AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO
THE VOICE OF TRANSPORTATION

Kirk T. Steudle, P.E., President
Director, Michigan Department of Transportation

John Horsley, Executive Director

444 North Capitol Street NW, Suite 249, Washington, DC 20001
(202) 624-5800 Fax: (202) 624-5806 • www.transportation.org

ERRATA

June 2012

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *AASHTO LRFD Bridge Design Specifications*, 6th Edition.

Please replace the existing text with the corrected text to ensure that your edition is both accurate and current.

AASHTO staff sincerely apologizes for any inconvenience.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
Section 3		
3-13	The last row of Table 3.4.1-1, column 1, reads "Fatigue I II"	Change "Fatigue I II" to "Fatigue II".
Section 4		
4-51	Eq. C4.6.2.5-3 reads: $G = \frac{\Sigma\left(\frac{E_c I_c}{L_c}\right)}{\Sigma\left(\frac{E_g I_c}{L_g}\right)}$	Revise denominator to read: $G = \frac{\Sigma\left(\frac{E_c I_c}{L_c}\right)}{\Sigma\left(\frac{E_g I_g}{L_g}\right)}$
4-60 through 4-61	Article includes the following extra content: Table 4.6.2.6.4-1, Figure 4.6.2.6.4-1, four specification paragraphs, and two commentary paragraphs.	In Article 4.6.2.6.4, delete the table, figure and last four paragraphs in the Article. In the commentary to Article 4.6.2.6.4, delete the last two paragraphs.
4-70	In Article 4.6.3.2.4, the second sentence of the first paragraph reads "The structural model should include all components and connections and consider local structural stress at fatigue prone details as shown in Table 6.6.1.2.3-3."	Change table number from "6.6.1.2.3-3" to "6.6.1.2.3-1".
	In Article C4.6.3.2.4, FHWA citation is shown as pending.	Update to "2012".
4-93	FHWA (2012) is cited (see above) but the reference is missing from Article 4.9.	Add the following reference: FHWA. 2012. *Manual for Design, Construction, and Maintenance of Orthotropic Steel Bridges*. Federal Highways Administration, U.S. Department of Transportation, Washington, DC.
Section 5		
5-26	The first bullet in Article 5.5.4.2.1 reads: • For shear and torsion: normal weight concrete……………. 0.90 lightweight concrete………………...0.80	Delete bullet.
	The fourth bullet reads: • For shear and torsion: normal weight concrete……………. 0.90 lightweight concrete………………...0.70	Change second value so bullet reads: • For shear and torsion: normal weight concrete……………. 0.90 lightweight concrete………………...0.80
5-39	In Eqs. 5.7.3.1.1-3 and 5.7.3.1.1-4, the subscript in "β_1" runs into the next variable in the expression.	Reformat the subscript so that it isn't overlooked.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
5-46	The second to last paragraph of Article 5.7.3.4 reads: The maximum spacing of the skin reinforcement shall not exceed either d_e /6 or 12.0 in.	Correct subscript from "*e*" to "ℓ" to read: The maximum spacing of the skin reinforcement shall not exceed either d_ℓ /6 or 12.0 in.
5-80 through 5-82	Most equations in Article 5.8.4.2 have the "÷" symbol.	Reset in standard algebraic notation.
5-81	Eq. 5.8.4.2-2 reads: $V_{ui} = v_{ui} A_{cv} = v_{ui} 12 b_v$	Revise to read: $V_{ui} = v_{ui} A_{cv} = v_{ui} 12 b_{vi}$
5-84	The second bullet of Article 5.8.4.4 includes the phrase "…provisions of Article 5.8.1.1 is…"	Revise the article number to read "…provisions of Article 5.8.2.5 is…"
5-108	Eq. 5.9.5.4.3b-1 reads: $\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b \left[\left(t_f, t_i \right) - \psi_b \left(t_d, t_i \right) \right] K_{df} + \frac{E_p}{E_c} \Delta f_{cd} \psi_b \left(t_f, t_d \right) K_{df}$	Revise flat bracket placement to read: $\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp} \left[\psi_b \left(t_f, t_i \right) - \psi_b \left(t_d, t_i \right) \right] K_{df} + \frac{E_p}{E_c} \Delta f_{cd} \psi_b \left(t_f, t_d \right) K_{df}$
5-121	Eq. C5.10.8-1 reads: $A_s \geq \frac{1.3 A_g}{Perimeter(fy)}$	Format the "Format the "*y*" as a subscript to read: $A_s \geq \frac{1.3 A_g}{Perimeter\left(f_y \right)}$
Section 6		
6-32	In Article 6.6.1.2.1, the second sentence of the third paragraph reads: In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination specified in Table 3.4.1-1.	Revise to read: In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than the maximum live load tensile stress caused by the Fatigue I load combination specified in Table 3.4.1-1.
6-34	In Article 6.6.1.2.3, a paragraph after the second paragraph is missing.	Insert the following: For components and details on fracture-critical members, the Fatigue I load combination specified in Table 3.4.1-1 should be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5.
6-34 and 6-45	Last paragraph of C6.6.1.2.3 is misplaced.	Move paragraph just after the fifth paragraph.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
6-35 through 6-36	In Table 6.6.1.2.3-1, the descriptions for Conditions 2.1 through 2.3 are incomplete.	Add the following to the condition descriptions: (Note: see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)
6-36	In Table 6.6.1.2.3-1, Condition 2.5 is missing.	Add Condition 2.5.
6-42	In Table 6.6.1.2.3-1, the description for Condition 7.1 is incomplete.	Add the following to the condition description: (Note: see Condition 7.2 for welded angle or tee section member connections to gusset or connection plates.)
	In Table 6.6.1.2.3-1, Condition 7.2 is missing.	Add Condition 7.2.
6-43 through 6-44	In Table 6.6.1.2.3-1, the figures for Conditions 8.1 through 8.9 display "$\Delta\sigma$".	Replace with figures that display "Δf".
6-46 through 6-47	Table 6.6.1.2.3-3 is redundant.	Delete table.
6-93	In the where list for Eq. 6.9.4.2.2-9, the definition of A_{eff} ends with "$\sum(b-b_e)\,t$".	Revise the definition for A_{eff} as follows: summation of the effective areas of the cross-section based on a reduced effective width for each slender stiffened element in the cross-section $A-\sum(b-b_e)\,t$ (in.^{2})
6-95	In the open-circle bullet immediately above Eq. 6.9.4.4-1, the expression "$\frac{\ell}{r_x} \leq 80$" is too small.	Remove the subscripting of "$\frac{\ell}{r_x} \leq 80$".
6-108	The last sentence of the first paragraph of Article 6.10.1.7 reads as follows: The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 60.0 ksi and a size not exceeding No. 6 bars.	Reword to read: The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 60.0 ksi; the size of the reinforcement should not exceed No. 6 bars.
6-109	The last sentence of the second paragraph of Article 6.10.1.7 reads as follows: The individual bars shall be spaced at intervals not exceeding 12.0 in.	Reword to read: The individual bars should be spaced at intervals not exceeding 12.0 in.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
6-134 through 6-140	The last sentence of the third paragraph of Article C6.10.6.2.3 reads as follows: Research has not yet been conducted to extend the provisions of Appendix A6.	Revise the sentence and add to the end of the paragraph to read as follows: Research has not yet been conducted to extend the provisions of Appendix A6 either to sections in kinked (chorded) continuous or horizontally curved steel bridges or to bridges with supports skewed more than 20 degrees from normal. Severely skewed bridges with contiguous cross-frames have significant transverse stiffness and thus already have large cross-frame forces in the elastic range. As interior-pier sections yield and begin to lose stiffness and shed their load, the forces in the adjacent cross-frames will increase. There is currently no established procedure to predict the resulting increase in the forces without performing a refined nonlinear analysis. With discontinuous cross-frames, significant lateral flange bending effects can occur. The resulting lateral bending moments and stresses are amplified in the bottom compression flange adjacent to the pier as the flange deflects laterally. There is currently no means to accurately predict these amplification effects as the flange is also yielding. Skewed supports also result in twisting of the girders, which is not recognized in plastic-design theory. The relative vertical deflections of the girders create eccentricities that are also not recognized in the theory. Thus, until further research is done to examine these effects in greater detail, a conservative approach has been taken in the specification.
6-174	The text immediately under the where list for Eq. 6.11.1.1-1 is shown as a new paragraph rather than a continuation.	Remove the indent at the beginning of the paragraph immediately under the where list for Eq. 6.11.1.1-1.
6-183	The text immediately under the bullet items in Article 6.11.5 is shown as a new paragraph rather than a continuation.	Remove the indent at the beginning of the paragraph immediately under the bullet items in Article 6.11.5.
	The third sentence of the text immediately under the bullet items in Article 6.11.5 is incomplete.	Reword to read: The allowables specified for nonredundant members are arbitarily reduced from those specified for redundant members due to the more severe consequences of failure of a nonredundant member.
6-190	In Article 6.11.8.2.2, the first paragraph reads: The nominal flexural resistance of the compression flange shall be taken as:	Add variable to read: The nominal flexural resistance of the compression flange, F_{nc}, shall be taken as:

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
6-191	Eq. 6.11.8.2.2-1 reads:	Remove "ϕ_f" just after equal sign to read:
	$F_{nc} = \phi_f F_{cb} \sqrt{1-\left(\frac{f_v}{\phi_v F_{cv}}\right)^2}$	$F_{nc} = F_{cb} \sqrt{1-\left(\frac{f_v}{\phi_v F_{cv}}\right)^2}$
	In Article 6.11.8.2.2, the variable description under "in which" reads:	Add missing phrase to read:
	F_{cb} = nominal axial compression buckling resistance of the flange calculated as follows:	F_{cb} = nominal axial compression buckling resistance of the flange under compression alone calculated as follows:
	Eq. 6.11.8.2.2-3 reads:	Replace "1" with "Δ" in two places to read:
	$F_{cb} = R_b R_h F_{yc}\left[1-\left(1-\frac{\Delta-0.3}{R_h}\right)\left(\frac{\lambda_f-\lambda_p}{\lambda_r-\lambda_p}\right)\right]$	$F_{cb} = R_b R_h F_{yc}\left[\Delta-\left(\Delta-\frac{\Delta-0.3}{R_h}\right)\left(\frac{\lambda_f-\lambda_p}{\lambda_r-\lambda_p}\right)\right]$
	The variable description following Eq. 6.11.8.2.2-4 reads:	Add missing phrase to read:
	F_{cv} = nominal shear buckling resistance of the flange calculated as follows:	F_{cv} = nominal shear buckling resistance of the flange under shear alone calculated as follows:
	In the second paragraph of Article C6.11.8.2.2, the sentence following Eq. C6.11.8.2.2-1 reads as follows:	Revise to show the complete equation number as follows:
	Rearranging Eq. C6.11.8.2.2-1 in terms of f_c and substituting F_{nc} for f_c facilitates the definition of the nominal flexural resistance of the compression flange as provided in Eq. 6.11.8.2.2.	Rearranging Eq. C6.11.8.2.2-1 in terms of f_c and substituting F_{nc} for f_c facilitates the definition of the nominal flexural resistance of the compression flange as provided in Eq. 6.11.8.2.2-1.
	In Article C6.11.8.2.2, the first sentence of the third paragraph reads as follows:	Revise to include omitted phrase as follows:
	The nominal axial compression buckling resistance of the flange, F_{cb}, is defined for three distinct regions based on the slenderness of the flange.	The nominal axial compression buckling resistance of the flange under compression alone, F_{cb}, is defined for three distinct regions based on the slenderness of the flange.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
6-191 (cont'd.)	In Article C6.11.8.2.2, the fifth paragraph reads: The equations for the nominal shear buckling resistance of the flange, F_{cv}, are determined from the equations for the constant, *C*, given in Article 6.10.9.3.2, where *C* is the ratio of the shear buckling resistance to the shear yield strength of the flange taken as $F_{yc}/\sqrt{3}$.	Revise to include omitted phrase as follows: The equations for the nominal shear buckling resistance of the flange under shear alone, F_{cv}, are determined from the equations for the constant, *C*, given in Article 6.10.9.3.2, where *C* is the ratio of the shear buckling resistance to the shear yield strength of the flange taken as $F_{yc}/\sqrt{3}$.
6-192	Eq. 6.11.8.2.2-13 reads: $(\Delta - 0.3)F_{yc} \leq F_{yw}$	Revise to read: $(\Delta - 0.3)F_{yc}$
6-202	In Eq. 6.12.2.2.1-4, there is an extra zero in the first term of equation, reading "0.038".	Delete zero to right of decimal point to read "0.38".
	In the where list for Eq. 6.12.2.2.2-1, there are separate definitions for *b* and *t* instead of one for *b*/*t*.	Replace definitions for *b* and *t* with the following: *b*/*t* = width of any flange or depth of any web component divided by its thickness neglecting any portions of flanges or webs that overhang the box perimeter
6-225	First paragraph of Article 6.13.2.10.3 is not indented.	Indent paragraph.
6-263	The second FHWA reference reads as follows: FHWA. 2011. *Manual for Design, Construction, and Maintenance of Orthotropic Steel Bridges*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.	Revise year and title to read: FHWA. 2012. *Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.
6-304	In Figure C6.4.5-1, the decision branch on the right side involving "Shored Construction" is incorrect.	Replace with a singular box in flowchart reading "Concrete compressive stress ≤ $0.6f'_c$".
6-315	The figure immediately below Table D6.1-2 is incorrect.	Replace figure.
Section 9		
9-26 through 9-30	Articles 9.8.3.6 and 9.8.3.7 need revisions.	After moving three paragraphs of Article C9.8.3.6 to Article C9.8.3.7, delete the rest of Article 9.8.3.6, renumber Article 9.8.3.7 as 9.8.3.6, and renumber object references and article cross references as needed.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Page	Existing Text	Corrected Text
9-28	Article 9.8.3.6.4, item d reads: Combined fillet-groove welds may have to be used in cases where the required size of fillet welds to satisfy the fatigue resistance requirements would be excessive, if used alone.	Revise to read: Combined fillet-groove welds may have to be used 1) in cases where the required size of fillet welds to satisfy the fatigue resistance requirements would be excessive if used alone or 2) to accomplish a ground termination.
9-44		
Section 12		
12-72	Eq. 12.12.2.2-2 is missing parentheses.	Revise to read: $\Delta_t = \dfrac{K_B\left(D_L P_{sp} + C_L P_L\right) D_o}{1000\left(E_p I_p / R^3 + 0.061\, M_s\right)} + \varepsilon_{sc} D$
12-74	Eq. 12.12.3.5-1 is missing parentheses.	Revise to read: $P_u = \eta_{EV}\left(\gamma_{EV} K_{\gamma E} K_2 VAF\, P_{sp} + \gamma_{WA} P_w\right) + \eta_{LL}\gamma_{LL} P_L\, C_L$
12-80	Eq. 12.12.3.9-1 is missing parentheses.	Revise to read: $P_L = \dfrac{P(1 + IM/100)m}{[L_0 + (12H + K_1)LLDF][W_0 + (12H + K_1)LLDF]}$
12-83	Eq. 12.12.3.10.1e-2 is missing parentheses.	Revise to read: $\varepsilon_{bck} = \dfrac{1.2C_n\left(E_p I_p\right)^{\frac{1}{3}}}{A_{eff} E_p}\left[\dfrac{\phi_s M_s\left(1 - 2\nu\right)}{\left(1 - \nu\right)^2}\right]^{\frac{2}{3}} R_h$

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

The load factor for settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{SE}, may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL + PS \quad (3.4.1\text{-}2)$$

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts and Structural Plate Culverts with Deep Corrugations		1.5	0.9
○ Thermoplastic culverts		1.3	0.9
○ All others		1.95	0.9
ES: Earth Surcharge		1.50	0.75

Table 3.4.1-3—Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effectuve}$	1.0	1.0
Steel Substructures	1.0	1.0

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

For unbraced frames:

$$\frac{G_a G_b \left(\frac{\pi}{K}\right)^2 - 36}{6\,(G_a + G_b)} = \frac{\frac{\pi}{K}}{\tan\left(\frac{\pi}{K}\right)} \qquad \text{(C4.6.2.5-2)}$$

where subscripts a and b refer to the two ends of the column under consideration

in which:

$$G = \frac{\Sigma\left(\frac{E_c I_c}{L_c}\right)}{\Sigma\left(\frac{E_g I_g}{L_g}\right)} \qquad \text{(C4.6.2.5-3)}$$

where:

Σ = summation of the properties of components rigidly connected to an end of the column in the plane of flexure
E_c = modulus of elasticity of column (ksi)
I_c = moment of inertia of column (in.^{4})
L_c = unbraced length of column (in.)
E_g = modulus of elasticity of beam or other restraining member (ksi)
I_g = moment of inertia of beam or other restraining member (in.^{4})
L_g = unsupported length of beam or other restraining member (in.)
K = effective length factor for the column under consideration

Figures C4.6.2.5-1 and C4.6.2.5-2 are graphical representations of the relationship among K, G_a, and G_b for Eqs. C4.6.2.5-1 and C4.6.2.5-2, respectively. The figures can be used to obtain values of K directly.

Eqs. C4.6.2.5-1, C4.6.2.5-2, and the alignment charts in Figures C4.6.2.5-1 and C4.6.2.5-2 are based on assumptions of idealized conditions. The development of the chart and formula can be found in textbooks such as Salmon and Johnson (1990) and Chen and Lui (1991). When actual conditions differ significantly from these idealized assumptions, unrealistic designs may result. Galambos (1988), Yura (1971), Disque (1973), Duan and Chen (1988), and AISC (1993) may be used to evaluate end conditions more accurately.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

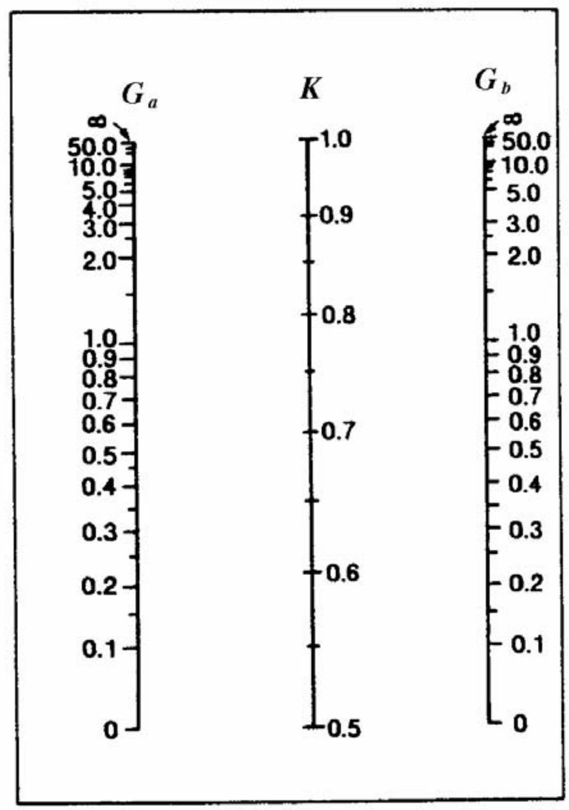


Figure C4.6.2.5-1—Alignment Chart for Determining Effective Length Factor, *K*, for Braced Frames

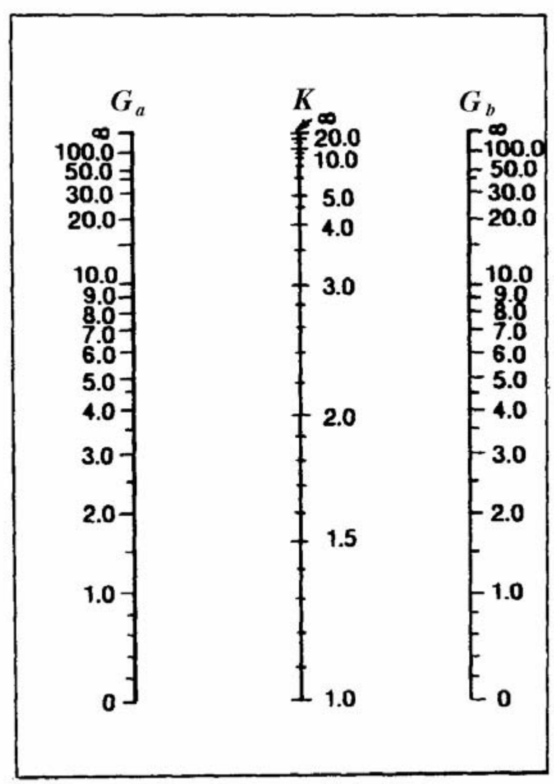


Figure C4.6.2.5-2—Alignment Chart for Determining Effective Length Factor, *K*, for Unbraced Frames

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

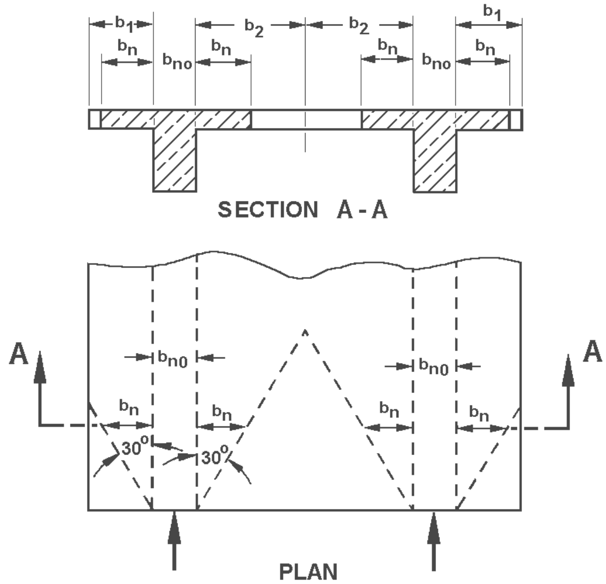


Figure 4.6.2.6.2-4—Effective Flange Widths, b_n, for Normal Forces

4.6.2.6.3—Cast-in-Place Multicell Superstructures

The effective width for cast-in-place multiweb cellular superstructures may be taken to be as specified in Article 4.6.2.6.1, with each web taken to be a beam, or it may be taken to be the full width of the deck slab. In the latter case, the effects of shear lag in the end zones shall be investigated.

4.6.2.6.4—Orthotropic Steel Decks

The effective width need not be determined when using refined analysis as specified in Article 4.6.3.2.4. For simplified analysis, the effective width of the deck, including the deck plate and ribs, acting as the top flange of a longitudinal superstructure component or a transverse beam may be taken as:

- $L/B \geq 5$: fully effective
- $L/B < 5$: $b_{od} = \frac{1}{5}L$

where:

L = span length of the orthotropic girder or transverse beam (in.)
B = spacing between orthotropic girder web plates or transverse beams (in.)
b_{od} = effective width of orthotropic deck (in.)

C4.6.2.6.4

Consideration of effective width of the deck plate can be avoided by application of refined analysis methods.

The procedures in *Design Manual for Orthotropic Steel Plate Deck Bridges* (AISC, 1963) may be used as an acceptable means of simplified analysis; however, it has been demonstrated that using this procedure can result in rib effective widths exceeding the rib spacing, which may be unconservative.

Tests (Dowling et al., 1977) have shown that for most practical cases, shear lag can be ignored in calculating the ultimate compressive strength of stiffened or unstiffened girder flanges (Lamas and Dowling, 1980; Burgan and Dowling, 1985; Jetteur et al., 1984; and Hindi, 1991). Thus, a flange may normally be considered to be loaded uniformly across its width. It necessary to consider the flange effectiveness in greater detail only in the case of flanges with particularly large aspect ratios ($L/B < 5$) or particularly slender edge panels or stiffeners (Burgan and Dowling, 1985 and Hindi, 1991) is it necessary to consider the flange effectiveness in greater detail.

Consideration of inelastic behavior can increase the effective width as compared to elastic analysis. At

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

for strength limit states for positive and negative flexure. For service and fatigue limit states in regions of high shear, the effective deck width can be determined by refined analysis or other accepted approximate methods.

ultimate loading, the region of the flange plate above the web can yield and spread the plasticity (and distribute stress) outward if the plate maintains local stability. Results from studies by Chen et al. (2005) on composite steel girders, which included several tub-girder bridges, indicate that the full slab width may be considered effective in both positive and negative moment regions.

Thus, orthotropic plates acting as flanges are considered fully effective for strength limit state evaluations from positive and negative flexure when the L/B ratio is at least 5. For the case of $L/B < 5$, only a width of one-fifth of the effective span should be considered effective. For service and fatigue limit states in regions of high shear, a special investigation into shear lag should be done.

4.6.2.6.5—Transverse Floorbeams and Integral Bent Caps

For transverse floorbeams and for integral bent caps designed with a composite concrete deck slab, the effective flange width overhanging each side of the transverse floorbeam or bent cap web shall not exceed six times the least slab thickness or one-tenth of the span length. For cantilevered transverse floorbeams or integral bent caps, the span length shall be taken as two times the length of the cantilever span.

C4.6.2.6.5

The provisions for the effective flange width for transverse floorbeams and integral bent caps are based on past successful practice, specified by Article 8.10.1.4 of the 2002 *AASHTO Standard Specifications*.

[*This space is intentionally left blank. —ed.*]

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

[*This page is intentionally left blank. —ed.*]

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

4.6.2.7—Lateral Wind Load Distribution in Multibeam Bridges

4.6.2.7.1—I-Sections

In bridges with composite decks, noncomposite decks with concrete haunches, and other decks that can provide horizontal diaphragm action, wind load on the upper half of the outside beam, the deck, vehicles, barriers, and appurtenances shall be assumed to be directly transmitted to the deck, acting as a lateral diaphragm carrying this load to supports. Wind load on the lower half of the outside beam shall be assumed to be applied laterally to the lower flange.

For bridges with decks that cannot provide horizontal diaphragm action, the lever rule shall apply for distribution of the wind load to the top and bottom flanges.

Bottom and top flanges subjected to lateral wind load shall be assumed to carry that load to adjacent brace points by flexural action. Such brace points occur at wind bracing nodes or at cross-frames and diaphragm locations.

The lateral forces applied at brace points by the flanges shall be transmitted to the supports by one of the following load paths:

- Truss action of horizontal wind bracing in the plane of the flange;
- Frame action of the cross-frames or diaphragms transmitting the forces into the deck or the wind bracing in the plane of the other flange, and then by diaphragm action of the deck, or truss action of the wind bracing, to the supports;
- Lateral bending of the flange subjected to the lateral forces and all other flanges in the same plane, transmitting the forces to the ends of the span, for example, where the deck cannot provide horizontal diaphragm action, and there is no wind bracing in the plane of either flange.

C4.6.2.7.1

Precast concrete plank decks and timber decks are not solid diaphragms and should not be assumed to provide horizontal diaphragm action unless evidence is available to show otherwise.

Unless a more refined analysis is made, the wind force, wind moment, horizontal force to be transmitted by diaphragms and cross-frames, and horizontal force to be transmitted by lateral bracing may be calculated as indicated below. This procedure is presented for beam bridges but may be adapted for other types of bridges.

The wind force, W, may be applied to the flanges of exterior members. For composite members and noncomposite members with cast-in-place concrete or orthotropic steel decks, W need not be applied to the top flange.

$$W = \frac{\eta_i \gamma P_D d}{2} \quad \text{(C4.6.2.7.1-1)}$$

where:

W = factored wind force per unit length applied to the flange (kip/ft)
P_D = design horizontal wind pressure specified in Article 3.8.1 (ksf)
d = depth of the member (ft)
γ = load factor specified in Table 3.4.1-1 for the particular group loading combination
η_i = load modifier relating to ductility, redundancy, and operational importance as specified in Article 1.3.2.1

For the first two load paths, the maximum wind moment on the loaded flange may be determined as:

$$M_w = \frac{W L_b^2}{10} \quad \text{(C4.6.2.7.1-2)}$$

where:

M_w = maximum lateral moment in the flange due to the factored wind loading (kip-ft)
W = factored wind force per unit length applied to the flange (kip/ft)
L_b = spacing of brace points (ft)

For the third load path, the maximum wind moment on the loaded flange may be computed as:

$$M_w = \frac{W L_b^2}{10} + \frac{W L^2}{8 N_b} \quad \text{(C4.6.2.7.1-3)}$$

where:

M_w = total lateral moment in the flange due to the factored wind loading (kip-ft)

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

- The longitudinal location of the design vehicular live load in each lane,
- The longitudinal axle spacing of the design vehicular live load,
- The transverse location of the design vehicular live load in each lane.

A structurally continuous railing, barrier, or median, acting compositely with the supporting components, may be considered to be structurally active at service and fatigue limit states.

This provision reflects the experimentally observed response of bridges. This source of stiffness has traditionally been neglected but exists and may be included, provided that full composite behavior is assured.

When a refined method of analysis is used, a table of live load distribution coefficients for extreme force effects in each span shall be provided in the contract documents to aid in permit issuance and rating of the bridge.

These live load distribution coefficients should be provided for each combination of component and lane.

4.6.3.2—Decks

4.6.3.2.1—General

Unless otherwise specified, flexural and torsional deformation of the deck shall be considered in the analysis but vertical shear deformation may be neglected.

Locations of flexural discontinuity through which shear may be transmitted should be modeled as hinges.

In the analysis of decks that may crack and/or separate along element boundaries when loaded, Poisson's ratio may be neglected. The wheel loads shall be modeled as patch loads distributed over an area, as specified in Article 3.6.1.2.5, taken at the contact surface. This area may be extended by the thickness of the wearing surface, integral or nonintegral, on all four sides. When such extension is utilized, the thickness of the wearing surface shall be reduced for any possible wear at the time of interest. Other extended patch areas may be utilized with the permission of the Owner provided that such extended area is consistent with the assumptions in, and application of, a particular refined method of analysis.

C4.6.3.2.1

In many solid decks, the wheel load-carrying contribution of torsion is comparable to that of flexure. Large torsional moments exist in the end zones of skewed girder bridges due to differential deflection. In most deck types, shear stresses are rather low, and their contribution to vertical deflection is not significant. In-plane shear deformations, which gave rise to the concept of effective width for composite bridge decks, should not be neglected.

4.6.3.2.2—Isotropic Plate Model

For the purpose of this section, bridge decks that are solid, have uniform or close to uniform depth, and whose stiffness is close to equal in every in-plane direction shall be considered isotropic.

C4.6.3.2.2

Analysis is rather insensitive to small deviations in constant depth, such as those due to superelevation, crown, and haunches. In slightly cracked concrete slabs, even a large difference in the reinforcement ratio will not cause significant changes in load distribution.

The torsional stiffness of the deck may be estimated using Eq. C4.6.2.2.1-1 with b equal to 1.0.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

4.6.3.2.3—Orthotropic Plate Model

In orthotropic plate modeling, the flexural rigidity of the elements may be uniformly distributed along the cross-section of the deck. Where the torsional stiffness of the deck is not contributed solely by a solid plate of uniform thickness, the torsional rigidity should be established by physical testing, three-dimensional analysis, or generally accepted and verified approximations.

C4.6.3.2.3

The accuracy of the orthotropic plate analysis is sharply reduced for systems consisting of a small number of elements subjected to concentrated loads.

4.6.3.2.4—Refined Orthotropic Deck Model

Refined analysis of orthotropic deck structures subjected to direct wheel loads should be accomplished using a detailed three-dimensional shell or solid finite element structural model. The structural model should include all components and connections and consider local structural stress at fatigue prone details as shown in Table 6.6.1.2.3-1. Structural modeling techniques that utilize the following simplifying assumptions may be applied:

- Linear elastic material behavior,
- Small deflection theory,
- Plane sections remain plane,
- Neglect residual stresses, and
- Neglect imperfections and weld geometry.

Meshing shall be sufficiently detailed to calculate local stresses at weld toes and to resolve the wheel patch pressure loading with reasonable accuracy.

C4.6.3.2.4

Further guidance on evaluating local structural stresses using finite element modeling is provided in *Manual for Design, Construction, and Maintenance of Orthotropic Steel Bridges* (FHWA, 2012).

4.6.3.3—Beam-Slab Bridges

4.6.3.3.1—General

The aspect ratio of finite elements and grid panels should not exceed 5.0. Abrupt changes in size and/or shape of finite elements and grid panels should be avoided.

Nodal loads shall be statically equivalent to the actual loads being applied.

C4.6.3.3.1

More restrictive limits for aspect ratio may be specified for the software used.

In the absence of other information, the following guidelines may be used at the discretion of the Engineer:

- A minimum of five, and preferably nine, nodes per beam span should be used.
- For finite element analyses involving plate and beam elements, it is preferable to maintain the relative vertical distances between various elements. If this is not possible, longitudinal and transverse elements may be positioned at the midthickness of the plate-bending elements, provided that the eccentricities are included in the equivalent properties of those sections that are composite.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

FHWA. 2012. *Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Frederick, G. R., C. V. Ardis, K. M. Tarhini, and B. Koo. 1988. "Investigation of the Structural Adequacy of C 850 Box Culverts," *Transportation Research Record* 1191, Transportation Research Board, National Research Council, Washington, DC.

Galambos, T. V., ed. 1998. *Guide to Stability Design for Metal Structures*, 5th Edition. Structural Stability Research Council. John Wiley and Sons, Inc., New York, NY.

Gasparini, D., and E. H Vanmarcke. 1976. *SIMQKE: A Program for Artificial Motion Generation*. Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, MA.

Ghali, A., and A. M. Neville. 1989. *Structural Analysis: A Unified Classical and Matrix Approach*, 3rd Edition. Chapman Hall, New York, NY.

Goel, S. C., and A. A. El-Tayem. 1986. "Cyclic Load Behavior of Angle X-Bracing." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 112, No. 11, November 1986, pp. 2528–2539.

Guyan, R. J. 1965. "Reduction of Stiffness and Mass Matrices." *AIAA Journal*, American Institute of Aeronautics and Astronautics, Reston, VA, Vol. 3, No. 2, February 1965, p. 380.

Hall, D. H., and C. H. Yoo. 1996. *I-Girder Curvature Study*. Interim Report, NCHRP Project 12-38 submitted to NCHRP, Transportation Research Board, Washington, DC, pp. 1–72 (or see Appendix A of NCHRP Report 424: Improved Design Specifications for Horizontally Curved Steel Highway Bridges, pp. 49–74).

Haroun, N. M., and R. Sheperd. 1986. "Inelastic Behavior of X-Bracing in Plane Frames." *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 112, No. 4, April 1986, pp. 764–780.

Higgins, C. 2003. "LRFD Orthotropic Plate Model for Determining Live Load Moments in Concrete Filled Grid Bridge Decks." *Journal of Bridge Engineering*, American Society of Civil Engineers, Reston, VA, January/February 2003, pp. 20–28.

Highway Engineering Division. 1991. *Ontario Highway Bridge Design Code*. Highway Engineering Division, Ministry of Transportation and Communications, Toronto, Canada.

Hindi, W. A. 1991. "Behavior and Design of Stiffened Compression Flanges of Steel Box Girder Bridges." Ph.D. Thesis. University of Surrey, Guilford, UK.

Homberg, H. 1968. *Fahrbahnplatten mit Verandlicher Dicke*. Springer-Verlag, New York, NY.

James, R. W. 1984. "Behavior of ASTM C 850 Concrete Box Culverts Without Shear Connectors," *Transportation Research Record* 1001, Transportation Research Board, National Research Council, Washington, DC.

Jetteur, P. et al. 1984. "Interaction of Shear Lag with Plate Buckling in Longitudinally Stiffened Compression Flanges," *Acta Technica CSAV*. Akademie Ved Ceske Republiky, Czech Republic, No. 3, p. 376.

Johnston, S. B., and A. H. Mattock. 1967. *Lateral Distribution of Load in Composite Box Girder Bridges*. Highway Research Record No. 167, Highway Research Board, Washington, DC.

Karabalis, D. L. 1983. "Static, Dynamic and Stability Analysis of Structures Composed of Tapered Beams." *Computers and Structures*, Vol. 16, No. 6, pp. 731–748.

Ketchum, M. S. 1986. "Short Cuts for Calculating Deflections." *Structural Engineering Practice: Analysis, Design, Management*, Vol. 3, No. 2, pp. 83–91.

King, Csagoly P. F., and J. W. Fisher. 1975. *Field Testing of the Aquasabon River Bridge*. Ontario, Canada.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Lamas, A. R. G. and P. J. Dowling. 1980. "Effect of Shear Lag on the Inelastic Buckling Behavior of Thin-Walled Structures." In *Thin-Walled Structures*, J. Rhodes & A.C. Walker, eds., Granada, London, p. 100.

Liu, H. 1991. *Wind Engineering: A Handbook for Structural Engineers*. Prentice Hall, Englewood Cliffs, NJ.

Manual for Design, Construction and Maintenance of Orthotropic Steel Bridges. In development.

McGrath, T. J., A. A. Liepins, J. L. Beaver, and B. P. Strohman. 2004. *Live Load Distribution Widths for Reinforced Concrete Box Culverts*. A Study for the Pennsylvania Department of Transportation, Simpson Gumpertz & Heger Inc., Waltham, MA.

Modjeski and Masters, Inc. 1994. *Report to Pennsylvania Department of Transportation.* Harrisburg, PA.

Moffatt, K. R., and P. J. Dowling. 1975. "Shear Lag in Steel Box Girder Bridges." *The Structural Engineer*, October 1975, pp. 439–447.

Moffatt, K. R., and P. J. Dowling. 1976. "Discussion." *The Structural Engineer*, August 1976, pp. 285–297.

Nassif, H., A.-A. Talat, and S. El-Tawil. 2006. "Effective Flange Width Criteria for Composite Steel Girder Bridges," Annual Meeting CD-ROM, Transportation Research Board, National Research Council, Washington, DC.

Nettleton, D. A. 1977. *Arch Bridges*. Bridge Division, Office of Engineering, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Nutt, Redfield and Valentine in association with David Evans and Associates and Zocon Consulting Engineers. 2008. Development of Design Specifications and Commentary for Horizontally Curved Concrete Box-Girder Bridges, NCHRP Report 620. Transportation Research Board, National Research Council, Washington, DC.

Paikowsky, S. G., with contributions from B. Birgisson, M. McVay, T. Nguyen, C. Kuo, G. Baecher, B. Ayyab, K. Stenersen, K. O'Malley, L. Chernauskas, and M. O'Neill. 2004. *Load and Resistance Factor Design (LRFD) for Deep Foundations*, NCHRP (Final) Report 507. Transportation Research Board, National Research Council, Washington, DC.

Paz, M. 1984. "Dynamic Condensation." *AIAA Journal*, American Institute of Aeronautics and Astronautics, Reston, VA, Vol. 22, No. 5, May 1984, pp. 724–727.

Paz, M. 1985. *Structural Dynamics*, 2nd Edition. Van Nosstrand Reinhold Company, New York, NY.

Peck, R. B., W. E. Hanson, and T. H. Thornburn. 1974. *Foundation Engineering*, 2nd Edition. John Wiley and Sons, Inc., New York, NY.

Podolny, W., and J. B. Scalzi. 1986. *Construction and Design of Cable-Stayed Bridges*, 2nd Edition. Wiley-Interscience, New York, NY.

Przemieniecki, J. S. 1968. *Theory of Matrix Structural Analysis*. McGraw Hill, New York, NY.

Pucher, A. 1964. *Influence Surfaces of Elastic Plates*, 4th Edition. Springer-Verlag, New York, NY.

Puckett, J. A. and M. Jablin. 2009. "AASHTO T-05 Report." Correspondence, March 2009.

Richardson, Gordon and Associates (presently HDR, Pittsburgh office). 1976. "Curved Girder Workshop Lecture Notes." Prepared under Contract No. DOT-FH-11-8815. Federal Highway Administration, U.S. Department of Transportation. Four-day workshop presented in Albany, Denver, and Portland, September–October 1976, pp. 31–38.

Salmon, C. G., and J. E. Johnson. 1990. *Steel Structures: Design and Behavior, Emphasizing Load, and Resistance Factor Design*, 3rd Edition. Harper and Row, New York, NY.

Scanlan, R. H. 1975. *Recent Methods in the Application of Test Results to the Wind Design of Long Suspended-Span Bridges*, FHWA-RD-75-115. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

The definition of the high-stress region for application of Eqs. 5.5.3.2-1 and 5.5.3.2-2 for flexural reinforcement shall be taken as one-third of the span on each side of the section of maximum moment.

Since the fatigue provisions were developed based primarily on ASTM A615 steel reinforcement, their applicability to other types of reinforcement is largely unknown. Consequently, a cautionary note is added to the Commentary.

5.5.3.3—Prestressing Tendons

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for prestressing tendons shall be taken as:

- 18.0 ksi for radii of curvature in excess of 30.0 ft, and
- 10.0 ksi for radii of curvature not exceeding 12.0 ft.

A linear interpolation may be used for radii between 12.0 and 30.0 ft.

C5.5.3.3

Where the radius of curvature is less than shown, or metal-to-metal fretting caused by prestressing tendons rubbing on hold-downs or deviations is apt to be a consideration, it will be necessary to consult the literature for more complete presentations that will allow the increased bending stress in the case of sharp curvature, or fretting, to be accounted for in the development of permissible fatigue stress ranges. Metal-to-metal fretting is not normally expected to be a concern in conventional pretensioned beams.

5.5.3.4—Welded or Mechanical Splices of Reinforcement

For welded or mechanical connections that are subject to repetitive loads, the constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, shall be as given in Table 5.5.3.4-1.

Table 5.5.3.4-1—Constant-Amplitude Fatigue Threshold of Splices

Type of Splice	$(\Delta F)_{TH}$ for greater than 1,000,000 cycles
Grout-filled sleeve, with or without epoxy coated bar	18 ksi
Cold-swaged coupling sleeves without threaded ends and with or without epoxy-coated bar; Integrally-forged coupler with upset NC threads; Steel sleeve with a wedge; One-piece taper-threaded coupler; and Single V-groove direct butt weld	12 ksi
All other types of splices	4 ksi

Where the total cycles of loading, N, as specified in Eq. 6.6.1.2.5-2, are less than one million, $(\Delta F)_{TH}$ in Table 5.5.3.4-1 may be increased by the quantity $24\,(6-\log N)$ ksi to a total not greater than the value given by Eq. 5.5.3.2-1 in Article 5.5.3.2. Higher values

C5.5.3.4

Review of the available fatigue and static test data indicates that any splice, that develops 125 percent of the yield strength of the bar will sustain one million cycles of a 4 ksi constant amplitude stress range. This lower limit is a close lower bound for the splice fatigue data obtained in NCHRP Project 10-35, and it also agrees well with the limit of 4.5 ksi for Category E from the provisions for fatigue of structural steel weldments. The strength requirements of Articles 5.11.5.2.2 and 5.11.5.2.3 also will generally ensure that a welded splice or mechanical connector will also meet certain minimum requirements for fabrication and installation, such as sound welding and proper dimensional tolerances. Splices that do not meet these requirements for fabrication and installation may have reduced fatigue performance. Further, splices designed to the lesser force requirements of Article 5.11.5.3.2 may not have the same fatigue performance as splices designed for the greater force requirement. Consequently, the minimum strength requirement indirectly provides for a minimum fatigue performance.

It was found in NCHRP Project 10-35 that there is substantial variation in the fatigue performance of different types of welds and connectors. However, all types of splices appeared to exhibit a constant amplitude fatigue limit for repetitive loading exceeding about one million cycles. The stress ranges for over one million cycles of loading given in Table 5.5.3.4-1 are based on statistical tolerance limits to constant amplitude staircase test data, such that there is a 95 percent level of

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

of $(\Delta F)_{TH}$, up to the value given by Eq. 5.5.3.2-1, may be used if justified by fatigue test data on splices that are the same as those that will be placed in service.

Welded or mechanical splices shall not be used with ASTM A1035/A1035M reinforcement.

confidence that 95 percent of the data would exceed the given values for five million cycles of loading. These values may, therefore, be regarded as a fatigue limit below which fatigue damage is unlikely to occur during the design lifetime of the structure. This is the same basis used to establish the fatigue design provisions for unspliced reinforcing bars in Article 5.5.3.2, which is based on fatigue tests reported in NCHRP Report 164, *Fatigue Strength of High-Yield Reinforcing Bars*.

5.5.4—Strength Limit State

5.5.4.1—General

The strength limit state issues to be considered shall be those of strength and stability.

Factored resistance shall be the product of nominal resistance as determined in accordance with the applicable provisions of Articles 5.6, 5.7, 5.8, 5.9, 5.10, 5.13, and 5.14, unless another limit state is specifically identified, and the resistance factor is as specified in Article 5.5.4.2.

C5.5.4.1

Additional resistance factors are specified in Article 12.5.5 for buried pipes and box structures made of concrete.

5.5.4.2—Resistance Factors

5.5.4.2.1—Conventional Construction

Resistance factor ϕ shall be taken as:

- For tension-controlled reinforced concrete sections as defined in Article 5.7.2.1 .... 0.90
- For tension-controlled prestressed concrete sections as defined in Article 5.7.2.1 .... 1.00
- For shear and torsion:
  - normal weight concrete............................... 0.90
  - lightweight concrete.................................... 0.80
- For compression-controlled sections with spirals or ties, as defined in Article 5.7.2.1, except as specified in Articles 5.10.11.3 and 5.10.11.4.1b for Seismic Zones 2, 3, and 4 at the extreme event limit state 0.75
- For bearing on concrete...................................... 0.70
- For compression in strut-and-tie models 0.70

C5.5.4.2.1

In applying the resistance factors for tension-controlled and compression-controlled sections, the axial tensions and compressions to be considered are those caused by external forces. Effects of primary prestressing forces are not included.

In editions of and interims to the LRFD Specifications prior to 2005, the provisions specified the magnitude of the resistance factor for cases of axial load or flexure, or both, it terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross-section, at nominal strength. The background and basis for these provisions are given in Mast (1992) and ACI 318-02.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

For sections subjected to axial load with flexure, factored resistances are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in Article 5.7.2.1 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

5.7.3—Flexural Members

5.7.3.1—Stress in Prestressing Steel at Nominal Flexural Resistance

5.7.3.1.1—Components with Bonded Tendons

For rectangular or flanged sections subjected to flexure about one axis where the approximate stress distribution specified in Article 5.7.2.2 is used and for which f_{pe} is not less than 0.5 f_{pu}, the average stress in prestressing steel, f_{ps}, may be taken as:

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad (5.7.3.1.1\text{-}1)$$

in which:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad (5.7.3.1.1\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c \beta_1 b_w + k A_{ps}\dfrac{f_{pu}}{d_p}} \quad (5.7.3.1.1\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{0.85 f'_c \beta_1 b + k A_{ps}\dfrac{f_{pu}}{d_p}} \quad (5.7.3.1.1\text{-}4)$$

where:

A_{ps} = area of prestressing steel (in.^{2})
f_{pu} = specified tensile strength of prestressing steel (ksi)
f_{py} = yield strength of prestressing steel (ksi)
A_s = area of mild steel tension reinforcement (in.^{2})
A'_s = area of compression reinforcement (in.^{2})
f_s = stress in the mild steel tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1
f'_s = stress in the mild steel compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.7.2.1
b = width of the compression face of the member; for a flange section in compression, the effective width of the flange as specified in Article 4.6.2.6 (in.)
b_w = width of web (in.)
h_f = depth of compression flange (in.)
d_p = distance from extreme compression fiber to the centroid of the prestressing tendons (in.)

C5.7.3.1.1

Equations in this Article and subsequent equations for flexural resistance are based on the assumption that the distribution of steel is such that it is reasonable to consider all of the tensile reinforcement to be lumped at the location defined by d_s and all of the prestressing steel can be considered to be lumped at the location defined by d_p. Therefore, in the case where a significant number of prestressing elements are on the compression side of the neutral axis, it is more appropriate to use a method based on the conditions of equilibrium and strain compatibility as indicated in Article 5.7.2.1.

The background and basis for Eqs. 5.7.3.1.1-1 and 5.7.3.1.2-1 can be found in Naaman (1985), Loov (1988), Naaman (1989), and Naaman (1990–1992).

Values of f_{py}/f_{pu} are defined in Table C5.7.3.1.1-1. Therefore, the values of k from Eq. 5.7.3.1.1-2 depend only on the type of tendon used.

Table C5.7.3.1.1-1—Values of *k*

Type of Tendon	f_{py}/f_{pu}	Value of k
Low relaxation strand	0.90	0.28
Stress-relieved strand and Type 1 high-strength bar	0.85	0.38
Type 2 high-strength bar	0.80	0.48

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

c = distance between the neutral axis and the compressive face (in.)
β_1 = stress block factor specified in Article 5.7.2.2

5.7.3.1.2—Components with Unbonded Tendons

For rectangular or flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.7.4.5, where the approximate stress distribution specified in Article 5.7.2.2 is used, the average stress in unbonded prestressing steel may be taken as:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py} \quad (5.7.3.1.2\text{-}1)$$

in which:

$$\ell_e = \left(\frac{2\,\ell_i}{2 + N_s}\right) \quad (5.7.3.1.2\text{-}2)$$

for T-section behavior:

$$c = \frac{A_{ps}\,f_{ps} + A_s\,f_s - A_s'\,f_s' - 0.85\,f_c'\,(b - b_w)\,h_f}{0.85\,f_c'\beta_1\,b_w} \quad (5.7.3.1.2\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps}f_{ps} + A_s f_s - A_s'\,f_s'}{0.85 f_c'\beta_1\,b} \quad (5.7.3.1.2\text{-}4)$$

where:

c = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded, given by Eqs. 5.7.3.1.2-3 and 5.7.3.1.2-4 for T-section behavior and rectangular section behavior, respectively (in.)
ℓ_e = effective tendon length (in.)
ℓ_i = length of tendon between anchorages (in.)
N_s = number of support hinges crossed by the tendon between anchorages or discretely bonded points
f_{py} = yield strength of prestressing steel (ksi)
f_{pe} = effective stress in prestressing steel at section under consideration after all losses (ksi)

C5.7.3.1.2

A first estimate of the average stress in unbonded prestressing steel may be made as:

$$f_{ps} = f_{pe} + 15.0 \text{ (ksi)} \quad (C5.7.3.1.2\text{-}1)$$

In order to solve for the value of f_{ps} in Eq. 5.7.3.1.2-1, the equation of force equilibrium at ultimate is needed. Thus, two equations with two unknowns (f_{ps} and c) need to be solved simultaneously to achieve a closed-form solution.

5.7.3.1.3—Components with Both Bonded and Unbonded Tendons

5.7.3.1.3a—Detailed Analysis

Except as specified in Article 5.7.3.1.3b, for components with both bonded and unbonded tendons, the stress in the prestressing steel shall be computed by detailed analysis. This analysis shall take into account the strain compatibility between the section and the bonded prestressing steel. The stress in the unbonded prestressing

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

5.7.3.4—Control of Cracking by Distribution of Reinforcement

The provisions specified herein shall apply to the reinforcement of all concrete components, except that of deck slabs designed in accordance with Article 9.7.2, in which tension in the cross-section exceeds 80 percent of the modulus of rupture, specified in Article 5.4.2.6, at applicable service limit state load combination specified in Table 3.4.1-1.

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad (5.7.3.4\text{-}1)$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

where:

γ_e = exposure factor
= 1.00 for Class 1 exposure condition
= 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)

h = overall thickness or depth of the component (in.)

d_ℓ = distance from the extreme compression fiber to the centroid of extreme tension steel element (in.)

C5.7.3.4

All reinforced concrete members are subject to cracking under any load condition, including thermal effects and restraint of deformations, which produces tension in the gross section in excess of the cracking strength of the concrete. Locations particularly vulnerable to cracking include those where there is an abrupt change in section and intermediate post-tensioning anchorage zones.

Provisions specified, herein, are used for the distribution of tension reinforcement to control flexural cracking.

Crack width is inherently subject to wide scatter, even in careful laboratory work, and is influenced by shrinkage and other time-dependent effects. Steps should be taken in detailing of the reinforcement to control cracking. From the standpoint of appearance, many fine cracks are preferable to a few wide cracks. Improved crack control is obtained when the steel reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are more effective in controlling cracking than one or two larger bars of equivalent area.

Extensive laboratory work involving deformed reinforcing bars has confirmed that the crack width at the service limit state is proportional to steel stress. However, the significant variables reflecting steel detailing were found to be the thickness of concrete cover and spacing of the reinforcement.

Eq. 5.7.3.4-1 is expected to provide a distribution of reinforcement that will control flexural cracking. The equation is based on a physical crack model (Frosch, 2001) rather than the statistically-based model used in previous editions of the specifications. It is written in a form emphasizing reinforcement details, i.e., limiting bar spacing, rather than crack width. Furthermore, the physical crack model has been shown to provide a more realistic estimate of crack widths for larger concrete covers compared to the previous equation (Destefano, 2003).

Eq. 5.7.3.4-1 with Class 1 exposure condition is based on an assumed crack width of 0.017 in. Previous research indicates that there appears to be little or no correlation between crack width and corrosion, however, the different classes of exposure conditions have been so defined in order to provide flexibility in the application of these provisions to meet the needs of the Authority having jurisdiction. Class 1 exposure condition could be thought of as an upper bound in regards to crack width for appearance and corrosion. Areas that the Authority having jurisdiction may consider for Class 2 exposure condition would include decks and substructures exposed to water. The crack width is directly proportional to the γ_e exposure factor, therefore, if the individual Authority with jurisdiction desires an alternate crack width, the γ_e factor can be adjusted directly. For example a γ_e factor of 0.5 will result in an approximate crack width of 0.0085 in.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and when there is increased concern of appearance and/or corrosion.

In the computation of d_c, the actual concrete cover thickness is to be used.

When computing the actual stress in the steel reinforcement, axial tension effects shall be considered, while axial compression effects may be considered.

The minimum and maximum spacing of reinforcement shall also comply with the provisions of Articles 5.10.3.1 and 5.10.3.2, respectively.

The effects of bonded prestressing steel may be considered, in which case the value of f_s used in Eq. 5.7.3.4-1, for the bonded prestressing steel, shall be the stress that develops beyond the decompression state calculated on the basis of a cracked section or strain compatibility analysis.

Where flanges of reinforced concrete T-girders and box girders are in tension at the service limit state, the flexural tension reinforcement shall be distributed over the lesser of:

- The effective flange width, specified in Article 4.6.2.6, or
- A width equal to 1/10 of the average of adjacent spans between bearings.

If the effective flange width exceeds 1/10 the span, additional longitudinal reinforcement, with area not less than 0.4 percent of the excess slab area, shall be provided in the outer portions of the flange.

If d_ℓ of nonprestressed or partially prestressed concrete members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance $d_\ell/2$ nearest the flexural tension reinforcement. The area of skin reinforcement A_{sk} in in.^{2}/ft of height on each side face shall satisfy:

$$A_{sk} \geq 0.012\ (d_\ell - 30) \leq \frac{A_s + A_{ps}}{4} \qquad (5.7.3.4\text{-}2)$$

where:

A_{ps} = area of prestressing steel (in.^{2})
A_s = area of tensile reinforcement (in.^{2})

However, the total area of longitudinal skin reinforcement (per face) need not exceed one-fourth of the required flexural tensile reinforcement $A_s + A_{ps}$.

The maximum spacing of the skin reinforcement shall not exceed either $d_\ell/6$ or 12.0 in.

Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires.

Where members are exposed to aggressive exposure or corrosive environments, additional protection beyond that provided by satisfying Eq. 5.7.3.4-1 may be provided by decreasing the permeability of the concrete and/or waterproofing the exposed surface.

Cracks in segmental concrete box girders may result from stresses due to handling and storing segments for precast construction and to stripping forms and supports from cast-in-place construction before attainment of the nominal f'_c.

The β_s factor, which is a geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level, has been incorporated into the basic crack control equation in order to provide uniformity of application for flexural member depths ranging from thin slabs in box culverts to deep pier caps and thick footings. The theoretical definition of β_s may be used in lieu of the approximate expression provided.

Distribution of the negative reinforcement for control of cracking in T-girders should be made in the context of the following considerations:

- Wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web.
- Close spacing near the web leaves the outer regions of the flange unprotected.

The 1/10 of the span limitation is to guard against an excessive spacing of bars, with additional reinforcement required to protect the outer portions of the flange.

The requirements for skin reinforcement are based upon ACI 318-95. For relatively deep flexural members, some reinforcement should be placed near the vertical faces in the tension zone to control cracking in the web. Without such auxiliary steel, the width of the cracks in the web may greatly exceed the crack widths at the level of the flexural tension reinforcement.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Reinforcement for interface shear may consist of single bars, multiple leg stirrups, or welded wire fabric.

All reinforcement present where interface shear transfer is to be considered shall be fully developed on both sides of the interface by embedment, hooks, mechanical methods such as headed studs or welding to develop the design yield stress.

Any reinforcement crossing the interface is subject to the same strain as the designed interface reinforcement. Insufficient anchorage of any reinforcement crossing the interface could result in localized fracture of the surrounding concrete.

When the required interface shear reinforcement in girder/slab design exceeds the area required to satisfy vertical (transverse) shear requirements, additional reinforcement must be provided to satisfy the interface shear requirements. The additional interface shear reinforcement need only extend into the girder a sufficient depth to develop the design yield stress of the reinforcement rather than extending the full depth of the girder as is required for vertical shear reinforcement.

The minimum area of interface shear reinforcement specified in Article 5.8.4.4 shall be satisfied.

The factored interface shear resistance, V_{ri}, shall be taken as:

$$V_{ri} = \phi V_{ni} \quad (5.8.4.1\text{-}1)$$

and the design shall satisfy:

$$V_{ri} \geq V_{ui} \quad (5.8.4.1\text{-}2)$$

where:

V_{ni} = nominal interface shear resistance (kip)

V_{ui} = factored interface shear force due to total load based on the applicable strength and extreme event load combinations in Table 3.4.1-1 (kip)

ϕ = resistance factor for shear specified in Article 5.5.4.2.1. In cases where different weight concretes exist on the two sides of an interface, the lower of the two values of ϕ shall be used.

Total load shall include all noncomposite and composite loads.

For the extreme limit state event ϕ may be taken as 1.0.

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \quad (5.8.4.1\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not be greater than the lesser of:

$$V_{ni} \leq K_1 f'_c A_{cv}, \text{ or} \quad (5.8.4.1\text{-}4)$$

$$V_{ni} \leq K_2 A_{cv} \quad (5.8.4.1\text{-}5)$$

in which:

$$A_{cv} = b_{vi} L_{vi} \quad (5.8.4.1\text{-}6)$$

A pure shear friction model assumes interface shear resistance is directly proportional to the net normal clamping force ($A_{vf} f_y + P_c$), through a friction coefficient (μ). Eq. 5.8.4.1-3 is a modified shear-friction model accounting for a contribution, evident in the experimental data, from cohesion and/or aggregate interlock depending on the nature of the interface under consideration given by the first term. For simplicity, the term “cohesion factor” is used throughout the body of this Article to capture the effects of cohesion and/or aggregate interlock such that Eq. 5.8.4.1-3 is analogous to the vertical shear resistance expression of $V_c + V_s$.

Eq. 5.8.4.1-4 limits V_{ni} to prevent crushing or shearing of aggregate along the shear plane.

Eqs. 5.8.4.1-3 and 5.8.4.1-4 are sufficient, with an appropriate value for K_1, to establish a lower bound for the available experimental data; however, Eq. 5.8.4.1-5 is necessitated by the sparseness of available experimental data beyond the limiting K_2 values provided in Article 5.8.4.3.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

The interface shear strength Eqs. 5.8.4.1-3, 5.8.4.1-4, and 5.8.4.1-5 are based on experimental data for normal weight, nonmonolithic concrete strengths ranging from 2.5 ksi to 16.5 ksi; normal weight, monolithic concrete strengths from 3.5 ksi to 18.0 ksi; sand-lightweight concrete strengths from 2.0 ksi to 6.0 ksi; and all-lightweight concrete strengths from 4.0 ksi to 5.2 ksi.

Composite section design utilizing full-depth precast deck panels is not addressed by these provisions. Design specifications for such systems should be established by, or coordinated with, the Owner.

where:

A_{cv} = area of concrete considered to be engaged in interface shear transfer (in.^{2})

A_{vf} = area of interface shear reinforcement crossing the shear plane within the area A_{cv} (in.^{2})

b_{vi} = interface width considered to be engaged in shear transfer (in.)

A_{vf} used in Eq. 5.8.4.1-3 is the interface shear reinforcement within the interface area A_{cv}. For a girder/slab interface, the area of the interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 5.8.4.1-3 with $12b_{vi}$ and P_c corresponding to the same one foot of girder length.

In consideration of the use of stay-in-place deck panels, or any other interface details, the Designer shall determine the width of interface, b_{vi}, effectively acting to resist interface shear.

L_{vi} = interface length considered to be engaged in shear transfer (in.)

c = cohesion factor specified in Article 5.8.4.3 (ksi)

μ = friction factor specified in Article 5.8.4.3 (dim.)

f_y = yield stress of reinforcement but design value not to exceed 60 (ksi)

The interface reinforcement is assumed to be stressed to its design yield stress, f_y. However, f_y used in determining the interface shear resistance is limited to 60 ksi because interface shear resistance computed using higher values have overestimated the interface shear resistance experimentally determined in a limited number of tests of pre-cracked specimens.

P_c = permanent net compressive force normal to the shear plane; if force is tensile, P_c = 0.0 (kip)

It is conservative to neglect P_c if it is compressive, however, if included, the value of P_c shall be computed as the force acting over the area, A_{cv}. If P_c is tensile, additional reinforcement is required to resist the net tensile force as specified in Article 5.8.4.2.

f'_c = specified 28-day compressive strength of the weaker concrete on either side of the interface (ksi)

K_1 = fraction of concrete strength available to resist interface shear, as specified in Article 5.8.4.3.

K_2 = limiting interface shear resistance specified in Article 5.8.4.3 (ksi)

5.8.4.2—Computation of the Factored Interface Shear Force, V_{ui}, for Girder/Slab Bridges

Based on consideration of a free body diagram and utilizing the conservative envelope value of V_{u1}, the factored interface shear stress for a concrete girder/slab bridge may be determined as:

$$V_{ui} = \frac{V_{u1}}{b_{vi} d_v} \quad (5.8.4.2\text{-}1)$$

where:

C5.8.4.2

The following illustrates a free body diagram approach to computation of interface shear in a girder/slab bridge. In reinforced concrete, or prestressed concrete, girder bridges, with a cast-in-place slab, horizontal shear forces develop along the interface between the girders and the slab. The classical strength of materials approach, which is based on elastic behavior of the section, has been used successfully in the past to determine the design interface shear force. As

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

d_v = the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress

The factored interface shear force in kips/ft for a concrete girder/slab bridge may be determined as:

$$V_{ui} = v_{ui} A_{cv} = v_{ui} 12 b_{vi} \qquad (5.8.4.2\text{-}2)$$

If the net force, P_c, across the interface shear plane is tensile, additional reinforcement, A_{vpc}, shall be provided as:

$$A_{vpc} = \frac{P_c}{\phi f_y} \qquad (5.8.4.2\text{-}3)$$

For beams and girders, the longitudinal spacing of the rows of interface shear transfer reinforcing bars shall not exceed 24.0 in.

an alternative to the classical elastic strength of materials approach, a reasonable approximation of the factored interface shear force at the strength or extreme event limit state for either elastic or inelastic behavior and cracked or uncracked sections, can be derived with the defined notation and the free body diagram shown in Figure C5.8.4.2-1 as follows:

M_{u2} = maximum factored moment at section 2
V_1 = the factored vertical shear at section 1 concurrent with M_{u2}
M_1 = the factored moment at section 1 concurrent with M_{u2}
Δl = unit length segment of girder
C_1 = compression force above the shear plane associated with M_1
C_{u2} = compression force above the shear plane associated with M_{u2}

$$M_{u2} = M_1 + V_1 \Delta l \qquad (C5.8.4.2\text{-}1)$$

$$C_{u2} = \frac{M_{u2}}{d_v} \qquad (C5.8.4.2\text{-}2)$$

$$C_{u2} = \frac{M_1}{d_v} + \frac{V_1 \Delta l}{d_v} \qquad (C5.8.4.2\text{-}3)$$

$$C_1 = \frac{M_1}{d_v} \qquad (C5.8.4.2\text{-}4)$$

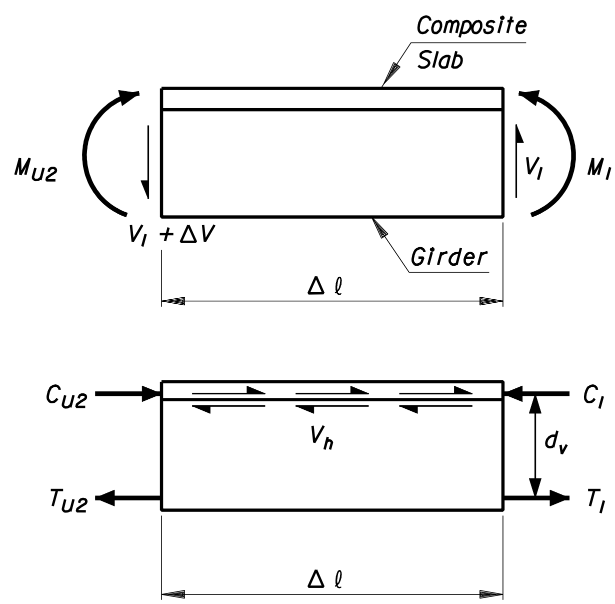


Figure C5.8.4.2-1—Free Body Diagrams

$$V_h = C_{u2} - C_1 \qquad (C5.8.4.2\text{-}5)$$

$$V_h = \frac{V_1 \Delta l}{d_v} \qquad (C5.8.4.2\text{-}6)$$

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Such that for a unit length segment:

$$V_{hi} = \frac{V_1}{d_v} \tag{C5.8.4.2-7}$$

where:

V_{hi} = factored interface shear force per unit length (kips/length)

The variation of V_1 over the length of any girder segment reflects the shear flow embodied in the classical strength of materials approach. For simplicity of design, V_1 can be conservatively taken as V_{u1} (since V_{u1}, the maximum factored vertical shear at section 1, is not likely to act concurrently with the factored moment at section 2); and further, the depth, d_v, can be taken as the distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress.

For design purposes, the computed factored interface shear stress of Eq. 5.8.4.2-1 is converted to a resultant interface shear force computed with Eq. 5.8.4.2-1 acting over an area, A_{cv}, within which the computed area of reinforcement, A_{vf}, shall be located. The resulting area of reinforcement, A_{vf}, then defines the area of interface reinforcement required per foot of girder for direct comparison with vertical shear reinforcement requirements.

5.8.4.3—Cohesion and Friction Factors

The following values shall be taken for cohesion, c, and friction factor, μ:

- For a cast-in-place concrete slab on clean concrete girder surfaces, free of laitance with surface roughened to an amplitude of 0.25 in.

  c = 0.28 ksi
  μ = 1.0
  K_1 = 0.3
  K_2 = 1.8 ksi for normal-weight concrete
  = 1.3 ksi for lightweight concrete

- For normal-weight concrete placed monolithically:

  c = 0.40 ksi
  μ = 1.4
  K_1 = 0.25
  K_2 = 1.5 ksi

C5.8.4.3

The values presented provide a lower bound of the substantial body of experimental data available in the literature (Loov and Patnaik, 1994; Patnaik, 1999; Mattock, 2001; Slapkus and Kahn, 2004). Furthermore, the inherent redundancy of girder/slab bridges distinguishes this system from other structural interfaces.

The values presented apply strictly to monolithic concrete. These values are not applicable for situations where a crack may be anticipated to occur at a Service Limit State.

The factors presented provide a lower bound of the experimental data available in the literature (Hofbeck, Ibrahim, and Mattock, 1969; Mattock, Li, and Wang, 1976; Mitchell and Kahn, 2001).

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

- For lightweight concrete placed monolithically, or nonmonolithically, against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25 in.:

  c = 0.24 ksi
  μ = 1.0
  K_1 = 0.25
  K_2 = 1.0 ksi

- For normal-weight concrete placed against a clean concrete surface, free of laitance, with surface intentionally roughened to an amplitude of 0.25 in.:

  c = 0.24 ksi
  μ = 1.0
  K_1 = 0.25
  K_2 = 1.5 ksi

- For concrete placed against a clean concrete surface, free of laitance, but not intentionally roughened:

  c = 0.075 ksi
  μ = 0.6
  K_1 = 0.2
  K_2 = 0.8 ksi

- For concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars where all steel in contact with concrete is clean and free of paint:

  c = 0.025 ksi
  μ = 0.7
  K_1 = 0.2
  K_2 = 0.8 ksi

For brackets, corbels, and ledges, the cohesion factor, c, shall be taken as 0.0.

Available experimental data demonstrates that only one modification factor is necessary, when coupled with the resistance factors of Article 5.5.4.2, to accommodate both all-lightweight and sand-lightweight concrete. Note this deviates from earlier specifications that distinguished between all-lightweight and sand-lightweight concrete.

Due to the absence of existing data, the prescribed cohesion and friction factors for nonmonolithic lightweight concrete are accepted as conservative for application to monolithic lightweight concrete.

Tighter constraints have been adopted for roughened interfaces, other than cast-in-place slabs on roughened girders, even though available test data does not indicate more severe restrictions are necessary. This is to account for variability in the geometry, loading and lack of redundancy at other interfaces.

Since the effectiveness of cohesion and aggregate interlock along a vertical crack interface is unreliable the cohesion component in Eq. 5.8.4.1-3 is set to 0.0 for brackets, corbels, and ledges.

5.8.4.4—Minimum Area of Interface Shear Reinforcement

Except as provided herein, the cross-sectional area of the interface shear reinforcement, A_{vf}, crossing the interface area, A_{cv}, shall satisfy:

$$Avf \geq \frac{0.05A_{cv}}{f_y} \qquad (5.8.4.4\text{-}1)$$

For a cast-in-place concrete slab on clean concrete girder surfaces free of laitance, the following provisions shall apply:

C5.8.4.4

For a girder/slab interface, the minimum area of interface shear reinforcement per foot of girder length is calculated by replacing A_{cv} in Eq. 5.8.4.4-1 with $12b_{vi}$.

Previous editions of these specifications and of the AASHTO Standard Specifications have required a minimum area of reinforcement based on the full

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

- The minimum interface shear reinforcement, A_{vf}, need not exceed the lesser of the amount determined using Eq. 5.8.4.4-1 and the amount needed to resist $1.33V_{ui}/\phi$ as determined using Eq. 5.8.4.1-3.

- The minimum reinforcement provisions specified herein shall be waived for girder/slab interfaces with surface roughened to an amplitude of 0.25 in. where the factored interface shear stress, v_{ui} of Eq. 5.8.4.2-1, is less than 0.210 ksi, and all vertical (transverse) shear reinforcement required by the provisions of Article 5.8.2.5 is extended across the interface and adequately anchored in the slab.

interface area; similar to Eq. 5.8.4.4-1, irrespective of the need to mobilize the strength of the full interface area to resist the applied factored interface shear. In 2006, the additional minimum area provisions, applicable only to girder/slab interfaces, were introduced. The intent of these provisions was to eliminate the need for additional interface shear reinforcement due simply to a beam with a wider top flange being utilized in place of a narrower flanged beam.

The additional provision establishes a rational upper bound for the area of interface shear reinforcement required based on the interface shear demand rather than the interface area as stipulated by Eq. 5.8.4.4-1. This treatment is analogous to minimum reinforcement provisions for flexural capacity where a minimum additional overstrength factor of 1.33 is required beyond the factored demand.

With respect to a girder/slab interface, the intent is that the portion of the reinforcement required to resist vertical shear which is extended into the slab also serves as interface shear reinforcement.

5.8.5—Principal Stresses in Webs of Segmental Concrete Bridges

The provisions specified herein shall apply to all types of segmental bridges with internal and/or external tendons.

The principal tensile stress resulting from the long-term residual axial stress and maximum shear and/or maximum shear combined with shear from torsion stress at the neutral axis of the critical web shall not exceed the tensile stress limit of Table 5.9.4.2.2-1 at the Service III limit state of Article 3.4.1 at all stages during the life of the structure, excluding those during construction. When investigating principal stresses during construction, the tensile stress limits of Table 5.14.2.3.3-1 shall apply.

The principal stress shall be determined using classical beam theory and the principles of Mohr's Circle. The width of the web for these calculations shall be measured perpendicular to the plane of the web.

Compressive stress due to vertical tendons provided in the web shall be considered in the calculation of the principal stress. The vertical force component of draped longitudinal tendons shall be considered as a reduction in the shear force due to the applied loads.

Local tensions produced in webs resulting from anchorage of tendons as discussed in Article 5.10.9.2 shall be included in the principal tension check.

Local transverse flexural stress due to out-of-plane flexure of the web itself at the critical section may be neglected in computing the principal tension in webs.

C5.8.5

This principal stress check is introduced to verify the adequacy of webs of segmental concrete bridges for longitudinal shear and torsion.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

where:

f_{pt} = stress in prestressing strands immediately after transfer, taken not less than $0.55f_{py}$ in Eq. 5.9.5.4.2c-1

K_L = 30 for low relaxation strands and 7 for other prestressing steel, unless more accurate manufacturer's data are available

The relaxation loss, Δf_{pR1}, may be assumed equal to 1.2 ksi for low-relaxation strands.

$$\Delta f_{pR1} = \left[\frac{f_{pt}}{K'_L}\frac{\log(24t)}{\log(24t_i)}\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right]K_{ia}$$

(C5.9.5.4.2c-1)

where the K'_L is a factor accounting for type of steel, equal to 45 for low relaxation steel and 10 for stress relieved steel, t is time in days between strand tensioning and deck placement. The term in the first square brackets is the intrinsic relaxation without accounting for strand shortening due to creep and shrinkage of concrete. The second term in square brackets accounts for relaxation reduction due to creep and shrinkage of concrete. The factor K_{id} accounts for the restraint of the concrete member caused by bonded reinforcement. It is the same factor used for the creep and shrinkage components of the prestress loss. The equation given in Article 5.9.5.4.2c is an approximation of the above formula with the following typical values assumed:

t_i = 0.75 day

t = 120 days

$$\left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right] = 0.67$$

K_{id} = 0.8

5.9.5.4.3—Losses: Time of Deck Placement to Final Time

5.9.5.4.3a—Shrinkage of Girder Concrete

The prestress loss due to shrinkage of girder concrete between time of deck placement and final time, Δf_{pSD}, shall be determined as:

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df} \quad (5.9.5.4.3a\text{-}1)$$

in which:

$$K_{df} = \frac{1}{1 + \frac{E_p}{E_{ci}}\frac{A_{ps}}{A_c}\left(1 + \frac{A_c e_{pc}^2}{I_c}\right)\left[1 + 0.7\psi_b\left(t_f, t_i\right)\right]}$$

(5.9.5.4.3a-2)

where:

ε_{bdf} = shrinkage strain of girder between time of deck placement and final time per Eq. 5.4.2.3.3-1

K_{df} = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between deck placement and final time

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

e_{pc} = eccentricity of prestressing force with respect to centroid of composite section (in.), positive in typical construction where prestressing force is below centroid of section

A_c = area of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio (in.^{2})

I_c = moment of inertia of section calculated using the gross composite concrete section properties of the girder and the deck and the deck-to-girder modular ratio at service (in.^{4})

5.9.5.4.3b—Creep of Girder Concrete

The prestress (loss is positive, gain is negative) due to creep of girder concrete between time of deck placement and final time, Δf_{pCD}, shall be determined as:

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp}\left[\psi_b\left(t_f,t_i\right)-\psi_b\left(t_d,t_i\right)\right]K_{df} + \frac{E_p}{E_c}\Delta f_{cd}\psi_b\left(t_f,t_d\right)K_{df} \quad (5.9.5.4.3b\text{-}1)$$

where:

Δf_{cd} = change in concrete stress at centroid of prestressing strands due to long-term losses between transfer and deck placement, combined with deck weight and superimposed loads (ksi)

$\Psi_b(t_f, t_d)$ = girder creep coefficient at final time due to loading at deck placement per Eq. 5.4.2.3.2-1

5.9.5.4.3c—Relaxation of Prestressing Strands

The prestress loss due to relaxation of prestressing strands in composite section between time of deck placement and final time, Δf_{pR2}, shall be determined as:

$$\Delta f_{pR2} = \Delta f_{pR1} \quad (5.9.5.4.3c\text{-}1)$$

C5.9.5.4.3.c

Research indicates that about one-half of the losses due to relaxation occur before deck placement; therefore, the losses after deck placement are equal to the prior losses.

5.9.5.4.3d—Shrinkage of Deck Concrete

The prestress gain due to shrinkage of deck composite section, Δf_{pSS}, shall be determined as:

$$\Delta f_{pSS} = \frac{E_p}{E_c}\Delta f_{cdf}K_{df}\left[1+0.7\psi_b\left(t_f,t_d\right)\right] \quad (5.9.5.4.3d\text{-}1)$$

in which:

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

No longitudinal bar or bundle shall be more than 24.0 in., measured along the tie, from a restrained bar or bundle. A restrained bar or bundle is one which has lateral support provided by the corner of a tie having an included angle of not more than 135 degrees. Where the column design is based on plastic hinging capability, no longitudinal bar or bundle shall be farther than 6.0 in. clear on each side along the tie from such a laterally supported bar or bundle and the tie reinforcement shall meet the requirements of Articles 5.10.11.4.1d through 5.10.11.4.1f. Where the bars or bundles are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

Columns in Seismic Zones 2, 3, and 4 are designed for plastic hinging. The plastic hinge zone is defined in Article 5.10.11.4.1c. Additional requirements for transverse reinforcement for bridges in Seismic Zones 2, 3, and 4 are specified in Articles 5.10.11.3 and 5.10.11.4.1. Plastic hinging may be used as a design strategy for other extreme events, such as ship collision.

5.10.7—Transverse Reinforcement for Flexural Members

Compression reinforcement in flexural members, except deck slabs, shall be enclosed by ties or stirrups satisfying the size and spacing requirements of Article 5.10.6 or by welded wire fabric of equivalent area.

5.10.8—Shrinkage and Temperature Reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement to ensure that the total reinforcement on exposed surfaces is not less than that specified herein.

Reinforcement for shrinkage and temperature may be in the form of bars, welded wire fabric, or prestressing tendons.

For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \quad (5.10.8\text{-}1)$$

$$0.11 \leq A_s \leq 0.60 \quad (5.10.8\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in.^{2}/ft)

b = least width of component section (in.)

h = least thickness of component section (in.)

f_y = specified yield strength of reinforcing bars ≤75 ksi

C5.10.8

The comparable equation in ACI was written for slabs with the reinforcement being distributed equally to both surfaces of the slabs.

The requirements of this Article are based on ACI 318 and 207.2R. The coefficient in Eq. 5.10.8-1 is the product of 0.0018, 60 ksi, and 12.0 in./ft and, therefore, has the units kips/in.-ft.

Eq. 5.10.8-1 is written to show that the total required reinforcement, A_s,= 0.0018bh, is distributed uniformly around the perimeter of the component. It provides a more uniform approach for components of any size. For example, a 30.0 ft high × 1.0 ft thick wall section requires 0.126 in.^{2}/ft in each face and each direction; a 4.0 ft × 4.0 ft component requires 0.260 in.^{2}/ft in each face and each direction; and a 5.0 ft × 20.0 ft footing requires 0.520 in.^{2}/ft in each face and each direction. For circular or other shapes the equation becomes:

$$A_s \geq \frac{1.3A_g}{Perimeter(f_y)} \quad (C5.10.8\text{-}1)$$

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section. Spacing shall not exceed:

- 3.0 times the component thickness, or 18.0 in.
- 12.0 in. for walls and footings greater than 18.0 in. thick
- 12.0 in. for other components greater than 36.0 in. thick

For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage and temperature steel shall not be required for:

- End face of walls 18 in. or less in thickness
- Side faces of buried footings 36 in. or less in thickness
- Faces of all other components, with smaller dimension less than or equal to 18.0 in.

If prestressing tendons are used as steel for shrinkage and temperature reinforcement, the tendons shall provide a minimum average compressive stress of 0.11 ksi on the gross concrete area through which a crack plane may extend, based on the effective prestress after losses. Spacing of tendons should not exceed either 72.0 in. or the distance specified in Article 5.10.3.4. Where the spacing is greater than 54.0 in., bonded reinforcement shall be provided between tendons, for a distance equal to the tendon spacing.

Permanent prestress of 0.11 ksi is equivalent to the resistance of the steel specified in Eq. 5.10.8-1 at the strength limit state. The 0.11 ksi prestress should not be added to that required for the strength or service limit states. It is a minimum requirement for shrinkage and temperature crack control.

The spacing of stress-relieving joints should be considered in determining the area of shrinkage and temperature reinforcement.

Surfaces of interior walls of box girders need not be considered to be exposed to daily temperature changes.

See also Article 12.14.5.8 for additional requirements for three-sided buried structures.

5.10.9—Post-Tensioned Anchorage Zones

5.10.9.1—General

Anchorages shall be designed at the strength limit states for the factored jacking forces as specified in Article 3.4.3.

For anchorage zones at the end of a component or segment, the transverse dimensions may be taken as the depth and width of the section but not larger than the longitudinal dimension of the component or segment. The longitudinal extent of the anchorage zone in the direction of the tendon shall not be less than the greater of the transverse dimensions of the anchorage zone and shall not be taken as more than one and one-half times that dimension.

For intermediate anchorages, the anchorage zone shall be considered to extend in the direction opposite to the anchorage force for a distance not less than the larger of the transverse dimensions of the anchorage zone.

C5.10.9.1

With slight modifications, the provisions of Article 5.10.9 are also applicable to the design of reinforcement under high-load capacity bearings.

The anchorage zone is geometrically defined as the volume of concrete through which the concentrated prestressing force at the anchorage device spreads transversely to a more linear stress distribution across the entire cross-section at some distance from the anchorage device.

Within the anchorage zone, the assumption that plane sections remain plane is not valid.

The dimensions of the anchorage zone are based on the principle of St. Venant. Provisions for components with a length smaller than one of its transverse dimensions were included to address cases such as transverse prestressing of bridge decks, as shown in Figure C5.10.9.1-1.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles $\phi_c = 0.50$
  - pipe piles $\phi_c = 0.60$
- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles $\phi_c = 0.60$
  - pipe piles $\phi_c = 0.70$
- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles $\phi_c = 0.70$
  - axial resistance for pipe piles $\phi_c = 0.80$
  - flexural resistance $\phi_f = 1.00$
- For shear connectors in tension $\phi_{st} = 0.75$

The basis for the resistance factors for driven steel piles is described in Article 6.15.2. Further limitations on usable resistance during driving are specified in Article 10.7.8.

Indicated values of ϕ_c and ϕ_f for combined axial and flexural resistance are for use in interaction equations in Article 6.9.2.2.

6.5.5—Extreme Event Limit State

All applicable extreme event load combinations in Table 3.4.1-1 shall be investigated. For Extreme Event I, γ_p for *DC* and *DW* loads shall be taken to be 1.0.

All resistance factors for the extreme event limit state, except those specified for bolts and shear connectors, shall be taken to be 1.0.

All resistance factors for ASTM A307 Grade C and ASTM F1554 bolts used as anchor bolts for the extreme event limit state shall be taken to be 1.0.

Bolted slip-critical connections within a seismic load path shall be proportioned according to the requirements of Article 6.13.2.1.1. The connections shall also be proportioned to provide shear, bearing, and tensile resistance in accordance with Articles 6.13.2.7, 6.13.2.9, and 6.13.2.10, as applicable, at the extreme event limit state. Standard holes or short-slotted holes normal to the line of force shall be used in such connections.

C6.5.5

During earthquake motion, there is the potential for full reversal of design load and inelastic deformations of members orconnections, or both. Therefore, slip of bolted joints located within a seismic load path cannot and need not be prevented during a seismic event. A special inspection of joints and connections, particularly in fracture critical members, should be performed as described in *The Manual for Bridge Evaluation* (2011) after a seismic event.

To prevent excessive deformations of bolted joints due to slip between the connected plies under earthquake motions, only standard holes or short-slotted holes normal to the line of force are permitted in bolted joints located within a seismic load path. For such holes, the upper limit of $2.4dtF_u$ on the bearing resistance is intended to prevent elongations due to bearing deformations from exceeding approximately 0.25 in. It should be recognized, however, that the actual bearing load in a seismic event may be much larger than that anticipated in design and the actual deformation of the holes may be larger than this theoretical value. Nonetheless, the specified upper limit on the nominal bearing resistance should effectively minimize damage in moderate seismic events.

6.6—FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1—Fatigue

6.6.1.1—General

Fatigue shall be categorized as load- or distortion-induced fatigue.

C6.6.1.1

In the AASHTO *Standard Specifications for Highway Bridges* (2002), the provisions explicitly relating to fatigue deal only with load-induced fatigue.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

6.6.1.2—Load-Induced Fatigue

6.6.1.2.1—Application

The force effect considered for the fatigue design of a steel bridge detail shall be the live load stress range. For flexural members with shear connectors provided throughout their entire length, and with concrete deck reinforcement satisfying the provisions of Article 6.10.1.7, live load stresses and stress ranges for fatigue design may be computed using the short-term composite section assuming the concrete deck to be effective for both positive and negative flexure.

Residual stresses shall not be considered in investigating fatigue.

These provisions shall be applied only to details subjected to a net applied tensile stress. In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than the maximum live load tensile stress caused by the Fatigue I load combination specified in Table 3.4.1-1.

C6.6.1.2.1

Concrete can provide significant resistance to tensile stress at service load levels. Recognizing this behavior will have a significantly beneficial effect on the computation of fatigue stress ranges in top flanges in regions of stress reversal and in regions of negative flexure. By utilizing shear connectors in these regions to ensure composite action in combination with the required one percent longitudinal reinforcement wherever the longitudinal tensile stress in the concrete deck exceeds the factored modulus of rupture of the concrete, crack length and width can be controlled so that full-depth cracks should not occur. When a crack does occur, the stress in the longitudinal reinforcement increases until the crack is arrested. Ultimately, the cracked concrete and the reinforcement reach equilibrium. Thus, the concrete deck may contain a small number of staggered cracks at any given section. Properly placed longitudinal reinforcement prevents coalescence of these cracks.

It has been shown that the level of total applied stress is insignificant for a welded steel detail. Residual stresses due to welding are implicitly included through the specification of stress range as the sole dominant stress parameter for fatigue design. This same concept of considering only stress range has been applied to rolled, bolted, and riveted details where far different residual stress fields exist. The application to nonwelded details is conservative. A complete stress range cycle may include both a tensile and compressive component. Only the live load plus dynamic load allowance effects need be considered when computing a stress range cycle; permanent loads do not contribute to the stress range. Tensile stresses propagate fatigue cracks. Material subjected to a cyclical loading at or near an initial flaw will be subject to a fully effective stress cycle in tension, even in cases of stress reversal, because the superposition of the tensile residual stress elevates the entire cycle into the tensile stress region.

Fatigue design criteria need only be considered for components or details subject to effective stress cycles in tension and/or stress reversal. If a component or detail is subject to stress reversal, fatigue is to be considered no matter how small the tension component of the stress cycle is since a flaw in the tensile residual stress zone could still be propagated by the small tensile component of stress. The decision on whether or not a tensile stress could exist is based on the Fatigue I Load Combination because this is the largest stress range a detail is expected to experience often enough to propogate a crack. When the tensile component of the stress range cycle resulting from this load combination exceeds the compressive stress due to the unfactored permanent loads, there is a net tensile stress in the component or at the detail under consideration, and therefore, fatigue must be considered. If the tensile component of the stress range does not exceed the

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

compressive stress due to the unfactored permanent loads there is no net tensile stress. In this case, the stress cycle is compression—compression and a fatigue crack will not propogate beyond a heat-affected zone.

Cross-frames and diaphragms connecting adjacent girders are stressed when one girder deflects with respect to the adjacent girder connected by the diaphragm or cross-frame. The sense of stress is reversed when the vehicle is positioned over the adjacent girder. Since it is the total stress range that produces fatigue, the effects of trucks in different transverse positions usually creates the largest stress range in these bracing members. To cause one cycle of the stress range so computed requires two vehicles to traverse the bridge in separate transverse positions with one vehicle leading the other. For cases where the force effects in these members are available from an analysis, such as in horizontally curved or sharply skewed bridges, it may be desirable in some instances to check fatigue-sensitive details on a bracing member subjected to a net applied tensile stress determined as specified herein. In lieu of more specific owner supplied guidance, it is recommended that one cycle of stress be taken as 75 percent of the stress range in the member determined by the passage of the factored fatigue load in the two different transverse positions just described. The factor of 0.75 is distinct from the load factor specified for the applicable fatigue load combination in Table 3.4.1-1; i.e., both factors may be applied simultaneously. The reduction is intended to approximate the low probability of two vehicles being located in the critical relative positions, such as outside of a striped lane, over millions of cycles. However, in no case should the calculated range of stress be less than the stress range caused by loading of only one lane. There is no provision in this recommended procedure to account for the need for two trucks to cause a single cycle of stress. For cases where the nominal fatigue resistance is calculated based on a finite life, the Engineer may wish to consider a reduction in the number of cycles whenever two trucks are required to cause a single cycle of stress.

6.6.1.2.2—Design Criteria

For load-induced fatigue considerations, each detail shall satisfy:

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad (6.6.1.2.2\text{-}1)$$

where:

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

(Δf) = force effect, live load stress range due to the passage of the fatigue load as specified in Article 3.6.1.4 (ksi)

$(\Delta F)_n$ = nominal fatigue resistance as specified in Article 6.6.1.2.5 (ksi)

C6.6.1.2.2

Eq. 6.6.1.2.2-1 may be developed by rewriting Eq. 1.3.2.1-1 in terms of fatigue load and resistance parameters:

$$\eta\gamma(\Delta f) \leq \phi(\Delta F)_n \qquad (C6.6.1.2.2\text{-}1)$$

but for the fatigue limit state,

$\eta = 1.0$
$\phi = 1.0$

6.6.1.2.3—Detail Categories

Components and details shall be designed to satisfy the requirements of their respective detail categories summarized in Table 6.6.1.2.3-1. Where bolt holes are

C6.6.1.2.3

Components and details susceptible to load-induced fatigue cracking have been grouped into eight categories, called detail categories, by fatigue resistance.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

depicted in Table 6.6.1.2.3-1, their fabrication shall conform to the provisions of Article 11.4.8.5 of the *AASHTO LRFD Bridge Construction Specifications*. Where permitted for use, unless specific information is available to the contrary, bolt holes in cross-frame, diaphragm, and lateral bracing members and their connection plates shall be assumed for design to be punched full size.

Except as specified herein for fracture critical members, where the projected 75-year single lane Average Daily Truck Traffic $(ADTT)_{SL}$ is less than or equal to that specified in Table 6.6.1.2.3-2 for the component or detail under consideration, that component or detail should be designed for finite life using the Fatigue II load combination specified in Table 3.4.1-1. Otherwise, the component or detail shall be designed for infinite life using the Fatigue I load combination. The single-lane Average Daily Truck Traffic $(ADTT)_{SL}$ shall be computed as specified in Article 3.6.1.4.2.

For components and details on fracture-critical members, the Fatigue I load combination specified in Table 3.4.1-1 should be used in combination with the nominal fatigue resistance for infinite life specified in Article 6.6.1.2.5.

Orthotropic deck components and details shall be designed to satisfy the requirements of their respective detail categories summarized in Table 6.6.1.2.3-1 for the chosen design level shown in the table and as specified in Article 9.8.3.4.

Experience indicates that in the design process the fatigue considerations for Detail Categories A through B′ rarely, if ever, govern. Nevertheless, Detail Categories A through B′ have been included in Table 6.6.1.2.3-1 for completeness. Investigation of components and details with a fatigue resistance based on Detail Categories A through B′ may be appropriate in unusual design cases.

Table 6.6.1.2.3-1 illustrates many common details found in bridge construction and identifies potential crack initiation points for each detail. In Table 6.6.1.2.3-1, “Longitudinal” signifies that the direction of applied stress is parallel to the longitudinal axis of the detail. “Transverse” signifies that the direction of applied stress is perpendicular to the longitudinal axis of the detail.

Category F for allowable shear stress range on the throat of a fillet weld has been eliminated from Table 6.6.1.2.3-1. When fillet welds are properly sized for strength considerations, Category F should not govern. Fatigue will be governed by cracking in the base metal at the weld toe and not by shear on the throat of the weld. Research on end-bolted cover plates is discussed in Wattar et al. (1985).

Where the design stress range calculated using the Fatigue I load combination is less than $(\Delta F)_{TH}$, the detail will theoretically provide infinite life. Except for Categories E and E′, for higher traffic volumes, the design will most often be governed by the infinite life check. Table 6.6.1.2.3-2 shows for each detail category the values of $(ADTT)_{SL}$ above which the infinite life check governs, assuming a 75-yr design life and one stress range cycle per truck.

The values in the second column of Table 6.6.1.2.3-2 were computed as follows:

$$75\_Year(ADTT)_{SL} = \frac{A}{\left[\frac{(\Delta F)_{TH}}{2}\right]^3 (365)(75)(n)} \qquad \text{(C6.6.1.2.3-1)}$$

using the values for A and $(\Delta F)_{TH}$ specified in Tables 6.6.1.2.5-1 and 6.6.1.2.5-3, respectively, a fatigue design life of 75 yr and a number of stress range cycles per truck passage, n, equal to one. These values were rounded up to the nearest five trucks per day. That is, the indicated values were determined by equating infinite life and finite life resistances with due regard to the difference in load factors used with the Fatigue I and Fatigue II load combinations. For other values of n, the values in Table 6.6.1.2.3-2 should be modified by dividing by the appropriate value of n taken from Table 6.6.1.2.5-2. For other values of the fatigue design life, the values in Table 6.6.1.2.3-2 should be modified by multiplying the values by the ratio of 75 divided by the fatigue life sought in years.

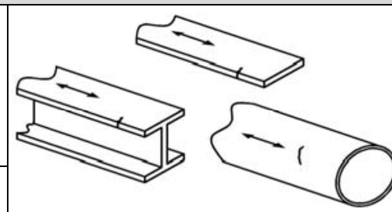
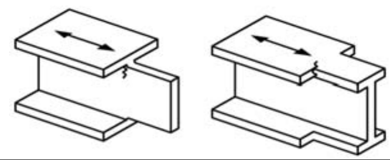
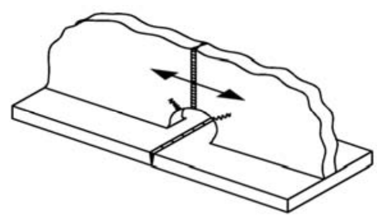
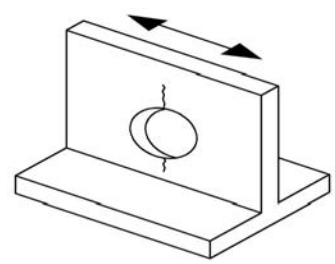
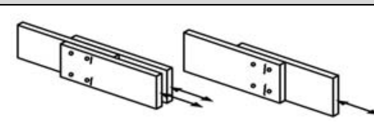
The procedures for load-induced fatigue are followed for orthotropic deck design. Although the local structural stress range for certain fatigue details can be caused by distortion of the deck plate, ribs, and floorbeams, research has demonstrated that load-induced fatigue analysis produces a reliable assessment of fatigue performance.

Considering the increased γ_{LL} and cycles per truck passage (n) in orthotropic decks, the 75-yr $ADTT_{SL}$ equivalent to infinite life (trucks per day) results in 870 for

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

deck plate details and 4350 for all other details, based on Category C. Thus, finite life design may produce more economical designs on lower-volume roadways.

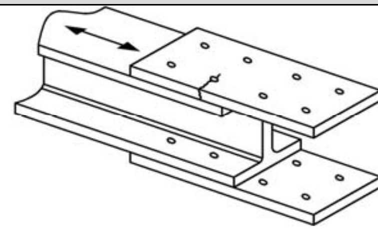
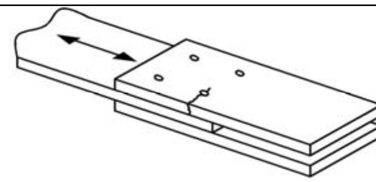
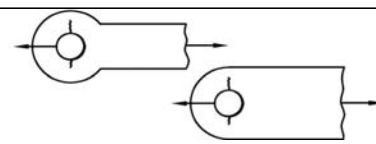
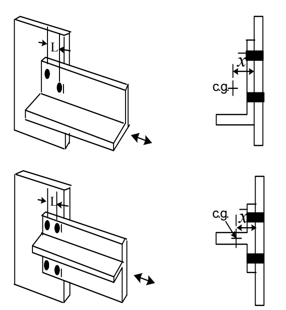
Table 6.6.1.2.3-1—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 1—Plain Material away from Any Welding					
1.1 Base metal, except noncoated weathering steel, with rolled or cleaned surfaces. Flame-cut edges with surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	A	250×10^8	24	Away from all welds or structural connections	
1.2 Noncoated weathering steel base metal with rolled or cleaned surfaces designed and detailed in accordance with FHWA (1989). Flame-cut edges with surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	B	120×10^8	16	Away from all welds or structural connections	
1.3 Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to the requirements of AASHTO/AWS D1.5, except weld access holes.	C	44×10^8	10	At any external edge	
1.4 Rolled cross sections with weld access holes made to the requirements of AASHTO/AWS D1.5, Article 3.2.4.	C	$44 \text{ x } 10^8$	10	In the base metal at the re-entrant corner of the weld access hole	
1.5 Open holes in members (Brown et al., 2007).	D	22×10^8	7	In the net section originating at the side of the hole	
Section 2—Connected Material in Mechanically Fastened Joints					
2.1 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size—e.g., bolted flange and web splices and bolted stiffeners. (Note: see Condition 2.3 for bolt holes punched full size; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)	B	120×10^8	16	Through the gross section near the hole	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

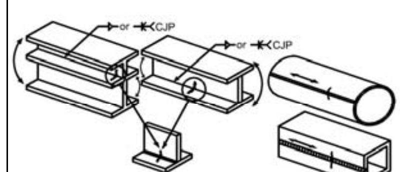
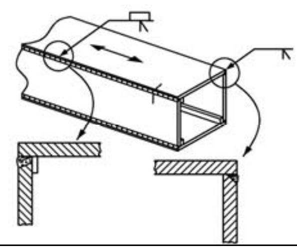
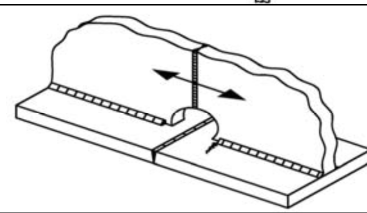
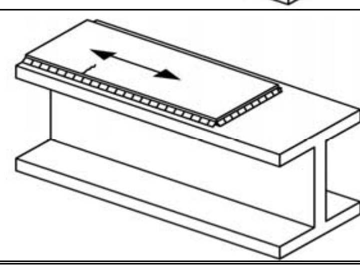
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 2—Connected Material in Mechanically Fastened Joints (continued)					
2.2 Base metal at the net section of high-strength bolted joints designed as bearing-type connections but fabricated and installed to all requirements for slip-critical connections with pretensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size. (Note: see Condition 2.3 for bolt holes punched full size; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)	B	120×10^8	16	In the net section originating at the side of the hole	
2.3 Base metal at the net section of all bolted connections in hot dipped galvanized members (Huhn and Valtinat, 2004); base metal at the appropriate section defined in Condition 2.1 or 2.2, as applicable, of high-strength bolted joints with pretensioned bolts installed in holes punched full size (Brown et al., 2007); and base metal at the net section of other mechanically fastened joints, except for eyebars and pin plates, e.g., joints using ASTM A307 bolts or non-pretensioned high-strength bolts. (Note: see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates).	D	22×10^8	7	In the net section originating at the side of the hole or through the gross section near the hole, as applicable	
2.4 Base metal at the net section of eyebar heads or pin plates (Note: for base metal in the shank of eyebars or through the gross section of pin plates, see Condition 1.1 or 1.2, as applicable.)	E	11×10^8	4.5	In the net section originating at the side of the hole	
2.5 Base metal in angle or tee section members connected to a gusset or connection plate with high-strength bolted slip-critical connections. The fatigue stress range shall be calculated on the effective net area of the member, $A_e = UA_g$, in which U=(1- $\bar{x}$ /L) and where A_g is the gross area of the member. $\bar{x}$ is the distance from the centroid of the member to the surface of the gusset or connection plate and L is the out-to-out distance between the bolts in the connection parallel to the line of force. The effect of the moment due to the eccentricities in the connection shall be ignored in computing the stress range (McDonald and Frank, 2009).	See applicable Category above	See applicable Constant above	See applicable Threshold above	Through the gross section near the hole, or in the net section originating at the side of the hole, as applicable	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

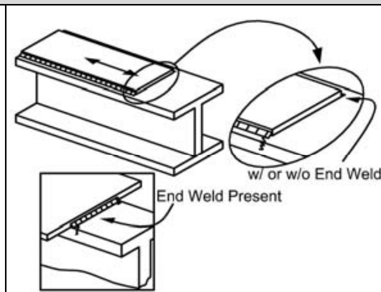
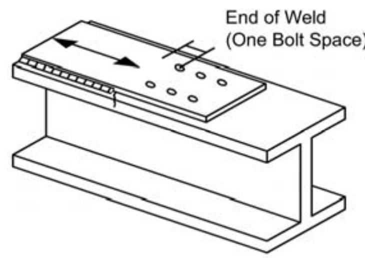
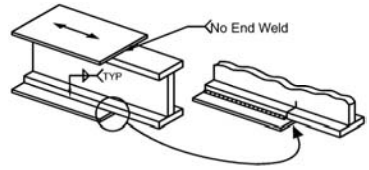
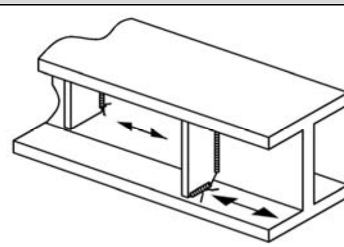
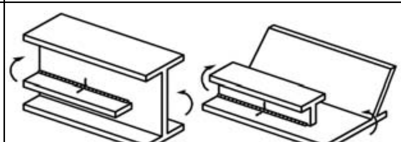
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
2.5 (continued) The fatigue category shall be taken as that specified for Condition 2.1. For all other types of bolted connections, replace A_g with the net area of the member, A_n, in computing the effective net area according to the preceding equation and use the appropriate fatigue category for that connection type specified for Condition 2.2 or 2.3, as applicable.					
Section 3—Welded Joints Joining Components of Built-Up Members					
3.1 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds back-gouged and welded from the second side, or by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From surface or internal discontinuities in the weld away from the end of the weld	
3.2 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds with backing bars not removed, or by continuous partial joint penetration groove welds parallel to the direction of applied stress.	B′	61×10^8	12	From surface or internal discontinuities in the weld, including weld attaching backing bars	
3.3 Base metal and weld metal at the termination of longitudinal welds at weld access holes made to the requirements of AASHTO/AWS D1.5, Article 3.2.4 in built-up members. (Note: does not include the flange butt splice).	D	22×10^8	7	From the weld termination into the web or flange	
3.4 Base metal and weld metal in partial length welded cover plates connected by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From surface or internal discontinuities in the weld away from the end of the weld	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

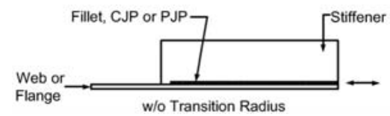
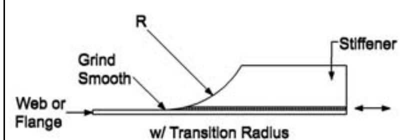
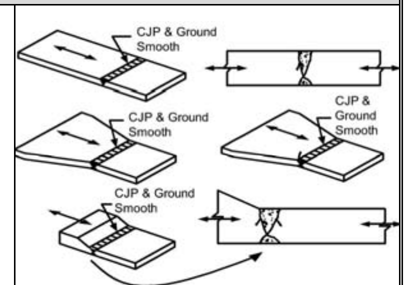
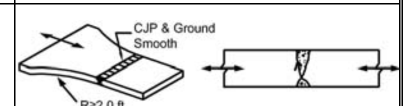
Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-Up Members (continued)					
3.5 Base metal at the termination of partial length welded cover plates having square or tapered ends that are narrower than the flange, with or without welds across the ends, or cover plates that are wider than the flange with welds across the ends:				In the flange at the toe of the end weld or in the flange at the termination of the longitudinal weld or in the edge of the flange with wide cover plates	
Flange thickness ≤ 0.8 in.	E	11×10^8	4.5		
Flange thickness > 0.8 in.	E′	3.9×10^8	2.6		
3.6 Base metal at the termination of partial length welded cover plates with slip-critical bolted end connections satisfying the requirements of Article 6.10.12.2.3.	B	120×10^8	16	In the flange at the termination of the longitudinal weld	
3.7 Base metal at the termination of partial length welded cover plates that are wider than the flange and without welds across the ends.	E′	3.9×10^8	2.6	In the edge of the flange at the end of the cover plate weld	
Section 4—Welded Stiffener Connections					
4.1 Base metal at the toe of transverse stiffener-to-flange fillet welds and transverse stiffener-to-web fillet welds. (Note: includes similar welds on bearing stiffeners and connection plates).	C′	44×10^8	12	Initiating from the geometrical discontinuity at the toe of the fillet weld extending into the base metal	
4.2 Base metal and weld metal in longitudinal web or longitudinal box-flange stiffeners connected by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From the surface or internal discontinuities in the weld away from the end of the weld	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

LRFDUS-6-E1: June 2012 Errata to LRFD Design, Sixth Edition

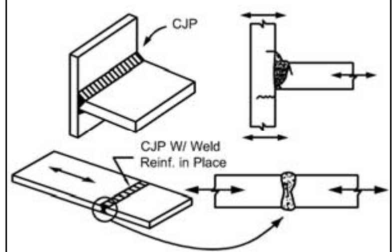
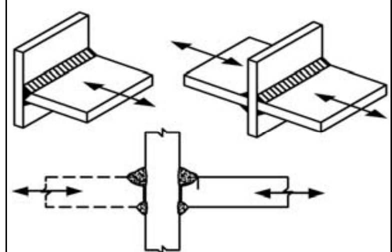
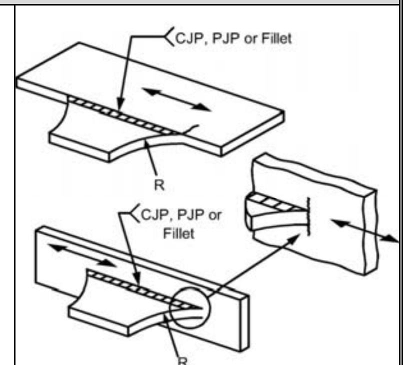
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 4—Welded Stiffener Connections (continued)					
4.3 Base metal at the termination of longitudinal stiffener-to-web or longitudinal stiffener-to-box flange welds:					
With the stiffener attached by fillet welds and with no transition radius provided at the termination:				In the primary member at the end of the weld at the weld toe	
Stiffener thickness < 1.0 in.	E	11 × 10^8	4.5		
Stiffener thickness ≥ 1.0 in.	E′	3.9 × 10^8	2.6		
With the stiffener attached by welds and with a transition radius R provided at the termination with the weld termination ground smooth:				In the primary member near the point of tangency of the radius	
$R \geq 24$ in.	B	120 × 10^8	16		
24 in. > $R \geq 6$ in.	C	44 × 10^8	10		
6 in. > $R \geq 2$ in.	D	22 × 10^8	7		
2 in. > R	E	11 × 10^8	4.5		
Section 5—Welded Joints Transverse to the Direction of Primary Stress					
5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices, with weld soundness established by NDT and with welds ground smooth and flush parallel to the direction of stress. Transitions in thickness or width shall be made on a slope no greater than 1:2.5 (see also Figure 6.13.6.2-1).				From internal discontinuities in the filler metal or along the fusion boundary or at the start of the transition	
F_y < 100 ksi	B	120 × 10^8	16		
F_y ≥ 100 ksi	B′	61 × 10^8	12		
5.2 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices, with weld soundness established by NDT and with welds ground parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft with the point of tangency at the end of the groove weld (see also Figure 6.13.6.2-1).	B	120 × 10^8	16	From internal discontinuities in the filler metal or discontinuities along the fusion boundary	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

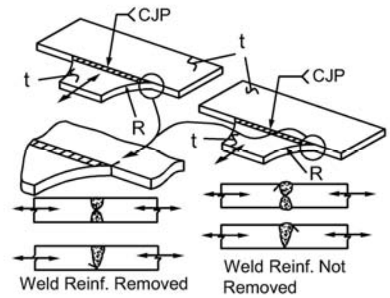
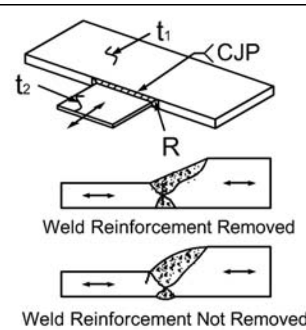
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
5.3 Base metal and weld metal in or adjacent to the toe of complete joint penetration groove welded T or corner joints, or in complete joint penetration groove welded butt splices, with or without transitions in thickness having slopes no greater than 1:2.5 when weld reinforcement is not removed. (Note: cracking in the flange of the "T" may occur due to out-of-plane bending stresses induced by the stem).	C	44×10^8	10	From the surface discontinuity at the toe of the weld extending into the base metal or along the fusion boundary	
5.4 Base metal and weld metal at details where loaded discontinuous plate elements are connected with a pair of fillet welds or partial joint penetration groove welds on opposite sides of the plate normal to the direction of primary stress.	C as adjusted in Eq. 6.6.1.2.5-4	44×10^8	10	Initiating from the geometrical discontinuity at the toe of the weld extending into the base metal or initiating at the weld root subject to tension extending up and then out through the weld	
Section 6—Transversely Loaded Welded Attachments					
6.1 Base metal in a longitudinally loaded component at a transversely loaded detail (e.g. a lateral connection plate) attached by a weld parallel to the direction of primary stress and incorporating a transition radius R with the weld termination ground smooth.				Near point of tangency of the radius at the edge of the longitudinally loaded component or at the toe of the weld at the weld termination if not ground smooth	
$R \geq 24$ in.	B	120×10^8	16		
24 in. > $R \geq 6$ in.	C	44×10^8	10		
6 in. > $R \geq 2$ in.	D	22×10^8	7		
2 in. > R	E	11×10^8	4.5		
For any transition radius with the weld termination not ground smooth (Note: Condition 6.2, 6.3 or 6.4, as applicable, shall also be checked.)	E	11×10^8	4.5		

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

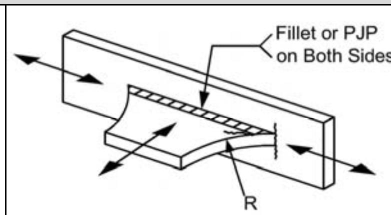
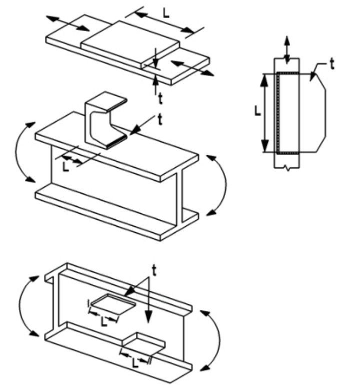
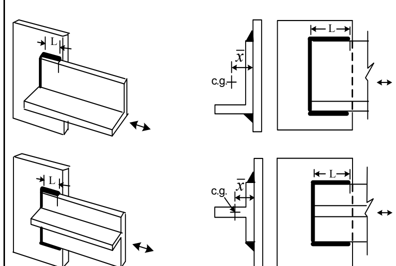
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments (continued)					
6.2 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component of equal thickness by a complete joint penetration groove weld parallel to the direction of primary stress and incorporating a transition radius R, with weld soundness established by NDT and with the weld termination ground smooth:					
With the weld reinforcement removed:					
$R \geq 24$ in.	B	120×10^8	16	Near points of tangency of the radius or in the weld or at the fusion boundary of the longitudinally loaded component or the transversely loaded attachment	
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
With the weld reinforcement not removed:				At the toe of the weld either along the edge of the longitudinally loaded component or the transversely loaded attachment	
$R \geq 24$ in.	C	44×10^8	10		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
(Note: Condition 6.1 shall also be checked.)					
6.3 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component of unequal thickness by a complete joint penetration groove weld parallel to the direction of primary stress and incorporating a weld transition radius R, with weld soundness established by NDT and with the weld termination ground smooth:				At the toe of the weld along the edge of the thinner plate In the weld termination of small radius weld transitions At the toe of the weld along the edge of the thinner plate	
With the weld reinforcement removed:					
R ≥ 2 in.	D	22×10^8	7		
$R < 2$ in.	E	11×10^8	4.5		
For any weld transition radius with the weld reinforcement not removed (Note: Condition 6.1 shall also be checked.)	E	11×10^8	4.5		

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

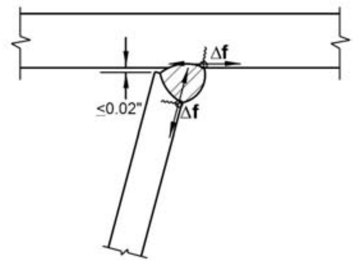
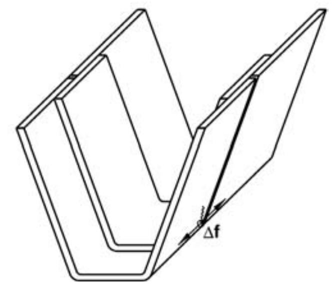
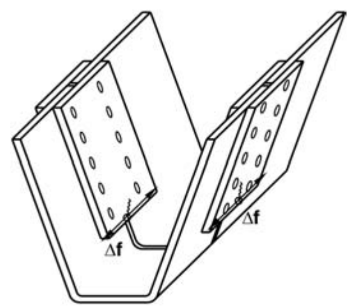
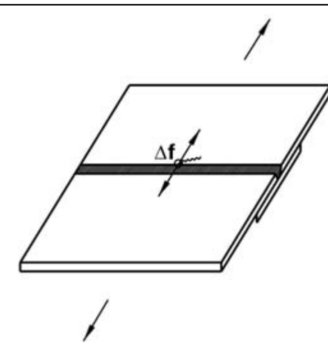
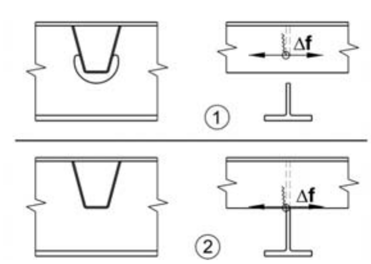
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments (continued)					
6.4 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component by a fillet weld or a partial joint penetration groove weld, with the weld parallel to the direction of primary stress (Note: Condition 6.1 shall also be checked.)	See Condition 5.4				
Section 7—Longitudinally Loaded Welded Attachments					
7.1 Base metal in a longitudinally loaded component at a detail with a length L in the direction of the primary stress and a thickness t attached by groove or fillet welds parallel or transverse to the direction of primary stress where the detail incorporates no transition radius:				In the primary member at the end of the weld at the weld toe	
$L < 2$ in.	C	44×10^8	10		
2 in. $\leq L \leq 12t$ or 4 in	D	22×10^8	7		
$L > 12t$ or 4 in.					
$t < 1.0$ in.	E	11×10^8	4.5		
$t \geq 1.0$ in.	E′	3.9×10^8	2.6		
(Note: see Condition 7.2 for welded angle or tee section member connections to gusset or connection plates.)					
7.2 Base metal in angle or tee section members connected to a gusset or connection plate by longitudinal fillet welds along both sides of the connected element of the member cross-section. The fatigue stress range shall be calculated on the effective net area of the member, $A_e = UA_g$, in which $U = (1- \bar{x}/L)$ and where A_g is the gross area of the member. $\bar{x}$ is the distance from the centroid of the member to the surface of the gusset or connection plate and L is the maximum length of the longitudinal welds. The effect of the moment due to the eccentricities in the connection shall be ignored in computing the stress range (McDonald and Frank, 2009).	E	11×10^8	4.5	Toe of fillet welds in connected element	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

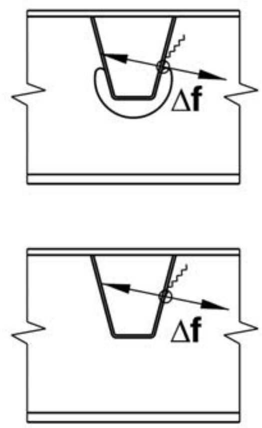
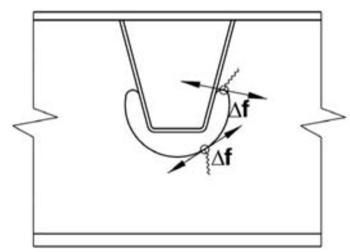
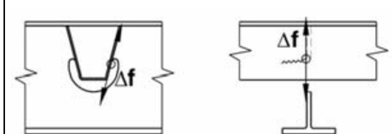
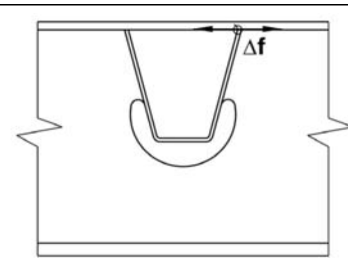
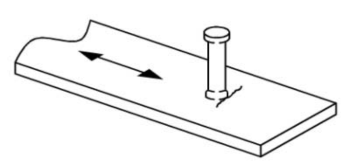
Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 8—Miscellaneous					
8.1 Rib to Deck Weld—One-sided 80% (70% min) penetration weld with root gap ≤ 0.02 in. prior to welding Allowable Design Level 1, 2, or 3	C	44×10^8	10	See Figure	
8.2 Rib Splice (Welded)—Single groove butt weld with permanent backing bar left in place. Weld gap > rib wall thickness Allowable Design Level 1, 2, or 3	D	22×10^8	7	See Figure	
8.3 Rib Splice (Bolted)—Base metal at gross section of high strength slip critical connection Allowable Design Level 1, 2, or 3	B	120×10^8	16	See Figure	
8.4 Deck Plate Splice (in Plane)—Transverse or Longitudinal single groove butt splice with permanent backing bar left in place Allowable Design Level 1, 2, or 3	D	22×10^8	7	See Figure	
8.5 Rib to FB Weld (Rib)—Rib wall at rib to FB weld (fillet or CJP) Allowable Design Level 1, 2, or 3	C	44×10^8	10	See Figure	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

Description	Category	Constant A (ksi^3)	Threshold $(\Delta f)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
8.6 Rib to FB Weld (FB Web)—FB web at rib to FB weld (fillet, PJP, or CJP) Allowable Design Level 1 or 3	C (see Note 1)	44×10^8	10	See Figure	
8.7 FB Cutout—Base metal at edge with "smooth" flame cut finish as per AWS D1.5 Allowable Design Level 1 or 3	A	250×10^8	24	See Figure	
8.8 Rib Wall at Cutout—Rib wall at rib to FB weld (fillet, PJP, or CJP) Allowable Design Level 1 or 3	C	44×10^8	10	See Figure	
8.9 Rib to Deck Plate at FB Allowable Design Level 1 or 3	C	44×10^8	10	See Figure	
Note 1: Where stresses are dominated by in-plane component at fillet or PJP welds, Eq. 6.6.1.2.5-4 shall be considered. In this case, Δf should be calculated at the mid-thickness and the extrapolation procedure as per Article 9.8.3.4.3 need not be applied.					
Section 9—Miscellaneous					
9.1 Base metal at stud-type shear connectors attached by fillet or automatic stud welding		44×10^8	10	At the toe of the weld in the base metal	

continued on next page

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table 6.6.1.2.3-1 (continued)—Detail Categories for Load-Induced Fatigue

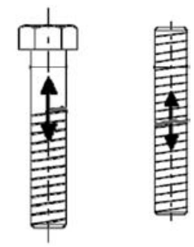
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 9—Miscellaneous (continued)					
9.2 Nonpretensioned high-strength bolts, common bolts, threaded anchor rods, and hanger rods with cut, ground, or rolled threads. Use the stress range acting on the tensile stress area due to live load plus prying action when applicable.				At the root of the threads extending into the tensile stress area	
(Fatigue II) Finite Life	E′	3.9×10^8	N/A		
(Fatigue I) Infinite Life	D	N/A	7		

Table 6.6.1.2.3-2—75-yr $(ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	75-yrs $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	530
B	860
B′	1035
C	1290
C′	745
D	1875
E	3530
E′	6485

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

[*This page is intentionally left blank. —ed.*]

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

[*This page is intentionally left blank. —ed.*]

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

6.6.1.2.4—Detailing to Reduce Constraint

To the extent practical, welded structures shall be detailed to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture. Welds that are parallel to the primary stress but interrupted by intersecting members shall be detailed to allow a minimum gap of 1 in. between weld toes.

C6.6.1.2.4

The objective of this Article is to provide recommended detailing guidelines for common joints to avoid details susceptible to brittle fracture.

The form of brittle fracture being addressed has been termed "constraint-induced fracture" and can occur without any perceptible fatigue crack growth and, more importantly, without any warning. This type of failure was documented during the Hoan Bridge failure investigation by Wright, Kaufmann, and Fisher (2003) and Kaufmann, Connor, and Fisher (2004). Criteria have been developed to identify bridges and details susceptible to this failure mode as discussed in Mahmoud, Connor and Fisher (2005).

Intersecting welds should be avoided.

Attached elements parallel to the primary stress are sometimes interrupted when intersecting a full-depth transverse member. These elements are less susceptible to fracture and fatigue if the attachment parallel to the primary stress is continuous and the transverse attachment is discontinuous as shown in Figure C6.6.1.2.4-1. Also shown is the space between the weld of the transverse stiffener to the web and the weld of the longitudinal stiffener to the web required to reduce constraint.

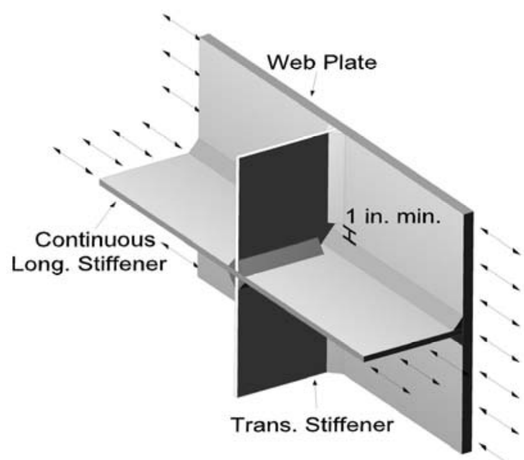


Figure C6.6.1.2.4-1—A Weld Detail where the Longitudinal Stiffener Is Continuous

6.6.1.2.5—Fatigue Resistance

Except as specified below, nominal fatigue resistance shall be taken as:

- For the Fatigue I load combination and infinite life:

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

- For the Fatigue II load combination and finite life:

C6.6.1.2.5

The requirement on higher-traffic-volume bridges that the maximum stress range experienced by a detail be less than the constant-amplitude fatigue threshold provides a theoretically infinite fatigue life. This requirement is reflected in Eq. 6.6.1.2.5-1.

The fatigue resistance above the constant amplitude fatigue threshold, in terms of cycles, is inversely proportional to the cube of the stress range, e.g., if the stress range is reduced by a factor of 2, the fatigue life increases by a factor of 2^3. This is reflected in

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

where:

A = total gross cross-sectional area of the member (in.^{2})

A_{eff} = summation of the effective areas of the cross-section based on a reduced effective width for each slender stiffened element in the cross-section = $A - \sum (b - b_e)\, t$ (in.^{2})

The effective width, b_e, shall be determined as follows:

- For flanges of square and rectangular box sections and HSS of uniform thickness; and nonperforated cover plates:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b \qquad (6.9.4.2.2\text{-}10)$$

- For webs; perforated cover plates; and all other stiffened elements:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b \qquad (6.9.4.2.2\text{-}11)$$

where:

f = $Q_s F_y$ (ksi)

Where all unstiffened elements, if any, in the cross-section are classified as nonslender, $Q_s = 1.0$.

For circular tubes, including round HSS, with D/t not exceeding $0.45\,E/F_y$, Q_a shall be taken as:

$$Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \qquad (6.9.4.2.2\text{-}12)$$

In the above, b, D, t, and k_c shall be taken as defined in Article 6.9.4.2.1 for the member element under consideration.

6.9.4.3—Built-up Members

6.9.4.3.1—General

The provisions of Article 6.9.4.2 shall apply. For built-up members composed of two or more shapes, the slenderness ratio of each component shape between connecting fasteners or welds shall not be more than 75 percent of the governing slenderness ratio of the built-up member. The least radius of gyration shall be used in computing the slenderness ratio of each component shape between the connectors.

C6.9.4.3.1

Two types of built-up members are commonly used for steel bridge construction: closely spaced steel shapes interconnected at intervals using welds or fasteners, and laced or battened members with widely spaced flange components.

The compressive resistance of built-up members is affected by the interaction between the global buckling mode of the member and the localized component buckling mode between lacing points or intermediate connectors.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, or batten plates shall be spaced so that the slenderness ratio of each component shape between the connectors shall not be more than 75 percent of the governing slenderness ratio of the built-up member.

The nominal compressive resistance of built-up members composed of two or more shapes shall be determined as specified in Article 6.9.4.1 subject to the following modification. If the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, $K\ell/r$ shall be replaced by $(K\ell/r)_m$ determined as follows for intermediate connectors that are welded or fully-tensioned bolted:

$$\left(\frac{K\ell}{r}\right)_m = \sqrt{\left(\frac{K\ell}{r}\right)_o^2 + 0.82\left(\frac{\alpha^2}{1+\alpha^2}\right)\left(\frac{a}{r_{ib}}\right)^2} \qquad (6.9.4.3.1\text{-}1)$$

where:

$\left(\frac{K\ell}{r}\right)_m$ = modified slenderness ratio of the built-up member

$\left(\frac{K\ell}{r}\right)_o$ = slenderness ratio of the built-up member acting as a unit in the buckling direction being considered

= separation ratio = $h/2r_{ib}$

a = distance between connectors (in.)

r_{ib} = radius of gyration of an individual component shape relative to its centroidal axis parallel to the member axis of buckling (in.)

h = distance between centroids of individual component shapes perpendicular to the member axis of buckling (in.)

6.9.4.3.2—Perforated Plates

Perforated plates shall satisfy the requirements of Articles 6.9.4.2 and 6.8.5.2 and shall be designed for the sum of the shear force due to the factored loads and an additional shear force taken as:

$$V = \frac{P_r}{100}\left(\frac{100}{(\ell/r) + 10} + \frac{8.8(\ell/r)F_y}{E}\right) \qquad (6.9.4.3.2\text{-}1)$$

where:

V = additional shear force (kip)

P_r = factored compressive resistance specified in Articles 6.9.2.1 or 6.9.2.2 (kip)

ℓ = member length (in.)

r = radius of gyration about an axis perpendicular to the perforated plate (in.)

Duan, Reno, and Uang (2002) refer to this type of buckling as compound buckling. For both types of built-up members, limiting the slenderness ratio of each component shape between connection fasteners or welds or between lacing points, as applicable, to 75 percent of the governing global slenderness ratio of the built-up member effectively mitigates the effect of compound buckling (Duan, Reno, and Uang, 2002).

The compressive resistance of both types of members is also affected by any relative deformation that produces shear forces in the connectors between the individual shapes. Eq. 6.9.4.3.1-1 is adopted from AISC (2005) and provides a modified slenderness ratio taking into account the effect of the shear forces. Eq. 6.9.4.3.1-1 applies for intermediate connectors that are welded or fully-tensioned bolted and was derived from theory and verified by test data (Aslani and Goel, 1991). For other types of intermediate connectors on built-up members, including riveted connectors on existing bridges, Eq. C6.9.4.3.1-1 as follows should instead be applied:

$$\left(\frac{K\ell}{r}\right)_m = \sqrt{\left(\frac{K\ell}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (C6.9.4.3.1\text{-}1)$$

where:

r_i = minimum radius of gyration of an individual component shape (in.)

Eq. C6.9.4.3.1-1 is based empirically on test results (Zandonini, 1985). In all cases, the connectors must be designed to resist the shear forces that develop in the buckled member.

Duan, Reno, and Lynch (2000) give an approach for determining the section properties of latticed built-up members, such as the moment of inertia and torsional constant.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

F_y = specified minimum yield strength (ksi)

In addition to checking the requirements of Article 6.9.4.2.1 for the clear distance between the two edge supports of the perforated cover plate utilizing a plate buckling coefficient k of 1.86, the requirements of Article 6.9.4.2.1 shall also separately be checked for the projecting width from the edge of the perforation to a single edge support utilizing a plate buckling coefficient k of 0.45.

6.9.4.4—Single-Angle Members

Single angles subject to combined axial compression and flexure about one or both principal axes and satisfying all of the following conditions, as applicable:

- End connections are to a single leg of the angle, and are welded or use a minimum of two bolts;
- The angle is loaded at the ends in compression through the same leg;
- The angle is not subjected to any intermediate transverse loads; and
- If used as web members in trusses, all adjacent web members are attached to the same side of the gusset plate or chord;

may be designed as axially loaded compression members for flexural buckling only according to the provisions of Articles 6.9.2.1, 6.9.4.1.1, and 6.9.4.1.2 provided the following effective slenderness ratio, $(K\ell/r)_{eff}$, is utilized in determining the nominal compressive resistance, P_n:

- For equal-leg angles and unequal-leg angles connected through the longer leg:
  - If $\frac{\ell}{r_x} \le 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{\ell}{r_x} \qquad (6.9.4.4\text{-}1)$$

  - If $\frac{\ell}{r_x} > 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 32 + 1.25\frac{\ell}{r_x} \qquad (6.9.4.4\text{-}2)$$

C**6.9.4.4**

Single angles are commonly used as compression members in cross-frames and lateral bracing for steel bridges. Since the angle is typically connected through one leg only, the member is subject to combined axial compression and flexure, or moments about both principal axes due to the eccentricities of the applied axial load. The angle is also usually restrained by differing amounts about its geometric x- and y-axes. As a result, the prediction of the nominal compressive resistance of these members under these conditions is difficult. The provisions contained herein provide significantly simplified provisions for the design of single-angle members satisfying certain conditions that are subject to combined axial compression and flexure. These provisions are based on the provisions for the design of single-angle members used in latticed transmission towers (ASCE, 2000). Similar provisions are also employed in Section E5 of AISC (2005).

In essence, these provisions permit the effect of the eccentricities to be neglected when these members are evaluated as axially loaded compression members for flexural buckling only using an appropriate specified effective slenderness ratio, $(K\ell/r)_{eff}$, in place of $(K\ell/r_s)$ in Eq. 6.9.4.1.2-1. The effective slenderness ratio indirectly accounts for the bending in the angles due to the eccentricity of the loading allowing the member to be proportioned according to the provisions of Article 6.9.2.1 as if it were a pinned-end concentrically loaded compression member. Furthermore, when the effective slenderness ratio is used, single angles need not be checked for flexural-torsional buckling. The actual maximum slenderness ratio of the angle, as opposed to $(K\ell/r)_{eff}$, is not to exceed the applicable limiting slenderness ratio specified in Article 6.9.3. Thus, if the actual maximum slenderness ratio of the angle exceeds the limiting ratio, a larger angle section must be selected until the ratio is satisfied. If $(K\ell/r)_{eff}$ exceeds the limiting ratio, but the actual maximum slenderness ratio of the angle does not, the design is satisfactory. The limiting ratios specified in Article 6.9.3 are well below the limiting ratio of 200 specified in AISC (2005).

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

- For unequal-leg angles that are connected through the shorter leg with the ratio of the leg lengths less than 1.7:

  - If $\frac{\ell}{r_x} \leq 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 72 + 0.75\frac{\ell}{r_x} + 4\left[\left(\frac{b_\ell}{b_s}\right)^2 - 1\right] \geq 0.95\frac{\ell}{r_z} \quad (6.9.4.4\text{-}3)$$

  - If $\frac{\ell}{r_x} > 80$, then:

$$\left(\frac{K\ell}{r}\right)_{eff} = 32 + 1.25\frac{\ell}{r_x} + 4\left[\left(\frac{b_\ell}{b_s}\right)^2 - 1\right] \geq 0.95\frac{\ell}{r_z} \quad (6.9.4.4\text{-}4)$$

where:

b_ℓ = length of the longer leg of an unequal-leg angle (in.)

b_s = length of the shorter leg of an unequal-leg angle (in.)

ℓ = distance between the work points of the joints measured along the length of the angle (in.)

r_x = radius of gyration about the geometric axis of the angle parallel to the connected leg (in.)

r_z = radius of gyration about the minor principal axis of the angle (in.)

The actual maximum slenderness ratio of the angle shall not exceed the applicable limiting slenderness ratio specified in Article 6.9.3. Single angles designed using $(K\ell/r)_{eff}$ shall not be checked for flexural-torsional buckling.

The expressions for the effective slenderness ratio presume significant end rotational restraint about the *y*-axis, or the axis perpendicular to the connected leg and gusset plate, as shown in Figure C6.9.4.4-1.

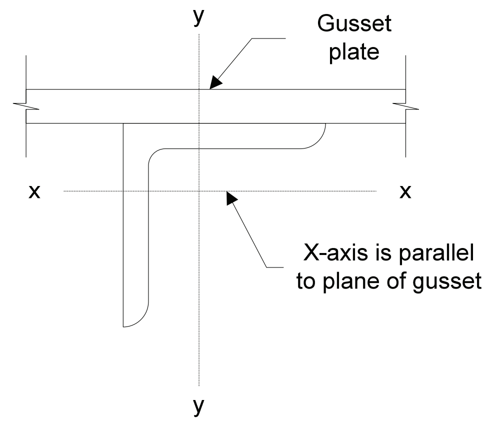


Figure C6.9.4.4-1—Single-Angle Geometric Axes Utilized in the Effective Slenderness Ratio Expressions

As a result, the angle tends to buckle primarily about the *x*-axis due to the eccentricity of the load about the *x*-axis coupled with the high degree of restraint about the *y*-axis (Usami and Galambos, 1971; Woolcock and Kitipornchai, 1986; Mengelkoch and Yura, 2002). Therefore, the radius of gyration in the effective slenderness ratio expressions is to be taken as r_x, or the radius of gyration about the geometric axis parallel to the connected leg, and not the minimum radius of gyration r_z about the minor principal axis of the angle. When an angle has significant rotational restraint about the *y*-axis, the stress along the connected leg will be approximately uniform (Lutz, 1996). Lutz (2006) compared the results from the effective slenderness ratio equations contained herein to test results for single-angle members in compression with essentially pinned-end conditions (Foehl, 1948; Trahair et al., 1969) and found an average value of P_n/P_{test} of 0.998 with a coefficient of variation of 0.109. A separate set of equations provided in AISC (2005), which assume a higher degree of *x*-axis rotational restraint and are thus intended for application only to single angles used as web members in box or space trusses, are not provided herein.

For the case of unequal-leg angles connected through the shorter leg, the limited available test data for this case gives lower capacities for comparable ℓ/r_x values than equal-leg angles (Lutz, 2006). Stiffening the shorter leg rotationally tends to force the buckling axis of the angle away from the *x*-axis and closer to the *z*-axis. Thus, $(K\ell/r)_{eff}$ for this case is modified by adding an additional term in Eqs. 6.9.4.4-3 and 6.9.4.4-4 along with a governing slenderness limit based on ℓ/r_z for slender unequal-leg angles. The upper limit on b_ℓ/b_s of 1.7 is based on the limits of the available physical tests. For an unequal-leg

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$f_\ell \leq 0.6F_{yf} \quad (6.10.1.6\text{-}1)$$

The flange lateral bending stress, f_ℓ, may be determined directly from first-order elastic analysis in discretely braced compression flanges for which:

$$L_b \leq 1.2L_p\sqrt{\frac{C_b R_b}{f_{bu}/F_{yc}}} \quad (6.10.1.6\text{-}2)$$

or equivalently:

$$L_b \leq 1.2L_p\sqrt{\frac{C_b R_b}{M_u/M_{yc}}} \quad (6.10.1.6\text{-}3)$$

where:

C_b = moment gradient modifier specified in Article 6.10.8.2.3 or Article A6.3.3, as applicable.

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (ksi)

L_b = unbraced length (in.)

L_p = limiting unbraced length specified in Article 6.10.8.2.3 (in.)

M_u = largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (kip-in.)

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

If Eq. 6.10.1.6-2, or Eq. 6.10.1.6-3 as applicable, is not satisfied, second-order elastic compression-flange lateral bending stresses shall be determined.

Second-order compression-flange lateral bending stresses may be approximated by amplifying first-order values as follows:

$$f_\ell = \left(\frac{0.85}{1-\frac{f_{bu}}{F_{cr}}}\right) f_{\ell 1} \;\geq\; f_{\ell 1} \quad (6.10.1.6\text{-}4)$$

or equivalently:

In lieu of a more refined analysis, Article C6.10.3.4 gives approximate equations for calculation of the maximum flange lateral bending moments due to eccentric concrete deck overhang loads acting on cantilever forming brackets placed along exterior members. Determination of flange wind moments is addressed in Article 4.6.2.7. The determination of flange lateral bending moments due to the effect of discontinuous cross-frames and/or support skew is best handled by a direct structural analysis of the bridge superstructure. The determination of flange lateral bending moments due to curvature is addressed in Article 4.6.1.2.4b.

In all resistance equations, f_{bu}, M_u, and f_ℓ are to be taken as positive in sign. However, for service and strength limit state checks at locations where the dead and live load contributions to f_{bu}, M_u or f_ℓ are of opposite sign, the signs of each contribution must be initially taken into account. In such cases, for both dead and live load, the appropriate net sum of the major-axis and lateral bending actions due to the factored loads must be computed, taking the signs into consideration that will result in the most critical response for the limit state under consideration.

The top flange may be considered continuously braced where it is encased in concrete or anchored to the deck by shear connectors satisfying the provisions of Article 6.10.10. For a continuously braced flange in tension or compression, flange lateral bending effects need not be considered. Additional lateral bending stresses are small once the concrete deck has been placed. Lateral bending stresses induced in a continuously braced flange prior to this stage need not be considered after the deck has been placed. The resistance of the composite concrete deck is generally adequate to compensate for the neglect of these initial lateral bending stresses. The Engineer should consider the non-composite lateral bending stresses in the top flange if the flange is not continuously supported by the deck.

The provisions of Article 6.10 for handling of combined vertical and flange lateral bending are limited to I-sections that are loaded predominantly in major-axis bending. For cases in which the elastically computed flange lateral bending stress is larger than approximately $0.6F_{yf}$, the reduction in the major-axis bending resistance due to flange lateral bending tends to be greater than that determined based on these provisions. The service and strength limit state provisions of these Specifications are sufficient to ensure acceptable performance of I-girders with elastically computed f_ℓ values somewhat larger than this limit.

Eq. 6.10.1.6-2, or equivalently Eq. 6.10.1.6-3 as applicable, simply gives a maximum value of L_b for which $f_\ell = f_{\ell 1}$ in Eq. 6.10.1.6-4 or 6.10.1.6-5. Eq. 6.10.1.6-4, or equivalently Eq. 6.10.1.6-5 as applicable, is an approximate formula that accounts for the amplification of the first-order compression-flange lateral bending stresses due to second-order effects. This equation, which is an established form for estimating the maximum second-order elastic moments in braced beam-column members whose ends are restrained by other framing, tends to be significantly conservative for

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$f_\ell = \left(\frac{0.85}{1 - \dfrac{M_u}{F_{cr} S_{xc}}} \right) f_{\ell 1} \geq f_{\ell 1} \qquad (6.10.1.6\text{-}5)$$

where:

f_{bu} = largest value of the compressive stress throughout the unbraced length in the flange under consideration, calculated without consideration of flange lateral bending (ksi)

$f_{\ell 1}$ = first-order compression-flange lateral bending stress at the section under consideration, or the maximum first-order lateral bending stress in the compression flange under consideration throughout the unbraced length, as applicable (ksi)

F_{cr} = elastic lateral torsional buckling stress for the flange under consideration determined from Eq. 6.10.8.2.3-8 or Eq. A6.3.3-8. Eq. A6.3.3-8 may only be applied for unbraced lengths in straight I-girder bridges in which the web is compact or noncompact.

M_u = largest value of the major-axis bending moment throughout the unbraced length causing compression in the flange under consideration (kip-in.)

S_{xc} = elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} (in.^{3})

larger unsupported lengths associated with f_{bu} approaching F_{cr} (White et al., 2001). This conservatism exists even when an effective length factor for lateral torsional buckling and/or a moment gradient factor C_b is considered in the calculation of F_{cr}, and even when one end of the unbraced segment under consideration is not restrained by an adjacent segment. Although Eqs. 6.10.1.6-4 and 6.10.1.6-5 are directed at estimating the maximum second-order lateral bending stress within the unbraced length, by use of the maximum first-order lateral bending stress for $f_{\ell 1}$, they may be applied for estimating the second-order lateral bending stresses at any cross-section within the unbraced length under consideration by use of the corresponding value of $f_{\ell 1}$ at that location.

The purpose of Eqs. 6.10.1.6-4 and 6.10.1.6-5 is to guard conservatively against large unbraced lengths in which the flange second-order lateral bending effects are significant. In construction situations where the amplification within these equations is large, the Engineer may wish to consider a direct geometric nonlinear analysis to more accurately determine the second-order effects within the superstructure, or using a lower value of the effective length factor for lateral torsional buckling to appropriately increase F_{cr} according to the procedure suggested in Article C6.10.8.2.3.

Note that the calculated value of F_{cr} for use in Eq. 6.10.1.6-4 is not limited to $R_b R_h F_{yc}$ as specified in Article 6.10.8.2.3, and that the calculated value of $F_{cr}S_{xc}$ for use in Eq. 6.10.1.6-5 is not limited to $R_{pc}M_{yc}$ as specified in Article A6.3.3. The elastic buckling stress is the appropriate stress for use in Eqs. 6.10.1.6-4 and 6.10.1.6-5 to estimate the elastic second-order amplification of the flange lateral bending stresses.

The definitions of a compact web and of a noncompact web are discussed in Article C6.10.6.2.3.

6.10.1.7—Minimum Negative Flexure Concrete Deck Reinforcement

Wherever the longitudinal tensile stress in the concrete deck due to either the factored construction loads or Load Combination Service II in Table 3.4.1-1 exceeds ϕf_r, the total cross-sectional area of the longitudinal reinforcement shall not be less than one percent of the total cross-sectional area of the concrete deck. ϕ shall be taken as 0.9 and f_r shall be taken as the modulus of rupture of the concrete determined as follows:

- For normal-weight concrete: $f_r = 0.24\sqrt{f'_c}$
- For lightweight concrete: f_r is calculated as specified in Article 5.4.2.6,

The longitudinal stresses in the concrete deck shall be determined as specified in Article 6.10.1.1.1d. The reinforcement used to satisfy this requirement shall have a specified minimum yield strength not less than 60.0 ksi; the size of the reinforcement should not exceed No. 6 bars.

The required reinforcement should be placed in two layers uniformly distributed across the deck width, and

C6.10.1.7

The use of one percent reinforcement with a size not exceeding No. 6 bars, a yield strength greater than or equal to 60.0 ksi, and spacing at intervals not exceeding 12.0 in. is intended to control concrete deck cracking. Pertinent criteria for concrete crack control are discussed in more detail in AASHTO (1991) and in Haaijer et al. (1987).

Previously, the requirement for one percent longitudinal reinforcement was limited to negative flexure regions of continuous spans, which are often implicitly taken as the regions between points of dead load contraflexure. Under moving live loads, the deck can experience significant tensile stresses outside the points of dead load contraflexure. Placement of the concrete deck in stages can also produce negative flexure during construction in regions where the deck already has been placed, although these regions may be subjected primarily to positive flexure in the final condition. Thermal and shrinkage strains can also cause tensile stresses in the deck in regions where such stresses otherwise might not be anticipated. To address these issues, the one percent

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

two-thirds should be placed in the top layer. The individual bars should be spaced at intervals not exceeding 12.0 in.

Where shear connectors are omitted from the negative flexure region, all longitudinal reinforcement shall be extended into the positive flexure region beyond the additional shear connectors specified in Article 6.10.10.3 a distance not less than the development length specified in Section 5.

longitudinal reinforcement is to be placed wherever the tensile stress in the deck due to either the factored construction loads, including loads during the various phases of the deck placement sequence, or due to Load Combination Service II in Table 3.4.1-1, exceeds ϕf_r. By satisfying the provisions of this Article to control the crack size in regions where adequate shear connection is also provided, the concrete deck may be considered to be effective in tension for computing fatigue stress ranges, as permitted in Article 6.6.1.2.1, and in determining flexural stresses on the composite section due to Load Combination Service II, as permitted in Article 6.10.4.2.1.

In addition to providing one percent longitudinal deck reinforcement, nominal yielding of this reinforcement should be prevented at Load Combination Service II (Carskaddan, 1980; AASHTO, 1991; Grubb, 1993) to control concrete deck cracking. The use of longitudinal deck reinforcement with a specified minimum yield strength not less than 60.0 ksi may be taken to preclude nominal yielding of the longitudinal reinforcement under this load combination in the following cases:

- Unshored construction where the steel section utilizes steel with a specified minimum yield strength less than or equal to 70.0 ksi in either flange, or
- Shored construction where the steel section utilizes steel with a specified minimum yield strength less than or equal to 50.0 ksi in either flange.

In these cases, the effects of any nominal yielding within the longitudinal reinforcing steel are judged to be insignificant. Otherwise, the Engineer should check to ensure that nominal yielding of the longitudinal reinforcement does not occur under the applicable Service II loads. The above rules are based on Carskaddan (1980) and apply for members that are designed by the provisions of Article 6.10 or Appendix A6, as well as for members that are designed for redistribution of the pier section moments at the Service II Load Combination using the provisions of Appendix B6.

Where feasible, approximately two-thirds of the required reinforcement should be placed in the top layer. When precast deck panels are used as deck forms, it may not be possible to place the longitudinal reinforcement in two layers. In such cases, the placement requirements may be waived at the discretion of the Engineer.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

6.10.1.8—Net Section Fracture

When checking flexural members at the strength limit state or for constructibility, the following additional requirement shall be satisfied at all cross-sections containing holes in the tension flange:

$$f_t \leq 0.84\left(\frac{A_n}{A_g}\right)F_u \leq F_{yt} \qquad (6.10.1.8\text{-}1)$$

where:

A_n = net area of the tension flange determined as specified in Article 6.8.3 (in.^{2})
A_g = gross area of the tension flange (in.^{2})
f_t = stress on the gross area of the tension flange due to the factored loads calculated without consideration of flange lateral bending (ksi)
F_u = specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi)

C6.10.1.8

If Eq. 6.10.1.8-1 is satisfied under the stated conditions at a cross-section containing holes in the tension flange, fracture on the net section of the flange is prevented. For holes larger than those typically used for connectors such as bolts, refer to Article 6.8.1.

At compact composite sections in positive flexure and at sections designed according to the optional provisions of Appendix A6 with no holes in the tension flange, the nominal flexural resistance is permitted to exceed the moment at first yield at the strength limit state. Pending the results from further research, it is conservatively required that Eq. 6.10.1.8-1 also be satisfied at the strength limit state at any such cross-sections containing holes in the tension flange. It has not yet been fully documented that complete plastification of the cross-section can occur at these sections prior to fracture on the net section of the tension flange. Furthermore, the splice design provisions of Article 6.13.6.1.4 do not consider the contribution of substantial web yielding to the flexural resistance of these sections. Eq. 6.10.1.8-1 will likely prevent holes from being located in the tension flange at or near points of maximum applied moment where significant yielding of the web, beyond the localized yielding permitted in hybrid sections, may occur.

The factor 0.84 in Eq. 6.10.1.8-1 is approximately equivalent to the ratio of the resistance factor for fracture of tension members, ϕ_u, to the resistance factor for yielding of tension members, ϕ_y, specified in Article 6.5.4.2.

6.10.1.9—Web Bend-Buckling Resistance

6.10.1.9.1—Webs without Longitudinal Stiffeners

The nominal bend-buckling resistance shall be taken as:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad (6.10.1.9.1\text{-}1)$$

but not to exceed the smaller of R_hF_{yc} and $F_{yw}/0.7$

in which:

k = bend-buckling coefficient

$$= \frac{9}{(D_c/D)^2} \qquad (6.10.1.9.1\text{-}2)$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.
R_h = hybrid factor specified in Article 6.10.1.10.1

C6.10.1.9.1

In subsequent Articles, the web theoretical bend-buckling resistance is checked generally against the maximum compression-flange stress due to the factored loads, calculated without consideration of flange lateral bending. The precision associated with making a distinction between the stress in the compression flange and the maximum compressive stress in the web is not warranted. The potential use of a value of F_{crw} greater than the specified minimum yield strength of the web, F_{yw}, in hybrid sections is justified since the flange tends to restrain the longitudinal strains associated with web bend-buckling for nominal compression-flange stresses up to R_hF_{yc}. A stable nominally elastic compression flange constrains the longitudinal and plate bending strains in the inelastic web at the web-flange juncture (ASCE, 1968). ASCE (1968) recommends that web bend-buckling does not need to be considered in hybrid sections with F_{yc} up to 100 ksi as long as the web slenderness does not exceed $5.87\sqrt{E/F_{yc}}$. Eq. 6.10.1.9.1-1 predicts $F_{crw} = F_{yc}$ at $2D_c/t_w = 5.7\sqrt{E/F_{yc}}$. For hybrid sections with $F_{yw}/F_{yc} < 0.7$, these provisions adopt a more conservative approach than recommended by ASCE (1968) by limiting F_{crw} to the smaller of R_hF_{yc} and $F_{yw}/0.7$. The

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Composite sections in straight bridges that satisfy the following requirements shall qualify as compact composite sections:

- The specified minimum yield strengths of the flanges do not exceed 70.0 ksi,
- The web satisfies the requirement of Article 6.10.2.1.1, and
- The section satisfies the web slenderness limit:

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

where:

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

Compact sections shall satisfy the requirements of Article 6.10.7.1. Otherwise, the section shall be considered noncompact and shall satisfy the requirements of Article 6.10.7.2.

Compact and noncompact sections shall satisfy the ductility requirement specified in Article 6.10.7.3.

the provisions of Article 6.10.7. The nominal flexural resistance of these sections, termed compact sections, is therefore more appropriately expressed in terms of moment. For composite sections in positive flexure in straight bridges not satisfying one or more of these requirements, or for composite sections in positive flexure in horizontally curved bridges, termed noncompact sections, the nominal flexural resistance is not permitted to exceed the moment at first yield. The nominal flexural resistance in these cases is therefore more appropriately expressed in terms of the elastically computed flange stress.

Composite sections in positive flexure in straight bridges with flange yield strengths greater than 70.0 ksi or with webs that do not satisfy Article 6.10.2.1.1 are to be designed at the strength limit state as noncompact sections as specified in Article 6.10.7.2. For concrete compressive strengths typically employed for deck construction, the use of larger steel yield strengths may result in significant nonlinearity and potential crushing of the deck concrete prior to reaching the flexural resistance specified for compact sections in Article 6.10.7.1. Longitudinal stiffeners generally must be provided in sections with webs that do not satisfy Article 6.10.2.1.1. Since composite longitudinally-stiffened sections tend to be deeper and used in longer spans with corresponding larger noncomposite dead load stresses, they tend to have D_c/t_w values that would preclude the development of substantial inelastic flexural strains within the web prior to bend-buckling at moment levels close to R_hM_y. Therefore, although the depth of the web in compression typically reduces as plastic strains associated with moments larger than R_hM_y are incurred, and D_{cp} may indeed satisfy Eq. 6.10.6.2.2-1 at the plastic moment resistance, sufficient test data do not exist to support the design of these types of sections for M_p. Furthermore, because of the relative size of the steel section to the concrete deck typical for these types of sections, M_p often is not substantially larger than R_hM_y. Due to these factors, composite sections in positive flexure in which the web does not satisfy Article 6.10.2.1.1 are categorized as noncompact sections. Composite sections in positive flexure in kinked (chorded) continuous or horizontally curved steel bridges are also to be designed at the strength limit state as noncompact sections as specified in Article 6.10.7.2. Research has not yet been conducted to support the design of these sections for a nominal flexural resistance exceeding the moment at first yield.

The web slenderness requirement of this Article is adopted from AISC (2005) and gives approximately the same allowable web slenderness as specified for compact sections in AASHTO (2002). Most composite sections in positive flexure without longitudinal web stiffeners will qualify as compact according to this criterion since the concrete deck causes an upward shift in the neutral axis, which reduces the depth of the web in compression. Also, D/t_w for these sections is limited to a maximum value of 150 based on the requirement of Article 6.10.2.1.1. The

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

location of the neutral axis of the composite section at the plastic moment may be determined using the equations listed in Table D6.1-1.

Compact composite sections in positive flexure must also satisfy the provisions of Article 6.10.7.3 to ensure a ductile mode of failure. Noncompact sections must also satisfy the ductility requirement specified in Article 6.10.7.3 to ensure a ductile failure. Satisfaction of this requirement ensures an adequate margin of safety against premature crushing of the concrete deck for sections utilizing up to 100-ksi steels and/or for sections utilized in shored construction. This requirement is also a key limit in allowing web bend-buckling to be disregarded in the design of composite sections in positive flexure when the web also satisfies Article 6.10.2.1.1, as discussed in Article C6.10.1.9.1.

6.10.6.2.3—Composite Sections in Negative Flexure and Noncomposite Sections

Sections in all kinked (chorded) continuous or horizontally curved steel girder bridges shall be proportioned according to the provisions specified in Article 6.10.8.

Sections in straight bridges whose supports are normal or skewed not more than 20° from normal, and with intermediate diaphragms or cross-frames placed in contiguous lines parallel to the supports, for which:

- The specified minimum yield strengths of the flanges do not exceed 70.0 ksi,
- The web satisfies the noncompact slenderness limit:

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

and:

- The flanges satisfy the following ratio:

$$\frac{I_{yc}}{I_{yt}} \geq 0.3 \quad (6.10.6.2.3\text{-}2)$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

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})

C6.10.6.2.3

For composite sections in negative flexure and noncomposite sections, the provisions of Article 6.10.8 limit the nominal flexural resistance to be less than or equal to the moment at first yield. As a result, the nominal flexural resistance for these sections is conveniently expressed in terms of the elastically computed flange stress.

For composite sections in negative flexure or noncomposite sections in straight bridges without skewed supports or with limited skews that satisfy the specified steel grade requirements and with webs that satisfy Eq. 6.10.6.2.3-1 and flanges that satisfy Eq. 6.10.6.2.3-2, the optional provisions of Appendix A6 may be applied to determine the nominal flexural resistance, which may exceed the moment at first yield. Therefore, the nominal flexural resistance determined from the provisions of Appendix A6 is expressed in terms of moment. Because these types of sections are less commonly used, the provisions for their design have been placed in an appendix in order to simplify and streamline the main design provisions. The provisions of Article 6.10.8 may be used for these types of sections to obtain an accurate to somewhat conservative determination of the nominal flexural resistance than would be obtained using Appendix A6.

For composite sections in negative flexure or noncomposite sections in straight bridges not satisfying one or more of these requirements, or for these sections in horizontally curved bridges, the provisions of Article 6.10.8 must be used. Research has not yet been conducted to extend the provisions of Appendix A6 either to sections in kinked (chorded) continuous or horizontally curved steel bridges or to bridges with supports skewed more than 20 degrees from normal. Severely skewed bridges with contiguous cross-frames have significant transverse stiffness and thus already have large cross-frame forces in the elastic range. As interior-pier sections yield and begin to lose stiffness and shed their load, the forces in the adjacent cross-frames will increase. There is currently no established procedure to predict the resulting increase in the forces without

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

may be proportioned according to the provisions for compact or noncompact web sections specified in Appendix A6. Otherwise, the section shall be proportioned according to provisions specified in Article 6.10.8.

For continuous span flexural members in straight bridges that satisfy the requirements of Article B6.2, a calculated percentage of the negative moments due to the factored loads at the pier section under consideration may be redistributed using the procedures of either Article B6.4 or B6.6.

performing a refined nonlinear analysis. With discontinuous cross-frames, significant lateral flange bending effects can occur. The resulting lateral bending moments and stresses are amplified in the bottom compression flange adjacent to the pier as the flange deflects laterally. There is currently no means to accurately predict these amplification effects as the flange is also yielding. Skewed supports also result in twisting of the girders, which is not recognized in plastic-design theory. The relative vertical deflections of the girders create eccentricities that are also not recognized in the theory. Thus, until further research is done to examine these effects in greater detail, a conservative approach has been taken in the specification.

Eq. 6.10.6.2.3-1 defines the slenderness limit for a noncompact web. A web with a slenderness ratio exceeding this limit is termed slender. The previous Specifications defined sections as compact or noncompact and did not explicitly distinguish between a noncompact and a slender web. For noncompact webs, theoretical web bend-buckling does not occur for elastic stress values, computed according to beam theory, smaller than the limit of the flexural resistance. Sections with slender webs rely upon the significant web post bend-buckling resistance under Strength Load Combinations. Specific values for the noncompact web slenderness limit for different grades of steel are listed in Table C6.10.1.10.2-2.

A compact web is one that satisfies the slenderness limit given by Eq. A6.2.1-1. Sections with compact webs and $I_{yc}/I_{yt} \geq 0.3$ are able to develop their full plastic moment capacity M_p provided that other steel grade, ductility, flange slenderness and/or lateral bracing requirements are satisfied. The web-slenderness limit given by Eq. A6.2.1-1 is significantly smaller than the limit shown in Table C6.10.1.10.2-2. It is generally satisfied by rolled I-shapes, but typically not by the most efficient built-up section proportions.

The flange yield stress, F_{yc}, is more relevant to the web buckling behavior and its influence on the flexural resistance than F_{yw}. For a section that has a web proportioned at the noncompact limit, a stable nominally elastic compression flange tends to constrain a lower-strength hybrid web at stress levels less than or equal to R_hF_{yc}. For a section that has a compact web, the inelastic strains associated with development of the plastic flexural resistance are more closely related to the flange rather than the web yield strength.

The majority of steel-bridge I-sections utilize either slender webs or noncompact webs that approach the slenderness limit of Eq. 6.10.6.2.3-1 represented by the values listed in Table C6.10.1.10.2-2. For these sections, the simpler and more streamlined provisions of Article 6.10.8 are the most appropriate for determining the nominal flexural resistance of composite sections in negative flexure and noncomposite sections. These provisions may also be applied to sections with compact webs or to sections with noncompact webs that are nearly compact, but at the

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

expense of some economy. Such sections are typically used in bridges with shorter spans. The potential loss in economy increases with decreasing web slenderness. The Engineer should give strong consideration to utilizing the provisions of Appendix A6 to compute the nominal flexural resistance of these sections in straight bridges, in particular, sections with compact webs.

Eq. 6.10.6.2.3-2 is specified to guard against extremely monosymmetric noncomposite I-sections, in which analytical studies indicate a significant loss in the influence of the St. Venant torsional rigidity *GJ* on the lateral-torsional buckling resistance due to cross-section distortion. The influence of web distortion on the lateral torsional buckling resistance is larger for such members. If the flanges are of equal thickness, this limit is equivalent to $b_{fc} \geq 0.67b_{ft}$.

Yielding in negative-flexural sections at interior piers at the strength limit state results in redistribution of the elastic moments. For continuous-span flexural members in straight bridges that satisfy the provisions of Article B6.2, the procedures of either Article B6.4 or B6.6 may be used to calculate redistribution moments at the strength limit state. These provisions replace the former ten-percent redistribution allowance and provide a more rational approach for calculating the percentage redistribution from interior-pier sections. When the redistribution moments are calculated according to these procedures, the flexural resistances at the strength limit state within the unbraced lengths immediately adjacent to interior-pier sections satisfying the requirements of Article B6.2 need not be checked. At all other locations, the provisions of Articles 6.10.7, 6.10.8.1 or A6.1, as applicable, must be satisfied after redistribution. The provisions of Article B6.2 are often satisfied by compact-flange unstiffened or transversely-stiffened pier sections that are otherwise designed by Article 6.10.8 or Appendix A6 using C_b = 1.0. Research has not yet been conducted to extend the provisions of Appendix B6 to kinked (chorded) continuous or horizontally curved steel bridges.

6.10.6.3—Shear

The provisions of Article 6.10.9 shall apply.

6.10.6.4—Shear Connectors

The provisions of Article 6.10.10.4 shall apply.

6.10.7—Flexural Resistance—Composite Sections in Positive Flexure

6.10.7.1—Compact Sections

6.10.7.1.1—General

At the strength limit state, the section shall satisfy:

$$M_u + \frac{1}{3} f_\ell S_{xt} \leq \phi_f M_n \qquad (6.10.7.1.1\text{-}1)$$

C6.10.7.1.1

For composite sections in positive flexure, lateral bending does not need to be considered in the compression flange at the strength limit state because the flange is continuously supported by the concrete deck.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
M_n = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in.)
M_u = bending moment about the major-axis of the cross-section determined as specified in Article 6.10.1.6 (kip-in.)
M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in.)
S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in.^{3})

Eq. 6.10.7.1.1-1 is an interaction equation that addresses the influence of lateral bending within the tension flange, represented by the elastically computed flange lateral bending stress, f_ℓ, combined with the major-axis bending moment, M_u. This equation is similar to the subsequent Eqs. 6.10.7.2.1-2 and 6.10.8.1.2-1, the basis of which is explained in Article C6.10.8.1.2. However, these other equations are expressed in an elastically computed stress format, and the resistance term on their right-hand side is generally equal to $\phi_f R_h F_{yt}$. Eq. 6.10.7.1.1-1 is expressed in a bending moment format, but alternatively can be considered in a stress format by dividing both sides of the equation by the elastic section modulus, S_{xt}.

The term M_n on the right-hand side of Eq. 6.10.7.1.1-1 is generally greater than the yield moment capacity, M_{yt}. Therefore, the corresponding resistance, written in the format of an elastically computed stress, is generally greater than F_{yt}. These Specifications use a moment format for all resistance equations which, if written in terms of an elastically computed stress, can potentially assume resistance values greater than the specified minimum yield strength of the steel. In these types of sections, the major-axis bending moment is physically a more meaningful quantity than the corresponding elastically computed bending stress.

Eq. 6.10.7.1.1-1 gives a reasonably accurate but conservative representation of the results from an elastic-plastic section analysis in which a fraction of the width from the tips of the tension flange is deducted to accommodate flange lateral bending. The rationale for calculation of S_{xt}, as defined in this Article for use in Eq. 6.10.7.1.1-1, is discussed in Article CA6.1.1.

6.10.7.1.2—Nominal Flexural Resistance

The nominal flexural resistance of the section shall be taken as:

If $D_p \leq 0.1\, D_t$, then:

$$M_n = M_p \tag{6.10.7.1.2-1}$$

Otherwise:

$$M_n = M_p \left(1.07 - 0.7\frac{D_p}{D_t}\right) \tag{6.10.7.1.2-2}$$

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 of the composite section determined as specified in Article D6.1 (kip-in.)

In a continuous span, the nominal flexural resistance of the section shall satisfy:

C6.10.7.1.2

Eq. 6.10.7.1.2-2 implements the philosophy introduced by Wittry (1993) that an additional margin of safety should be applied to the theoretical nominal flexural resistance of compact composite sections in positive flexure when the depth of the plastic neutral axis below the top of the deck, D_p, exceeds a certain value. This additional margin of safety, which increases approximately as a linear function of D_p/D_t, is intended to protect the concrete deck from premature crushing, thereby ensuring adequate ductility of the composite section. Sections with D_p/D_t less than or equal to 0.1 can reach as a minimum the plastic moment, M_p, of the composite section without any ductility concerns.

Eq. 6.10.7.1.2-2 gives approximately the same results as the comparable equation in previous Specifications, but is a simpler form that depends only on the plastic moment resistance M_p and on the ratio D_p/D_t, as also suggested in Yakel and Azizinamini (2005). Both equations implement the above philosophy justified by Wittry (1993). Eq. 6.10.7.1.2-2 is somewhat more restrictive than the equation in previous Specifications for sections with small values of M_p/M_y, such as sections with hybrid webs, a relatively small deck area and a high-strength tension flange. It is somewhat less restrictive for sections with large values

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$M_n \leq 1.3 R_h M_y \qquad (6.10.7.1.2\text{-}3)$$

where:

M_n = nominal flexural resistance determined from Eq. 6.10.7.1.2-1 or 6.10.7.1.2-2, as applicable (kip-in.)
M_y = yield moment determined as specified in Article D6.2 (kip-in.)
R_h = hybrid factor determined as specified in Article 6.10.1.10.1

unless:

- the span under consideration and all adjacent interior-pier sections satisfy the requirements of Article B6.2,

and:

- the appropriate value of θ_{RL} from Article B6.6.2 exceeds 0.009 radians at all adjacent interior-pier sections,

- in which case the nominal flexural resistance of the section is not subject to the limitation of Eq. 6.10.7.1.2-3.

of M_p/M_y. Wittry (1993) considered various experimental test results and performed a large number of parametric cross-section analyses. The smallest experimental or theoretical resistance of all the cross-sections considered in this research and in other subsequent studies is $0.96M_p$. Eq. 6.10.7.1.2-2 is based on the target additional margin of safety of 1.28 specified by Wittry at the maximum allowed value of D_p combined with an assumed theoretical resistance of $0.96M_p$ at this limit. At the maximum allowed value of D_p specified by Eq. 6.10.7.3-1, the resulting nominal design flexural resistance is $0.78M_p$.

The limit of $D_p < 0.1D_t$ for the use of Eq. 6.10.7.1.2-1 is obtained by use of a single implicit β value of 0.75 in the comparable equations from AASHTO (2004). AASHTO (2004) specifies $\beta = 0.7$ for $F_y = 50$ and 70.0 ksi and $\beta = 0.9$ for $F_y = 36.0$ ksi. The value of $\beta = 0.75$ is justifiable for all cases based on the scatter in strain-hardening data. The derived β values are sensitive to the assumed strain-hardening characteristics.

The shape factor, M_p/M_y, for composite sections in positive flexure can be somewhat larger than 1.5 in certain cases. Therefore, a considerable amount of yielding and resulting inelastic curvature is required to reach M_p in these situations. This yielding reduces the effective stiffness of the positive flexural section. In continuous spans, the reduction in stiffness can shift moment from the positive to the negative flexural regions. If the interior-pier sections in these regions do not have additional capacity to sustain these larger moments and are not designed to have ductile moment-rotation characteristics according to the provisions of Appendix B6, the shedding of moment to these sections could result in incremental collapse under repeated live load applications. Therefore, for cases where the span or either of the adjacent interior-pier sections do not satisfy the provisions of Article B6.2, or where the appropriate value of θ_{RL} from Article B6.6.2 at either adjacent pier section is less than or equal to 0.009 radians, the positive flexural sections must satisfy Eq. 6.10.7.1.2-3.

It is possible to satisfy the above concerns by ensuring that the pier section flexural resistances are not exceeded if the positive flexural section moments above R_hM_y are redistributed and combined with the concurrent negative moments at the pier sections determined from an elastic analysis. This approach is termed the Refined Method in AASHTO (2004). However, concurrent moments are not typically tracked in the analysis and so this method is not included in these Specifications.

Eq. 6.10.7.1.2-3 is provided to limit the amount of additional moment allowed above R_hM_y at composite sections in positive flexure to 30 percent of R_hM_y in continuous spans where the span or either of the adjacent pier sections do not satisfy the requirements of Article B6.2. The $1.3R_hM_y$ limit is the same as the limit specified for the Approximate Method in AASHTO (2004). The nominal flexural resistance determined from Eq. 6.10.7.1.2-3 is not to exceed the resistance determined from either Eq. 6.10.7.1.2-1 or 6.10.7.1.2-2, as applicable,

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

to ensure adequate strength and ductility of the composite section. In cases where D_p/D_t is relatively large and M_p/M_y is relatively small, Eq. 6.10.7.1.2-2 may govern relative to Eq. 6.10.7.1.2-3. However, for most practical cases, Eq. 6.10.7.1.2-3 will control.

Interior-pier sections satisfying the requirements of Article B6.2 and for which the appropriate value of θ_{RL} from Article B6.6.2 exceeds 0.009 radians have sufficient ductility and robustness such that the redistribution of moments caused by partial yielding within the positive flexural regions is inconsequential. The value of 0.009 radians is taken as an upper bound for the potential increase in the inelastic rotations at the interior-pier sections due to positive-moment yielding. Thus, the nominal flexural resistance of positive flexural sections in continuous spans that meet these requirements is not limited due to the effect of potential moment shifting. These restrictions are often satisfied by compact-flange unstiffened or transversely-stiffened pier sections designed by Article 6.10.8 or Appendix A6 using C_b = 1.0. All current ASTM A6 rolled I-shapes satisfying Eqs. B6.2.1-3, B6.2.2-1, and B6.2.4-1 meet these restrictions. All built-up sections satisfying Article B6.2 that also either have $D/b_{fc} < 3.14$ or satisfy the additional requirements of Article B6.5.1 meet these restrictions.

The Engineer is not required to redistribute moments from the pier sections in order to utilize the additional resistance in positive flexure, but only to satisfy the stated restrictions from Appendix B6 that ensure significant ductility and robustness of the adjacent pier sections. Redistribution of the pier moments is permitted in these cases, if desired, according to the provisions of Appendix B6.

Assuming the fatigue and fracture limit state does not control, under the load combinations specified in Table 3.4.1-1 and in the absence of flange lateral bending, the permanent deflection service limit state criterion given by Eq. 6.10.4.2.2-2 will often govern the design of the bottom flange of compact composite sections in positive flexure wherever the nominal flexural resistance at the strength limit state is based on either Eq. 6.10.7.1.2-1, 6.10.7.1.2-2, or 6.10.7.1.2-3. Thus, it is prudent and expedient to initially design these types of sections to satisfy this permanent deflection service limit state criterion and then to subsequently check the nominal flexural resistance at the strength limit state according to the applicable Eq. 6.10.7.1.2-1, 6.10.7.1.2-2, or 6.10.7.1.2-3.

6.10.7.2—Noncompact Sections

6.10.7.2.1—General

At the strength limit state, the compression flange shall satisfy:

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

where:

C6.10.7.2.1

For noncompact sections, the compression flange must satisfy Eq. 6.10.7.2.1-1 and the tension flange must satisfy Eq. 6.10.7.2.1-2 at the strength limit state. The basis for Eq. 6.10.7.2.1-2 is explained in Article C6.10.8.1.2. For composite sections in positive flexure, lateral bending does not need to be considered in the compression flange at the strength limit state because the flange is continuously

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)
F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.7.2.2 (ksi)

The tension flange shall satisfy:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nt} \quad (6.10.7.2.1\text{-}2)$$

where:

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
F_{nt} = nominal flexural resistance of the tension flange determined as specified in Article 6.10.7.2.2 (ksi)

The maximum longitudinal compressive stress in the concrete deck at the strength limit state, determined as specified in Article 6.10.1.1.1d, shall not exceed $0.6f'_c$.

supported by the concrete deck.

For noncompact sections, the longitudinal stress in the concrete deck is limited to $0.6f'_c$ to ensure linear behavior of the concrete, which is assumed in the calculation of the steel flange stresses. This condition is unlikely to govern except in cases involving: (1) shored construction, or unshored construction where the noncomposite steel dead load stresses are low, combined with (2) geometries causing the neutral axis of the short-term and long-term composite section to be significantly below the bottom of the concrete deck.

6.10.7.2.2—Nominal Flexural Resistance

The nominal flexural resistance of the compression flange shall be taken as:

$$F_{nc} = R_b R_h F_{yc} \quad (6.10.7.2.2\text{-}1)$$

where:

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2
R_h = hybrid factor determined as specified in Article 6.10.1.10.1

The nominal flexural resistance of the tension flange shall be taken as:

$$F_{nt} = R_h F_{yt} \quad (6.10.7.2.2\text{-}2)$$

C6.10.7.2.2

The nominal flexural resistance of noncompact composite sections in positive flexure is limited to the moment at first yield. Thus, the nominal flexural resistance is expressed simply in terms of the flange stress. For noncompact sections, the elastically computed stress in each flange due to the factored loads, determined in accordance with Article 6.10.1.1.1a, is compared with the yield stress of the flange times the appropriate flange-strength reduction factors.

6.10.7.3—Ductility Requirement

Compact and noncompact sections shall satisfy:

$$D_p \leq 0.42\, D_t \quad (6.10.7.3\text{-}1)$$

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.)

C6.10.7.3

The ductility requirement specified in this Article is intended to protect the concrete deck from premature crushing. The limit of $D_p < 5D'$ in AASHTO (2004) corresponds to $D_p/D_t < 0.5$ for $\beta = 0.75$. The D_p/D_t ratio is lowered to 0.42 in Eq. 6.10.7.3-1 to ensure significant yielding of the bottom flange when the crushing strain is reached at the top of concrete deck for all potential cases. In checking this requirement, D_t should be computed using a lower bound estimate of the actual thickness of the concrete haunch, or may be determined conservatively by neglecting the thickness of the haunch.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

and that the web heights at a given cross-section be kept equal. If the bridge is to incrementally launched, a constant depth box is recommended.

The provisions of Article 6.11 provide a unified approach for consideration of combined major-axis bending and flange lateral bending from any source in the design of top flanges of tub sections during construction. These provisions also provide a unified approach for consideration of the combined effects of normal stress and St. Venant torsional shear stress in closed-box and tub sections both during construction and in the final constructed condition. General design equations are provided for determining the nominal flexural resistance of box flanges under the combined effects of normal stress and torsional shear stress. The provisions also allow for the consideration of torsional shear in the design of the box-section webs and shear connectors. For straight boxes, the effects of torsional shear are typically relatively small unless the bridge is subjected to large torques. For example, boxes resting on skewed supports are usually subjected to large torques. For horizontally curved boxes, flange lateral bending effects due to curvature and the effects of torsional shear must always be considered at all limit states and also during construction.

For cases where the effects of the flange lateral bending and/or torsional shear are judged to be insignificant or incidental, or are not to be considered, the terms related to these effects are simply set equal to zero in the appropriate equations. The format of the equations then simply reduces to the format of the more familiar equations given in previous Specifications for checking the nominal flexural resistance of box sections in the absence of flange lateral bending and St. Venant torsion.

Fundamental calculations for flexural members previously found in Article 6.10.3 of AASHTO (2004) have been placed in Appendix D6.

6.11.1.1—Stress Determinations

Box flanges in multiple and single box sections shall be considered fully effective in resisting flexure if the width of the flange does not exceed one-fifth of the effective span. For simple spans, the effective span shall be taken as the span length. For continuous spans, the effective span shall be taken equal to the distance between points of permanent load contraflexure, or between a simple support and a point of permanent load contraflexure, as applicable. If the flange width exceeds one-fifth of the effective span, only a width equal to one-fifth of the effective span shall be considered effective in resisting flexure.

For multiple box sections in straight bridges satisfying the requirements of Article 6.11.2.3, the live-load flexural moment in each box may be determined in accordance with the applicable provisions of Article 4.6.2.2.2b. Shear due to St. Venant torsion and transverse bending and longitudinal warping stresses due to cross-section distortion may also be neglected for sections within these

C6.11.1.1

Stress analyses of actual box girder bridge designs were carried out to evaluate the effective width of a box flange using a series of folded plate equations (Goldberg and Leve, 1957). Bridges for which the span-to-flange width ratio varied from 5.65 to 35.3 were included in the study. The effective flange width as a ratio of the total flange width covered a range from 0.89 for the bridge with the smallest span-to-width ratio to 0.99 for the bridge with the largest span-to-width ratio. On this basis, it is reasonable to permit a box flange to be considered fully effective and subject to a uniform longitudinal stress, provided that its width does not exceed one-fifth of the span of the bridge. For extremely wide box flanges, a special investigation for shear lag effects may be required.

Although the results quoted above were obtained for simply-supported bridges, this criterion would apply equally to continuous bridges using the appropriate effective span defined in this Article for the section under consideration.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

bridges that have fully effective box flanges. The section of an exterior member assumed to resist horizontal factored wind loading within these bridges may be taken as the bottom box flange acting as a web and 12 times the thickness of the web acting as flanges.

The provisions of Article 4.6.2.2.2b shall not apply to:

- Single box sections in straight or horizontally curved bridges,
- Multiple box sections in straight bridges not satisfying the requirements of Article 6.11.2.3, or
- Multiple box sections in horizontally curved bridges.

For these sections, and for sections that do not have fully effective box flanges, the effects of both flexural and St. Venant torsional shear shall be considered. The St. Venant torsional shear stress in box flanges due to the factored loads at the strength limit state shall not exceed the factored torsional shear resistance of the flange, F_{vr}, taken as:

$$F_{vr} = 0.75\phi_v \frac{F_{yf}}{\sqrt{3}} \qquad (6.11.1.1\text{-}1)$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

In addition, transverse bending stresses due to cross-section distortion shall be considered for fatigue as specified in Article 6.11.5, and at the strength limit state. Transverse bending stresses due to the factored loads shall not exceed 20.0 ksi at the strength limit state. Longitudinal warping stresses due to cross-section distortion shall be considered for fatigue as specified in Article 6.11.5, but may be ignored at the strength limit state. Transverse bending and longitudinal warping stresses shall be determined by rational structural analysis in conjunction with the application of strength-of-materials principles. Transverse stiffeners attached to the webs or box flanges should be considered effective in resisting transverse bending.

The effective box-flange width should be used when calculating the flexural stresses in the section due to the factored loads. The full flange width should be used to calculate the nominal flexural resistance of the box flange.

Closed-box sections are capable of resisting torsion with limited distortion of the cross-section. Since distortion is generally limited by providing sufficient internal bracing in accordance with Article 6.7.4.3, torsion is resisted mainly by St. Venant torsional shear flow. The warping constant for closed-box sections is approximately equal to zero. Thus, warping shear and normal stresses due to warping torsion are typically quite small and are usually neglected.

Transverse bending stresses in box flanges and webs due to distortion of the box cross-section occur due to changes in direction of the shear flow vector. The transverse bending stiffness of the webs and flanges alone is not sufficient to retain the box shape so internal cross bracing is required. Longitudinal warping stresses due to cross-section distortion are also best controlled by internal cross bracing, as discussed further in Article C6.7.4.3.

Top flanges of tub girders subject to torsional loads need to be braced so that the section acts as a pseudo-box for noncomposite loads applied before the concrete deck hardens or is made composite. Top-flange bracing working with internal cross bracing retains the box shape and resists lateral force induced by inclined webs and torsion.

As discussed further in Article C6.11.2.3, for multiple box sections in straight bridges that conform to the restrictions specified in Article 6.11.2.3, the effects of St. Venant torsional shear and secondary distortional stresses may be neglected unless the box flange is very wide. The live-load distribution factor specified in Article 4.6.2.2.2b for straight multiple steel box sections may also be applied in the analysis of these bridges. Bridges not satisfying one or more of these restrictions must be investigated using one of the available methods of refined structural analysis, or other acceptable methods of approximate structural analysis as specified in Articles 4.4 or 4.6.2.2.4, since the specified live-load distribution factor does not apply to such bridges. The effects of St. Venant torsional shear and secondary distortional stresses are also more significant and must therefore be considered for sections in these bridges. Included in this category are all types of bridges containing single-box sections, and horizontally curved bridges containing multiple-box sections. Transverse bending stresses are of particular concern in boxes that may be subjected to large torques; e.g. single box sections, sharply curved boxes, and boxes resting on skewed supports. For other cases, the distortional stresses may be ignored if it can be demonstrated that the torques are of comparable magnitude to the torques for cases in which research has shown that these stresses are small enough to be neglected (Johnston and Mattock, 1967), e.g., a straight bridge of similar proportion satisfying the requirements of Article 6.11.2.3 or if the torques are deemed small enough in the judgment of the Owner and the Engineer. In such

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

The applicability of the optional provisions of Appendix B6 to box sections has not been demonstrated. Therefore, these provisions may not be used in the design of box sections.

6.11.5—Fatigue and Fracture Limit State

Except as specified herein, the provisions of Article 6.10.5 shall apply.

For fatigue in shear connectors, the provisions of Article 6.11.10 shall also apply, as applicable. The provisions for fatigue in shear connectors specified in Article 6.10.10.3 shall not apply.

When checking the shear requirement specified in Article 6.10.5.3, the provisions of Article 6.11.9 shall also apply, as applicable.

Longitudinal warping stresses and transverse bending stresses due to cross-section distortion shall be considered for:

- Single box sections in straight or horizontally curved bridges,
- Multiple box sections in straight bridges not satisfying the requirements of Article 6.11.2.3,
- Multiple box sections in horizontally curved bridges, or
- Any single or multiple box section with a box flange that is not fully effective according to the provisions of Article 6.11.1.1.

The stress range due to longitudinal warping shall be considered in checking the fatigue resistance of the base metal at all details on the box section according to the provisions specified in Article 6.6.1. The transverse bending stress range shall be considered separately in evaluating the fatigue resistance of the base metal adjacent to flange-to-web fillet welds and adjacent to the termination of fillet welds connecting transverse elements to webs and box flanges. In determining the longitudinal warping and transverse bending stress ranges, one cycle of stress shall be defined as 75 percent of the stress range determined by the passage of the factored fatigue load in two different critical transverse positions. In no case shall the stress range calculated in this manner be less than the calculated stress range due to the passage of the factored fatigue load in only one lane. The need for a bottom transverse member within the internal cross-frames to resist the transverse bending stress range in the bottom box flange at the termination of fillet welds connecting cross-frame connection plates to the flange shall be investigated. Transverse cross-frame members next to box flanges shall be attached to the box flange unless longitudinal flange stiffeners are used, in which case the transverse members shall be attached to the longitudinal stiffeners by bolting. The moment of inertia of these transverse cross-frame members shall not be less than the moment of inertia of the largest connection plate

C6.11.5

When a box section is subjected to a torsional load, its cross-section distorts and is restored at diaphragms or cross-frames. This distortion gives rise to secondary bending stresses. A torsional loading in the opposite direction produces a reversal of these distortional secondary bending stresses. In certain cases, as defined herein, these distortional stresses are to be considered when checking fatigue. Situations for which these stresses are of particular concern and for which these stresses may potentially be ignored are discussed in Article C6.11.1.1.

Transverse bending and longitudinal warping stress ranges due to cross-section distortion can be determined using the BEF analogy, as discussed in Article C6.11.1.1. Longitudinal warping stresses are considered additive to the longitudinal major-axis bending stresses.

The largest distortional stress range is usually caused by positioning the live load on one side and then on the opposite side of a box. To cause one cycle of the stress range so computed requires two vehicles to traverse the bridge in separate transverse positions, with one vehicle leading the other. To account for the unlikely event of this occurring over millions of cycles, the provisions permit application of a factor of 0.75 to the computed range of distortionally-induced stresses, except when the maximum stress range is caused by loading of only one lane. This 0.75 factor is distinct from the load factor specified for the applicable fatigue load combination in Table 3.4.1-1, i.e., when applicable, both factors may be applied simultaneously. There is no provision to account for the need for two trucks to cause a single cycle of stress in this case. For cases where the nominal fatigue resistance is calculated based on a finite life, the Engineer may wish to consider a reduction in the number of cycles since two cycles are required to cause a single cycle of stress.

The most critical case for transverse bending is likely to be the base metal at the termination of fillet welds connecting transverse elements to webs and box flanges. A stress concentration occurs at the termination of these welds as a result of the transverse bending. The fatigue resistance of this detail when subject to transverse bending is not currently quantified, but is anticipated to be perhaps as low as Category E.

Should this situation be found critical in the web at transverse web stiffeners not serving as connection plates, the transverse bending stress range may be reduced by welding the stiffeners to the flanges. Attaching transverse stiffeners to the flanges reduces the sharp through-thickness bending stresses within the unstiffened portions of the web at the termination of the stiffener-to-web welds, which is usually the most critical region for this check. Cross-frame connection plates already are required to be

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

for the internal cross-frame under consideration taken about the edge in contact with the web.

For single box sections, box flanges in tension shall be considered fracture-critical, unless analysis shows that the section can support the full dead and an appropriate portion of the live load after sustaining a hypothetical complete fracture of the flange and webs at any point.

Unless adequate strength and stability of a damaged structure can be verified by refined analysis, in cross-sections comprised of two box sections, only the bottom flanges in the positive moment regions should be designated as fracture-critical. Where cross-sections contain more than two box girder sections, none of the components of the box sections should be considered fracture-critical.

attached to the flanges according to the provisions of Article 6.6.1.3.1 for this reason.

Should it become necessary to reduce the transverse bending stress range in the box flange adjacent to the cross-frame connection plate welds to the flange, the provision of transverse cross-frame members across the bottom of the box or tub as part of the internal cross-bracing significantly reduces the transverse bending stress range at the welds and ensures that the cross-section shape is retained. Closer spacing of cross-frames also leads to lower transverse bending stresses. Where bottom transverse cross-frame members are provided, they are to be attached to the box flange or to the longitudinal flange stiffeners, as applicable. For closed-box sections, the top transverse cross-frame members should be similarly attached. Where transverse bracing members are welded directly to the box flange, the stress range due to transverse bending should also be considered in checking the fatigue resistance of the base metal adjacent to the termination of these welds. Where transverse bracing members are connected to longitudinal flange stiffeners, the box flange may be considered stiffened when computing the transverse bending stresses. In such cases, the transverse connection plates must still be attached to both flanges as specified in Article 6.6.1.3.1.

Load-induced fatigue is usually not critical for top lateral bracing in tub sections since the concrete deck is much stiffer and resists more of the load than does the bracing. Since the deck resists the majority of the torsional shear in these cases, it is advisable to check the reinforcement in the deck for the additional horizontal shear. Severely skewed supports may cause critical horizontal deck shear.

It is advisable to connect the lateral bracing to the top flanges to eliminate a load path through the web. Although removable deck forms are problematic in tub girders, they are sometimes required by the Owner. In such cases, it may be necessary to lower the lateral bracing by attaching it to the box webs. In these cases, connections to the webs must be made according to the requirements of Article 6.6.1.3.2 to prevent potential problems resulting from fatigue. An adequate load path, with fatigue considered, must be provided between the bracing-to-web connections and the top flanges. Connections of the lateral bracing to the web can be avoided by using metal stay-in-place deck forms.

Fatigue of the base metal at the net section of access holes should be considered. The fatigue resistance at the net section of large access holes is not currently specified; however, base metal at the net section of open bolt holes has been shown to satisfy Category D (Brown et al., 2007). This assumes a stress concentration, or ratio of the elastic tensile stress adjacent to the hole to the average stress on the net area, of 3.0. A less severe fatigue category might be considered if the proper stress concentration at the edges of the access hole is evaluated.

Refer to Article C6.6.2 for further discussion regarding the use of refined analyses to demonstrate that part of a structure is not fracture-critical.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$\Delta = \sqrt{1 - 3\left(\frac{f_v}{F_{yt}}\right)^2} \qquad (6.11.7.2.2\text{-}6)$$

f_v = St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi)

$$= \frac{T}{2A_o t_{ft}} \qquad (6.11.7.2.2\text{-}7)$$

6.11.8—Flexural Resistance—Sections in Negative Flexure

6.11.8.1—General

6.11.8.1.1—Box Flanges in Compression

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

$$f_{bu} \leq \phi_f F_{nc} \qquad (6.11.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_{bu} = longitudinal flange stress due to the factored loads at the section under consideration calculated without consideration of longitudinal warping (ksi)

F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.11.8.2 (ksi)

C6.11.8.1.1

Eq. 6.11.8.1.1-1 ensures that box flanges in compression have sufficient strength with respect to flange local buckling. Flange lateral bending and lateral torsional buckling are not a consideration for box flanges.

In general, bottom box flanges at interior-pier sections are subjected to biaxial stresses due to major-axis bending of the box section and major-axis bending of the internal diaphragm over the bearing sole plate. The flange is also subject to shear stresses due to the internal diaphragm vertical shear, and in cases where it must be considered, the St. Venant torsional shear. Bending of the internal diaphragm over the bearing sole plate can be particularly significant for boxes supported on single bearings. For cases where the shear stresses and/or bending of the internal diaphragm are deemed significant, the following equation may be used to check this combined stress state in the box flange at the strength limit state:

$$\sqrt{f_{bu}^2 - f_{bu}f_{by} + f_{by}^2 + 3\left(f_d + f_v\right)^2} \leq \phi_f R_b R_h F_{yc} \qquad (C6.11.8.1.1\text{-}1)$$

where:

f_{by} = stress in the flange due to the factored loads caused by major-axis bending of the internal diaphragm over the bearing sole plate (ksi)

f_d = shear stress in the flange caused by the internal diaphragm vertical shear due to the factored loads (ksi)

f_v = St. Venant torsional shear stress in the flange due to the factored loads (ksi)

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

Eq. C6.11.8.1.1-1 represents the general form of the Huber-von Mises-Hencky yield criterion (Ugural and Fenster, 1975).

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

For a box supported on two bearings, f_{by} in Eq. C6.11.8.1.1-1 is typically relatively small and can often be neglected.

The box flange may be considered effective with the internal diaphragm at interior-pier sections in making this check. A flange width equal to six times its thickness may be considered effective with the internal diaphragm. The shear stress in the flange, f_d, caused by the internal diaphragm vertical shear due to the factored loads can then be estimated as:

$$f_d = \frac{VQ}{It_{fc}} \quad \text{(C6.11.8.1.1-2)}$$

where:

V = vertical shear in the internal diaphragm due to flexure plus St. Venant torsion (kip)

Q = first moment of one-half the effective box-flange area about the neutral axis of the effective internal diaphragm section (in.^{3})

I = moment of inertia of the effective internal diaphragm section (in.^{4})

Wherever an access hole is provided within the internal diaphragm, the effect of the hole should be considered in computing the section properties of the effective diaphragm section.

6.11.8.1.2—Continuously Braced Flanges in Tension

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

$$f_{bu} \leq \phi_f F_{nt} \quad \text{(6.11.8.1.2-1)}$$

where:

F_{nt} = nominal flexural resistance of the flange determined as specified in Article 6.11.8.3 (ksi)

C6.11.8.1.2

For continuously braced top flanges of tub sections, lateral flange bending need not be considered. St. Venant torsional shears are also typically neglected. The torsional shears may not be neglected, however, in a continuously braced box flange.

6.11.8.2—Flexural Resistance of Box Flanges in Compression

6.11.8.2.1—General

The nominal flexural resistance of box flanges in compression without flange longitudinal stiffeners shall be determined as specified in Article 6.11.8.2.2. The nominal flexural resistance of box flanges in compression with flange longitudinal stiffeners shall be determined as specified in Article 6.11.8.2.3.

6.11.8.2.2—Unstiffened Flanges

The nominal flexural resistance of the compression flange, F_{nc}, shall be taken as:

C6.11.8.2.2

For unstiffened flanges, the slenderness is based on the full flange width between webs, b_{fc}.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$F_{nc} = F_{cb}\sqrt{1-\left(\frac{f_v}{\phi_v F_{cv}}\right)^2} \quad (6.11.8.2.2\text{-}1)$$

in which:

F_{cb} = nominal axial compression buckling resistance of the flange under compression alone calculated as follows:

- If $\lambda_f \le \lambda_p$, then:

$$F_{cb} = R_b R_h F_{yc}\Delta \quad (6.11.8.2.2\text{-}2)$$

- If $\lambda_p < \lambda_f \le \lambda_r$, then:

$$F_{cb} = R_b R_h F_{yc}\left[\Delta - \left(\Delta - \frac{\Delta - 0.3}{R_h}\right)\left(\frac{\lambda_f - \lambda_p}{\lambda_r - \lambda_p}\right)\right] \quad (6.11.8.2.2\text{-}3)$$

- If $\lambda_f > \lambda_r$, then:

$$F_{cb} = \frac{0.9 E R_b k}{\lambda_f^2} \quad (6.11.8.2.2\text{-}4)$$

F_{cv} = nominal shear buckling resistance of the flange under shear alone calculated as follows:

- If $\lambda_f \le 1.12\sqrt{\frac{Ek_s}{F_{yc}}}$, then:

$$F_{cv} = 0.58 F_{yc} \quad (6.11.8.2.2\text{-}5)$$

- If $1.12\sqrt{\frac{Ek_s}{F_{yc}}} < \lambda_f \le 1.40\sqrt{\frac{Ek_s}{F_{yc}}}$, then:

$$F_{cv} = \frac{0.65\sqrt{F_{yc} E k_s}}{\lambda_f} \quad (6.11.8.2.2\text{-}6)$$

- If $\lambda_f > 1.40\sqrt{\frac{Ek_s}{F_{yc}}}$, then:

$$F_{cv} = \frac{0.9 E k_s}{\lambda_f^2} \quad (6.11.8.2.2\text{-}7)$$

λ_f = slenderness ratio for the compression flange

$$= \frac{b_{fc}}{t_{fc}} \quad (6.11.8.2.2\text{-}8)$$

$$\lambda_p = 0.57\sqrt{\frac{Ek}{F_{yc}\Delta}} \quad (6.11.8.2.2\text{-}9)$$

For flanges under combined normal stress and torsional shear stress, the following nonlinear interaction curve is used to derive the resistance of the flange (NHI, 2011):

$$\left(\frac{f_v}{\phi_v F_{cv}}\right)^2 + \left(\frac{f_c}{\phi_f F_{cb}}\right)^2 \le 1.0 \quad (C6.11.8.2.2\text{-}1)$$

Rearranging Eq. C6.11.8.2.2-1 in terms of f_c and substituting F_{nc} for f_c facilitates the definition of the nominal flexural resistance of the compression flange as provided in Eq. 6.11.8.2.2-1. A general discussion of the problem of reduction of critical local buckling stresses due to the presence of torsional shear may be found in Galambos (1998).

The nominal axial compression buckling resistance of the flange under compression alone, F_{cb}, is defined for three distinct regions based on the slenderness of the flange. The elastic buckling resistance of the flange given by Eq. 6.11.8.2.2-4 is based on the theoretical elastic Euler buckling equation for an infinitely long plate under a uniform normal normal stress (Timoshenko and Gere, 1961). For stocky plates, full yielding of the plate as defined by the von Mises yield criterion for combined normal and shear stress (Boresi et al., 1978) can be achieved. For such plates, F_{cb} is defined by Eq. 6.11.8.2.2-2. In between these two regions is a transition region that reflects the fact that partial yielding due to residual stresses and initial imperfections does not permit the attainment of the elastic buckling stress. The nominal flexural resistance of the flange in this region is expressed in Eq. 6.11.8.2.2-3 as a linear function of the flange slenderness. A residual stress level equal to $0.3F_{yc}$ is assumed in the presence of no shear.

The limiting flange slenderness, λ_p, defining whether to use Eq. 6.11.8.2.2-2 or 6.11.8.2.2-3 is defined as 0.6 times the flange slenderness at which the elastic buckling stress given by Eq. 6.11.8.2.2-4 equals $R_b F_{yc}\Delta$. The limiting flange slenderness, λ_r, defining whether to use Eq. 6.11.8.2.2-3 or 6.11.8.2.2-4 is defined as the flange slenderness at which the elastic buckling stress given by Eq. 6.11.8.2.2-4 equals $R_b F_{yr}$, where F_{yr} is given by Eq. 6.11.8.2.2-13.

The equations for the nominal shear buckling resistance of the flange under shear alone, F_{cv}, are determined from the equations for the constant, C, given in Article 6.10.9.3.2, where C is the ratio of the shear buckling resistance to the shear yield strength of the flange taken as $F_{yc}/\sqrt{3}$.

The computation of the flange torsional shear stress, f_v, from Eq. 6.11.8.2.2-12 due to torques applied separately to the noncomposite and composite sections is discussed in Article C6.11.1.1. In cases where f_v is relatively small, consideration might be given to assuming Δ equal to 1.0 and F_{nc} equal to F_{cb} for preliminary design.

The specified plate-buckling coefficient for uniform normal stress, k, and shear-buckling coefficient, k_s, assume simply-supported boundary conditions at the edges of the flanges (Timoshenko and Gere, 1961).

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$\lambda_r = 0.95\sqrt{\frac{Ek}{F_{yr}}} \quad (6.11.8.2.2\text{-}10)$$

$$\Delta = \sqrt{1-3\left(\frac{f_v}{F_{yc}}\right)^2} \quad (6.11.8.2.2\text{-}11)$$

f_v = St. Venant torsional shear stress in the flange due to the factored loads at the section under consideration (ksi)

$$= \frac{T}{2A_o t_{fc}} \quad (6.11.8.2.2\text{-}12)$$

F_{yr} = smaller of the compression-flange stress at the onset of nominal yielding, with consideration of residual stress effects, or the specified minimum yield strength of the web (ksi)

$$= (\Delta - 0.3)F_{yc} \quad (6.11.8.2.2\text{-}13)$$

k = plate-buckling coefficient for uniform normal stress
= 4.0

k_s = plate-buckling coefficient for shear stress
= 5.34

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

b_{fc} = compression-flange width between webs (in.)

A_o = enclosed area within the box section (in.^{2})

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

T = internal torque due to the factored loads (kip-in.)

The term R_b is a postbuckling strength reduction factor that accounts for the reduction in the section flexural resistance caused by the shedding of compressive stresses from a slender web and the corresponding increase in the flexural stress within the compression flange. The R_h factor accounts for the reduced contribution of the web to the nominal flexural resistance at first yield in any flange element, due to earlier yielding of the lower strength steel in the web of a hybrid section. The R_b and R_h factors are discussed in greater detail in Articles C6.10.1.10.2 and C6.10.1.10.1, respectively. In calculating R_b and R_h for a tub section, use one-half of the effective box flange width in conjunction with one top flange and a single web, where the effective box flange width is defined in Article 6.11.1.1. For a closed-box section, use one-half of the effective top and bottom box flange width in conjunction with a single web.

6.11.8.2.3—Longitudinally Stiffened Flanges

The nominal flexural resistance of the compression flange shall be taken as equal to the nominal flexural resistance for the compression flange without longitudinal stiffeners, determined as specified in Article 6.11.8.2.2, with the following substitutions:

- w shall be substituted for b_{fc},
- The plate-buckling coefficient for uniform normal stress, k, shall be taken as:
- If $n = 1$, then:

C6.11.8.2.3

When a noncomposite unstiffened box flange becomes so slender that nominal flexural resistance of the flange decreases to an impractical level, longitudinal stiffeners can be added to the flange.

The nominal flexural resistance of a longitudinally-stiffened box flange is determined using the same basic equations specified for unstiffened box flanges in Article 6.11.8.2.2. The width, w, must be substituted for b_{fc} in the equations. The shear-buckling coefficient, k_s, for a stiffened plate to be used in the equations is given by Eq. 6.11.8.2.3-3, which comes from Culver (1972). The

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

where:

A_g = gross area of the section based on the design wall thickness (in.)
D = outside diameter of the tube (in.)
L_v = distance between points of maximum and zero shear (in.)
t = design wall thickness taken equal to 0.93 times the nominal wall thickness for electric-resistance-welded round HSS and taken equal to the nominal wall thickness for all others (in.)

6.12.2—Nominal Flexural Resistance

6.12.2.1—General

Except as specified herein, provisions for lateral torsional buckling need not be applied to composite members, noncomposite box-shaped members, noncomposite I- and H-shaped members bent about their weak axis, and circular tubes.

6.12.2.2—Noncomposite Members

6.12.2.2.1—I- and H-Shaped Members

The provisions of this Article apply to I- and H-shaped members and members consisting of two channel flanges connected by a web plate.

The provisions of Article 6.10 shall apply to flexure about an axis perpendicular to the web.

The nominal flexural resistance for flexure about the weak axis shall be taken as:

- If $\lambda_f \leq \lambda_{pf}$, then:

$$M_n = M_p \tag{6.12.2.2.1-1}$$

- If $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$, then:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_{yf}}}}\right)\right] F_{yf} Z_y \tag{6.12.2.2.1-2}$$

in which:

λ_f = slenderness ratio for the flange

$$= \frac{b_f}{2t_f} \tag{6.12.2.2.1-3}$$

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

C6.12.2.2.1

Eqs. 6.12.2.2.1-1 and 6.12.2.2.1-2 are taken from Appendix F of AISC (1999), except that the flange slenderness λ_{rf} corresponding to the transition from inelastic to elastic flange local buckling is consistently set based on the yield moment in weak-axis bending $F_{yf}S_y$. AISC (1999) uses $F_{yf}S_y$ as the moment corresponding to the inelastic-to-elastic flange local buckling transition, but then specifies λ_{rf} based on a smaller moment level. The approach adopted in these provisions is interpreted as a corrected form of the AISC (1999) equations and is conservative relative to the AISC (1999) equations as printed. The yield moment $F_{yf}S_y$ may be taken conservatively as the moment at the inelastic-to-elastic flange local buckling transition because of the beneficial effects of the stress gradient in the flange associated with weak-axis bending.

For H-shaped members $M_p = 1.5F_yS$, where S is the elastic section modulus about this axis.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$= 0.38\sqrt{\frac{E}{F_{yf}}} \qquad (6.12.2.2.1\text{-}4)$$

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

$$= 0.83\sqrt{\frac{E}{F_{yf}}} \qquad (6.12.2.2.1\text{-}5)$$

where:

F_{yf} = specified minimum yield strength of the lower-strength flange (ksi)
M_p = plastic moment about the weak axis (kip-in.)
S_y = elastic section modulus about the weak axis (in.^{3})
Z_y = plastic section modulus about the weak axis (in.^{3})

6.12.2.2.2—Box-Shaped Members

Except as specified herein, for homogeneous doubly symmetric box-shaped members bent about either axis, the nominal flexural resistance shall be taken as:

$$M_n = F_y S\left[1 - \frac{0.064\, F_y S \ell}{AE}\left(\frac{\Sigma\left(\frac{b}{t}\right)}{I_y}\right)^{0.5}\right] \qquad (6.12.2.2.2\text{-}1)$$

where:

S = section modulus about the flexural axis (in.^{3})
A = area enclosed within the centerlines of the plates comprising the box (in.^{2})
ℓ = unbraced length (in.)
I_y = moment of inertia about an axis perpendicular to the axis of bending (in.^{4})
b/t = width of any flange or depth of any web component divided by its thickness neglecting any portions of flanges or webs that overhang the box perimeter

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable.

For yielding, the nominal flexural resistance for square and rectangular HSS shall be taken as:

$$M_n = M_p = F_y Z \qquad (6.12.2.2.2\text{-}2)$$

where:

M_p = plastic moment (kip-in.)
Z = plastic section modulus about the axis of bending (in.^{3})

C6.12.2.2.2

The lateral-torsional resistance of box shapes is usually quite high and its effect is often ignored. For truss members, frame members, arch ribs, and other situations in which long unbraced lengths are possible, this expediency may not be adequate. Eq. 6.12.2.2.2-1 was derived from the elastic lateral torsional buckling moment, M_{CR}, given by:

$$M_{CR} = \frac{\pi}{\ell}\sqrt{EI_y GJ} \qquad (C6.12.2.2.2\text{-}1)$$

in which:

$$G = 0.385E\text{, and} \qquad (C6.12.2.2.2\text{-}2)$$

$$J = \frac{4A^2}{\Sigma \frac{b}{t}} \qquad (C6.12.2.2.2\text{-}3)$$

After substitution of Eqs. C6.12.2.2.2-2 and C6.12.2.2.2-3 into C6.12.2.2.2-1:

$$M_{CR} = \frac{3.90\, EA}{\ell}\sqrt{\frac{I_y}{\Sigma \frac{b}{t}}} \qquad (C6.12.2.2.2\text{-}4)$$

It was assumed that buckling would be in the inelastic range so the CRC column equation was used to estimate the effect of inelastic buckling as:

$$M_I = F_y S\left[1 - \frac{F_y S}{4M_{CR}}\right] \qquad (C6.12.2.2.2\text{-}5)$$

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in.)

6.13.2.10—Tensile Resistance

6.13.2.10.1—General

High-strength bolts subjected to axial tension shall be tensioned to the force specified in Table 6.13.2.8-1. The applied tensile force shall be taken as the force due to the external factored loadings, plus any tension resulting from prying action produced by deformation of the connected parts, as specified in Article 6.13.2.10.4.

6.13.2.10.2—Nominal Tensile Resistance

The nominal tensile resistance of a bolt, T_n, independent of any initial tightening force shall be taken as:

$$T_n = 0.76 A_b F_{ub} \quad (6.13.2.10.2\text{-}1)$$

where:

A_b = area of bolt corresponding to the nominal diameter (in.^{2})

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

C6.13.2.10.2

The recommended design strength is approximately equal to the initial tightening force; thus, when loaded to the service load, high-strength bolts will experience little, if any, actual change in stress. For this reason, bolts in connections, in which the applied loads subject the bolts to axial tension, are required to be fully tensioned.

6.13.2.10.3—Fatigue Resistance

Where high-strength bolts in axial tension are subject to fatigue, the stress range, Δf, in the bolt, due to the fatigue design live load, plus the dynamic load allowance for fatigue loading specified in Article 3.6.1.4, plus the prying force resulting from cyclic application of the fatigue load, shall satisfy Eq. 6.6.1.2.2-1.

The nominal diameter of the bolt shall be used in calculating the bolt stress range. In no case shall the calculated prying force exceed 30 percent of the externally applied load.

Low carbon ASTM A307 bolts shall not be used in connections subjected to fatigue.

C6.13.2.10.3

Properly tightened A325 and A490 bolts are not adversely affected by repeated application of the recommended service load tensile stress, provided that the fitting material is sufficiently stiff that the prying force is a relatively small part of the applied tension. The provisions covering bolt tensile fatigue are based upon study of test reports of bolts that were subjected to repeated tensile load to failure (Kulak et al., 1987).

6.13.2.10.4—Prying Action

The tensile force due to prying action shall be taken as:

$$Q_u = \left[\frac{3b}{8a} - \frac{t^3}{20}\right] P_u \quad (6.13.2.10.4\text{-}1)$$

where:

Q_u = prying tension per bolt due to the factored loadings taken as 0 when negative (kip)

C6.13.2.10.4

Eq. 6.13.2.10.4-1 for estimating the magnitude of the force due to prying is a simplification given in ASCE (1971) of a semiempirical expression (Douty and McGuire, 1965). This simplified formula tends to overestimate the prying force and provides conservative design results (Nair et al., 1974).

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

P_u = direct tension per bolt due to the factored loadings (kip)
a = distance from center of bolt to edge of plate (in.)
b = distance from center of bolt to the toe of fillet of connected part (in.)
t = thickness of thinnest connected part (in.)

6.13.2.11—Combined Tension and Shear

The nominal tensile resistance of a bolt subjected to combined shear and axial tension, T_n, shall be taken as:

- If $\frac{P_u}{R_n} \leq 0.33$, then:

$$T_n = 0.76\, A_b F_{ub} \tag{6.13.2.11-1}$$

- Otherwise:

$$T_n = 0.76 A_b F_{ub} \sqrt{1 - \left(\frac{P_u}{\phi_s R_n}\right)^2} \tag{6.13.2.11-2}$$

where:

A_b = area of the bolt corresponding to the nominal diameter (in.^{2})
F_{ub} = specified minimum tensile strength of the bolt specified in Article 6.4.3 (ksi)
P_u = shear force on the bolt due to the factored loads (kip)
R_n = nominal shear resistance of a bolt specified in Article 6.13.2.7 (kip)

The nominal resistance of a bolt in slip-critical connections under Load Combination Service II, specified in Table 3.4.1-1, subjected to combined shear and axial tension, shall not exceed the nominal slip resistance specified in Article 6.13.2.8 multiplied by:

$$1 - \frac{T_u}{P_t} \tag{6.13.2.11-3}$$

where:

T_u = tensile force due to the factored loads under Load Combination Service II (kip)
P_t = minimum required bolt tension specified in Table 6.13.2.8-1 (kip)

C6.13.2.11

The nominal tensile resistance of bolts subject to combined axial tension and shear is provided by elliptical interaction curves, which account for the connection length effect on bolts loaded in shear, the ratio of shear strength to tension strength of threaded bolts, and the ratios of root area to nominal body area and tensile stress area to nominal body area (Chesson et al., 1965). Eqs. 6.13.2.11-1 and 6.13.2.11-2 are conservative simplifications of the set of elliptical curves. The equations representing the set of elliptical curves for various cases may be found in AISC (1988). No reduction in the nominal tensile resistance is required when the applied shear force on the bolt due to the factored loads is less than or equal to 33 percent of the nominal shear resistance of the bolt.

6.13.2.12—Shear Resistance of Anchor Bolts

The nominal shear resistance of an ASTM F1554 or an ASTM A307 Grade C anchor bolt at the strength limit state shall be taken as:

- Where threads are included in the shear plane:

C6.13.2.12

Conditions typically exist in connections with anchor bolts such that the full resistance of each bolt is probably not entirely utilized when resisting applied shear forces. Oversize holes and other factors tend to cause nonuniformity in anchor bolt stresses and thus, connection

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Duan, L., M. Reno, and L. Lynch. 2000. "Section Properties for Latticed Members of San Francisco-Oakland Bay Bridges," *Journal of Bridge Engineering*. American Society of Civil Engineering, Reston, VA, Vol. 4, No. 2, May, pp. 156–164.

Duan, L., M. Reno, and C. M. Uang. 2002. "Effect of Compound Buckling on Compression Strength of Built-Up Members," *AISC Engineering Journal*. American Institute of Steel Construction, Chicago, IL, Vol. 39, No. 1, 1st Qtr., 2002, pp. 30–37.

Dubas, C. 1948. *A Contribution to the Buckling of Stiffened Plates*. IABSE Preliminary Publication. Third Congress, International Association for Bridge and Structural Engineers, Zurich, Switzerland.

El Darwish, I. A., and B. G. Johnston. 1965. "Torsion of Structural Shapes," *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 91, No. ST1, February.

Elgaaly, M., and R. Salkar. 1991. "Web Crippling Under Edge Loading," *Proceedings of AISC National Steel Construction Conference*. Washington, DC, pp. 7-1–7-21.

Ellifritt, D. S., G. Wine, T. Sputo, and S. Samuel. 1992. "Flexural Strength of WT Sections," *Engineering Journal*. American Institute of Steel Construction, Chicago, IL, Vol. 29, No. 2, 2nd Qtr.

El-Tayem, A., and S. C. Goel. 1986. "Effective Length Factor for the Design of X-Bracing Systems," *AISC Engineering Journal*. American Institute of Steel Construction, Chicago, IL, Vol. 23, No. 1, 1st Qtr.

Fan Z., and T. A. Helwig. 1999. "Behavior of Steel Box Girders with Top Flange Bracing," *Journal of Structural Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 125, No. 8, August, pp. 829–837.

FHWA. 1980. "Proposed Design Specifications for Steel Box Girder Bridges." *FHWA-TS-80-205*. Federal Highway Administration, Washington, DC.

FHWA. 1989. *Technical Advisory on Uncoated Weathering Steel in Structures*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, October.

FHWA. 2012. *Manual for Design, Construction and Maintenance of Orthotropic Steel Deck Bridges*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Fisher, J. W., J. Jin, D. C. Wagner, and B. T. Yen. 1990. *Distortion-Induced Fatigue Cracking in Steel Bridges*, NCHRP Report 336. Transportation Research Board, National Research Council, Washington, DC, December.

Foehl, P. J. 1948. "Direct Method of Designing Single Angle Struts in Welded Trusses," *Design Book for Welding*. Lincoln Electric Company, Cleveland, OH.

Frank, K. H., and J. W. Fisher. 1979. "Fatigue Strength of Fillet Welded Cruciform Joints," *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 105, No. ST9, September, pp. 1727–1740.

Frank, K. H., and T. A. Helwig. 1995, "Buckling of Webs in Unsymmetric Plate Girders," *AISC Engineering Journal*. American Institute of Steel Construction, Chicago, IL, Vol. 32, 2nd Qtr., pp. 43–53.

Frank, K. H., and J. A. Yura. 1981. *An Experimental Study of Bolted Shear Connections*, FHWA/RD-81/148. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, December.

Frost, R. W., and C. G. Schilling. 1964. "Behavior of Hybrid Beams Subjected to Static Loads," *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 90, No. ST3, June, pp. 55–88.

Galambos, T. V., ed. 1998. *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition. Structural Stability Research Council, John Wiley and Sons, Inc., New York, NY.

Galambos, T. V. 2006. "Reliability of the Member Stability Criteria in the 2005 AISC Specification," *Engineering Journal*. AISC, Fourth Quarter, 257–265.

Galambos, T. V., and J. Chapuis. 1980. *LRFD Criteria for Composite Columns and Beam Columns*. Revised draft. Washington University Department of Civil Engineering, St. Louis, MO, December.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Galambos, T. V., R. T. Leon, C. E. French, M. G. Barker, and B. E. Dishongh. 1993. *Inelastic Rating Procedures for Steel Beam and Girder Bridges*, NCHRP Report 352, Transportation Research Board, National Research Council, Washington, DC.

Gaylord, E.H. 1963. "Discussion of K. Basler 'Strength of Plate Girders in Shear," *Transaction ASCE*. American Society of Civil Engineers, New York, NY, Vol. 128 No. II, pp. 712–719.

Goldberg, J. E., and H. L. Leve. 1957. "Theory of Prismatic Folded Plate Structures," *IABSE*. International Association for Bridge and Structural Engineers, Zurich, Switzerland, Vol. 16, pp. 59–86.

Graham, J. D., A. N. Sherbourne, R. N. Khabbaz, and C. D. Jensen. 1959. *Welded Interior Beam-to-Column Connections*. American Institute of Steel Construction, New York, NY.

Grubb, M. A. 1993. "Review of Alternate Load Factor (Autostress) Design," *Transportation Research Record 1380*. Transportation Research Board, National Research Council, Washington, DC, pp. 45–48.

Grubb, M. A., and R. E. Schmidt. 2004. *Three-Span Continuous Straight Composite I Girder, Load and Resistance Factor Design*, Third Edition—Customary U.S. Units. Available from the National Steel Bridge Alliance, Chicago, IL, pp. C1–C5.

Haaijer, G. 1981. "Simple Modeling Technique for Distortion Analysis for Steel Box Girders." *Proceedings of the MSC/NASTRAN Conference on Finite Element Methods and Technology*. MacNeal/Schwendler Corporation, Los Angeles, CA.

Haaijer, G., P. S. Carskaddan, and M. A. Grubb. 1987. "Suggested Autostress Procedures for Load Factor Design of Steel Beam Bridges," *AISI Bulletin*. American Iron and Steel Institute, Washington, DC, No. 29, April.

Hall, D. H., and C. H. Yoo. 1996. *I-Girder Curvature Study*. Interim Report, NCHRP Project 12-38. Submitted to NCHRP, Transportation Research Board, Washington, DC, pp. 1–72 (or see Appendix A of NCHRP Report 424, *Improved Design Specifications for Horizontally Curved Steel Highway Bridges*, pp. 49–74).

Hanshin Expressway Public Corporation and Steel Structure Study Subcommittee. 1988. *Guidelines for the Design of Horizontally Curved Girder Bridges (Draft)*. Hanshin Expressway Public Corporation, October, pp. 1–178.

Heins, C. P. 1975. *Bending and Torsional Design in Structural Members*. Lexington Books, D. C. Heath and Company, Lexington, MA, pp. 17–24.

Heins, C. P. 1978. "Box Girder Bridge Design—State of the Art." *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, 4th Qrt., pp. 126–142.

Heins, C.P., and D. H. Hall. 1981. *Designer's Guide to Steel Box-Girder Bridges*, Booklet No. 3500, Bethlehem Steel Corporation, Bethlehem, PA, pp. 20–30.

Helwig, T. A., K. H. Frank, and J. A. Yura. 1997. "Lateral-Torsional Buckling of Singly Symmetric I-Beams," *Journal of Structural Engineering*. American Society of Civil Engineers, New York, NY, Vol. 123, No. 9, pp. 1172–1179.

Horne, M. R. and W. R. Grayson. 1983. "Parametric Finite Element Study of Transverse Stiffeners for Webs in Shear. Instability and Plastic Collapse of Steel Structures." In *Proc., the Michael R. Horne Conference*, L. J. Morris, ed., Granada Publishing, London, pp. 329–341.

Horne, M. R. and R. Narayanan. 1977. "Design of Axially Loaded Stiffened Plates," ASCE *Journal of the Structural Division*. American Society of Civil Engineers, Reston, VA, Vol. 103, ST11, pp. 2243–2257.

Huhn, H., and G. Valtinat. 2004. "Bolted Connections with Hot Dip Galvanized Steel Members with Punched Holes." *Proceedings of the ECCS/AISC Workshop, Connections in Steel Structures V: Innovative Steel Connections*, June 3–5, 2004. European Convention for Constructional Steelwork/American Institute of Steel Construction, Amsterdam.

IIW. 2007. *Recommendations for Fatigue Design of Welded Joints and Components*, doc. XII-1965-03/XV-1127-03. International Institute of Welding, Paris, France.

Itani, A. M. 1995. Cross-Frame Effect on Seismic Behavior of Steel Plate Girder Bridges. In *Proc., Annual Technical Session*, Structural Stability Research Council, Kansas City, MO. University of Missouri, Rolla, MO.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

C6.4.4—Flowchart for LRFD Article 6.10.6

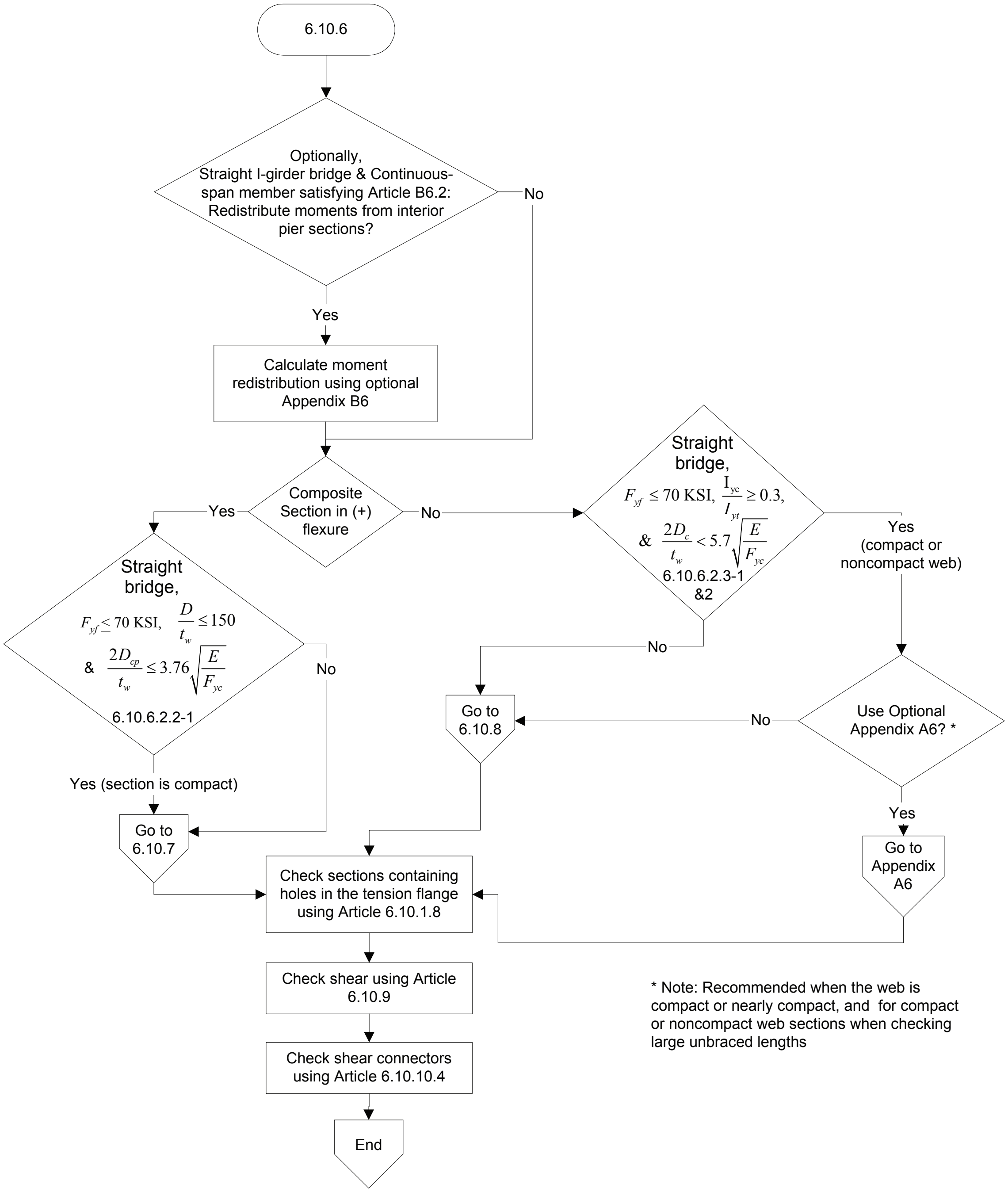


Figure C6.4.4-1—Flowchart for LRFD Article 6.10.6—Strength Limit State

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

C6.4.5—Flowchart for LRFD Article 6.10.7

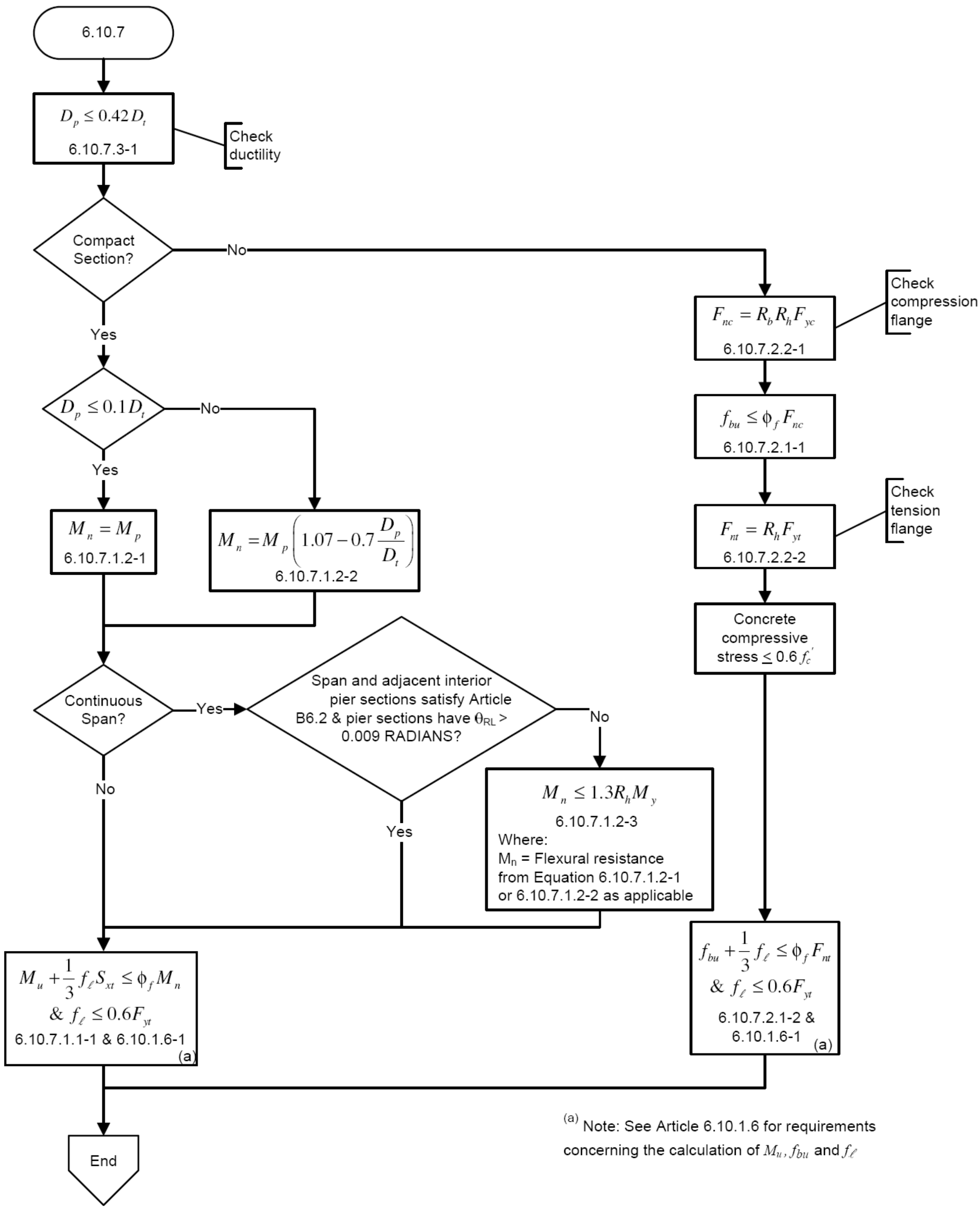
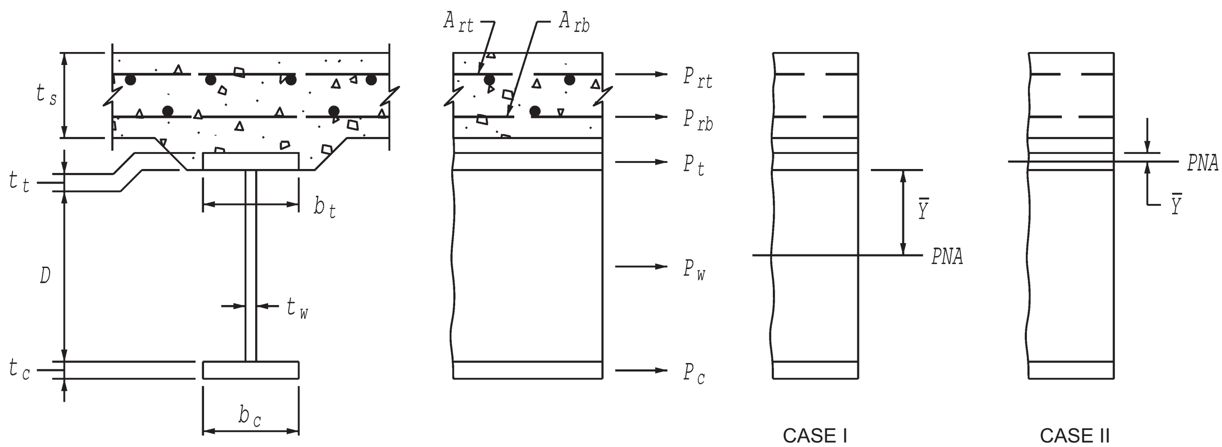


Figure C6.4.5-1—Flowchart for LRFD Article 6.10.7—Composite Sections in Positive Flexure

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Table D6.1-2—Calculation of $\bar{Y}$ and M_p for Sections in Negative Flexure

Case	PNA	Condition	$\bar{Y}$ and M_p
I	In Web	$P_c + P_w \geq P_t + P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{D}{2}\right)\left[\frac{P_c - P_t - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_p = \frac{P_w}{2D}\left[\bar{Y}^2 + \left(D - \bar{Y}\right)^2\right] + [P_{rt}d_{rt} + P_{rb}d_{rb} + P_t d_t + P_c d_c]$
II	In Top Flange	$P_c + P_w + P_t \geq P_{rb} + P_{rt}$	$\bar{Y} = \left(\frac{t_t}{2}\right)\left[\frac{P_w + P_c - P_{rt} - P_{rb}}{P_t} + 1\right]$ $M_p = \frac{P_t}{2t_t}\left[\bar{Y}^2 + \left(t_t - \bar{Y}\right)^2\right] + [P_{rt}d_{rt} + P_{rb}d_{rb} + P_w d_w + P_c d_c]$



in which:

$P_{rt} = F_{yrt} A_{rt}$

$P_s = 0.85 f'_c b_s t_s$

$P_{rb} = F_{yrb} A_{rb}$

$P_c = F_{yc} b_c t_c$

$P_w = F_{yw} D t_w$

$P_t = F_{yt} b_t t_t$

D6.2—YIELD MOMENT

D6.2.1—Noncomposite Sections

The yield moment, M_y, of a noncomposite section shall be taken as the smaller of the moment required to cause nominal first yielding in the compression flange, M_{yc}, and the moment required to cause nominal first yielding in the tension flange, M_{yt}, at the strength limit state. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

D6.2.2—Composite Sections in Positive Flexure

The yield moment of a composite section in positive flexure shall be taken as the sum of the moments applied separately to the steel and the short-term and long-term composite sections to cause nominal first yielding in either steel flange at the strength limit state. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.

The yield moment of a composite section in positive flexure may be determined as follows:

- Calculate the moment M_{D1} caused by the factored permanent load applied before the concrete deck has hardened or is made composite. Apply this moment to the steel section.
- Calculate the moment M_{D2} caused by the remainder of the factored permanent load. Apply this moment to the long-term composite section.
- Calculate the additional moment M_{AD} that must be applied to the short-term composite section to cause nominal yielding in either steel flange.
- The yield moment is the sum of the total permanent load moment and the additional moment.

Symbolically, the procedure is:

1) Solve for M_{AD} from the equation:

$$F_{yf} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{(D6.2.2-1)}$$

2) Then calculate:

$$M_y = M_{D1} + M_{D2} + M_{AD} \qquad \text{(D6.2.2-2)}$$

where:

S_{NC} = noncomposite section modulus (in.^{3})
S_{ST} = short-term composite section modulus (in.^{3})
S_{LT} = long-term composite section modulus (in.^{3})
M_{D1}, M_{D2} & M_{AD} = moments due to the factored loads applied to the appropriate sections (kip-in.)

M_y shall be taken as the lesser value calculated for the compression flange, M_{yc}, or the tension flange, M_{yt}.

D6.2.3—Composite Sections in Negative Flexure

For composite sections in negative flexure, the procedure specified in Article D6.2.2 is followed, except that the composite section for both short-term and long-term moments shall consist of the steel section and the longitudinal reinforcement within the effective width of the concrete deck. Thus, S_{ST} and S_{LT} are the same value.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$f_{lss} = 1.5 f_{0.5} - 0.50 f_{1.5} \quad (9.8.3.4.4\text{-}1)$$

where:

$f_{0.5}$ = the surface stress at a distance of 0.5t from the weld toe (ksi)

$f_{1.5}$ = the surface stress at a distance of 1.5t from the weld toe (ksi)

Design Level 3 shall be required for all bridge redecking applications unless the redecking procedure can be demonstrated as meeting the requirements of Article 9.8.3.4.1 and is approved by the Owner.

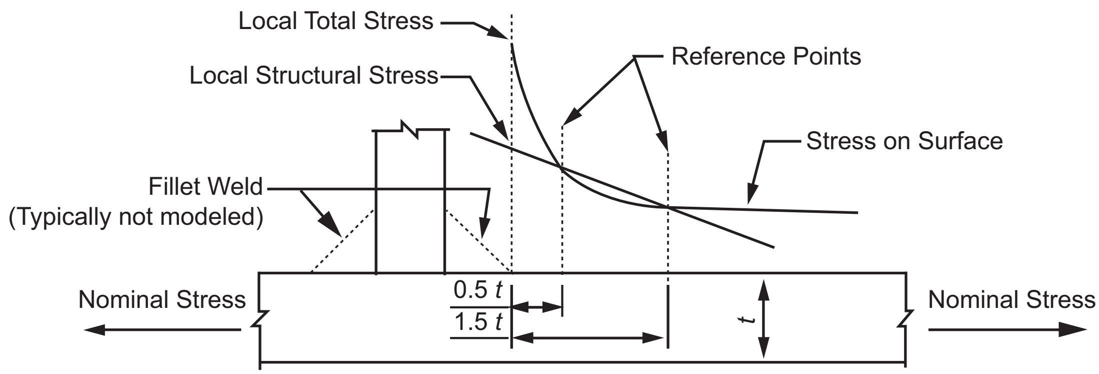


Figure 9.8.3.4.4-1—Local Structural Stress

9.8.3.5—Design

9.8.3.5.1—Superposition of Local and Global Effects

In calculating extreme force effects in the deck, the combination of local and global effects should be determined as specified in Article 6.14.3.

C9.8.3.5.1

The orthotropic deck is part of the global structural system, and, therefore, participates in distributing global stresses. These stresses may be additive to those generated in the deck locally. The axles of the design truck or the design tandem is used for the design of decks, whereas the rest of the bridge is proportioned for combinations of the design truck, the design tandem, and the design lane load. The governing positions of the same load for local and global effects could be quite different. Therefore, the Designer should analyze the bridge for both load regimes separately, apply the appropriate dynamic load allowance factor, and use the one that governs.

9.8.3.5.2—Limit States

9.8.3.5.2a—General

Orthotropic decks shall be designed to meet the requirements of Section 6 at all applicable limit states unless otherwise specified herein.

C9.8.3.5.2a

Tests indicate a large degree of redundancy and load redistribution between first yield and failure of the deck. The large reduction in combined force effects is a reflection of this performance.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

9.8.3.5.2b—Service Limit State

At the service limit state, the deck shall satisfy the requirements as specified in Article 2.5.2.6.

C9.8.3.5.2b

Service Limit State I must be satisfied of overall deflection limits and is intended to prevent premature deterioration of the wearing surface.

Service Limit State II is for the design of bolted connections against slip for overload and should be considered for the design of the rib and floorbeam splices. The remaining limit states are for tensile stresses in concrete structures and can be ignored.

9.8.3.5.2c—Strength Limit State

At the strength limit state for the combination of local and global force effects, the provisions of Article 6.14.3 shall apply.

The effects of compressive instability shall be investigated at the strength limit state. If instability does not control, the resistance of orthotropic plate deck shall be based on the attainment of yield strength at any point in the cross-section.

C9.8.3.5.2c

The deck, because it acts as part of the global structural system, is exposed to in-plane axial tension or compression. Consequently, buckling should be investigated.

Strength design must consider the following demands: rib flexure and shear, floorbeam flexure and shear, and panel buckling. The rib, including the effective portion of deck plate, must be evaluated for flexural and shear strength for its span between the floorbeams. The floorbeam, including the effective portion of the deck plate, must be evaluated for flexural and shear strength for its span between primary girders or webs. The reduction in floorbeam cross-section due to rib cutouts must be considered. When the panel is part of a primary girder flange, the panel must be evaluated for axial strength based on stability considerations.

Strength Limit IV condition is only expected to control where the orthotropic deck is integral with a long-span bridge superstructure.

9.8.3.5.2d—Fatigue Limit State

Structural components shall be checked for fatigue in accordance with the appropriate design level as specified in Article 9.8.3.4. The provisions of Article 6.6.1.2 shall apply for load-induced fatigue.

With the Owner's approval, application of less stringent fatigue design rules for interior traffic lanes of multilane decks subjected to infrequent traffic may be considered.

C9.8.3.5.2d

Experience has shown that fatigue damage on orthotropic decks occurs mainly at the ribs under the truck wheel paths in the exterior lanes.

For Level 1 design, test loads should be representative of the fatigue truck factored for the Fatigue I load combination and the critical details of the test specimen(s) should simulate both the expected service conditions and the appropriate boundary conditions; verification of these details is sufficient in lieu of a detailed refined fatigue analysis.

9.8.3.6—Detailing Requirements

9.8.3.6.1—Minimum Plate Thickness

Minimum plate thickness shall be determined as specified in Article 6.7.3.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

9.8.3.6.2—Closed Ribs

The one-sided weld between the web of a closed rib and the deck plate shall have a target penetration of 80 percent, with 70 percent minimum and no blow-through, and shall be placed with a tight fit of less than 0.02 in. gap prior to welding.

C9.8.3.6.2

Historically, the rib-to-deck plate weld has been specified as a one-sided partial penetration weld with minimum 80 percent penetration. Achieving a minimum of 80 percent penetration without blow-through is very difficult and fabricators have often failed to consistently satisfy this requirement. A review of the literature suggests that it is the maximum penetration that could be achieved without regularly resulting in weld blow-through. It has been suggested that the weld throat should, at a minimum, be of the same size as the rib wall and that the penetration be between 50 and 80 percent (Kolstein, 2007). However, a lower penetration limit of only 50 percent results in a rather large lack of fusion plane and increases the risk of cracks initiating from the root. Levels between 75 and 95 percent, with a target of 80 percent, are achievable and the lower bound of 70 percent is supported by research (Xiao, 2008).

The root gap is also a parameter that may influence performance. Research has shown that fatigue resistance of the weld is clearly improved when the root gap is closed in the final condition. When there is full contact, it appears that the root is protected and cracking is prevented. Shop experience indicates that using a tight fit prior to welding will also help prevent weld blow-through. Kolstein (2007) suggests the limit of 0.02 in. and this is adopted in these Specifications.

Additionally, melt-through of the weld is a quality issue that must be controlled. Fatigue tests on a limited number of samples (Sim and Uang, 2007) indicate that the performance of locations of melt-through is greater than or equal to those created by the notch condition of the 80 percent penetration. However, there are legitimate concerns that excessive melt-through may provide potential fatigue initiation sites and as such it should be avoided if possible. As such, the proposed detailing criteria is that the rib to deck shall be one-sided nominal 80 percent penetration, with 70 percent minimum and no blow-through, and with a tight fit less than 0.02 in. prior to welding. Additional details of the weld joint should be left for the fabricator to develop.

9.8.3.6.3—Welding to Orthotropic Decks

Welding of attachments, utility supports, lifting lugs, or shear connectors to the deck plate or ribs shall require approval by an Engineer.

9.8.3.6.4—Deck and Rib Details

Deck and rib splices shall either be welded or mechanically fastened by high-strength bolts~~.~~ Ribs shall be run continuously through cutouts in the webs of floorbeams, as shown in Figure 9.8.3.6.4-1. The following fabrication details shall be required by the contract documents as identified in Figure 9.8.3.6.4-1:

C9.8.3.6.4

Closed ribs may be trapezoidal, U-shaped, or V-shaped; the latter two are most efficient.

The floorbeam web cutouts at the intersections with the ribs may be with or without an additional free cutout at the bottom of the ribs. The former detail is generally preferable since it minimizes the rib restraint against

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

a) No snipes (cutouts) in floorbeam web

b) Welds to be wrapped around

c) Grind smooth

d) Combined fillet-groove welds may have to be used 1) in cases where the required size of fillet welds to satisfy the fatigue resistance requirements would be excessive if used alone or 2) to accomplish a ground termination.

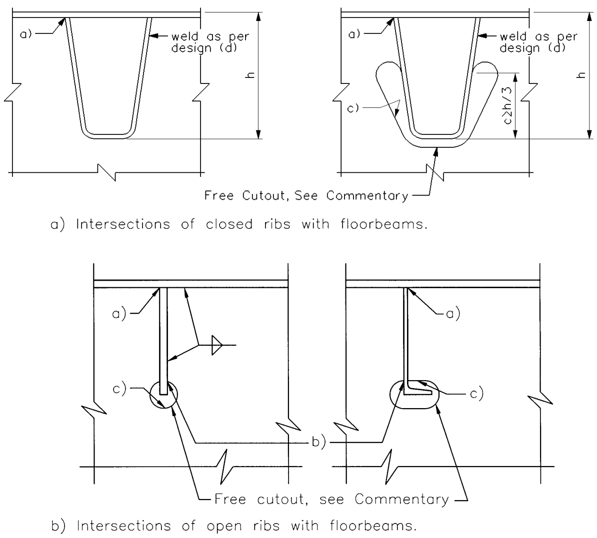


Figure 9.8.3.6.4-1—Detailing Requirements for Orthotropic Decks

rotation in its plane and associated stresses in the welds and in the floorbeam web.

If the bottom cutout depth *c* is small enough, the rotation of the rib is restrained and considerable out-of-plane stresses are introduced in the floorbeam web when the floorbeam is shallow. Local secondary stresses are also introduced in the rib walls by the interaction forces between the floorbeam webs and the rib walls and by secondary effects due to the small depth of cutout *c* (Wolchuk and Ostapenko, 1992).

If the floorbeam web is deep and flexible, or where additional depth of the cutout would unduly reduce the shear strength of the floorbeam, welding all around the rib periphery may be appropriate (*ECSC Report on Fatigue*, 1995, Wolchuk, 1999).

Fatigue test suggested that open snipes in the floorbeam webs at the junctions of the rib walls with the deck plate may cause cracks in the rib walls. Therefore, a tight-fitting snipe and a continuous weld between the floorbeam web and the deck and rib wall plates appear to be preferable.

Open ribs may be flat bars, angles, tees or bulb bars. Open-rib decks are less efficient and require more welding but are generally considered less risky to fabricate.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

[*This page is intentionally left blank. —ed.*]

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

[*This page is intentionally left blank. —ed.*]

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

9.9.8.3—Chip Seal

When a chip seal wearing surface is used on wood decks, a minimum of two layers should be provided.

C9.9.8.3

Laminated decks may have offset laminations creating irregularities on the surface, and it is necessary to provide an adequate depth of wearing surface to provide proper protection to the wood deck. Chip seal wearing surfaces have a good record as applied to stress laminated decks due to their behavior approaching that of solid slabs.

9.10—REFERENCES

Ahlskog, J. 2000. "Vibration and Deflection Criteria for Lightweight Decks Designed Using the LRFD Code." Being submitted for publication.

AISC. 1963. *Design Manual for Orthotropic Steel Plate Deck Bridges*. American Institute of Steel Construction, Chicago, IL.

Baker, T. H. 1991. *Volume I, Plate Stiffness Constants for Concrete Filled Steel Grid Decks, Static and Fatigue Strength Determination of Design Properties for Grid Bridge Decks*, Research Report ST-9, Department of Civil Engineering, University of Pittsburgh, Pittsburgh, PA.

Bieschke, L. A., and R. E. Klingner. 1982. *The Effect of Transverse Strand Extensions on the Behavior of Precast Prestressed Panel Bridges*, FHWA/TX-82/18-303-1F. Federal Highway Administration, Washington, DC, University of Texas, Austin, TX.

Buth, E., H. L. Furr, and H. L. Jones. 1992. *Evaluation of a Prestressed Panel, Cast-in-Place Bridge*, TTI-2-5-70-145-3. Texas Transportation Institute, College Station, TX.

Connor, R. J. 2002. "A Comparison of the In-service Response of an Orthotropic Steel Deck with Laboratory Studies and Design Assumptions." Ph.D. dissertation, Department of Civil Engineering, Lehigh University, Bethlehem, PA, May 2002.

Connor, R. J. and J. W. Fisher. 2004. *Results of Field Measurements Made on the Prototype Orthotropic Deck on the Bronx-Whitestone Bridge—Final Report*, ATLSS Report No. 04-03. Center for Advanced Technology for Large Structural Systems, Lehigh University, Bethlehem PA.

Connor, R. J., and J. W. Fisher. 2006. "Consistent Approach to Calculating Stresses for Fatigue Design of Welded Rib-to-Web Connections in Steel Orthotropic Bridge Decks," *Journal of Bridge Engineering*. American Society of Civil Engineers, Reston, VA, Vol. 11, No. 5, September/October 2006, pp. 517–525.

Csagoly, P. F. 1979. *Design of Thin Concrete Deck Slabs by the Ontario Highway Bridge Design Code*. Ministry of Transportation of Ontario, Downsville, Ontario, Canada.

Csagoly, P. F., and J. M. Lybas. 1989. "Advanced Design Method for Concrete Bridge Deck Slabs," *Concrete International*. American Concrete Institute, Farmington Hills, MI, Vol. 11, No. 5, May 1989, pp. 53–64.

Csagoly, P. F., and W. N. Nickas. 1987. "Florida Bulb-Tee and Double-Tee Beams," *Concrete International*. American Concrete Institute, Farmington Hills, MI, Vol. 9, No. 11, November 1987, pp. 18–23.

Csagoly, P. F., and R. J. Taylor. 1979. *A Development Program for Wood Highway Bridges*. Ministry of Transportation of Ontario, Downsville, Ontario, Canada.

CEN (European Committee for Standardization). 1992. *Eurocode 3 Design of Steel Structures*. European Committee for Standardization, Brussels, Belgium, Section 9, Fatigue.

Connor, R. J. and J. W. Fisher. 2004. *Results of Field Measurements Made on the Prototype Orthotropic Deck on the Bronx-Whitestone Bridge—Final Report*, ATLSS Report No. 04-03. Center for Advanced Technology for Large Structural Systems, Lehigh University, Bethlehem PA.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

Daniels, J., and R. Slutter. 1985. *Behavior of Modular Unfilled Composite Steel Grid Bridge Deck Panels*, Report No. 200.84.795.1, Lehigh University, Bethlehem, PA, January 1985.

Darlow, M., and N. Bettigole. 1989. "Instrumentation and Testing of Bridge Rehabilitated with Exodermic Deck," *Journal of Structural Engineering*. American Society of Civil Engineers, New York, NY, Vol. 115, No. 10, October 1989, pp. 2461–2480.

deV Batchelor, B., K. V. Dalen, T. Morrison, and R. J. Taylor. 1981. *Structural Characteristics of Red-Pine and Hem-Fir in Prestressed Laminated Wood Bridge Decks*. Queens University, Ontario, Canada.

deV Batchelor, B., B . E. Hewitt, and P. F. Csagoly. 1978. "Investigation of the Fatigue Strength of Deck Slabs of Composite Steel/Concrete Bridges." In *Transportation Research Record 664*. Transportation Research Board, National Research Council, Washington, DC.

Dexter, R. T., J. E. Tarquinio, and J. W. Fisher. 1994. Application of Hot Spot Stress Fatigue Analysis to Attachments on Flexible Plate. In *Proc., 13th International Conference on Offshore Mechanics and Arctic Engineering*, ASME, Vol. III, Material Engineering, pp. 85–92.

DiCesare, A., and J. Pensiero. 1992. *Bridge Analysis Report: High Street Bridge over Metro-North Railroad*, *Dobbs Ferry*. BIN 2265160, M.G. McLaren, P.C., West Nyack, NY, July 1992.

ECSC. 1995. "Measurements and Interpretations of Dynamic Loads and Bridges." In *Phase 4: Fatigue Strength of Steel Bridges*. Common Synthesis Report, edited by A. Bruls. Brussels, Belgium, September 1995, Annex F.

Fang, K. I. 1985. "Behavior of Ontario-Type Bridge Deck on Steel Girders." Ph.D. diss. University of Texas, Austin, TX, December 1985.

Fang, K. I., J. Worley, N. H. Burns, and R. E. Klingner. 1990. "Behavior of Isotropic Reinforced Concrete Bridge Decks on Steel Girders," *Journal of Structural Engineering*, American Society of Civil Engineers, New York, NY, Vol. 116, No. 3, March 1990, pp. 659–678.

FHWA. 2012. *Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges, Federal Highway Administration*, U.S. Department of Transportation, Washington, DC.

Gangarao, H. V. S., P. R. Raju, and R. Kannekanti. 1993. *Final Report: Static and Fatigue Behavior of Filled and Unfilled Composite Steel Grid Bridge Decks,* Report CFC92-150. West Virginia University, Constructed Facilities Center, Morgantown, WV, December 1993, Volume III.

Gangarao, H. V. S., P. R. Raju, and N. R. Koppula. 1992. "Behavior of Concrete-Filled Steel Grid Decks." In *Transportation Research Record 1371*. Transportation Research Board, National Research Council, Washington, DC.

Hays, C. O., J. M. Lybas, and J. O. Guevara. 1989. *Test of Punching Shear Strength of Lightly Reinforced Orthotropic Bridge Decks*. University of Florida, Gainesville, FL.

Hewitt, B. E., and B. deV Batchelor. 1975. "Punching Shear Strength of Restraint Slabs," *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 101, No. ST9, pp. 1837–1853.

Higgins, C. 2003. "LRFD Orthotropic Plate Model for Determining Live Load Moments in Concrete Filled Grid Bridge Decks," *Journal of Bridge Engineering*. American Society of Civil Engineers, Reston, VA, January/February 2003, pp. 20–28.

Higgins, C. "LRFD Compatible Orthotropic Plate Model for Estimating Deflections in Concrete Filled Grid Bridge Decks." Being submitted for publication.

Higgins, C., and H. Mitchell. 1997. *Tests of a Revised Exodermic Bridge Deck Design*, Report No. 97-16. Clarkson University, Potsdam, NY.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

12.11.4.4—Minimum Cover for Precast Box Structures

The provisions of Article 5.12.3 shall apply unless modified herein for precast box structures.

If the height of the fill is <2.0 ft, the minimum cover in the top slab shall be 2.0 in. for all types of reinforcement.

Where welded wire fabric is used, the minimum cover shall be the greater of three times the diameter of the wire or 1.0 in.

12.11.5—Construction and Installation

The contract documents shall require that construction and installation conform to Section 27, "Concrete Culverts," *AASHTO LRFD Bridge Construction Specifications*.

12.12—THERMOPLASTIC PIPES

12.12.1—General

The provisions herein shall apply to the structural design of buried thermoplastic pipe with solid, corrugated, or profile wall, manufactured of PE or PVC.

C12.12.1

These structures become part of a composite system comprised of the plastic pipe and the soil envelope.

The following specifications are applicable:

For PE:

- Solid Wall—ASTM F714,
- Corrugated—AASHTO M 294, and
- Profile—ASTM F894.

For PVC:

- Solid Wall—AASHTO M 278 and
- Profile—AASHTO M 304.

12.12.2—Service Limit States

12.12.2.1—General

The allowable maximum localized distortion of installed plastic pipe shall be limited based on the service requirements and overall stability of the installation. The extreme fiber tensile strain shall not exceed the allowable long-term strain in Table 12.12.3.3-1. The net tension strain shall be the numerical difference between the bending tensile strain and ring compression strain.

C12.12.2.1

The allowable long-term strains should not be reached in pipes designed and constructed in accordance with this Specification. Deflections resulting from conditions imposed during pipe installation should also be considered in design.

12.12.2.2—Deflection Requirement

Total deflection, Δ_t, shall be less than the allowable deflection, Δ_A, as follows:

C12.12.2.2

Deflection is controlled through proper construction in the field, and construction contracts should place responsibility for control of deflections on

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$\Delta_t \leq \Delta_A \quad (12.12.2.2\text{-}1)$$

where:

Δ_t = total deflection of pipe expressed as a reduction of the vertical diameter taken as positive for reduction of the vertical diameter and expansion of horizontal diameter. (in.)

Δ_A = total allowable deflection of pipe, reduction of vertical diameter (in.)

Total deflection, calculated using Spangler's expression for predicting flexural deflection in combination with the expression for circumferential shortening, shall be determined as:

$$\Delta_t = \frac{K_B\left(D_L P_{sp} + C_L P_L\right) D_o}{1000\left(E_p I_p / R^3 + 0.061\, M_s\right)} + \varepsilon_{sc} D \quad (12.12.2.2\text{-}2)$$

in which:

$$\varepsilon_{sc} = \frac{T_s}{1000\left(A_{eff} E_p\right)} \quad (12.12.2.2\text{-}3)$$

$$T_s = P_s\left(\frac{D_o}{2}\right) \quad (12.12.2.2\text{-}4)$$

where:

ε_{sc} = service compressive strain due to thrust, as specified in Article 12.12.3.10.1c and taken as positive for compression

T_s = service thrust per unit length (lb/in.)

D_L = deflection lag factor , a value of 1.5 is typical

K_B = bedding coefficient , a value of 0.10 is typical

P_{sp} = soil prism pressure (EV), evaluated at pipe springline (psi)

C_L = live load distribution coefficient

P_L = design live load pressure including vehicle, dynamic load allowance, and multiple presence effect (psi)

D_o = outside diameter of pipe (in.) as shown in Figure C12.12.2.2-1

E_p = short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi)

I_p = moment of inertia of pipe profile per unit length of pipe (in.^{4}/in.)

R = radius from center of pipe to centroid of pipe profile (in.) as shown in Figure C12.12.2.2-1

the contractor. However, feasibility of a specified installation needs to be checked prior to writing the project specifications.

The construction specifications set the allowable deflection, Δ_A, for thermoplastic pipe at five percent as a generally appropriate limit. The Engineer may allow alternate deflection limits for specific projects if calculations using the design method in this section show that the pipe meets all of the strength-limit-state requirements.

Eq. 12.12.2.2-2 uses the constrained soil modulus, M_s, as the soil property. Note that the soil prism load is used as input, rather than the reduced load used to compute thrust.

This check should be completed to determine that the expected field deflection based on thrust and flexure is lower than the maximum allowable deflection for the project.

Thrust and hoop strain in the pipe wall are defined positive for compression.

There are no standard values for the deflection lag factor. Values from 1.0 to 6.0 have been recommended. The highest values are for installations with quality backfill and low initial deflections and do not generally control designs. A value of 1.5 provides some allowance for increase in deflection over time for installations with initial deflection levels of several percent.

The bedding coefficient, K_B varies from 0.083 for full support to 0.110 for line support at the invert. Haunching is always specified to provide good support; however, it is still common to use a value of K_B equal to 0.10 to account for inconsistent haunch support.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

D = diameter to centroid of pipe profile (in.) as shown in Figure C12.12.2.2-1

M_s = secant constrained soil modulus, as specified in Article 12.12.3.5-1 (ksi)

P_s = design service load (psi)

A_{eff} = effective area of pipe wall per unit length of pipe as specified in Article 12.12.3.10.1b (in.^{2}/in.)

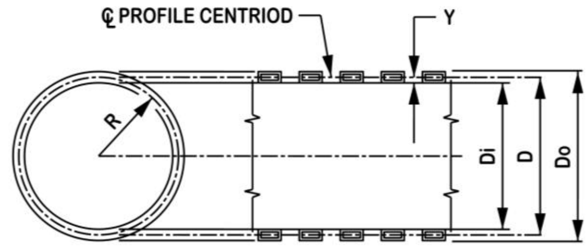


Figure C12.12.2.2-1—Schematic for Thermoplastic Pipe Terms

12.12.3—Safety against Structural Failure

12.12.3.1—General

Buried thermoplastic culverts shall be investigated at the strength limit states for thrust, general and local buckling, and combined strain.

C12.12.3.1

Total compressive strain in a thermoplastic pipe can cause yielding or buckling, and total tensile strain can cause cracking.

12.12.3.2—Section Properties

Section properties for thermoplastic pipe, including wall area, moment of inertia, and profile geometry should be determined from cut sections of pipe or obtained from the pipe manufacturer.

C12.12.3.2

Historically, AASHTO bridge specifications have contained minimum values for the moment of inertia and wall area of thermoplastic pipe; however, these values have been minimum values and are not meaningful for design. This is particularly so since provisions to evaluate local buckling were introduced in 2001. These provisions require detailed profile geometry that varies with manufacturer. Thus, there is no way to provide meaningful generic information on section properties. A convenient method for determining section properties for profile wall pipe is to make optical scans of pipe wall cross-sections and determine the properties with a computer drafting program.

12.12.3.3—Chemical and Mechanical Requirements

Mechanical properties for design shall be as specified in Table 12.12.3.3-1.

Except for buckling, the choice of either initial or long-term mechanical property requirements, as appropriate for a specific application, shall be determined by the Engineer. Investigation of general buckling shall be based on the value of modulus of elasticity that represents the design life of the project.

C12.12.3.3

Properties in Table 12.12.3.3-1 include “initial” and long-term values. No product standard requires determining the actual long-term properties; thus, there is some uncertainty in the actual values. However, pipe designed with the Table 12.12.3.3-1 values for 50-yr modulus of elasticity have performed well, and the properties are assumed to be reasonably conservative. Estimated values for a modulus of elasticity for a 75-yr design life have been estimated from relaxation tests on PVC and PE in parallel plate tests. The tests were conducted for over two years and show that the modulus of elasticity reduces approximately linearly with the logarithm of time. Further, with a log-linear extrapolation, the differences between 50-yr and 75-yr modulus values are very small. These values should be reasonably conservative, with the same reliability as the 50-yr values. Pipe and thermoplastic resin suppliers should be asked to provide confirmation of long-term modulus values for any

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

particular product. Values should meet or exceed those provided in Table 12.12.3.3-1. Where service life is in excess of 75 yr, test data may be used for the desired life.

The service long-term tension strain limit and the factored compression strain limit in Table 12.12.3.3-1 need to be multiplied by the appropriate resistance factors to obtain the strain limits.

Table 12.12.3.3-1—Mechanical Properties of Thermoplastic Pipe

Type of Pipe	Minimum Cell Class	Service Long-Term Tension Strain Limit, ε_{yt} (%)	Factored Compr. Strain Limit, ε_{yc} (%)	Initial		50-yr		75-yr	
				F_u min (ksi)	E min (ksi)	F_u min (ksi)	E min (ksi)	F_u min (ksi)	E min (ksi)
Solid Wall PE Pipe – ASTM F714	ASTM D3350, 335434C	5.0	4.1	3.0	110.0	1.44	22	1.40	21
Corrugated PE Pipe – AASHTO M 294	ASTM D3350, 435400C	5.0	4.1	3.0	110.0	0.90	22	0.90	21
Profile PE Pipe – ASTM F894	ASTM D3350, 334433C	5.0	4.1	3.0	80.0	1.12	20	1.10	19
	ASTM D3350, 335434C	5.0	4.1	3.0	110.0	1.44	22	1.40	21
Solid Wall PVC Pipe – AASHTO M 278, ASTM F679	ASTM D1784, 12454C	5.0	2.6	7.0	400.0	3.70	140	3.60	137
	ASTM D1784, 12364C	3.5	2.6	6.0	440.0	2.60	158	2.50	156
Profile PVC Pipe – AASHTO M 304	ASTM D1784, 12454C	5.0	2.6	7.0	400.0	3.70	140	3.60	137
	ASTM D1784, 12364C	3.5	2.6	6.0	440.0	2.60	158	2.50	156

12.12.3.4—Thrust

Loads on buried thermoplastic pipe shall be based on the soil prism load, modified as necessary to consider the effects of pipe-soil interaction. Calculations shall consider the duration of a load when selecting pipe properties to be used in design. Live loads need not be considered for the long-term loading condition.

C12.12.3.4

Because of the time-dependent nature of thermoplastic pipe properties, the load will vary with time.

Time of loading is an important consideration for some types of thermoplastic pipe. Live loads and occasional flood conditions are normally considered short-term loads. Earth loads or permanent high groundwater are normally considered long-term loads.

12.12.3.5—Factored and Service Loads

The factored load, P_u, in psi shall be taken as:

$$P_u = \eta_{EV}\left(\gamma_{EV} K_{\gamma E} K_2 VAF\, P_{sp} + \gamma_{WA} P_w\right) + \eta_{LL}\gamma_{LL} P_L\, C_L \qquad (12.12.3.5\text{-}1)$$

The service load, P_s, in psi shall be taken as:

C12.12.3.5

For η factors, refer to Article 12.5.4 regarding assumptions about redundancy for earth loads and live loads.

The factor K_2 is introduced to consider variation in thrust around the circumference, which is necessary when combining thrust with moment or thrust due to earth and live load under shallow fill. K_2 is set at 1.0 to determine thrust at the springline and 0.6 to determine

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$FF = \frac{S^2}{EI} \qquad (12.12.3.6\text{-}1)$$

where:

I = moment of inertia (in.^{4}/in.)
E = initial modulus of elasticity (ksi)
S = diameter of pipe (in.)

The flexibility factor, FF, shall be limited as specified in Article 12.5.6.3.

12.12.3.7—Soil Prism

The soil-prism load shall be calculated as a pressure representing the weight of soil above the pipe springline. The pressure shall be calculated for three conditions:

- If the water table is above the top of the pipe and at or above the ground surface:

$$P_{sp} = \frac{\left(H + 0.11\frac{D_o}{12}\right)\gamma_b}{144} \qquad (12.12.3.7\text{-}1)$$

- If the water table is above the top of the pipe and below the ground surface:

$$P_{sp} = \frac{1}{144}\left[\begin{array}{c}\left[\left(H_W - \frac{D_o}{24}\right) + 0.11\frac{D_o}{12}\right]\gamma_b + \\ \left[H - \left(H_w - \frac{D_o}{24}\right)\right]\gamma_s\end{array}\right] \qquad (12.12.3.7\text{-}2)$$

- If the water table is below the top of the pipe:

$$P_{sp} = \frac{\left(H + 0.11\frac{D_o}{12}\right)\gamma_s}{144} \qquad (12.12.3.7\text{-}3)$$

where:

P_{sp} = soil-prism pressure (EV), evaluated at pipe springline (psi)
D_o = outside diameter of pipe (in.)
γ_b = unit weight of buoyant soil (lb/ft^3)
H = depth of fill over top of pipe (ft)
H_w = depth of water table above springline of pipe (ft)
γ_s = wet unit weight of soil (lb/ft^3)

C12.12.3.7

The soil prism load and vertical arching factor, VAF, serve as a common reference for the load on all types of pipe.

The soil prism calculation needs to consider the unit weight of the backfill over the pipe. Use the wet unit weight above the water table and the buoyant unit weight below the water table. In cases where the water table fluctuates, multiple conditions may need to be evaluated.

Figure C3.11.3-1 shows the effect of groundwater on the earth pressure. See Table 3.5.1-1 for common unit weights.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

12.12.3.8—Hydrostatic Pressure

The pressure due to ground water shall be calculated as:

$$P_w = \frac{\gamma_w K_{wa} H_w}{144} \quad (12.12.3.8\text{-}1)$$

where:

P_w = hydrostatic water pressure at the springline of the pipe (psi)
γ_w = unit weight of water (lb/ft^3)
K_a = factor for uncertainty in level of groundwater table

C12.12.3.8

Hydrostatic loading due to external water pressure should be calculated in all cases where water table may be above the pipe springline at any time. This load contributes to hoop thrust but does not affect deflection.

There is often uncertainty in the level of the groundwater table and its annual variations. The designer may use the factor K_{wa} with values up to 1.3 to account for this uncertainty or may select conservative values of H_w with a lower value of K_{wa} but not less than 1.

12.12.3.9—Live Load

The live load shall be determined as a pressure applied at the pipe crown. The live load magnitude shall be based on the design vehicular live load in Article 3.6.1.2 and shall include modifiers for multiple presence/overload, dynamic load allowance, and distribution through cover soils.

The live load pressure, P_L, shall be taken as:

$$P_L = \frac{P(1 + IM/100)m}{\left[L_0 + (12H + K_1)LLDF\right]\left[W_0 + (12H + K_1)LLDF\right]} \quad (12.12.3.9\text{-}1)$$

where:

P_L = service live load on culvert (psi)
P = design wheel load as specified in Article 3.6.1.2 (lbs)
IM = dynamic load allowance as specified in Article 3.6.2.2 (%)
m = multiple presence factor as specified in Table 3.6.1.1.2-1
L_0 = length of live load surface contact area parallel to pipe diameter as specified in Article 3.6.1.2.5 (in.)
H = depth of fill over top of pipe (ft)
$LLDF$ = factor for distribution of live load through earth fills as specified in Article 3.6.1.2.6
W_0 = width of live load ground surface contact area parallel to flow in pipe as specified in Article 3.6.1.2.5 (in.)
K_1 = coefficient to consider design location (in.)
= 0 for live load at the crown of the pipe
= $D_0/2$ for live load at the springline

C12.12.3.9

Live load calculations are included here to demonstrate the computation of live load thrust at the crown and springline. NCHRP Project 15-29 to revise this is nearing completion. This project is proposing no changes to the live load distribution.

Increase as necessary if depth is sufficient for wheels and/or axles to interact.

Add axle spacing if depth is sufficient for axles to interact.

Add wheel spacing if depth is sufficient for wheels to interact.

Setting the term K_1 to 0 is the normal assumption in distributing live loads to the pipe and accounts for the load attenuating to the top of the pipe; however, the load continues to spread longitudinally along the pipe as it attenuates from the crown to the springline. Using the term $K_1 = D_0/2$ provides a means to account for this.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$\varepsilon_{sc} = \frac{T_s}{1000\left(A_{eff} E_p\right)} \quad (12.12.3.10.1c\text{-}2)$$

in which:

$$T_u = P_u\left(\frac{D_o}{2}\right) \quad (12.12.3.10.1c\text{-}3)$$

where:

ε_{uc} = factored compressive strain due to thrust
ε_{sc} = service compressive strain due to thrust
T_u = factored thrust per unit length (lb/in.)
T_s = service thrust per unit length (lb/in.)
A_{eff} = effective area of pipe wall per unit length of pipe (in.^{2}/in.)
E_p = short-term modulus for short-term loading or long-term modulus of pipe material for long-term loading as specified in Table 12.12.3.3-1 (ksi)
D_o = outside diameter of pipe (in.)
P_u = factored load as specified in Eq. 12.12.3.5-1

12.12.3.10.1d—Thrust Strain Limits

The factored compression strain due to thrust, ε_{uc}, shall satisfy:

$$\varepsilon_{uc} \leq \phi_T \varepsilon_{yc} \quad (12.12.3.10.1d\text{-}1)$$

where:

ε_{uc} = factored compressive strain due to thrust
ϕ_T = resistance factor for thrust effects
ε_{yc} = factored compression strain limit of the pipe wall material as specified in Table 12.12.3.3-1

12.12.3.10.1e—General Buckling Strain Limits

The factored compression strain due to thrust, incorporating local buckling effects, ε_{uc}, shall satisfy:

$$\varepsilon_{uc} \leq \phi_{bck} \varepsilon_{bck} \quad (12.12.3.10.1e\text{-}1)$$

The nominal strain capacity for general buckling of the pipe shall be determined as:

$$\varepsilon_{bck} = \frac{1.2C_n\left(E_p I_p\right)^{\frac{1}{3}}}{A_{eff} E_p}\left[\frac{\phi_s M_s\left(1-2\nu\right)}{\left(1-\nu\right)^2}\right]^{\frac{2}{3}} R_h \quad (12.12.3.10.1e\text{-}2)$$

in which:

C12.12.3.10.1e

The equations for global resistance presented here are a conservative simplification of the continuum buckling theory presented by Moore (1990). Detailed analysis using the full theory may be applied in lieu of the calculations in this section.

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.

$$R_h = \frac{11.4}{11 + \frac{D}{12H}} \qquad (12.12.3.10.1e\text{-}3)$$

where:

ε_{uc} = factored compressive strain due to thrust
ϕ_{bck} = resistance factor for global buckling
ε_{bck} = nominal strain capacity for general buckling

The term ϕ_s appears in this expression for ε_{bck} to account for backfills compacted to levels below that specified in the design. Lower levels of compaction increases the thrust force in the pipe.

For designs meeting all other requirements of these specifications and the *AASHTO LRFD Bridge Construction Specifications*, the correction for backfill soil geometry, R_h, is equal to value at left.

R_h = correction factor for backfill soil geometry

The complete theory proposed by Moore (1990) provides variations in R_h that consider nonuniform backfill support. In the extreme case where the width of structural backfill at the side of the culvert is 0.1 times the span and the modulus of the soil outside of the structural backfill is 0.1 times the modulus of the backfill, then:

$$R_h = \frac{20}{56 + \frac{D}{12H}} \qquad (C12.12.3.10.1e\text{-}1)$$

C_n = calibration factor to account for nonlinear effects = 0.55
E_p = short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi)
I_p = moment of inertia of pipe profile per unit length of pipe (in.^{4}/in.)
A_{eff} = effective area of pipe profile per unit length of pipe (in.^{2}/in.)
ϕ_s = resistance factor for soil pressure
M_s = secant constrained soil modulus as specified in Table 12.12.3.5-1 (ksi)
ν = Poisson's ratio of soil

Poisson's ratio is used to convert the constrained modulus of elasticity to the plane strain modulus. Values for Poisson's ratio of soils are provided in many geotechnical references. One reference is Selig (1990).

D = diameter to centroid of pipe profile (in.)
H = depth of fill over top of pipe (ft)

12.12.3.10.2—Bending and Thrust Strain Limits

12.12.3.10.2a—General

To ensure adequate flexural capacity the combined strain at the extreme fibers of the pipe profile must be evaluated at the allowable deflection limits against the limiting strain values.

12.12.3.10.2b—Combined Strain

If summation of axial strain, ε_{uc}, and bending strain, ε_f, produces tensile strain in the pipe wall, the combined

C12.12.3.10.2b

The criteria for combined compressive strain are based on limiting local buckling. A higher strain limit is

© 2012 by the American Association of State Highway and Transportation Officials.
All rights reserved. Duplication is a violation of applicable law.