flange equal to 18 times its thickness may be considered effective with the diaphragm in resisting bending. For cases where bending of the diaphragm is considered, more than one loading condition may need to be investigated in order to determine the critical principal stresses in the diaphragm.

For the two-bearing arrangement selected in this example, the section through the access hole is not critical. However, the designer should ensure that sufficient section is provided around the access hole to carry the torsional shear flow without reinforcement of the hole. For a box supported on a single bearing, the section through the access hole is critical and additional stiffening and/or reinforcement around the hole may be necessary.

E.10 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36
DESIGN OF BEARING STIFFENERS

Compute the factored reactions.

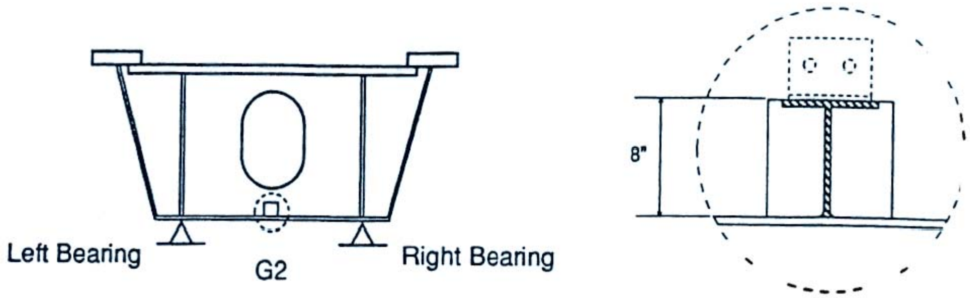

Figure E.10.1 Internal Diaphragm and Bearing Stiffeners at Pier of Girder 2 Looking Upstation

Load	Reaction Location Left	Right		Source
Steel	79	93		3D Finite Element Analysis
Deck	238	370		(Not tabulated)
SupImp	198	26		
Total DL	515 k	489 k		
Live Load HS25 + Impact	252 k	288 k		
	–78 k	–17 k	uplift	

Note that the reported live load plus impact reactions at the left and right bearings are not caused by coincident loads.

$$R_{left} = 1.3\left[515 + \frac{5}{3}(252)\right] = 1{,}216 \text{ kips}$$

$$R_{right} = 1.3\left[489 + \frac{5}{3}(288)\right] = 1{,}260 \text{ kips (controls)}$$

Ignore uplift.

Assume that the bearings are fixed at the piers. Thus, there will be no expansion causing eccentric loading on the bearing stiffeners. Design the bearing stiffeners at this location according to the provisions of Article 6.7.

Use bars with $F_y = 50$ ksi. Compute the maximum permissible width-to-thickness ratio of the stiffener plates according to Eq. (6-13).

$$\frac{b_s}{t_s} \le 0.48\sqrt{\frac{E}{F_y}} = 0.48\sqrt{\frac{29{,}000}{50}} = 11.6$$

where:

F_y = specified minimum yield stress of the stiffeners

Compute the effective area of the diaphragm to which the stiffeners are attached (t_w = 1.0 in) according to the provisions of Article 6.7.

$$A_d = 18t_w^2 = 18 \text{ x } 1.0^2 = 18.0 \text{ in}^2$$

Eq. (A6) tacitly assumes that cross bracing is effective in both compression and tension. If the bracing slenderness is large, the bracing is only effective in tension, and A_b in Equation A6 should be one-half the area of one brace.

The stresses derived from distortion of the box can be determined analogously by solving the BEF problem. Moment in the BEF is analogous to normal distortional stress, σ_{dw}, and deflection in the BEF is analogous to distortional transverse bending stress, σ_t. The reactions in the BEF are analogous to the forces in cross bracing, F_b. Solutions for these three components are presented in graphical form in Figures A2 through A10. [Ed. note: only the pages containing Figures A4 through A9 are reproduced here. Figure A6 is used in this example.] These figures give a "C" value which is used in appropriate equations —A8, A10, A11. These graphs show relationships for uniform torque, m, or concentrated torque, T, at midpanel or diaphragms. The figures give the appropriate "C" values for a given box geometry, β, loading, diaphragm stiffness, q, and spacing, *l*. The designer is able to determine the distortion-related stresses, and estimate how diaphragm spacing and stiffness may be best modified if necessary.

Eq. (A8) gives transverse bending stresses at the top or bottom corners of the box section, depending on the determination of F_d in Equations A9a or A9b. The critical stress may be in either the web or flange. The AASHTO Specification limits the range of the transverse bending stresses to 20,000 psi. [Ed. note: the preceding limit is specified only in **AASHTO Article 10.39.3.2.2** for straight boxes and not in the 1993 **Guide Spec** or the **Guide Specifications**.] Therefore, the torsion in both directions often must be determined. The stress range is the sum of absolute values of stresses due to opposite torques.

$$\sigma_t = C_t\ F_d\ \beta \frac{1}{2a}\left(ml \text{ or } T\right) \qquad \textbf{(A8)}$$

where:

m = uniform torque per unit length
T = concentrated torque

$$F_d = \frac{bv}{2S} \text{ for bottom corner of box} \qquad \textbf{(A9a)}$$

$$F_d = \frac{a}{2S}\left(\frac{b}{a+b} - v\right) \text{ for top corner of box} \qquad \textbf{(A9b)}$$

where:

S = section modulus of transverse member (see Figure A1c)

Eq. (A10) gives the normal distortional warping stress at any point in the cross section. The value of C_w is obtained from either Figure A2, A3, or A9.

$$\sigma_{dw} = \frac{C_w y}{I\beta a}\left(m\ell \text{ or } T\right) \qquad \textbf{(A10)}$$

where: y = distance along the transverse vertical axis of the box from the neutral axis to the point under consideration

Eq. (A11) gives the axial (brace) force due to distortional forces applied to the box. The value of C_b is obtained from either Figure A5 or A10.

$$F_b = C_b\left[\frac{\sqrt{1+\left(\frac{a+b}{2h}\right)^2}}{2\left(1+\frac{a}{b}\right)}\right]\frac{\left(m\ell \text{ or } T\right)}{a}$$

(A11)

Since only two loading positions for concentrated loads are considered in the charts, it is often necessary to interpolate between figures. The principle of superposition applies for more than one torque. Figure A11 shows the effect of β on the influence line for diaphragm forces when the diaphragm is rigid.

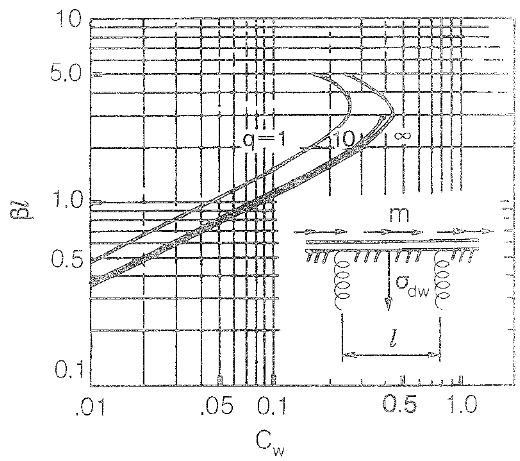

Figure A2. Uniform Torque on Continuous Beam—Normal Distortional Warping Stress at Midpanel

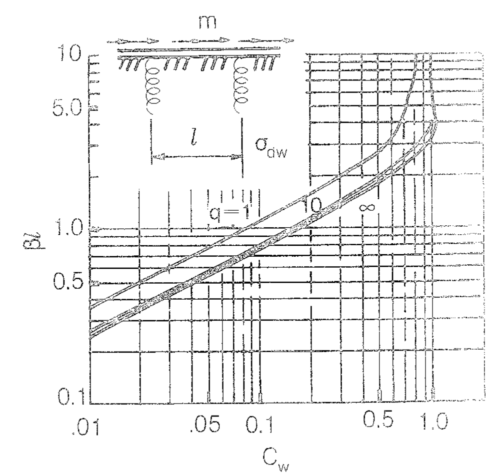

Figure A3. Uniform Torque on Continuous Beam—Normal Distortional Warping Stress at Diaphragm

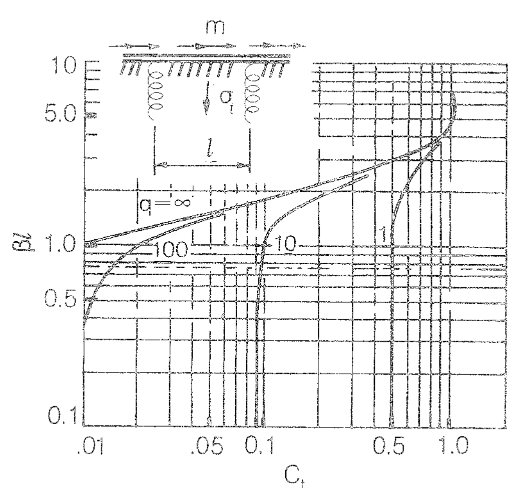

Figure A4. Uniform Torque on Continuous Beam–Distortional Transverse Bending Stress at Midpanel

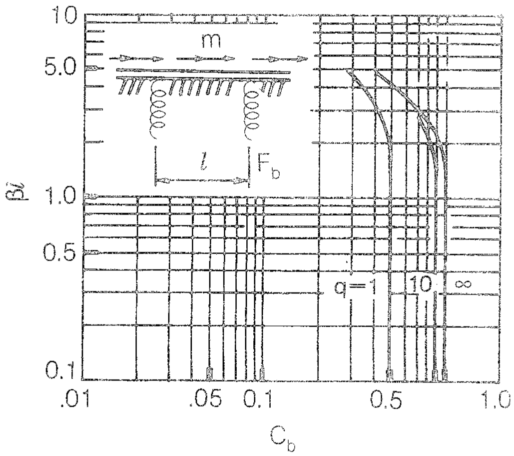

Figure A5. Uniform Torque on Continuous Beam—Diaphragm Force

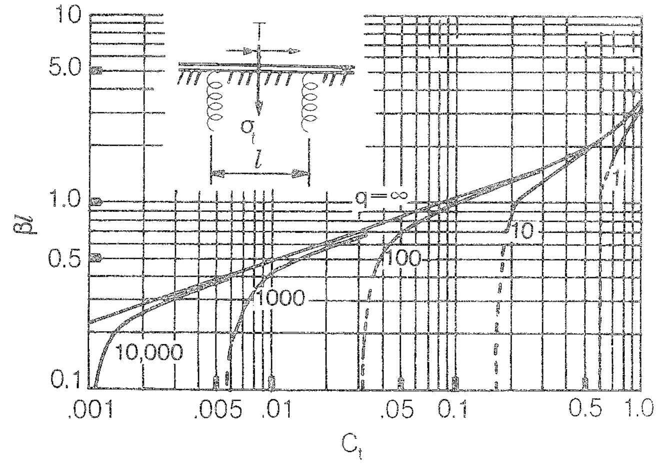

Figure A6. Concentrated Torque at Midpanel on Continuous Beam–Distortional Transverse Bending Stress at Load

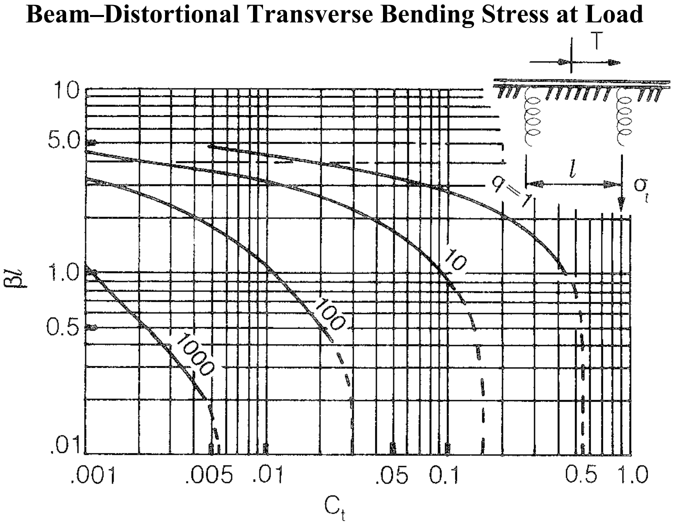

Figure A7. Concentrated Torque at Midpanel on Continuous Beam—Distortional Transverse Bending Stress at Diaphragm

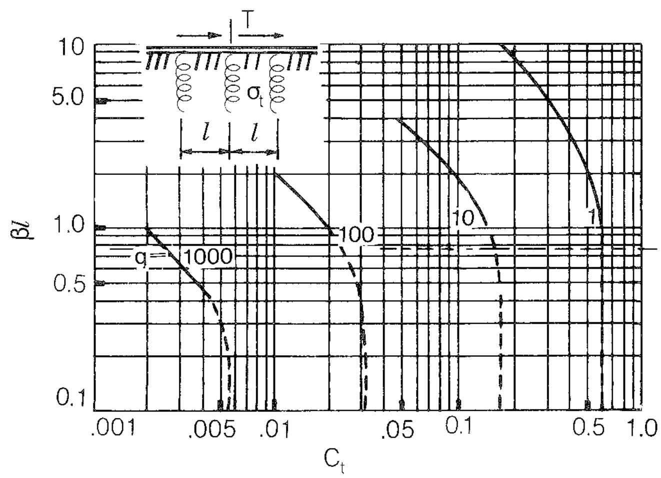

Figure A8. Concentrated Torque at Diaphragm on Continuous Beam—Distortional Transverse Bending Stress at Diaphragm

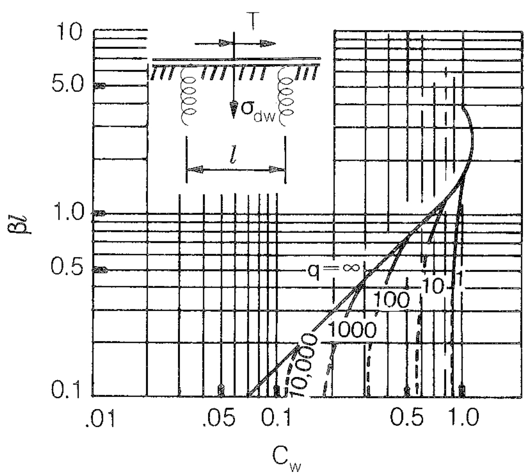

Figure A9. Concentrated Torque at Midpanel on Continuous Beam—Normal Distortional Warping Stress at Midpanel

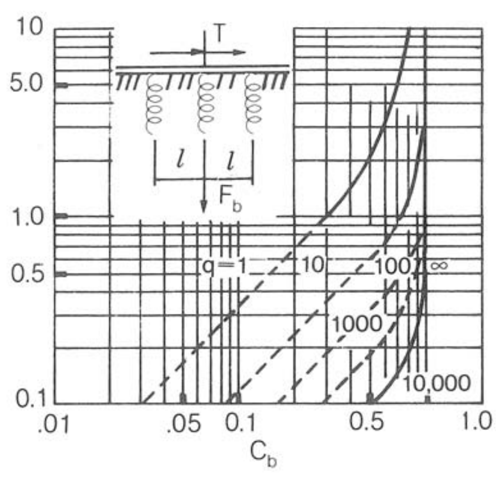

Figure A10. Load at Diaphragm on Continuous Beam—Diaphragm Force

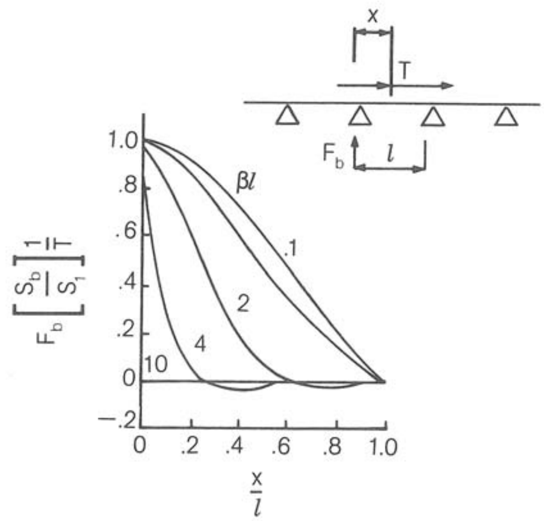

Figure A11. Influence Line for Rigid Diaphragm With Concentrated Torque

E.13 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 COMPOSITE BOTTOM FLANGE OPTION

Assume that the bottom box flange in the negative moment regions will be redesigned using an unstiffened plate with 8 inches of composite concrete according to the provisions of Article 10.4.3. The girder moment due to the added weight of the flange concrete is considered. It is assumed that the concrete is placed after the steel has been erected and before the deck is cast. Thus, the bottom flange will be composite for the deck load. The superimposed dead load is added after the deck and the bottom flange concrete have hardened. Live load plus impact will be applied to the composite section. Only the rebars in the deck will be considered to act compositely with the steel section (with the composite box flange) at the Strength limit state.

Table E.13.1 Section Properties of G2 with 8 inches of 6,000 psi Concrete in Bottom Flange

	Top Flg 18 x 3 Bot Flg 81 x 1.25 (Node 36)		Top Flg 18 x 1.5 Bot Flg 81 x 0.75 (Node 32)	
Section	I (in^4)	NA (in)	I (in^4)	NA (in)
1. Noncomposite	396,155	41.99	238,466	39.59
2. Composite flange 3n w/o deck rebars	438,765	38.26	275,140	34.69
3. Composite flange n w/o deck rebars	502,477	32.68	324,442	28.11
4. Composite flange 3n w/deck rebars	454,801	39.20	293,094	36.06
5. Composite flange n w/deck rebars	560,391	35.22	390,313	31.61

NA is distance from bottom of section to the neutral axis.
Effective deck rebar area for 3n equals 20.0/3 = 6.67 in^2

1. Noncomposite
2. Steel with bottom flange concrete at 3n without deck rebars
3. Steel with bottom flange concrete at n without deck rebars
4. Steel with bottom flange concrete at 3n and deck rebars at 3n
5. Steel with bottom flange concrete at n and deck rebars at n

Properties of bottom flange concrete:

$$f_c' = 6{,}000 \text{ psi} ;\ E_c = 4{,}696 \text{ ksi} ;\ n = 6.2$$

The bottom flange concrete is assumed to be continuous through the interior support diaphragm as required in Article 10.2.2.2. Bottom transverse bracing members (i.e. the bottom struts of the interior cross frames) are assumed to be located above the concrete in this region.

Try a 1.25-inch thick unstiffened bottom flange plate.

Check the transverse bending stress in the bottom flange plate due to the self-weight of the plate and the wet concrete.

Compute the section modulus of a one-foot wide section of the bottom flange plate.

$$S = \frac{1}{6} b t_f^2 = \frac{1}{6} \text{ x } 12 \text{ x } 1.25^2 = 3.13 \text{ in}^3/\text{ft}$$

Compute the moment applied to the flange plate due to the weight of the steel and flange concrete. Assume a simple span between webs.

Steel weight per foot of width, $w_s = \dfrac{1.25 \text{ x } 12 \text{ in x } 3.4}{12} = 4.3$ pounds/in/ft

$$M_{steel} = \frac{1}{8} w_s \ell^2 = \frac{1}{8} \text{ x } 0.0043 \text{ x } 81^2 = 3.53 \text{ k-in/ft}$$

Concrete weight per foot of width, $w_c = 1' \text{ x } 150 \text{ x } \frac{8}{12}\frac{1}{12} = 8.33$ pounds/in/ft

$$M_{conc} = \frac{1}{8} w_c \ell^2 = \frac{1}{8} \text{ x } 0.00833 \text{ x } 81^2 = 6.83 \text{ k-in/ft}$$

Compute the maximum transverse bending stress in the flange plate at the Constructibility limit state. Load factor = 1.4 (Article 3.3).

$$f_{tran} = \frac{M}{S} = \frac{(3.53 + 6.83)}{3.13} \text{ x } 1.4 = 4.63 \text{ ksi} < 20 \text{ ksi} \qquad \text{OK}$$

Although not checked here, Article 10.4.2.1 also limits the maximum vertical deflection of the box flange due to self-weight and the applied permanent loads to 1/360 times the transverse span between webs.

The concrete is to be placed on the bottom flange of the field section over each pier.

The bottom flange concrete causes a longitudinal girder moment of –880 k-ft at Section 5-5 in G2 from the finite element analysis. The longitudinal girder moment due to steel weight = –3,154 k-ft from Table D.1.

Compute the total moment applied to the steel section.

$$M = -3{,}154 + (-880) = -4{,}034 \text{ k-ft}$$

Check the vertical bending stress in the unstiffened plate at the Constructibility limit state.

Compute the vertical bending stress in the bottom flange due to steel weight and concrete. Section properties are from Table E.13.1.

As specified in Article 10.4.3.1, for loads applied prior to hardening of the concrete, composite box flanges are to be designed as non-composite box flanges according to the provisions of Article 10.4.2.

$$f_{bot\ flg} = \frac{-4{,}034 \text{ x } 41.99 \text{ x } 12 \text{ x } 1.4}{396{,}155} = -7.18 \text{ ksi}$$

The critical buckling stress for the non-composite bottom flange with no stiffening according to Article 10.4.2.4.1 equals 24.9 ksi (calculations not shown).

$$\frac{|-7.18|}{24.9} = 0.29 < 1.00 \qquad \text{OK}$$

Compute the factored bottom flange vertical bending stress in the non-composite section at the Strength limit state.

$$f_b = -7.18 \text{ x } \left(\frac{1.3}{1.4}\right) = -6.67 \text{ ksi}$$

Compute the factored bottom flange vertical bending stress in the composite section due to the deck weight.

$$M_{deck} = -12{,}272 \text{ k-ft (Table D.1)}$$

As specified in Article 10.4.3.3, concrete creep is to be considered when checking the compressive steel stress. Concrete compressive stress are to be checked without considering creep. Therefore, use 3n section properties to check the steel stress and n section properties to check the concrete stress.

Compute the factored vertical bending stress in the extreme fiber of the composite bottom flange due to the deck weight. Use creep properties (3n) from Table E.13.1.

$$f_{bot} = \frac{-12{,}272 \text{ x } 38.26}{438{,}765} \text{ x } 12 \text{ x } 1.3 = -16.69 \text{ ksi}$$

Compute the factored vertical bending stress in the extreme fiber of the composite bottom flange concrete due to the deck weight. Use no creep properties (n) from Table E.13.1.

$$f_{bot} = \frac{-12,272 \text{ x } (32.68 - 1.25)}{502,477} \text{ x } 12 \text{ x } 1.3 \text{ x } \frac{1}{6.2} = -1.93 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the superimposed dead load after the deck has hardened.

Superimposed dead load moment = –4,473 k-ft from Table D.1.

Compute the factored vertical bending stress in the extreme fiber of the composite bottom flange due to the superimposed dead load. Use the appropriate creep properties (3n) from Table E.13.1.

$$f_{bot} = \frac{-4,473 \text{ x } 39.20}{454,801} \text{ x } 12 \text{ x } 1.3 = -6.01 \text{ ksi}$$

Compute the factored vertical bending stress in the extreme fiber of the composite bottom flange concrete due to the superimposed dead load. Use the appropriate no creep properties (n) from Table E.13.1.

$$f_{bot} = \frac{-4,473 \text{ x } (35.22 - 1.25)}{560,391} \text{ x } 12 \text{ x } 1.3 \text{ x } \frac{1}{6.2} = -0.68 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the live load. Use the appropriate n section properties from Table E.13.1.

Moment due to live load = –8,566 k-ft from Table D.1.

$$f_{bot} = \frac{-8,566(5/3) \text{ x } 35.22}{560,391} \text{ x } 12 \text{ x } 1.3 = -14.00 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange concrete due to the live load. Use the appropriate n section properties from Table E.13.1.

$$f_{bot} = \frac{-8,566(5/3) \text{ x } (35.22 - 1.25) \text{ x } 12 \text{ x } 1.3}{560,391} \text{ x } \frac{1}{6.2} = -2.18 \text{ ksi}$$

Check the total factored vertical bending stress in the steel bottom flange at the Strength limit state.

$$f_{tot} = -6.67 + (-16.69) + (-6.01) + (-14.00) = -43.37 \text{ ksi}$$

As specified in Article 10.4.3.3, the critical stress for the steel bottom flange in compression at the Strength limit state is given by Eq. (10-4) as follows:

$$F_{cr} = F_y \Delta \qquad \text{Eq. (10-4)}$$

where: $$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_y}\right)^2} \qquad \text{Eq. (10-3)}$$

From Table D.3, the torque due to the steel weight is –22 k-ft (the torque due to the wet bottom concrete is neglected since the load is symmetrical and the curvature effect is relatively small). Using calculations similar to those shown on page 331, the enclosed area of the non-composite box is computed to be $A_o = 55.9 \text{ ft}^2$.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-22|}{2(55.9)(1.25)12} \text{ x } 1.3 = 0.017 \text{ ksi}$$

Subsequent calculations (page 366) show that the factored torque on the composite section at the Strength limit state is equal to –2,481 k-ft. To account for the possibility of concrete creep, it is conservatively assumed that all of this torque is resisted by the steel flange in this computation, as required by Article 10.4.3.4 at the Strength limit state. The enclosed area of the composite box is computed to be $A_o = 61.0\,\text{ft}^2$.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-2,481|}{2(61.0)(1.25)(12)} = 1.36\,\text{ksi}$$

$$(f_v)_{tot} = 0.017 + 1.36 = 1.38\,\text{ksi}$$

The total shear stress satisfies Eq (10-1) in Article 10.4.2.2 by inspection.

$$\Delta = \sqrt{1 - 3\left(\frac{1.38}{50}\right)^2} = 0.999$$

$$F_{cr} = 50(0.999) = 49.95\,\text{ksi}$$

$$\frac{|-43.37|}{49.95} = 0.87 < 1.00 \qquad \text{OK}$$

Compute the factored vertical bending stress in the bottom of the concrete at the Strength limit state assuming no creep.

$$F_{cr\ conc} = 0.85 \text{ x } 6 = 5.10\,\text{ksi} \;\; \text{(Article 10.4.3.3)}$$

$$f_{tot} = -1.93 + (-0.68) + (-2.18) = -4.79\,\text{ksi}$$

$$\frac{|-4.79|}{5.10} = 0.94 < 1.00 \qquad \text{OK}$$

E.14 GIRDER STRESS CHECK SECTION 4-4 G2 NODE 32 COMPOSITE BOTTOM FLANGE OPTION

Try a 0.75-inch thick flange plate at this section with 8 inches of concrete.

Check the transverse bending stress in the bottom flange plate due to the self-weight of the plate and the wet concrete.

Compute the section modulus of a one-foot wide section of the bottom flange plate.

$$S = \frac{1}{6} b t_f^2 = \frac{1}{6} \text{ x } 12 \text{ x } 0.75^2 = 1.13 \text{ in}^3/\text{ft}$$

Compute the moment applied to the flange plate due to the weight of the steel and flange concrete. Assume a simple span between webs.

Steel weight per foot of width, $w_s = \frac{0.75 \text{ x } 12 \text{ in x } 3.4}{12} = 2.6$ pounds/in/ft

$$M_{steel} = \frac{1}{8} w_s \ell^2 = \frac{1}{8} \text{ x } 0.0026 \text{ x } 81^2 = 2.13 \text{ k-in/ft}$$

Compute the maximum transverse bending stress in the flange plate at the Constructibility limit state. Load factor = 1.4.

M_{conc} = 6.83 k-in/ft due to the weight of the concrete from page 359.

$$f_{tran} = \frac{M}{S} = \frac{(2.13 + 6.83)}{1.13 \text{ in}^3} \text{ x } 1.4 = 11.10 \text{ ksi} < 20 \text{ ksi} \qquad \text{OK}$$

The longitudinal girder moment due to the additional concrete in the bottom flange = –358 k-ft from the finite element analysis. The longitudinal girder moment due to the steel weight = –1,896 k-ft (Table D.1).

$$M = -1{,}896 + (-358) = -2{,}254 \text{ k-ft}$$

Check the vertical bending stress in the bottom flange due to the total moment applied to the non-composite section at the Constructibility limit state. Section properties are from Table E.13.1.

$$f_{bot\ flg} = \frac{-2{,}254 \text{ x } 39.59 \text{ x } 12 \text{ x } 1.4}{238{,}466} = -6.29 \text{ ksi}$$

Determine the critical stress for the unstiffened box flange according to Article 10.4.2.4.1.

$$F_{cr} = 8.8 \text{ ksi (calculations not shown)}$$

$$\frac{|-6.29|}{8.8} = 0.71 < 1.00 \qquad \text{OK}$$

Adjust the stress for the Strength limit state load factor.

$$-6.29 \text{ x } \frac{1.3}{1.4} = -5.84 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the deck weight. Use 3n section properties from Table E.13.1.

Moment due to deck weight = –7,599 k-ft from Table D.1.

$$f_{bot} = \frac{-7{,}599 \text{ x } 34.69}{275{,}140} \text{ x } 12 \text{ x } 1.3 = -14.95 \text{ ksi}$$

Compute the factored vertical bending stress in the extreme fiber of the composite bottom flange concrete due to the deck weight. Use n section properties from Table E.13.1.

$$f_{bot} = \frac{-7,599 \text{ x } (28.11 - 0.75)}{324,442} \text{ x } 12 \text{ x } 1.3 \text{ x } \frac{1}{6.2} = -1.61 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the superimposed dead load. Use the appropriate 3n section properties from Table E.13.1.

Superimposed dead load moment = –2,610 k-ft from Table D.1.

$$f_{bot} = \frac{-2,610 \text{ x } 36.06}{293,094} \text{ x } 12 \text{ x } 1.3 = -5.01 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange concrete due to superimposed dead load. Use the appropriate n section properties from Table E.13.1.

$$f_{bot} = \frac{-2,610 \text{ x } (31.61 - 0.75)}{390,313} \text{ x } 12 \text{ x } 1.3 \text{ x } \frac{1}{6.2} = -0.52 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange due to the live load. Use the appropriate n section properties from Table E.13.1.

Moment due to live load = –5,612 k-ft from Table D.1.

$$f_{bot} = \frac{-5,612(5/3) \text{ x } 31.61}{390,313} \text{ x } 12 \text{ x } 1.3 = -11.82 \text{ ksi}$$

Compute the factored vertical bending stress in the composite bottom flange concrete due to live load. Use the appropriate n section properties from Table E.13.1.

$$f_{bot} = \frac{-5,612(5/3) \text{ x } (31.61 - 0.75)}{390,313} \text{ x } 12 \text{ x } 1.3 \text{ x } \frac{1}{6.2} = -1.86 \text{ ksi}$$

Compute the total factored vertical bending stress in the steel bottom flange at the Strength limit state.

$$f_{tot} = -5.84 + (-14.95) + (-5.01) + (-11.82) = -37.62 \text{ ksi}$$

The torque due to the steel weight is -10 k-ft (Table D.3). The enclosed area of the non-composite box is computed to be A_o = 55.2 ft^2.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-10|}{2(55.2)(0.75)(12)} \text{ x } 1.3 = 0.013 \text{ ksi}$$

The factored torque on the composite section at the Strength limit state is computed to be

Loading	Torque (Table D.3)	
Deck	63 x 1.3	82 k-ft
Superimposed DL	–273 x 1.3	–355 k-ft
Live Load HS25 + Impact	–688(5/3) x 1.3	–1,491 k-ft
		–1,764 k-ft

The enclosed area of the composite box is computed to be A_o = 60.8 ft^2.

$$f_v = \frac{T}{2A_o t_f} = \frac{|-1,764|}{2(60.8)(0.75)(12)} = 1.61 \text{ ksi}$$

$$(f_v)_{tot} = 0.013 + 1.61 = 1.62 \text{ ksi}$$

which satisfies Eq. (10-1) in Article 10.4.2.2 by inspection.

$$\Delta = \sqrt{1 - 3\left(\frac{1.62}{50}\right)^2} = 0.998$$

$$F_{cr} = 50(0.998) = 49.90\text{ ksi}$$

$$\frac{|-37.62|}{49.90} = 0.75 < 1.00 \qquad \text{OK}$$

Compute the total factored vertical bending stress in the bottom flange concrete at the Strength limit state.

$$f_{tot} = -1.61 + (-0.52) + (-1.86) = -3.99\text{ ksi}$$

$$F_{cr\ conc} = 5.10\text{ ksi}$$ (from page 361)

$$\frac{|-3.99|}{5.10} = 0.78 < 1.00 \qquad \text{OK}$$

E.15 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36
COMPOSITE BOTTOM FLANGE OPTION DESIGN OF SHEAR CONNECTORS—STRENGTH

Check the ultimate strength of the shear connectors on the composite bottom flange according to the provisions of Article 10.4.3.5, which refer back to the provisions of Article 7.2.1. As specified in Article 10.4.3.5, the radial force due to curvature is ignored. The longitudinal force to be developed is given by Eq. (7-4), with b_d taken as the full width of the bottom flange concrete.

$$\text{Required capacity} = 0.85 f_c' b_d t_d \qquad \text{Eq. (7-4)}$$

$$P = 0.85 \text{ x } 6 \text{ ksi x } 81 \text{ in x } 8 \text{ in } = 3{,}305 \text{ kips}$$

Compute the capacity of one shear connector.

$$\frac{H}{d} = \frac{6}{0.875} = 6.86 > 4.0$$

$$S_u = 0.4d^2\sqrt{f_c' E_c} \le 60{,}000 A_{sc} \qquad \textbf{AASHTO Eq. (10-67)}$$

(Note: the upper limit of $60{,}000A_{sc}$ in the above equation is incorporated in the 2000 Interims to the Standard Specifications and is included here.)

$$A_{sc} = \pi(0.875)^2/4 = 0.6 \text{ in}^2$$

$$S_u = 0.4 \text{ x } 0.875^2\sqrt{6 \text{ x } 4{,}696} = 51.4 \text{ kips} > 60(0.6) = 36 \text{ kips}$$

$$\therefore S_u = 36 \text{ kips}$$

Compute the minimum number of shear connectors required on each side of the pier.

$$\text{No. of shear connectors req'd.} = \frac{P}{\phi_{sc} S_u} = \frac{3{,}305}{0.85(36)} = 108.0$$

Try six studs uniformly spaced across the flange (Figure E.15.1) with 18 rows on each side of the pier (for 108 shear connectors per flange on each side of the pier). The studs must be spaced transversely so that the steel plate slenderness limit of R_1 in Eq (10-4) is satisfied, where b_f is taken as the transverse spacing between the shear connectors (Article 10.4.3.5).

Check the computed force on the critical studs at Node 36.

Compute the axial force in the bottom flange concrete due to the vertical moment. Compute the stresses in the top of the composite bottom flange concrete due to the deck weight without creep. Use the ratio of the distances to the neutral axis.

$$f_{top} = -1.93\left(\frac{32.68 - 9.25}{32.68 - 1.25}\right) = -1.44 \text{ ksi}$$

Compute the stress in the top of the composite bottom flange concrete due to superimposed dead load and live load without creep.

$$f_{top} = \left[-0.68 + (-2.18)\right]\left(\frac{35.22 - 9.25}{35.22 - 1.25}\right) = -2.19 \text{ ksi}$$

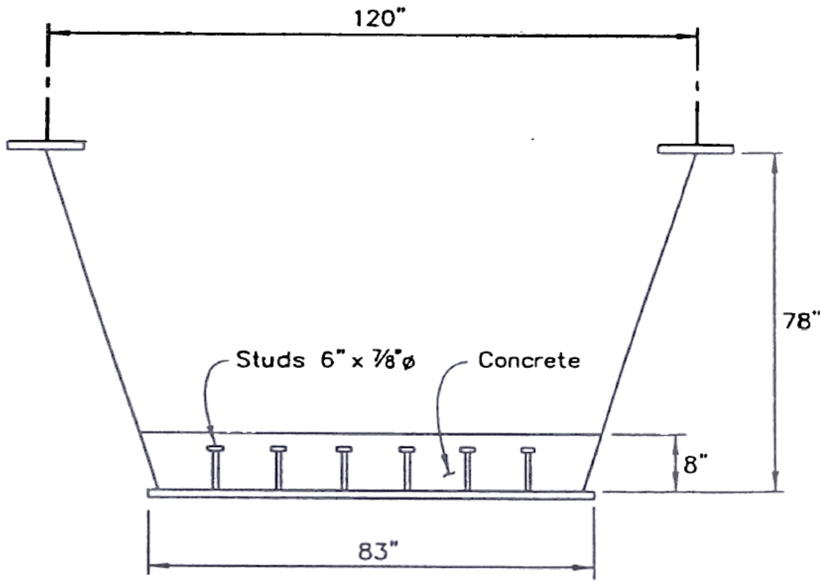

Figure E.15.1 Shear Studs on Composite Bottom Flange

Compute the longitudinal force in the composite bottom flange concrete due to vertical bending. Use the average bending stress in the concrete times the area of the concrete.

$$F = \left(\frac{1.93 + 1.44}{2} + \frac{2.86 + 2.19}{2}\right) \text{x } 81 \text{ x } 8 = 2,728 \text{ kips}$$

Compute the longitudinal force per stud.

$$F_L = 2,728/108 = 25.26 \text{ kips/stud}$$

Compute the St. Venant torsional shear in the concrete at the Strength limit state. Assume that a single row of studs across the flange will resist the torsional shear in the flange concrete.

Loading	Torque (Table D.3)	
Deck	48 x 1.3	62 k-ft
Superimposed DL	–346 x 1.3	–450 k-ft
Live load HS25 + Impact	–966(5/3) x 1.3	–2,093 k-ft
	Total	–2,481 k-ft

Assume all torsion is applied to the uncracked section without creep.
Effective concrete thickness = Thickness/n = 8 in /6.2 = 1.29 in.

Using calculations similar to those shown on page 332, the enclosed area of the composite box is computed to be $A_o = 61.0 \text{ ft}^2$.

$$V = \frac{T}{2A_o} b_f$$

$$V = \frac{|-2,481|}{2 \text{ x } 61.0} \frac{81}{12} = 137 \text{ kips}$$

Compute the portion of the torsional shear resisted by the concrete by taking the ratio of the effective concrete thickness to the total thickness of the steel flange plus the effective concrete.

$$V_{conc} = 137 \text{ kips x } \frac{1.29 \text{ in}}{(1.25 \text{ in} + 1.29 \text{ in})} = 70 \text{ kips}$$