American Association of State Highway and Transportation Officials

Jack Lettiere, Jr., President
Commissioner
New Jersey Department of Transportation

John Horsley
Executive Director

ERRATA

Dear Customer:

Due to errors found after the publication had been completed, AASHTO has reprinted the pages listed below and made the following errata changes to the *Standard Specifications for Highway Bridges*, 17th Edition:

Page No(s).	Affected Article	Errata Change
Front Matter		
p. lxxiii/p. lxxiv	Contents	Add "Figure 4.12.3.2.1-1 Location of Equivalent Footing after Duncan and Buchignami (1976)…104.1" to the list of figures
p. lxxxiii/p. lxxxiv	Contents	Add the following paragraph: "As referenced in Section 4.12.3.3.7b and 4.13.2, the following figures have been reprinted from the 1993 Commentary of the 1993 Interims to the *Standard Specifications for Highway Bridges*:"
		Add "Figure C4.12.3.7.2-1 Uplift of Group of Closely-Spaced Piles in Cohesionless Soils…104.2" directly below Commentary references
		Add "Figure C4.12.3.7.2-2 Uplift of Group of Piles in Cohesive Soils after Tomlinson (1987)…104.2" directly below Commentary references
		Add "Figure C4.13.3.3.4-1 Elastic Settlement Influence Factor as a Function of Embedment Ratio and Modulus Ratio after Donald, Sloan, and Chiu, 1980, as presented by Reese and O'Neill (1988)…104.2" directly below Commentary references
		Add "Figure C4.13.3.3.4-4 Bearing Capacity Coefficient, K_{sp} after Canadian Geotechnical Society (1985)…104.2" directly below Commentary references
Division I – Design		
p. 17/p.18	Article 3.1	Reference to "Figure 3.7.3A" should read "Figure 3.7.7A"

HB-17-E4 March 2005

Executive Office: 444 N. Capitol St., N.W., Suite 249, Washington, D.C. 20001
Telephone (202) 624-5800 Telefax (202) 624-5806 Telex 49000095800 HTO

Page No(s).	Affected Article	Errata Change
p. 39/p. 40	Article 3.26.1.1	Reference to “Article 20.19.1” should read “Article 16.3.14”
p. 53/p. 54	Article 4.4.7.1.1.4	Insert correct Figure 4.4.7.1.1.4A
p. 95/p. 96	Article 4.11.2	Remove the letter “C” from the following references: “Article C4.10.4” and “Article C4.11.4.1.1”
p. 115/p. 116	Article 5.2.2.4	Reference to “1996 Commentary, Division 1A, Article 6, in particular Equation C6-10 of these specifications” should read “*AASHTO LRFD Bridge Design Specifications*, 2nd Edition”
p. 157/p. 158	Article 5.8.7.1	Reference to “Article 8.5.4.2” should read “Article 5.8.4.2”
p. 239/p. 240	Article 9.20.3.2	Add missing square root symbol to “ $4\ f_c' b'\ d$ ” so that it reads “ $4 \sqrt{f_c'}\ b'\ d$ ”
p. 249/p. 250	Article 9.28.1	Add the multiplier “1.6” so that the equation reads “ $1.6\left(f_{su}^{*} - \frac{2}{3} f_{se}\right) D$ ”
p. 253/p. 254	Article 10.1.1	Reference to “Article 10.38.17” for notation F_v should read “Article 10.38.1.7”
p. 255/p. 256	Article 10.1.1	Reference to “Article 10.53.1.4” for notation V_u should read “Article 10.53.3”
p. 289/p. 290	Article 10.32.1	In Table 10.32.1A, “ $\frac{135,000,740}{(KL/r)^2}$ ” should read “ $\frac{135,008,740}{(KL/r)^2}$ ”
p. 339/p. 340	Article 12.1.2	Remove the following: “C_{dl} = dead load adjustment coefficient (Article 12.8.4.3.2)”
		Reference to “Article 12.8.4.3.3” for $M_{d\ell}$ should read “Article 12.8.4.3.1”
		Reference to “Article 12.8.4.3.3” for $M_{\ell\ell}$ should read “Article 12.8.4.3.2”
		Reference to “Articles 12.3.1 and 12.3.3” for ϕ should read “Articles 12.3.1, 12.3.3, 12.5.3.1, 12.6.1.3, and 12.8.4.2”
p. 343/p. 344	Article 12.4.1.5	Reference to “Article 23.10–Division II” should read “Division II, Article 26.6”
p. 355/p. 356	Article 12.8.4.3	Three references to “Table 12.8.4D” should read “Table 12.8.4B”
	Article 12.8.4.3.1	Add “factored” in first paragraph before “crown and haunch dead load moments”
		Two references to “M_{DL}” should read “$M_{d\ell}$”

Page No(s).	Affected Article	Errata Change
p. 355/p. 356	Article 12.8.4.3.1	Add "$\times(\text{Dead load, load factor})$" to end of Equation 12-12
		Reference to "nominal" for $M_{d\ell}$ should read "factored"
	Article 12.8.4.3.2	Add "factored" in first paragraph before "crown and haunch live load moments"
p. 355/p. 356	Article 12.8.4.3.2	Two references to "M_{LL}" should read "$M_{\ell\ell}$"
		Add parentheses around "$C_{\ell\ell}K_1S/K_2$" and "$\times(\text{Live load, load factor})$" to end of Equation 12-13
		Reference to "nominal" for $M_{\ell\ell}$ should read "factored"
	Article 12.8.4.3.3	Add "ϕ" to beginning of Equations 12-19 and 12-20
	Article 12.8.4.33	Delete "C_{dl}" and "C_{ll}" from Equation 12-19
		Delete "C_{dl}" and "C_{ll}" from Equation 12-20
	Article 12.8.5	Reference to "Articles 23.3.1.4" should read "Division II,"
p. 395/p. 396	Article 14.6.5.2	Reference to "Figure 14.6.5.2-2" should read "Figure 14.6.5.2-1"

Division IA – Seismic Design

p. 467/p. 468	Article 7.3.1	Reference to "Figure 5" should read "Figure 3.10"

Division II – Construction

p. 509/p. 510	Article 6.4.3	Reference to "Article 10.3.1.4.3, 'Anchorage Devices with Distribution Plates.'" should read "Division I, Article 9.21.7.2, 'Bearing Strength.' "
p. 561/p. 562	Article 10.10.2	Reference to "Article 10.5.1.4" should read "Article 10.5.1"
p. 579/p. 580	Article 11.5.6.4.3	Reference to "7/8 inch" should read "7/8"; the last bullet should be a new paragraph, not a bullet.
p. 631/p. 632	Article 18.9.1	Reference to "Article 18.4.10" should read "Article 18.4.9"
p. 665/p. 666	Article 26.5.4.1	Reference to "Figure 26.5.1D" should read "Figure 26.5.2D"
p. 687/p. 688	Article 30.1.1	Reference to "Division I, Section 18" should read "Division I, Section 17"

The following new pages have been added:

p. 104.1/p. 104.2		Add Figure 4.12.3.2.1-1 Location of Equivalent Footing after Duncan and Buchignami (1976) to p. 104.1
		Add the following paragraph: "As referenced in Section 4.12.3.3.7b and 4.13.2, the following figures have been reprinted from the 1993 Commentary of the 1993 Interims to the *Standard Specifications for Highway Bridges*:" to p. 104.2

p. 104.1/p. 104.2	Add Figure C4.12.3.7.2-1 Uplift of Group of Closely-Spaced Piles in Cohesionless Soils to p. 104.2
	Add Figure C4.12.3.7.2-2 Uplift of Group of Piles in Cohesive Soils after Tomlinson (1987) to p. 104.2
p. 104.1/p. 104.2	Add Figure C4.13.3.3.4-1 Elastic Settlement Influence Factor as a Function of Embedment Ratio and Modulus Ratio after Donald, Sloan, and Chiu, 1980, as presented by Reese and O'Neill (1988) to p. 104.2
	Add Figure C4.13.3.3.4-4 Bearing Capacity Coefficient, K_{sp} after Canadian Geotechnical Society (1985) to p. 104.2

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

AASHTO Publications Staff

SECTION 4—FOUNDATIONS

SECTION 11—STEEL STRUCTURES

SECTION 13—PAINTING

SECTION 15—CONCRETE BLOCK AND BRICK MASONRY

SECTION 16—TIMBER STRUCTURES

SECTION 18—BEARINGS

SECTION 26—METAL CULVERTS

SECTION 27—CONCRETE CULVERTS

APPENDICES:

As referenced in Section 4.12.3.3.7b and 4.13.2, the following figures have been reprinted from the 1993 Commentary of the 1993 Interims to the *Standard Specifications for Highway Bridges:*

Section 3
LOADS

Part A
TYPES OF LOADS

3.1 NOTATIONS

A = maximum expected acceleration of bedrock at the site
a = length of short span of slab (Article 3.24.6)
B = buoyancy (Article 3.22)
b = width of pier or diameter of pile (Article 3.18.2.2.4)
b = length of long span of slab (Article 3.24.6)
C = combined response coefficient
C = stiffness parameter = K(W/L) (Article 3.23.4.3)
C = centrifugal force in percent of live load (Article 3.10.1)
CF = centrifugal force (Article 3.22)
C_n = coefficient for nose inclination (Article 3.18.2.2.1)
C_M = steel bending stress coefficient (Article 3.25.1.5)
C_R = steel shear stress coefficient (Article 3.25.1.5)
D = parameter used in determination of load fraction of wheel load (Article 3.23.4.3)
D = degree of curve (Article 3.10.1)
D = dead load (Article 3.22)
D.F. = fraction of wheel load applied to beam (Article 3.28.1)
DL = contributing dead load
E = width of slab over which a wheel load is distributed (Article 3.24.3)
E = earth pressure (Article 3.22)
EQ = equivalent static horizontal force applied at the center of gravity of the structure
E_c = modulus of elasticity of concrete (Article 3.26.3)
E_s = modulus of elasticity of steel (Article 3.26.3)
E_w = modulus of elasticity of wood (Article 3.26.3)
F = horizontal ice force on pier (Article 3.18.2.2.1)
F_b = allowable bending stress (Article 3.25.1.3)
F_v = allowable shear stress (Article 3.25.1.3)
g = 32.2 ft./sec.^{2}
I = impact fraction (Article 3.8.2)
I = gross flexural moment of inertia of the precast member (Article 3.23.4.3)
ICE = ice pressure (Article 3.22)
J = gross Saint-Venant torsional constant of the precast member (Article 3.23.4.3)
K = stream flow force constant (Article 3.18.1)
K = stiffness constant (Article 3.23.4)
K = wheel load distribution constant for timber flooring (Article 3.25.1.3)
k = live load distribution constant for spread box girders (Article 3.28.1)
L = loaded length of span (Article 3.8.2)
L = loaded length of sidewalk (Article 3.14.1.1)

L = live load (Article 3.22)
L = span length (Article 3.23.4)
LF = longitudinal force from live load (Article 3.22)
M_D = moment capacity of dowel (Article 3.25.1.4)
M_x = primary bending moment (Article 3.25.1.3)
$\overline{M}_y$ = total transferred secondary moment (Article 3.25.1.4)
N_B = number of beams (Article 3.28.1)
N_L = number of traffic lanes (Article 3.23.4)
n = number of dowels (Article 3.25.1.4)
P = live load on sidewalk (Article 3.14.1.1)
P = stream flow pressure (Article 3.18.1)
P = total uniform force required to cause unit horizontal deflection of whole structure
P = load on one rear wheel of truck (Article 3.24.3)
P = wheel load (Article 3.24.5)
P = design wheel load (Article 3.25.1.3)
P_{15} = 12,000 pounds (Article 3.24.3)
P_{20} = 16,000 pounds (Article 3.24.3)
p = effective ice strength (Article 3.18.2.2.1)
p = proportion of load carried by short span (Article 3.24.6.1)
R = radius of curve (Article 3.10.1)
R = normalized rock response
R = rib shortening (Article 3.22)
R_D = shear capacity of dowel (Article 3.25.1.4)
R_x = primary shear (Article 3.25.1.3)
$\overline{R}_y$ = total secondary shear transferred (Article 3.25.1.4)
S = design speed (Article 3.10.1)
S = soil amplification spectral ratio
S = shrinkage (Article 3.22)
S = average stringer spacing (Article 3.23.2.3.1)
S = spacing of beams (Article 3.23.3)
S = width of precast member (Article 3.23.4.3)
S = effective span length (Article 3.24.1)
S = span length (Article 3.24.8.2)
S = beam spacing (Article 3.28.1)
s = effective deck span (Article 3.25.1.3)
SF = stream flow (Article 3.22)
T = period of vibration
T = temperature (Article 3.22)
t = thickness of ice (Article 3.18.2.2.4)
t = deck thickness (Article 3.25.1.3)
V = variable spacing of truck axles (Figure 3.7.7A)
V = velocity of water (Article 3.18.1)
W = combined weight on the first two axles of a standard HS Truck (Figure 3.7.7A)
W = width of sidewalk (Article 3.14.1.1)
W = wind load on structure (Article 3.22)
W = total dead weight of the structure
W_e = width of exterior girder (Article 3.23.2.3.2)
W = overall width of bridge (Article 3.23.4.3)
W = roadway width between curbs (Article 3.28.1)
WL = wind load on live load (Article 3.22)
w = width of pier or diameter of circular-shaft pier at the level of ice action (Article 3.18.2.2.1)
X = distance from load to point of support (Article 3.24.5.1)
x = subscript denoting direction perpendicular to longitudinal stringers (Article 3.25.1.3)

$$\overline{R_y} = 6Ps/1{,}000 \text{ for } s \le 50 \text{ inches} \qquad (3\text{-}28)$$

or,

$$\overline{R_y} = \frac{P}{2s}(s-20) \text{ for } s > 50 \text{ inches} \qquad (3\text{-}29)$$

$\overline{M_y}$ = total secondary moment transferred, in inch-pound, determined by the relationship,

$$\overline{M_y} = \frac{Ps}{1{,}600}(s-10) \text{ for } s \le 50 \text{ inches} \qquad (3\text{-}30)$$

$$\overline{M_y} = \frac{Ps}{20}\frac{(s-30)}{(s-10)} \text{ for } s > 50 \text{ inches} \qquad (3\text{-}31)$$

R_D and M_D = shear and moment capacities, respectively, as given in the following table:

Diameter of Dowel	Shear Capacity R_D	Moment Capacity M_D	Steel Stress Coefficients C_R	C_M	Total Dowel Length Required
in.	lb.	in.-lb.	l/in.^{2}	l/in.^{3}	in.
0.5	600	850	36.9	81.5	8.50
.625	800	1,340	22.3	41.7	10.00
.75	1,020	1,960	14.8	24.1	11.50
.875	1,260	2,720	10.5	15.2	13.00
1.0	1,520	3,630	7.75	10.2	14.50
1.125	1,790	4,680	5.94	7.15	15.50
1.25	2,100	5,950	4.69	5.22	17.00
1.375	2,420	7,360	3.78	3.92	18.00
1.5	2,770	8,990	3.11	3.02	19.50

3.25.1.5 In addition, the dowels shall be checked to ensure that the allowable stress of the steel is not exceeded using the following equation:

$$\sigma = \frac{1}{n}(C_R\overline{R_y} + C_M\overline{M_y}) \qquad (3\text{-}32)$$

where,

σ = minimum yield point of steel pins in pounds per square inch (see Table 10.32.1A);

n, $\overline{R_y}$, $\overline{M_y}$ = as previously defined;

C_R, C_M = steel stress coefficients as given in preceding table.

3.25.2 Plank and Nail Laminated Longitudinal Flooring

3.25.2.1 In the direction of the span, the wheel load shall be distributed over 10 inches.

3.25.2.2 Normal to the direction of the span the wheel load shall be distributed as follows:

Plank floor: 20 inches;

Non-interconnected nail laminated floor: width of tire plus thickness of floor, but not to exceed panel width. Continuous nail laminated floor and interconnected nail laminated floor, with adequate shear transfer between panels*, not less than 6 inches thick: width of tire plus twice thickness of floor.

3.25.2.3 For longitudinal flooring the span shall be taken as the clear distance between floor beams plus one-half the width of one beam but shall not exceed the clear span plus the floor thickness.

3.25.3 Longitudinal Glued Laminated Timber Decks

3.25.3.1 Bending Moment

In calculating bending moments in glued laminated timber longitudinal decks, no longitudinal distribution of wheel loads shall be assumed. The lateral distribution shall be determined as follows.

The live load bending moment for each panel shall be determined by applying to the panel the fraction of a wheel load determined from the following equations:

TWO OR MORE TRAFFIC LANES

$$\text{Load Fraction} = \frac{W_p}{3.75 + \frac{L}{28}} \text{ or } \frac{W_p}{5.00}, \text{ whichever is}$$

greater.

ONE TRAFFIC LANE

$$\text{Load Fraction} = \frac{W_p}{4.25 + \frac{L}{28}} \text{ or } \frac{W_p}{5.50}, \text{ whichever is}$$

greater.

where, W_p = Width of Panel; in feet ($3.5 \le W_p \le 4.5$)

L = Length of span for simple span bridges and the length of the shortest span for continuous bridges in feet.

*This shear transfer may be accomplished using mechanical fasteners, splines, or dowels along the panel joint or spreader beams located at intervals along the panels or other suitable means.

3.25.3.2 Shear

When calculating the end shears and end reactions for each panel, no longitudinal distribution of the wheel loads shall be assumed. The lateral distribution of the wheel load at the supports shall be that determined by the equation:

Wheel Load Fraction per Panel

$$= \frac{W_p}{4.00} \text{ but not less than 1.}$$

For wheel loads in other positions on the span, the lateral distribution for shear shall be determined by the method prescribed for moment.

3.25.3.3 Deflections

The maximum deflection may be calculated by applying to the panel the wheel load fraction determined by the method prescribed for moment.

3.25.3.4 Stiffener Arrangement

The transverse stiffeners shall be adequately attached to each panel, at points near the panel edges, with either steel plates, thru-bolts, C-clips or aluminum brackets. The stiffener spacing required will depend upon the spacing needed in order to prevent differential panel movement; however, a stiffener shall be placed at mid-span with additional stiffeners placed at intervals not to exceed 10 feet. The stiffness factor EI of the stiffener shall not be less than 80,000 kip-in^2.

3.25.4 Continuous Flooring

If the flooring is continuous over more than two spans, the maximum bending moment shall be assumed as being 80% of that obtained for a simple span.

3.26 DISTRIBUTION OF WHEEL LOADS AND DESIGN OF COMPOSITE WOOD-CONCRETE MEMBERS

3.26.1 Distribution of Concentrated Loads for Bending Moment and Shear

3.26.1.1 For freely supported or continuous slab spans of composite wood-concrete construction, as described in Article 16.3.14, Division II, the wheel loads shall be distributed over a transverse width of 5 feet for bending moment and a width of 4 feet for shear.

3.26.1.2 For composite T-beams of wood and concrete, as described in Article 20.19.2, Division II, the effective flange width shall not exceed that given in Article 10.38.3. Shear connectors shall be capable of resisting both vertical and horizontal movement.

3.26.2 Distribution of Bending Moments in Continuous Spans

3.26.2.1 Both positive and negative moments shall be distributed in accordance with the following table:

Maximum Bending Moments—Percent of Simple Span Moment

	Maximum Uniform Dead Load Moments				Maximum Live Load Moments			
	Wood Subdeck		Composite Slab		Concentrated Load		Uniform Load	
Span	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
Interior	50	50	55	45	75	25	75	55
End	70	60	70	60	85	30	85	65
2-Span[a]	65	70	60	75	85	30	80	75

[a]Continuous beam of 2 equal spans.

3.26.2.2 Impact should be considered in computing stresses for concrete and steel, but neglected for wood.

3.26.3 Design

The analysis and design of composite wood-concrete members shall be based on assumptions that account for the different mechanical properties of the components. A suitable procedure may be based on the elastic properties of the materials as follows:

$\frac{E_c}{E_w}$ = 1 for slab in which the net concrete thickness is less than half the overall depth of the composite section

$\frac{E_c}{E_w}$ = 2 for slab in which the net concrete thickness is at least half the overall depth of the composite section

$\frac{E_s}{E_w}$ = 18.75 (for Douglas fir and Southern pine)

in which,

E_c = modulus of elasticity of concrete;
E_w = modulus of elasticity of wood;
E_s = modulus of elasticity of steel.

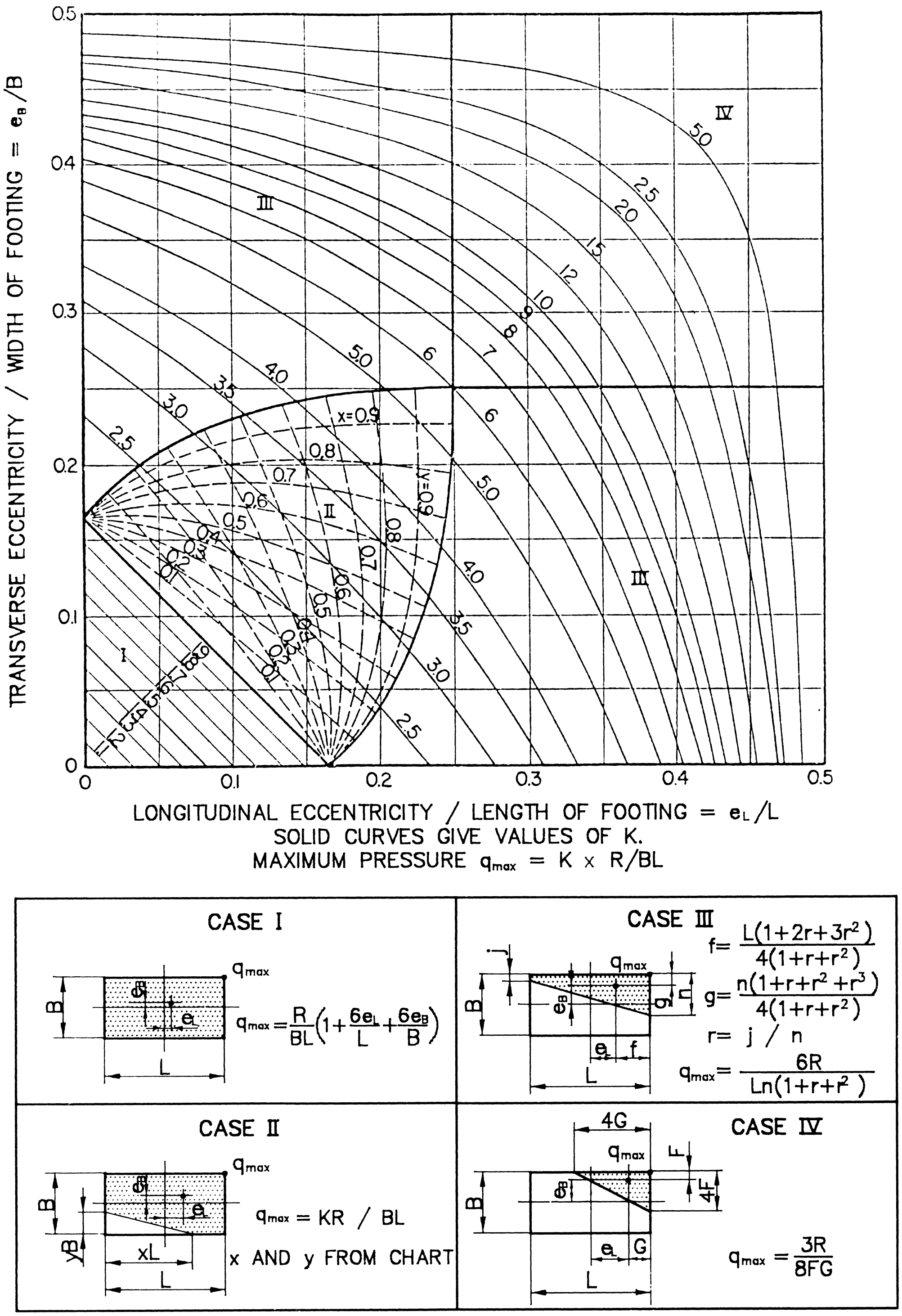


FIGURE 4.4.7.1.1.1C Contact Pressure for Footing Loaded Eccentrically About Two Axes Modified after AREA (1980)

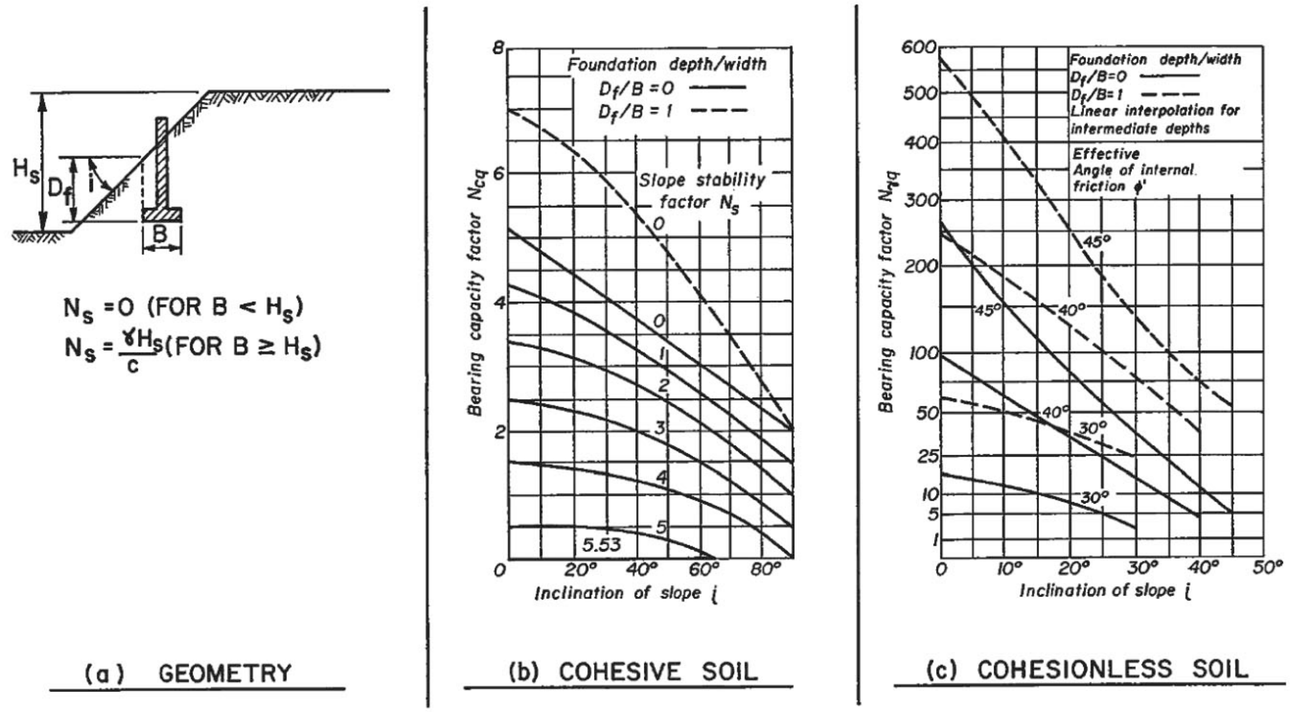


FIGURE 4.4.7.1.1.4A Modified Bearing Capacity Factors for Footing on Sloping Ground Modified after Meyerhof (1957)

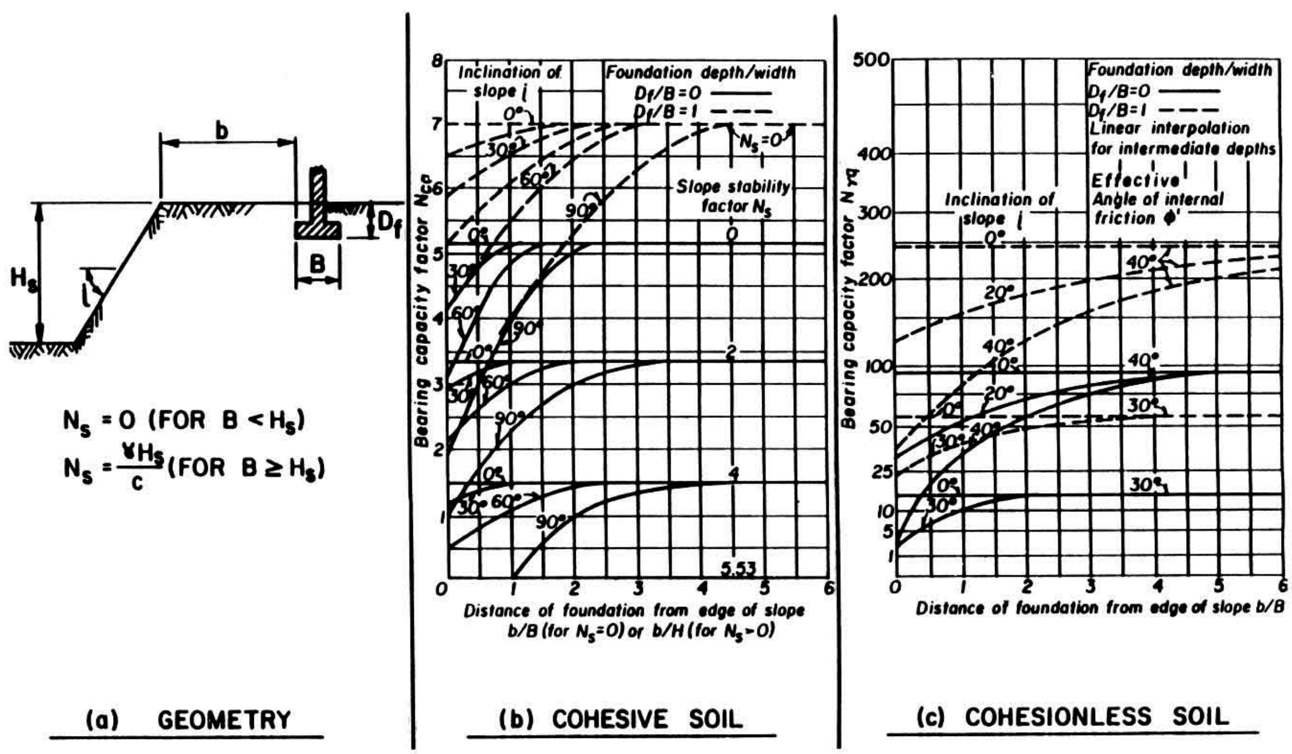


FIGURE 4.4.7.1.1.4B Modified Bearing Capacity Factors for Footing Adjacent Sloping Ground Modified after Meyerhof (1957)

TABLE 4.10.6-2 Performance Factors for Geotechnical Strength Limit States in Axially Loaded Piles

Method/Soil/Condition			Performance Factor
Ultimate bearing capacity of single piles	Skin friction	α-method	0.70
		β-method	0.50
		λ-method	0.55
	End bearing	Clay (Skempton, 1951)	0.70
		Sand (Kulhawy, 1983)	
		ϕ_f from CPT	0.45
		ϕ_f from SPT	0.35
		Rock (Canadian Geotech. Society, 1985)	0.50
	Skin friction and end bearing	SPT-method	0.45
		CPT-method	0.55
		Load test	0.80
		Pile driving analyzer	0.70
Block failure		Clay	0.65
Uplift capacity of single piles		α-method	0.60
		β-method	0.40
		λ-method	0.45
		SPT-method	0.35
		CPT-method	0.45
		Load Test	0.80
Group uplift capacity		Sand	0.55
		Clay	0.55

ysis of soil and groundwater samples should be considered.

4.11.1.9 Nearby Structures

In cases where foundations are placed adjacent to existing structures, the influence of the existing structures on the behavior of the foundation, and the effect of the foundation on the existing structures, shall be investigated.

4.11.2 Notations

B = footing width (in length units)
B' = reduced effective footing width (see Article 4.11.4.1.5) (in length units)
c = soil cohesion (in units of force/length^2)
C_{w1}, C_{w2} = correction factors for groundwater effect (dimensionless)
D_f = depth to footing base (in length units)
D_w = depth to groundwater table (in length units)
E_m = elastic modulus of rock masses (in units of force/length^2)
i = type of load
L' = reduced effective length (see Article 4.11.4.1.5) (in length units)
L_i = load type i
$\overline{N}$ = average value of standard penetration test blow count (dimensionless)
N_m, N_{cm}, N_{qm} = modified bearing capacity factors used in analytic theory (dimensionless)
q_c = cone resistance (in units of force/length^2)
q_{ult} = ultimate bearing capacity (in units of force/length^2)
R_I = reduction factor due to the effect of load inclination (dimensionless)
R_n = nominal resistance
RQD = rock quality designation
s = span length (in length units)
s_u = undrained shear strength of soil (in units of force/length^2)
β_i = load factor coefficient for load type i (see Article 4.10.4)
γ = load factor (see Article 4.10.4)
γ = total (moist) unit weight of soil (see Article 4.11.4.1.1)

TABLE 4.10.6-3 Performance Factors for Geotechnical Strength Limit States in Axially Loaded Drilled Shafts

Method/Soil/Condition			Performance Factor
Ultimate bearing capacity of single drilled shafts	Side resistance in clay	α-method (Reese & O'Neill)	0.65
	Base resistance in clay	Total Stress (Reese & O'Neill)	0.55
	Side resistance in sand	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See discussion in article 4.13.3.3.3
	Base resistance in sand	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See discussion in article 4.13.3.3.3
	Side resistance in rock	Carter & Kulhawy Horvath and Kenney	0.55 0.65
	Base resistance in rock	Canadian Geotechnical Society Pressuremeter Method (Canadian Geotechnical Society)	0.50 0.50
	Side resistance and end bearing	Load test	0.80
Block failure		Clay	0.65
Uplift capacity of single drilled shafts	Clay	α-method (Reese & O'Neill) Belled Shafts (Reese & O'Neill)	0.55 0.50
	Sand	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See discussion in section 4.13.3.3.3
	Rock	Carter & Kulhawy Horvath & Kenney	0.45 0.55
		Load test	0.80
Group uplift capacity		Sand Clay	0.55 0.55

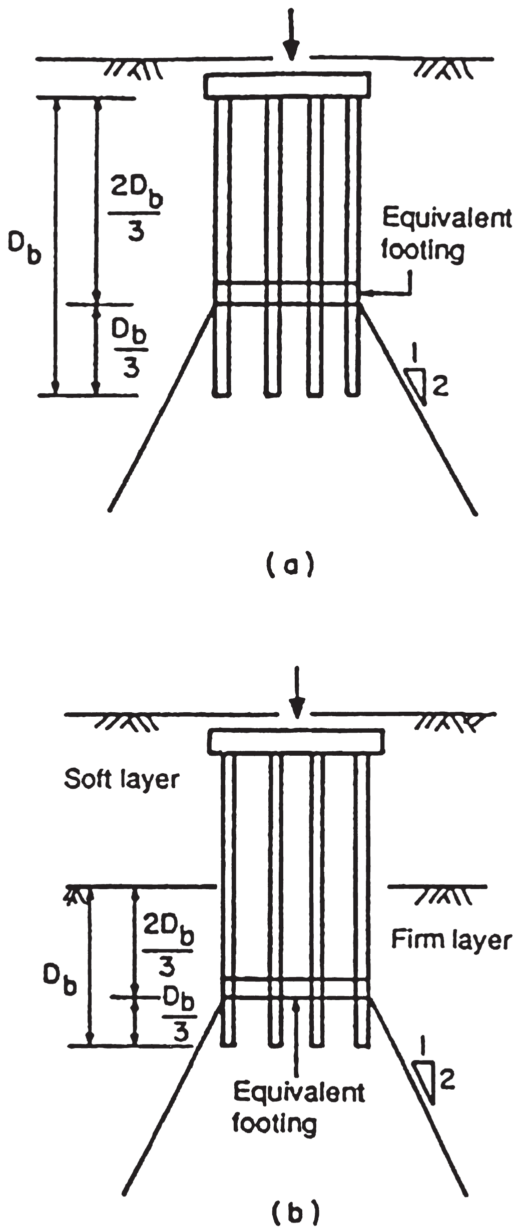


FIGURE C4.12.3.2.1-1 Location of Equivalent Footing (After Duncan and Buchignani, 1976)

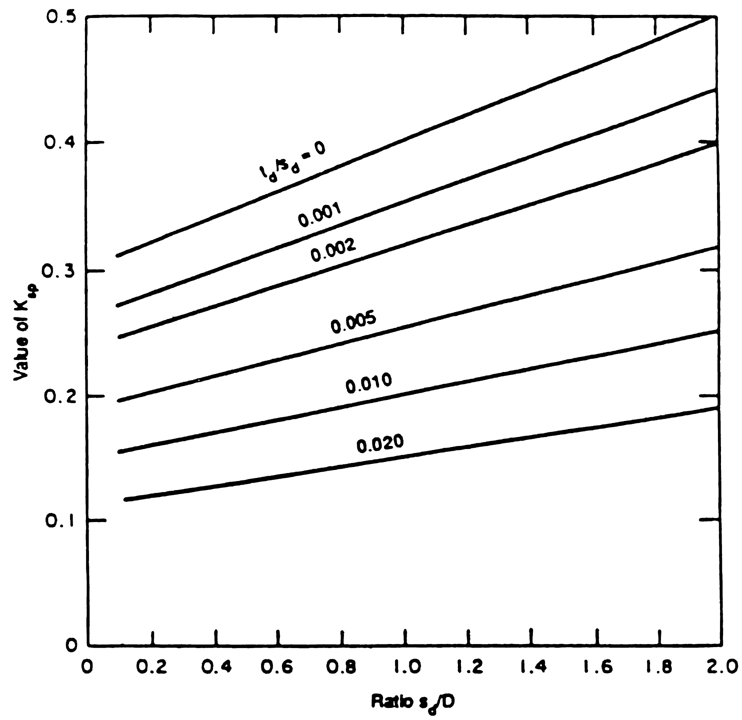


FIGURE C4.12.3.3.4-1 Bearing capacity coefficient, K_{sp} (After Canadian Foundation Engineering Manual, 1985)

As referenced in Section 4.12.3.3.7b and 4.13.2, the following figures have been reprinted from the 1993 Commentary of the 1993 Interims to the *Standard Specifications for Highway Bridges.*

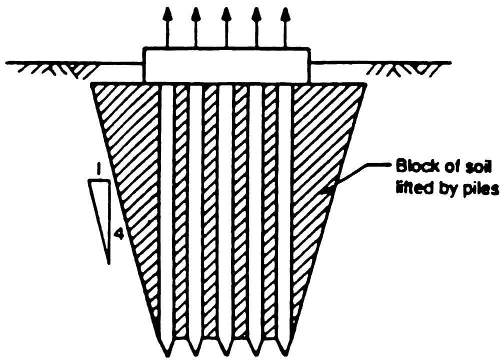


FIGURE C4.12.3.7.2-1 Uplift of group of closely-spaced piles in cohesionless soils

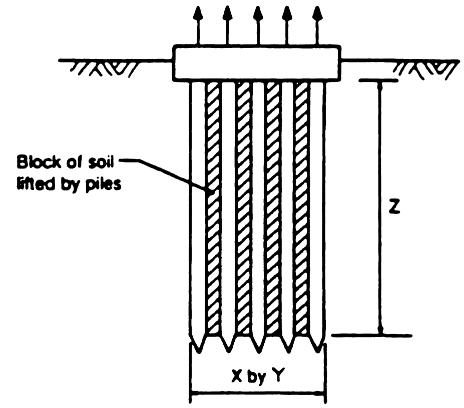


FIGURE C4.12.3.7.2-2 Uplift of group of piles in cohesive soils (After Tomlinson, 1987)

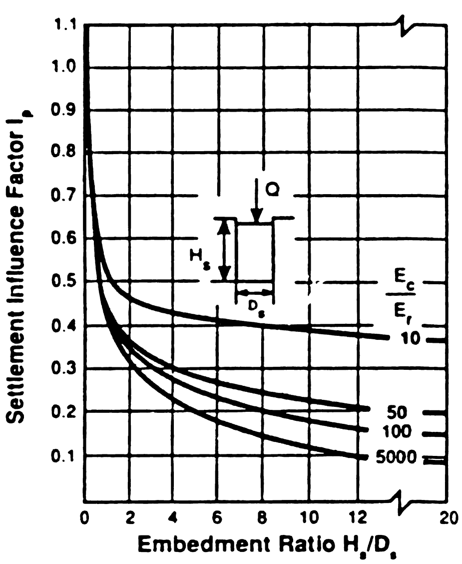


FIGURE C4.13.3.3.4-1 Elastic Settlement Influence Factor as a Function of Embedment Ratio and Modulus Ratio (After Donald, Sloan and Chiu, 1980, as presented by Reese and O'Neill, 1988)

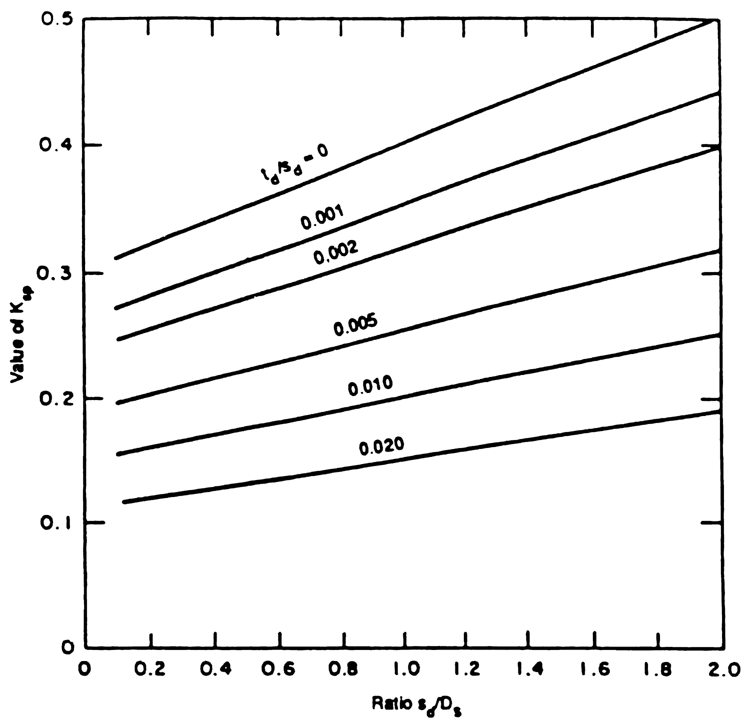


FIGURE C4.13.3.3.4-4 Bearing Capacity Coefficient, K_{sp} (After Canadian Geotechnical Society, 1985)

have been successfully used in both fill and cut wall applications. However, they are most effective in fill wall applications. MSE walls shall not be used under the following conditions.

- When utilities other than highway drainage must be constructed within the reinforced zone if future access to the utilities would require that the reinforcement layers be cut, or if there is potential for material which can cause degradation of the soil reinforcement to leak out of the utilities into the wall backfill.
- With soil reinforcements exposed to surface or ground water contaminated by acid mine drainage, other industrial pollutants, or other environmental conditions which are defined as aggressive as described in Division II, Article 7.3.6.3, unless environment specific long-term corrosion or degradation studies are conducted.
- When floodplain erosion may undermine the reinforced fill zone or facing column, or where the depth of scour cannot be reliably determined.

MSE walls may be considered for use under the following special conditions:

- When two intersecting walls form an enclosed angle of 70° or less, the affected portion of the wall is designed as an internally tied bin structure with at-rest earth pressure coefficients.
- Where metallic reinforcements are used in areas of anticipated stray currents within 60 meters (200 feet) of the structure, a corrosion expert should evaluate the potential need for corrosion control requirements.

5.2.1.5 Prefabricated Modular Walls

Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soil-filled reinforced concrete or steel modules or bins, rock filled gabion baskets, precast concrete units, or dry cast segmental masonry concrete units (without soil reinforcement) which resist earth pressures by acting as gravity retaining walls. Prefabricated modular walls may also use their structural elements to mobilize the dead weight of a portion of the wall backfill through soil arching to provide resistance to lateral loads. Prefabricated modular systems may be used where conventional gravity, cantilever or counterfort concrete retaining walls are considered.

Steel modular systems shall not be used where the steel will be exposed to surface or subsurface water which is contaminated by acid mine drainage, other industrial pollutants, other environmental conditions which are defined as aggressive as described in Division II, Article 7.3.6.3, or where deicing spray is anticipated.

5.2.2 Wall Capacity

Retaining walls shall be designed to provide adequate structural capacity with acceptable movements, adequate foundation bearing capacity with acceptable settlements, and acceptable overall stability of slopes adjacent to walls. The tolerable level of wall lateral and vertical deformations is controlled by the type and location of the wall structure and surrounding facilities.

5.2.2.1 Bearing Capacity

The bearing capacity of wall foundation support systems shall be estimated using procedures described in Articles 4.4, 4.5, or 4.6, or other generally accepted theories. Such theories are based on soil and rock parameters measured by in situ and/or laboratory tests.

5.2.2.2 Settlement

The settlement of wall foundation support systems shall be estimated using procedures described in Articles 4.4, 4.5, or 4.6, or other generally accepted methods. Such methods are based on soil and rock parameters measured directly or inferred from the results of in situ and/or laboratory test.

5.2.2.3 Overall Stability

The overall stability of slopes in the vicinity of walls shall be considered as part of the design of retaining walls. The overall stability of the retaining wall, retained slope, and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis such as the Modified Bishop, simplified Janbu or Spencer methods of analysis. A minimum factor of safety of 1.3 shall be used for walls designed for static loads, except the factor of safety shall be 1.5 for walls that support abutments, buildings, critical utilities, or for other installations with a low tolerance for failure. A minimum factor of safety of 1.1 shall be used when designing walls for seismic loads. In all cases, the subsurface conditions and soil/rock properties of the wall site shall be adequately characterized through in-situ exploration and testing and/or laboratory testing as described in Article 5.3.

Seismic forces applied to the mass of the slope shall be based on a horizontal seismic coefficient k_h equal to one-half the ground acceleration coefficient A, with the vertical seismic coefficient k_v equal to zero.

It must be noted that, even if overall stability is satisfactory, special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft subsoils where consolidation and/or lateral flow of the soft soil could result in unacceptable long-term settlements or horizontal movements.

Stability of temporary construction slopes needed to construct the wall shall also be evaluated.

5.2.2.4 Tolerable Deformations

Tolerable vertical and lateral deformation criteria for retaining walls shall be developed based on the function and type of wall, unanticipated service life, and consequences of unacceptable movements (i.e., both structural and aesthetic).

Allowable total and differential vertical deformations for a particular retaining wall are dependent on the ability of the wall to deflect without causing damage to the wall elements or exhibiting unsightly deformations. The total and differential vertical deformation of a retaining wall should be small for rigid gravity and semi-gravity retaining walls, and for soldier pile walls with a cast-in-place facing. For walls with anchors, any downward movement can cause significant destressing of the anchors.

MSE walls can tolerate larger total and differential vertical deflections than rigid walls. The amount of total and differential vertical deflection that can be tolerated depends on the wall facing material, configuration, and timing of facing construction. A cast-in-place facing has the same vertical deformation limitations as the more rigid retaining wall systems. However, an MSE wall with a cast-in-place facing can be specified with a waiting period before the cast-in-place facing is constructed so that vertical (as well as horizontal) deformations have time to occur. An MSE wall with welded wire or geosynthetic facing can tolerate the most deformation. An MSE wall with multiple precast concrete panels cannot tolerate as much vertical deformation as flexible welded wire or geosynthetic facings because of potential damage to the precast panels and unsightly panel separation.

Horizontal movements resulting from outward rotation of the wall or resulting from the development of internal equilibrium between the loads applied to the wall and the internal structure of the wall must be limited to prevent overstress of the structural wall facing and to prevent the wall face batter from becoming negative. In general, if vertical deformations are properly controlled, horizontal deformations will likely be within acceptable limits. For MSE walls with extensible reinforcements, reinforcement serviceability criteria, the wall face batter, and the facing type selected (i.e., the flexibility of the facing) will influence the horizontal deformation criteria required.

Vertical wall movements shall be estimated using conventional settlement computational methods (see Articles 4.4, 4.5, and 4.6. For gravity and semi-gravity walls, lateral movement results from a combination of differential vertical settlement between the heel and the toe of the wall and the rotation necessary to develop active earth pressure conditions (see Table 5.5.2A). If the wall is designed for at-rest earth pressure conditions, the deflections in Table 5.5.2A do not need to be considered. For anchored walls, deflections shall be estimated in accordance with Article 5.7.2. For MSE walls, deflections may be estimated in accordance with Article 5.8.10.

Where a wall is used to support a structure, tolerable movement criteria shall be established in accordance with Articles 4.4.7.2.5, 4.5 and 4.6. Where a wall supports soil on which an adjacent structure is founded, the effects of wall movements and associated backfill settlement on the adjacent structure shall be evaluated.

For seismic design, seismic loads may be reduced, as result of lateral wall movement due to sliding, for what is calculated based on Division 1A using the Mononobe-Okabe method if both of the following conditions are met:

- the wall system and any structures supported by the wall can tolerate lateral movement resulting from sliding of the structure,
- the wall base is unrestrained regarding its ability to slide, other than soil friction along its base and minimal soil passive resistance.

Procedures for accomplishing this reduction in seismic load are provided in the *AASHTO LRFD Bridge Design Specifications,* 2nd Edition. In general, this only applies to gravity and semi-gravity walls. Though the specifications in Division 1A regarding this issue are directed at structural gravity and semi-gravity walls, these specifications may also be applicable to other types of gravity walls regarding this issue provided the two conditions listed above are met.

5.2.3 Soil, Rock, and Other Problem Conditions

Geologic and environmental conditions can influence the performance of retaining walls and their foundations, and may require special consideration during design. To the extent possible, the presence and influence of such conditions shall be evaluated as part of the subsurface exploration program. A representative, but not exclusive, listing of problem conditions requiring special consideration is presented in Table 4.2.3A for general guidance.

5.3 SUBSURFACE EXPLORATION AND TESTING PROGRAMS

The elements of the subsurface exploration and testing programs shall be the responsibility of the Designer, based

TABLE 5.8.6.1.2B Default and Minimum Values for the Total Geosynthetic Ultimate Limit State Strength Reduction Factor, RF

Application	Total Reduction Factor, RF
All applications, but with product specific data obtained and analyzed in accordance with FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines"—Appendix B, and FHWA Publication No. FHWA SA-96-072 "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes"	All reduction factors shall be based on product specific data. RF_{ID} and RF_D shall not be less than 1.1.
Permanent applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 5.8.6.1.2A, provided product specific data is not available	7.0
Temporary applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 5.8.6.1.2A, provided product specific data is not available	3.5

Values for RF_{ID}, RF_{CR}, and RF_D shall be determined from product specific test results. Even with product specific test results, RF_{ID} and RF_D shall be no less than 1.1 each. Guidelines for how to determine RF_{ID}, RF_{CR}, and RF_D from product specific data are provided in FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines"—Appendix B, and in FHWA Publication No. FHWA SA-96-072 "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes." For wall applications which are defined as not having severe consequences should poor performance or failure occur, having nonaggressive soil conditions, and if the geosynthetic product meets the minimum requirements listed in Table 5.8.6.1.2A, the long-term tensile strength of the reinforcement may be determined using a default reduction factor for RF as provided in Table 5.8.6.1.2B in lieu of product specific test results.

5.8.6.2 Allowable Stresses

5.8.6.2.1 Steel Reinforcements

The allowable tensile stress for steel reinforcements and connections for permanent structures (i.e., design lives of 75 to 100 years) shall be in accordance with Article 10.32, in particular Table 10.32.1A. These requirements result in an allowable tensile stress for steel strip reinforcement, in the wall backfill away from the wall face connections, of $0.55F_y$. For grid reinforcing members connected to a rigid facing element (e.g., a concrete panel or block), the allowable tensile stress shall be reduced to $0.48F_y$. Transverse and longitudinal grid members shall be sized in accordance with AASHTO M 55 (ASTM A 185). For temporary structures (i.e., design lives of 3 years or less), the allowable tensile stress may be increased by 40 %. The global safety factor of 0.55 applied to F_y for permanent structures accounts for uncertainties in structure geometry, fill properties, externally applied loads, the potential for local overstress due to load nonuniformities, and uncertainties in long-term reinforcement strength. Safety factors less than 0.55, such as the 0.48 factor applied to grid members, account for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars.

The allowable reinforcement tension is determined by multiplying the allowable stress by the cross-sectional area of the steel reinforcement after corrosion losses. (See Figure 5.8.6A.) The loss in steel cross-sectional area due to corrosion shall be determined in accordance with Article 5.8.6.1.1. Therefore,

$$T_a = FS\frac{A_c F_y}{b} \qquad (5.8.6.2.1\text{-}1)$$

where, all variables are as defined in Figure 5.8.6A.

5.8.6.2.2 Geosynthetic Reinforcements

The allowable tensile load per unit of reinforcement width for geosynthetic reinforcements for permanent structures (i.e., design lives of 75 to 100 years) is determined as follows: (See Figure 5.8.6B.)

$$T_a = \frac{T_{ult}}{FS \times RF} \qquad (5.8.6.2.2\text{-}1)$$

where, FS is a global safety factor which accounts for uncertainties in structure geometry, fill properties, externally applied loads, the potential for local overstress due to load nonuniformities, and uncertainties in long-term reinforcement strength. For ultimate limit state conditions for permanent walls, a FS of 1.5 shall be used. Note that the uncertainty of determining long-term reinforcement strength is taken into account through an additional factor of safety, which is typically about 1.2, depending on the amount of creep data available, through the creep extrapolation protocol provided in Appendix B of the FHWA-SA-96-071, "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines."

5.8.7 Soil Reinforcement/Facing Connection Strength Design

5.8.7.1 Connection Strength for Steel Soil Reinforcements

Connections shall be designed to resist stresses resulting from active forces (T_0, as described in Article 5.8.4.2) as well as from differential movements between the reinforced backfill and the wall facing elements.

Elements of the connection which are embedded in the facing element shall be designed with adequate bond length and bearing area in the concrete to resist the connection forces. The capacity of the embedded connector shall be checked by tests as required in Article 8.31. Connections between steel reinforcement and the wall facing units (e.g., welds, bolts, pins, etc.) shall be designed in accordance with Article 10.32.

Connection materials shall be designed to accommodate losses due to corrosion in accordance with Article 5.8.6.1.1. Potential differences between the environment at the face relative to the environment within the reinforced soil mass shall be considered when assessing potential corrosion losses.

5.8.7.2 Connection Strength for Geosynthetic Reinforcements

To evaluate the long-term geosynthetic strength at the connection with the wall facing, reduce T_{ult} using the connection/seam strength determined in accordance with ASTM D 4884 for structural (i.e., not partial or full friction) connections. ASTM D 4884 will produce a short-term connection strength equal to $T_{ult} \times CR_u$. (See Equation 5.8.7.2-1.) Note that ASTM D 4884 will need to be modified to accommodate geogrid joints such as a Bodkin joint. The portion of the connection embedded in the concrete facing shall be designed in accordance with Article 8.31.

For reinforcements connected to the facing through embedment between facing elements using a partial or full friction connection (e.g., segmental concrete block faced walls), the capacity of the connection shall be reduced from T_{ult} for the backfill reinforcement using the connection strength determined from laboratory tests. (See Equation 5.8.7.2-1.) This connection strength is based on the lessor of the pullout capacity of the connection, the long-term rupture strength of the connection and T_{al} as determined in Article 5.8.6.1.2. An appropriate laboratory testing and interpretation procedure, which is a modification of NCMA Test Method SRWU-1 (Simac, et. al., 1993), is discussed in Appendix A of FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized

TABLE 5.8.7.2A Default and Minimum Values for the Total Geosynthetic Ultimate Limit State Strength Reduction Factor at the Facing Connection, RF_c

Application	Total Reduction Factor, RF_c
All applications, but with product specific data obtained and analyzed in accordance with FHWA Publication No. FHWA SA-96-071 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines"—Appendix B, and FHWA Publication No. FHWA SA-96-072 "Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes."	All reduction factors shall be based on product specific data. RF_{ID} and RF_D shall not be less than 1.1.
Permanent applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 5.8.6.1.2A, provided product specific data is not available. If using polyester reinforcement, the pH regime at the connection must be investigated and determined to be within the pH requirements for a nonaggressive environment. (See Division II, Article 7.3.6.3.)	4.0
Temporary applications not having severe consequences should poor performance or failure occur, nonaggressive soils, and polymers meeting the requirements listed in Table 5.8.6.1.2A, provided product specific data is not available.	2.5

shall be computed from the load combination causing maximum moment at the section.

9.20.2.3 The shear strength, V_{cw}, shall be computed by

$$V_{CW} = (3.5\sqrt{f'_c} + 0.3\, f_{pc})\, b'd + V_p \qquad (9\text{-}29)$$

but d need not be taken less than 0.8h.

9.20.2.4 For a pretensioned member in which the section at a distance h/2 from the face of support is closer to the end of the member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V_{cw}. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

9.20.2.5 The provisions for computing the shear strength provided by concrete, V_{ci} and V_{cw}, apply to normal weight concrete. When lightweight aggregate concretes are used (see definition, concrete, structural lightweight, Article 8.1.3), one of the following modifications shall apply:

(a) When f_{ct} is specified, the shear strength, V_{ci} and V_{cw}, shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_{ci}}$, but the value of $f_{ct}/6.7$ used shall not exceed $\sqrt{f'_c}$.
(b) When f_{ct} is not specified, V_{ci} and V_{cw} shall be modified by multiplying each term containing $\sqrt{f'_c}$ by 0.75 for "all lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation may be used when partial sand replacement is used.

9.20.3 Shear Strength Provided by Web Reinforcement

9.20.3.1 The shear strength provided by web reinforcement shall be taken as:

$$V_s = \frac{A_v f_{sy} d}{s} \qquad (9\text{-}30)$$

where A_v is the area of web reinforcement within a distance s. V_s shall not be taken greater than $8\sqrt{f'_c}\, b'\, d$ and d need not be taken less than 0.8h.

9.20.3.2 The spacing of web reinforcing shall not exceed 0.75h or 24 inches. When V_s exceeds $4\sqrt{f'_c}\, b'\, d$, this maximum spacing shall be reduced by one-half.

9.20.3.3 The minimum area of web reinforcement shall be

$$A_v = \frac{50\, b's}{f_{sy}} \qquad (9\text{-}31)$$

where b′ and s are in inches and f_{sy} is in psi.

9.20.3.4 The design yield strength of web reinforcement, f_{sy}, shall not exceed 60,000 psi.

9.20.4 Horizontal Shear Design—Composite Flexural Members

9.20.4.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

9.20.4.2 Design of cross sections subject to horizontal shear may be in accordance with provisions of Article 9.20.4.3 or 9.20.4.4, or any other shear transfer design method that results in prediction of strength in substantial agreement with results of comprehensive tests.

9.20.4.3 Design of cross sections subject to horizontal shear may be based on:

$$V_u \le \phi V_{nh} \qquad (9\text{-}31a)$$

where V_u is factored shear force at section considered, V_{nh} is nominal horizontal shear strength in accordance with the following, and where d is for the entire composite section.

(a) When contact surface is clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_vd$, in pounds.
(b) When minimum ties are provided in accordance with Article 9.20.4.5, and contact surface is clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $80b_vd$, in pounds.
(c) When minimum ties are provided in accordance with Article 9.20.4.5, and contact surface is clean, free of laitance, and intentionally roughened to a full amplitude of approximately ¼ inch, shear strength V_{nh} shall not be taken greater than $350b_vd$, in pounds.
(d) For each percent of tie reinforcement crossing the contact surface in excess of the minimum required by Article 9.20.4.5, shear strength V_{nh} may be increased by $(160f_y/40{,}000)b_vd$, in pounds.

9.20.4.4 Horizontal shear may be investigated by computing, in any segment not exceeding one-tenth of the span, the change in compressive or tensile force to be transferred, and provisions made to transfer that force as horizontal shear between interconnected elements. The factored horizontal shear force shall not exceed horizontal shear strength ϕV_{nh} in accordance with Article 9.20.4.3, except that length of segment considered shall be substituted for d.

9.20.4.5 Ties for Horizontal Shear

(a) When required, a minimum area of tie reinforcement shall be provided between interconnected elements. Tie area shall not be less than 50 $b_v s/f_y$, and tie spacing "s" shall not exceed four times the least web width of support element, nor 24 inches.
(b) Ties for horizontal shear may consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric. All ties shall be adequately anchored into interconnected elements by embedment or hooks.

9.21 POST-TENSIONED ANCHORAGE ZONES

9.21.1 Geometry of the Anchorage Zone

9.21.1.1 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 linear stress distribution across the entire cross section.

9.21.1.2 For anchorage zones at the end of a member or segment, the transverse dimensions may be taken as the depth and width of the section. The longitudinal extent of the anchorage zone in the direction of the tendon (ahead of the anchorage) shall be taken as not less than the larger transverse dimension but not more than 1½ times that dimension.

9.21.1.3 For intermediate anchorages in addition to the length of Article 9.21.1.2 the anchorage zone shall be considered to also extend in the opposite direction for a distance not less than the larger transverse dimension.

9.21.1.4 For multiple slab anchorages, both width and length of the anchorage zone shall be taken as equal to the center-to-center spacing between stressed tendons, but not more than the length of the slab in the direction of the tendon axis. The thickness of the anchorage zone shall be taken equal to the thickness of the slab.

9.21.1.5 For design purposes, the anchorage zone shall be considered as comprised of two regions; the *general zone* as defined in Article 9.21.2.1 and the *local zone* as defined in Article 9.21.2.2.

9.21.2 General Zone and Local Zone

9.21.2.1 General Zone

9.21.2.1.1 The geometric extent of the general zone is identical to that of the overall anchorage zone as defined in Article 9.21.1 and includes the local zone.

9.21.2.1.2 Design of general zones shall meet the requirements of Articles 9.14 and 9.21.3.

9.21.2.2 Local Zone

9.21.2.2.1 The local zone is defined as the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete surrounding and immediately ahead of the anchorage device and any integral confining reinforcement. The dimensions of the local zone are defined in Article 9.21.7.

9.21.2.2.2 Design of local zones shall meet the requirements of Articles 9.14 and 9.21.7 or shall be based on the results of experimental tests required in Article 9.21.7.3 and described in Article 10.3.2.3 of Division II. Anchorage devices based on the acceptance test of Division II, Article 10.3.2.3, are referred to as *special anchorage devices.*

9.21.2.3 Responsibilities

9.21.2.3.1 The engineer of record is responsible for the overall design and approval of working drawings for the general zone, including the specific location of the tendons and anchorage devices, general zone reinforcement, and the specific stressing sequence. The engineer of record is also responsible for the design of local zones based on Article 9.21.7.2 and for the approval of special anchorage devices used under the provisions of Article 9.21.7.3. All working drawings for the local zone must be approved by the engineer of record.

9.21.2.3.2 Anchorage device suppliers are responsible for furnishing anchorage devices which satisfy the anchor efficiency requirements of Division II, Article 10.3.2. In addition, if special anchorage devices are used, the anchorage device supplier is responsible for furnishing anchorage devices that satisfy the acceptance test requirements of Article 9.21.7.3 and of Division II, Article 10.3.2.3. This acceptance test and the anchor efficiency test shall be conducted by an independent testing agency acceptable to the engineer of record. The anchorage device supplier shall provide records of the acceptance test in conformance with Division II, Article 10.3.2.3.12 to the

9.27.4 Couplings of unbonded tendons shall be used only at locations specifically indicated and/or approved by the Engineer. Couplings shall not be used at points of sharp tendon curvature. All couplings shall develop at least 95% of the minimum specified ultimate strength of the prestressing steel without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplings and/or coupling components shall be enclosed in housings long enough to permit the necessary movements. All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

9.27.5 Anchorages, end fittings, couplers, and exposed tendons shall be permanently protected against corrosion.

9.28 EMBEDMENT OF PRESTRESSED STRAND

9.28.1 Three- or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length in inches not less than

$$1.6\left(f_{su}^{*} - \frac{2}{3} f_{se}\right) D \qquad (9\text{-}42)$$

where D is the nominal diameter in inches, f_{su}^{*} and f_{se} are in kips per square inch, and the parenthetical expression is considered to be without units.

9.28.2 Investigations may be limited to those cross sections nearest each end of the member which are required to develop their full ultimate capacity.

9.28.3 Where strand is debonded at the end of a member and tension at service load is allowed in the precompressed tensile zone, the development length required above shall be doubled.

9.29 BEARINGS

Bearing devices for prestressed concrete structures shall be designed in accordance with Article 10.29 and Section 14.

F'_e = Euler stress divided by a factor of safety (Article 10.36)

F_{ncf} = design stress for the noncontrolling flange at a point of splice (Article 10.18.2.2.3)

F_{ncu} = design stress for the noncontrolling flange at a point of splice (Article 10.18.2.2.1)

F_p = computed bearing stress due to design load (Table 10.32.3B)

F_s = limiting bending stress (Article 10.34.4)

F_{sr} = allowable range of stress (Table 10.3.1A)

F'_t = reduced allowable tensile stress on rivet or bolt due to the applied shear stress, ksi (Articles 10.32.3.3.4 and 10.56.1.3.3)

F^r_y = specified minimum yield point of the reinforcing steel (Article 10.38.5.1.2)

F.S. = factor of safety (Table 10.32.1A and Articles 10.32.1 and 10.36)

F_u = specified minimum tensile strength (Tables 10.2A, 10.32.1A and 10.32.3B and Article 10.18.4)

F_u = tensile strength of electrode classification (Table 10.56A and Article 10.32.2)

F_u = maximum bending strength of the flange (Articles 10.48.8.2, 10.50.1.2.1, and 10.50.2.2)

F_v = allowable shear stress (Table 10.32.1A and 10.32.3B and Articles 10.18.2.3.6, 10.32.2, 10.32.3, 10.34.4, 10.38.1.7, and 10.40.2.2)

F_v = shear strength of a fastener (Article 10.56.1.3)

F_{vc} = combined tension and shear in bearing-type connections (Article 10.56.1.3)

F_w = design shear stress in the web at a point of splice (Articles 10.18.2.3.6, 10.18.2.3.7, and 10.18.2.3.9)

F_y = specified minimum yield point of steel (Articles 10.15.2.1, 10.15.3, 10.16.11, 10.32.1, 10.32.4, 10.34, 10.35, 10.37.1.3, 10.38.1.7, 10.38.5, 10.39.4, 10.40.2.2, 10.41.4.6, 10.46, 10.48, 10.49, 10.50, 10.51.5, 10.54, and 10.61.4)

F_{yf} = specified minimum yield strength of the flange (Articles 10.18.2.2.1, 10.48.1.1, 10.53.1, 10.57.1, and 10.57.2)

$F_{y\ stiffener}$ = specified minimum yield strength of a transverse stiffener (Articles 10.34.4.7 and 10.48.5.3)

F_{yw} = specified minimum yield strength of the web (Articles 10.18.2.2.1, 10.18.2.2.2, 10.18.2.3.4, 10.53.1, and 10.61.1)

$F_{y\ web}$ = specified mimimum yield strength of the web (Articles 10.34.4.7 and 10.48.5.3)

f = the lesser of (f_b/R_b) or F_y (Articles 10.48.2.1(b), 10.48.2.2, and 10.53)

f_a = computed axial compression stress (Articles 10.35.2.10, 10.36, 10.37, 10.55.2, and 10.55.3)

f_b = computed compressive bending stress (Articles 10.34.2, 10.34.3, 10.34.5.2, 10.37, 10.39, and 10.55)

f_b = factored bending stress in the compression flange (Articles 10.48, 10.48.2.1(b), 10.48.4.1, 10.50.1.2.1, 10.50.2.2, 10.53, and 10.53.1.2)

f_b = maximum factored noncomposite dead load compressive bending stress in the web (Article 10.61.1)

f'_c = unit ultimate compressive strength of concrete as determined by cylinder tests at age of 28 days, psi (Articles 10.38.1, 10.38.5.1.2, 10.45.3, and 10.50.1.1.1)

f_{cf} = maximum flexural stress at the mid-thickness of the flange under consideration at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8)

f_{cu} = maximum flexural stress due to the factored loads at the mid-thickness of the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4)

f_{DL} = noncomposite dead load stress in the compression flange (Articles 10.34.5.1 and 10.49.3.2(a))

f_{DL} = top flange compressive stress due to the factored noncomposite dead load divided by the factor R_b (Article 10.61.4)

f_{DL+LL} = total noncomposite and composite dead-load plus composite live-load stress in the compression flange at the most highly stressed section of the web (Articles 10.34.5.1 and 10.49.3.2(a))

$f_{d\ell 1}$ = top flange compressive stress due to noncomposite dead load (Articles 10.34.2.1 and 10.34.2.2)

f_{ncf} = flexural stress at the mid-thickness of the noncontrolling flange concurrent with f_{cf} (Articles 10.18.2.2.3 and 10.18.2.3.8)

f_{ncu} = flexural stress due to the factored loads at the mid-thickness of the noncontrolling flange at a point of splice concurrent with f_{cu} (Articles 10.18.2.2.1 and 10.18.2.3.4)

f_o = maximum flexural stress due to $D + \beta_L(L + I)$ at the mid-thickness of the flange under consideration at a point of splice (Articles 10.18.2.2.2 and 10.18.2.3.5)

f_{of} = flexural stress due to $D + \beta_L(L + I)$ at the mid-thickness of the other flange at a point of splice concurrent with f_o in the flange under consideration (Article 10.18.2.3.5)

f_r = range of stress due to live load plus impact, in the slab reinforcement over the support (Article 10.38.5.1.3)
f_r = modulus of rupture of concrete specified in Article 8.15.2.1.1 (Article 10.50.2.3)
f_s = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffener (Article 10.39.4.4)
f_s = factored bending stress in either the top or bottom flange, whichever flange has the larger ratio of (f_s/F_u) (Article 10.48.8.2)
f_t = tensile stress due to applied loads (Articles 10.32.3.3.3 and 10.56.1.3.2)
f_t = allowable tensile stress in the concrete specified in Article 8.15.2.1.1 (Article 10.38.4.3)
f_v = unit shear stress (Articles 10.32.3.2.3, 10.34.4.4, and 10.34.4.7)
f_v = maximum shear stress in the web at a point of splice (Article 10.18.2.3.6)
f_{bx} = computed compressive bending stress about the x axis (Article 10.36)
f_{by} = computed compressive bending stress about the y axis (Article 10.36)
g = gage between fasteners, in. (Articles 10.16.14, 10.24.5, and 10.24.6)
H = height of stud, in. (Article 10.38.5.1.1)
H_w = horizontal design force resultant in the web at a point of splice (Articles 10.18.2.3.8 and 10.18.2.3.9)
H_{wo} = overload horizontal design force resultant in the web at a point of splice (Article 10.18.2.3.5)
H_{wu} = horizontal design force resultant in the web at a point of splice (Articles 10.18.2.3.4 and 10.18.2.3.5)
h = average flange thickness of the channel flange, in. (Article 10.38.5.1.2)
I = moment of inertia, $in.^4$ (Articles 10.34.4, 10.34.5, 10.38.5.1.1, 10.48.5.3, and 10.48.6.3)
I_s = moment of inertia of stiffener (Articles 10.37.2, 10.39.4.4.1, and 10.51.5.4)
I_t = moment of inertia of transverse stiffeners (Article 10.39.4.4.2)
I_y = moment of inertia of member about the vertical axis in the plane of the web, in^4 (Article 10.48.4.1)
I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in^4 (Table 10.32.1A and Article 10.48.4.1)
J = required ratio of rigidity of one transverse stiffener to that of the web plate (Articles 10.34.4.7 and 10.48.5.3)
J = St. Venant torsional constant, in^4 (Table 10.32.1A and Article 10.48.4.1)
K = effective length factor in plane of buckling (Table 10.32.1A and Articles 10.37, 10.54.1, and 10.54.2)
K_b = effective length factor in the plane of bending (Article 10.36)
k = constant: 0.75 for rivets; 0.6 for high-strength bolts with thread excluded from shear plane (Article 10.32.3.3.4)
k = buckling coefficient (Articles 10.34.3.2.1, 10.34.4, 10.39.4.3, 10.48.4.1, 10.48.8, 10.51.5.4, and 10.61.1)
k = distance from outer face of flange to toe of web fillet of member to be stiffened (Article 10.56.3)
k_l = buckling coefficient (Article 10.39.4.4)
L = distance between bolts in the direction of the applied force (Table 10.32.3B)
L = actual unbraced length (Table 10.32.1A and Articles 10.7.4, 10.15.3, and 10.55.1)
L = 1/2 of the length of the arch rib (Article 10.37.1)
L = distance between transverse beams (Article 10.41.4.6)
L_b = unbraced length (Table 10.48.2.1.A and Articles 10.36, 10.48.1.1, 10.48.2.1, 10.48.4.1, and 10.53.1.3)
L_c = length of member between points of support, in. (Article 10.54.1.1)
L_c = clear distance between the holes, or between the hole and the edge of the material in the direction of the applied bearing force, in. (Table 10.32.3B and Article 10.56.1.3.2)
L_p = limiting unbraced length (Article 10.48.4.1)
L_r = limiting unbraced length (Article 10.48.4.1)
ℓ = member length (Table 10.32.1A and Article 10.35.1)
M = maximum bending moment (Articles 10.48.8, 10.54.2.1, and 10.50.1.1.2)
M_1 = smaller moment at the end of the unbraced length of the member (Article 10.48.1.1(c))
M_1 & M_2 = moments at two adjacent braced points (Tables 10.32.1A and 10.36A and Articles 10.48.4.1 and 10.50.2.2)
M_c = column moment (Article 10.56.3.2)
M_p = full plastic moment of the section (Articles 10.50.1.1.2 and 10.54.2.1)
M_r = lateral torsional buckling moment or yield moment (Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2, and 10.53.1.3)
M_s = elastic pier moment for loading producing maximum positive moment in adjacent span (Article 10.50.1.1.2)
M_u = maximum bending strength (Articles 10.18.2.2.1, 10.48, 10.49, 10.50.1, 10.50.2, 10.51.1, 10.53.1, 10.54.2.1, and 10.61.3)

M_v = design moment due to the eccentricity of the design shear at a point of splice (Articles 10.18.2.3.7 and 10.18.2.3.9)
M_{vo} = overload design moment due to the eccentricity of the overload design shear at a point of splice (Article 10.18.2.3.5)
M_{vu} = design moment due to the eccentricity of the design shear at a point of splice (Articles 10.18.2.3.3 and 10.18.2.3.5)
M_w = design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Articles 10.18.2.3.8 and 10.18.2.3.9)
M_{wo} = overload design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Article 10.18.2.3.5)
M_{wu} = design moment at a point of splice representing the portion of the flexural moment assumed to be resisted by the web (Articles 10.18.2.3.4 and 10.18.2.3.5)
M_y = moment capacity at first yield (Articles 10.18.2.2.1, 10.50.1.1.2, and 10.61.3)
N_1 & N_2 = number of shear connectors (Article 10.38.5.1.2)
N_c = number of additional connectors for each beam at point of contraflexure (Article 10.38.5.1.3)
N_s = number of slip planes in a slip-critical connection (Articles 10.32.3.2.1 and 10.57.3.1)
N_w = number of roadway design lanes (Article 10.39.2)
n = ratio of modulus of elasticity of steel to that of concrete (Article 10.38.1)
n = number of longitudinal stiffeners (Articles 10.39.4.3, 10.39.4.4, and 10.51.5.4)
P = allowable compressive axial load on members (Article 10.35.1)
P = axial compression on the member (Articles 10.48.1.1, 10.48.2.1, and 10.54.2.1)
P, P_1, P_2, & P_3 = force in the slab (Article 10.38.5.1.2)
P_{cf} = design force in the controlling flange at a point of splice (Article 10.18.2.2.3)
P_{cu} = design force for the controlling flange at a point of splice (Article 10.18.2.2.1)
P_{fo} = overload design force in the flange at a point of splice (Article 10.18.2.2.2)
P_{ncf} = design force for the noncontrolling flange at a point of splice (Article 10.18.2.2.3)
P_{ncu} = design force in the noncontrolling flange at a point of splice (Article 10.18.2.2.1)
P_o = design force for checking slip of a bolted splice in a tension member (Article 10.18.4.2)
P_s = allowable slip resistance (Article 10.32.3.2.1)
P_u = maximum axial compression capacity (Article 10.54.1.1)
P_u = design force for checking the strength of a bolted splice in a tension member (Article 10.18.4.1)
p = allowable bearing (Article 10.32.4.2)
Q = prying tension per bolt (Articles 10.32.3.3.2 and 10.56.2)
Q = statical moment about the neutral axis (Article 10.38.5.1.1)
R = radius (Article 10.15.2.1)
R = number of design lanes per box girder (Article 10.39.2.1)
R = reduction factor for hybrid girders (Articles 10.18.2.2.1, 10.18.2.2.2, 10.18.2.2.3, 10.18.2.3.4, 10.18.2.3.8, 10.40.2.1.1, 10.53.1.2, and 10.53.1.3)
R = reduction factor applied to the design shear strength of fasteners passing through fillers (Article 10.18.1.2.1)
R_b = bending capacity reduction factor (Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2, 10.53.1.2, 10.53.1.3, and 10.61.4)
R_{cf} = absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange at a point of splice (Articles 10.18.2.2.3 and 10.18.2.3.8)
R_{cu} = the absolute value of the ratio of F_{cu} to f_{cu} for the controlling flange at a point of splice (Articles 10.18.2.2.1 and 10.18.2.3.4)
Rev = a range of stress involving both tension and compression during a stress cycle (Table 10.3.1B)
R_s = vertical force at connections of vertical stiffeners to longitudinal stiffeners (Article 10.39.4.4.8)
R_w = vertical web force (Article 10.39.4.4.7)
r = radius of gyration, in (Articles 10.35.1, 10.37.1, 10.41.4.6, 10.48.6.3, 10.54.1.1, 10.54.2.1, and 10.55.1)
r_b = radius of gyration in plane of bending, in. (Article 10.36)
r_y = radius of gyration with respect to the Y-Y axis, in. (Article 10.48.1.1)
r' = radius of gyration of the compression flange about the axis in the plane of the web, in. (Table 10.32.1A and Article 10.48.4.1)
S = allowable rivet or bolt unit stress in shear (Article 10.32.3.3.4)
S = section modulus, in.^{3} (Articles 10.48.2, 10.51.1, 10.53.1.2, and 10.53.1.3)
S = pitch of any two successive holes in the chain (Article 10.16.14.2)
S_r = range of horizontal shear (Article 10.38.5.1.1)

S_s = section modulus of transverse stiffener, in.^{3} (Articles 10.39.4.4 and 10.48.6.3)
S_t = section modulus of longitudinal or transverse stiffener, in.^{3} (Article 10.48.6.3)
S_u = ultimate strength of the shear connector (Article 10.38.5.1.2)
S_{xc} = section modulus with respect to the compression flange, in.^{3} (Table 10.32.1A and Articles 10.48.2, 10.48.4.1, 10.50.1.2.1, 10.50.2.2 and 10.53.1.2)
S_{xt} = section modulus with respect to the tension flange, in.^{3} (Articles 10.48.2 and 10.53.1.2)
s = computed rivet or bolt unit stress in shear (Article 10.32.3.3.4)
T = range in tensile stress (Table 10.3.1B)
T = direct tension per bolt due to external load (Articles 10.32.3 and 10.56.2)
T = arch rib thrust at the quarter point from dead+live+impact loading (Articles 10.37.1 and 10.55.1)
t = thickness of the thinner outside plate or shape (Article 10.35.2)
t = thickness of members in compression (Article 10.35.2)
t = thickness of thinnest part connected, in (Articles 10.32.3.3.2 and 10.56.2)
t = computed rivet or bolt unit stress in tension, including any stress due to prying action (Article 10.32.3.3.4)
t = thickness of the wearing surface, in. (Article 10.41.2)
t = flange thickness, in. (Articles 10.18.2.2.4, 10.34.2.1, 10.34.2.2, 10.39.4.2, 10.48, 10.48.1.1, 10.48.2, 10.48.2.1, 10.51.5.1, and 10.61.4)
t = thickness of a flange angle (Article 10.34.2.2)
t = thickness of the web of a channel, in. (Article 10.38.5.1.2)
t = thickness of stiffener (Articles 10.34.4.7 and 10.48.5.3)
t_b = thickness of flange delivering concentrated force (Article 10.56.3.2)
t_c = thickness of flange of member to be stiffened (Article 10.56.3.2)
t_f = thickness of the flange (Articles 10.37.3, 10.55.3, and 10.39.4.3)
t_h = thickness of the concrete haunch above the beam or girder top flange (Article 10.50.1.1.2)
t_s = thickness of stiffener (Article 10.37.2 and 10.55.2)
t_s = slab thickness (Articles 10.38.5.1.2, 10.50.1.1.1, and 10.50.1.1.2)
t_w = web thickness, in. (Articles 10.15.2.1, 10.18.2.3.4, 10.18.2.3.7, 10.18.2.3.8, 10.18.2.3.9, 10.34.3, 10.34.4, 10.34.5, 10.37.2, 10.48, 10.49.2, 10.49.3, 10.55.2, 10.56.3, and 10.61.1)
t_{tf} = thickness of top flange (Article 10.50.1.1.1)
t' = thickness of outstanding stiffener element (Articles 10.39.4.5.1 and 10.51.5.5)
V = shearing force (Articles 10.35.1, 10.48.5.3, 10.48.8, and 10.51.3)
V = maximum shear in the web at a point of splice due to the factored loads (Article 10.18.2.3.2)
V_o = maximum shear in the web at the point of splice due to $D + \beta_L (L + I)$ (Article 10.18.2.3.5)
V_p = shear yielding strength of the web (Articles 10.48.8 and 10.53.1.4)
V_r = range of shear due to live loads and impact, kips (Article 10.38.5.1.1)
V_u = maximum shear force (Articles 10.18.2.3.2, 10.34.4, 10.48.5.3, 10.48.8, and 10.53.3)
V_v = vertical shear (Article 10.39.3.1)
V_w = design shear for a web (Articles 10.39.3.1 and 10.51.3)
V_w = design shear in the web at a point of splice (Articles 10.18.2.3.2, 10.18.2.3.3, and 10.18.2.3.5)
V_{wo} = overload design shear in the web at a point of splice (Article 10.18.2.3.5)
V_{wu} = design shear in the web at a point of splice (Articles 10.18.2.3.2, 10.18.2.3.3, and 10.18.2.3.5)
W = length of a channel shear connector, in. (Article 10.38.5.1.2)
W_c = roadway width between curbs in feet or barriers if curbs are not used (Article 10.39.2.1)
W_n = least net width of a flange (Article 10.18.2.2.4)
W_L = fraction of a wheel load (Article 10.39.2)
w = length of a channel shear connector in inches measured in a transverse direction on the flange of a girder (Article 10.38.5.1.1)
w = unit weight of concrete, lb per cu ft (Article 10.38.5.1.2)
w = width of flange between longitudinal stiffeners (Articles 10.39.4.3, 10.39.4.4, and 10.51.5.4)
Y_o = distance from the neutral axis to the extreme outer fiber, in. (Article 10.15.3)
$\bar{y}$ = location of steel sections from neutral axis (Article 10.50.1.1.1)
Z = plastic section modulus (Articles 10.48.1, 10.53.1.1, and 10.54.2.1)
Z_r = allowable range of horizontal shear, in pounds on an individual connector (Article 10.38.5.1)
α = constant based on the number of stress cycles (Article 10.38.5.1.1)

TABLE 10.32.1A Allowable Stresses—Structural Steel (In pounds per square inch) (Continued)

Type		Structural Carbon Steel	High-Strength Low-Alloy Steel		Quenched and Tempered Low-Alloy Steel	High-Yield Strength Quenched and Tempered Alloy Steel[a]	
when $KL/r > C_c$ $F_a = \frac{\pi^2 E}{F.S.(KL/r)^2} =$ with F.S. = 2.12			$\frac{135,008,740}{(KL/r)^2}$				
Shear in girder webs, gross section	$F_v = 0.33F_y$	12,000	17,000	17,000	23,000	33,000	30,000
Bearing on milled stiffeners and other steel parts in contact (rivets and bolts excluded)	$0.80F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Stress in extreme fiber of pins[g]	$0.80F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Shear in pins	$F_v = 0.40F_y$	14,000	20,000	20,000	28,000	40,000	36,000
Bearing on pins not subject to rotation[h]	$0.80F_y$	29,000	40,000	40,000	56,000	80,000	72,000
Bearing on pins subject to rotation (such as used in rockers and hinges)	$0.40F_y$	14,000	20,000	20,000	28,000	40,000	36,000
Bearing on connected material at Low Carbon Steel Bolts (ASTM A 307), Turned Bolts, Ribbed Bolts, and Rivets (ASTM A 502 Grades 1 and 2)—Governed by Table 10.32.3A							

[a] Quenched and tempered alloy steel structural shapes and seamless mechanical tubing meeting all mechanical and chemical requirements of A 709 Grades 100/100W except that the specified maximum tensile strength may be 140,000 psi for structural shapes and 145,000 psi for seamless mechanical tubing, shall be considered as A 709 Grades 100/100W steel.

[b] Except for the mandatory notch toughness and weldability requirements, the ASTM designations are similar to the AASHTO designations. Steels meeting the AASHTO requirements are prequalified for use in welded bridges.

[c] M 270 Gr. 36 and A 709 Gr. 36 are equivalent to M 183 and A 36
M 270 Gr. 50 and A 709 Gr. 50 are equivalent to M 223 Gr. 50 and A 572 Gr. 50
M 270 Gr. 50W and A 709 Gr. 50W are equivalent to M 222 and A 588
M 270 Gr. 70W and A 709 Gr. 70W are equivalent to A 852
M 270 Gr. 100/100W and A 709 Gr. 100/100W are equivalent to M 244 and A 514

[d] For the use of larger C_b values, see Structural Stability Research Council *Guide to Stability Design Criteria for Metal Structures*, 3rd Ed., p. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

[e] ℓ = length in inches, of unsupported flange between lateral connections, knee braces, or other points of support.
I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web in.^{4}
d = depth of girder, in.
$J = \frac{[(bt^3)_c + (bt^3)_t + Dt_w^3]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, respectively (in.^{4}).
S_{xc} = section modulus with respect to compression flange (in.^{3}).

[f] E = modulus of elasticity of steel
r = governing radius of gyration
L = actual unbraced length
K = effective length factor (see Appendix C.)
F.S. = factor of safety = 2.12
For graphic representation of these formulas, see Appendix C.
The formulas do not apply to members with variable moment of inertia. Procedures for designing members with variable moments of inertia can be found in the following references: "Engineering Journal," American Institute of Steel Construction, January 1969, Volume 6, No. 1, and October 1972, Volume 9, No. 4; and "Steel Structures," by William McGuire, 1968, Prentice-Hall, Inc., Englewood Cliffs, New Jersey. For members with eccentric loading, see Article 10.36. Singly symmetric and unsymmetric compression members, such as angles or tees, and doubly symmetric compression members, such as cruciform or built-up members with very thin walls, may also require consideration of flexural-torsional and torsional buckling. Refer to the *Manual of Steel Construction*, Ninth Edition, 1989, American Institute of Steel Construction.

[g] See also Article 10.32.4.

[h] This shall apply to pins used primarily in axially loaded members, such as truss members and cable adjusting links. It shall not apply to pins used in members having rotation caused by expansion or deflection.

TABLE 10.32.3A Allowable Stresses for Low-Carbon Steel Bolts and Power Driven Rivets (in psi)

Type of Fastener	Tension[a]	Bearing[b]	Shear Bearing-Type Connection[a]
(A) Low-Carbon Steel Bolts[c] Turned Bolts (ASTM A 307) Ribbed Bolts	18,000	20,000	11,000[d]
(B) Power-Driven Rivets (rivets driven by pneumatically or electrically operated hammers are considered power driven)			
Structural Steel Rivet Grade 1 (ASTM A 502 Grade 1)	—	40,000	13,500
Structural Steel Rivet (high-strength) Grade 2 (ASTM A 502 Grade 2)	—	40,000	20,000

[a] Applies to fastener cross-sectional area based upon nominal body diameter.

[b] Applies to nominal diameter of fastener multiplied by the thickness of the metal.

[c] ASTM A 307 bolts shall not be used in connections subject to fatigue.

[d] In connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 inches, the tabulated value shall be reduced 20 percent.

10.32.3 Fasteners (Rivets and Bolts)

Allowable stresses for fasteners shall be as listed in Tables 10.32.3.A and 10.32.3.B, and the allowable force on a slip-critical connection shall be as provided by Article 10.32.3.2.1.

10.32.3.1 General

10.32.3.1.1 In proportioning fasteners for shear or tension, the cross-sectional area based upon the nominal diameter shall be used except as otherwise noted.

10.32.3.1.2 The effective bearing area of a fastener shall be its diameter multiplied by the thickness of the metal on which it bears. In metal less than ⅜ inch thick, countersunk fasteners shall not be assumed to carry stress. In metal ⅜ inch thick and over, one-half of the depth of the countersink shall be omitted in calculating the bearing area.

10.32.3.1.3 In determining whether the bolt threads are excluded from the shear planes of the contact surfaces, thread length of bolts shall be calculated as two thread pitches greater than the specified thread length as an allowance for thread runout.

10.32.3.1.4 In bearing-type connections, pull-out shear in a plate should be investigated between the end of the plate and the end row of fasteners. (See Table 10.32.3B, footnote g.)

10.32.3.1.5 All bolts except high-strength bolts, tensioned to the requirements of Division II. Table 11.5A or Table 11.5B, shall have single self-locking nuts or double nuts.

10.32.3.1.6 Joints, utilizing high-strength bolts, required to resist shear between their connected parts are designated as either slip-critical (See Article 10.24.1.4) or bearing-type connections. Shear connections subjected to stress reversal, or where slippage would be undesirable, shall be slip-critical connections. Potential slip

TABLE 10.32.3B Allowable Stresses on High-Strength Bolts or Connected Material (ksi)

Load Condition	AASHTO M 164 (ASTM A 325)[a]	AASHTO M 253 (ASTM A 490)[a]
Applied Static Tension[b,c]	38[d]	47
Shear, F_v, on bolt with threads in shear plane[e,f]	19[d]	24
Bearing, F_p, on connected material in standard, oversize, short-slotted holes loaded in any direction, or long-slotted holes parallel to the applied bearing force	$\frac{0.5L_cF_u}{d} \leq F_u$[g,h,i,j]	
Bearing, F_p, on connected material in long-slotted holes perpendicular to the applied bearing force	$\frac{0.4L_cF_u}{d} \leq 0.8F_u$[g,h,i,j]	

[a] AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) high-strength bolts are available in three types, designated as types 1, 2, or 3. Type 3 shall be required on the plans when using unpainted AASHTO M 270 Grade 50W (ASTM A 709 Grade 50W).

[b] Bolts must be tensioned to requirements of Table 11.5A, Div II.

[c] See Article 10.32.3.4 for bolts subject to tensile fatigue.

[d] The tensile strength of M 164 (A 325) bolts decreases for diameters greater than 1 inch. The design values listed are for bolts up to 1 inch diameter. The design values shall be multiplied by 0.875 for diameters greater than 1 inch.

[e] In connections transmitting axial force whose length between extreme fasteners measured parallel to the line of force exceeds 50 inches, tabulated values shall be reduced 20 percent. For flange splices, the 50-inch length is to be measured between the extreme bolts on only one side of the connection.

[f] If material thickness or joint details preclude threads in the shear plane, multiply tabulated values by 1.25.

[g] F_u = specified minimum tensile strength of connected material.

[h] Connections using high-strength bolts in slotted holes with the load applied in a direction other than approximately normal (between 80 and 100 degrees) to the axis of the hole and connections with bolts in oversized holes shall be designed for resistance against slip in accordance with Article 10.32.3.2.1.

[i] L_c is equal to the clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force, in. and d is the nominal diameter of the bolt, in.

[j] The allowable bearing force for the connection is equal to the sum of the allowable bearing forces for the individual bolts in the connection.

Section 12

SOIL-CORRUGATED METAL STRUCTURE INTERACTION SYSTEMS

12.1 GENERAL

12.1.1 Scope

The specifications of this Section are intended for the structural design of corrugated metal structures. It must be recognized that a buried flexible structure is a composite structure made up of the metal ring and the soil envelope, and that both materials play a vital part in the structural design of flexible metal structures.

Only Article 12.7 is applicable to structural plate box culverts.

12.1.2 Notations

A = required wall area (Article 12.2.1)
A = area of pipe wall (Article 12.3.1)
AL = total axle load on single axle or tandem axles (Articles 12.8.4.3.2 and 12.8.4.4)
C_1 = number of axles coefficient (Article 12.8.4.3.2)
C_2 = number of wheels per axle coefficient (Article 12.8.4.3.2)
$C_{\ell\ell}$ = live load adjustment coefficient (Article 12.8.4.3.2)
D = straight leg of haunch (Article 12.8.2)
E_m = modulus of elasticity of metal (Articles 12.2.2 and 12.3.2)
E_m = modulus of elasticity of pipe material (Articles 12.2.4 and 12.3.4)
FF = flexibility factor (Articles 12.2.4 and 12.3.4)
f_a = allowable stress—specified minimum yield point divided by safety factor (Article 12.2.1)
f_{cr} = critical buckling stress (Articles 12.2.2 and 12.3.2)
f_u = specified minimum tensile strength (Articles 12.2.2 and 12.3.2)
f_y = specified minimum yield point (Article 12.3.1)
H = height of cover above crown (Article 12.8.4.4)
I = moment of inertia, per unit length, of cross section of the pipe wall (Articles 12.2.4 and 12.3.4)
k = soil stiffness factor (Articles 12.2.2 and 12.3.2)
$M_{d\ell}$ = dead load factored moment (Article 12.8.4.3.1)
$M_{\ell\ell}$ = live load factored moment (Article 12.8.4.3.2)
M_{pc} = crown plastic moment capacity (Article 12.8.4.3.3)
M_{ph} = haunch plastic moment capacity (Article 12.8.4.3.3)
P = design load (Article 12.1.4)
P = proportion of total moment carried by the crown. Limits for P are given in Table 12.7.4D (Article 12.8.4.3.3)
r = radius of gyration of corrugation (Articles 12.2.2 and 12.3.2)
r_c = radius of crown (Table 12.8.2A)
r_h = radius of haunch (Table 12.8.2A)
R = rise of box culvert (Articles 12.7.2 and 12.8.4.4)
R_h = haunch moment reduction factor (Article 12.8.4.3.3)
S = diameter of span (Articles 12.1.4, 12.2.2, 12.8.2, and 12.8.4.4)
s = pipe diameter or span (Articles 12.2.4, 12.3.2, and 12.3.4)
SF = safety factor (Article 12.2.3)
SS = required seam strength (Articles 12.2.3 and 12.3.3)
T = thrust (Article 12.1.4)
T_L = thrust, load factor (Articles 12.3.1 and 12.3.3)
T_s = thrust, service load (Articles 12.2.1 and 12.2.3)
t = length of stiffening rib on leg (Article 12.8.2)
V = reaction acting in leg direction (Article 12.8.4.4)
Δ = haunch radius included angle (Table 12.8.2A)
γ = unit weight of backfill (Articles 12.8.4.3.2 and 12.8.4.4)
ϕ = capacity modification factor (Articles 12.3.1, 12.3.3, 12.5.3.1, 12.6.1.3, and 12.8.4.2)

12.1.3 Loads

Design load, P, shall be the pressure acting on the structure. For earth pressures, see Article 3.20. For live load, see Articles 3.4 to 3.7, 3.11, 3.12, and 6.4, except that the words “When the depth of fill is 2 feet or more” in Article 6.4.1 need not be considered. For loading combinations, see Article 3.22.

12.1.4 Design

12.1.4.1 The thrust in the wall shall be checked by three criteria. Each considers the mutual function of the metal wall and the soil envelope surrounding it. The criteria are:

(a) Wall area;
(b) Buckling stress;
(c) Seam strength (structures with longitudinal seams).

12.1.4.2 The thrust in the wall is:

$$T = P \times \frac{S}{2} \qquad (12\text{-}1)$$

where:

P = design load, in pounds per square foot;
S = diameter or span, in feet;
T = thrust, in pounds per foot.

12.1.4.3 Handling and installation strength shall be sufficient to withstand impact forces when shipping and placing the pipe.

12.1.5 Materials

The materials shall conform to the AASHTO specifications referenced herein.

12.1.6 Soil Design

12.1.6.1 Soil Parameters

The performance of a flexible culvert is dependent on soil structure interaction and soil stiffness.

The following must be considered:

(a) Soils:
(1) The type and anticipated behavior of the foundation soil must be considered; i.e., stability for bedding and settlement under load.
(2) The type, compacted density, and strength properties of the soil envelope immediately adjacent to the pipe must be established. Good side fill is obtained from a granular material with little or no plasticity and free of organic material, i.e., AASHTO classification groups A-1, A-2, and A-3, compacted to a minimum 90% of standard density based on AASHTO Specification T 99 (ASTM D 698).
(3) The density of the embankment material above the pipe must be determined. See Article 6.2.
(b) Dimensions of soil envelope.

The general recommended criteria for lateral limits of the culvert soil envelope are as follows:

(1) *Trench installations*—2-feet minimum each side of culvert. This recommended limit should be modified as necessary to account for variables such as poor in situ soils.
(2) *Embankment installations*—one diameter or span each side of culvert.
(3) The minimum upper limit of the soil envelope is 1 foot above the culvert.

12.1.6.2 Pipe Arch Design

The design of the corner backfill shall account for corner pressure which shall be considered to be approximately equal to thrust divided by the radius of the pipe arch corner. The soil envelope around the corners of pipe arches shall be capable of supporting this pressure.

12.1.6.3 Arch Design

12.1.6.3.1 Special design considerations may be applicable; a buried flexible structure may raise two important considerations. The first is that it is undesirable to make the metal arch relatively unyielding or fixed compared with the adjacent sidefill. The use of massive footings or piles to prevent any settlement of the arch is generally not recommended.

Where poor materials are encountered, consideration should be given to removing some or all of this poor material and replacing it with acceptable material.

The footing should be designed to provide uniform longitudinal settlement, of acceptable magnitude from a functional aspect. Providing for the arch to settle will protect it from possible drag down forces caused by the consolidation of the adjacent sidefill.

The second consideration is bearing pressure of soils under footings. Recognition must be given to the effect of depth of the base of footing and the direction of the footing reaction from the arch.

Footing reactions for the metal arch are considered to act tangential to the metal plate at its point of connection to the footing. The value of the reaction is the thrust in the metal arch plate at the footing.

12.1.6.3.2 Invert slabs and other appropriate measures shall be provided to anticipate scour.

12.4.1.4 Flexibility Factor

(a) For steel conduits, FF should generally not exceed the following values:
¼-in. and ½-in. depth corrugation,
FF = 4.3×10^{-2}
1-in. depth corrugation, FF = 3.3×10^{-2}
(b) For aluminum conduits, FF should generally not exceed the following values:
¼-in. and ½-in. depth corrugations,
FF = 3.1×10^{-2} for 0.060 in. material thickness
FF = 6.1×10^{-2} for 0.075 in. material thickness
FF = 9.2×10^{-2} for all other material thicknesses
1-in. depth corrugation, FF = 6×10^{-2}

12.4.1.5 Minimum Cover

The minimum cover for design loads shall be Span/8 but not less than 12 inches. (The minimum cover shall be measured from the top of a rigid pavement or the bottom of a flexible pavement.) For construction requirements, see Division II, Article 26.6.

12.4.2 Seam Strength

Minimum Longitudinal Seam Strength

2 × 1/2 and 2-2/3 × 1/2 Corrugated Steel Pipe—Riveted or Spot Welded				3 × 1 Corrugated Steel Pipe—Riveted or Spot Welded		
Thickness (in.)	Rivet Size (in.)	Single Rivets (kips/ft)	Double Rivets (kips/ft)	Thickness (in.)	Rivet Size (in.)	Double Rivets (kips/ft)
0.064	5/16	16.7	21.6	0.064	3/8	28.7
0.079	5/16	18.2	29.8	0.079	3/8	35.7
0.109	3/8	23.4	46.8	0.109	7/16	53.0
0.138	3/8	24.5	49.0	0.138	7/16	63.7
0.168	3/8	25.6	51.3	0.168	7/16	70.7

2 × 1/2 and 2-2/3 × 1/2 Corrugated Aluminum Pipe—Riveted			
Thickness (in.)	Rivet Size (in.)	Single Rivets (kips/ft)	Double Rivets (kips/ft)
0.060	5/16	9.0	14.0
0.075	5/16	9.0	18.0
0.105	3/8	15.6	31.5
0.135	3/8	16.2	33.0
0.164	3/8	16.8	34.0

3 × 1 Corrugated Aluminum Pipe—Riveted			6 × 1 Corrugated Aluminum Pipe—Riveted		
Thickness (in.)	Rivet Size (in.)	Double Rivets (kips/ft)	Thickness (in.)	Rivet Size (in.)	Double Rivets (kips/ft)
0.060	3/8	16.5	0.060	1/2	16.0
0.075	3/8	20.5	0.075	1/2	19.9
0.105	1/2	28.0	0.105	1/2	27.9
0.135	1/2	42.0	0.135	1/2	35.9
0.164	1/2	54.5	0.167	1/2	43.5

12.4.3 Section Properties

12.4.3.1 Steel Conduits

	1-1/2 × 1/4 Corrugation			2-2/3 × 1/2 Corrugation		
Thickness (in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.028	0.304					
0.034	0.380					
0.040	0.456	0.0816	0.253	0.465	0.1702	1.121
0.052	0.608	0.0824	0.344	0.619	0.1707	1.500
0.064	0.761	0.0832	0.439	0.775	0.1712	1.892
0.079	0.950	0.0846	0.567	0.968	0.1721	2.392
0.109	1.331	0.0879	0.857	1.356	0.1741	3.425
0.138	1.712	0.0919	1.205	1.744	0.1766	4.533
0.168	2.098	0.0967	1.635	2.133	0.1795	5.725

	3 × 1 Corrugation			5 × 1 Corrugation		
Thickness (in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.064	0.890	0.3417	8.659	0.794	0.3657	8.850
0.079	1.113	0.3427	10.883	0.992	0.3663	11.092
0.109	1.560	0.3448	15.459	1.390	0.3677	15.650
0.138	2.008	0.3472	20.183	1.788	0.3693	20.317
0.168	2.458	0.3499	25.091	2.186	0.3711	25.092

12.4.3.2 Aluminum Conduits

	1-1/2 × 1/4 Corrugation			2-2/3 × 1/2 Corrugation		
Thickness (in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.048	0.608	0.0824	0.344			
0.060	0.761	0.0832	0.349	0.775	0.1712	1.892
0.075				0.968	0.1721	2.392
0.105				1.356	0.1741	3.425
0.135				1.745	0.1766	4.533
0.164				2.130	0.1795	5.725

	3 × 1 Corrugation			6 × 1			
Thickness (in.)	A_s (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)	A_s (sq in./ft)	Effective Area (sq in./ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.060	0.890	0.3417	8.659	0.775	0.387	0.3629	8.505
0.075	1.118	0.3427	10.883	0.968	0.484	0.3630	10.631
0.105	1.560	0.3448	15.459	1.356	0.678	0.3636	14.340
0.135	2.088	0.3472	20.183	1.744	0.872	0.3646	19.319
0.164	2.458	0.3499	25.091	2.133	1.066	0.3656	23.760

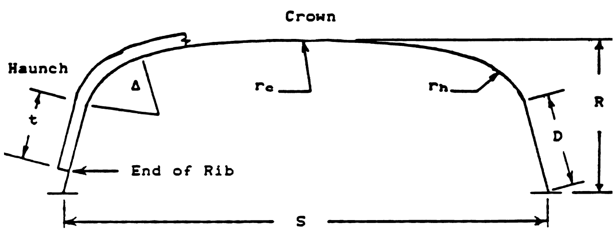


FIGURE 12.8.2A Standard Terminology of Structural Plate Box Culvert Shapes

12.8.4 Design

12.8.4.1 Analytical Basis for Design

Structural requirements for box culverts have been developed from finite element analyses covering the range of structures allowed by Article 12.8.2.

12.8.4.1.1 Structural requirements are based on analyses using two dimensional live loads equivalent to HS 20, 4-wheel, single-axle vehicles. Dead load of soil equals 120 pounds per cubic foot. Coefficients to adjust for other load conditions are contained in Article 12.8.4.3.2.

12.8.4.1.2 Backfill required in Article 12.8.3 is dense granular material. The analyses that provide the basis for this specification were based on conservative soil properties of low plasticity clay (CL) compacted to 90% of standard AASHTO T 99.

12.8.4.2 Load Factor Method

The combined gamma and beta factors to be applied are

Dead load, load factor = 1.5
Live load, load factor = 2.0

The capacity modification factor ϕ is 1.00.

12.8.4.3 Plastic Moment Requirements

Analyses covering the range of box culvert shapes described in Article 12.8.2 have shown moment requirements govern the design in all cases. Effects of thrust were found to be negligible when combined with moment.

Metal box culverts act similar to rigid frames, distributing moment between the crown and haunch on the basis of their relative stiffness. Within limits, increasing the stiffness of one component of the box (either crown or haunch) reduces the portion of the total moment carried by the other.

Article 12.8 provides for this moment distribution within the allowable limits of the moment proportioning factor (P). P represents the proportion of the total moment that can be carried by the crown of the box culvert and varies with the relative moment capacities of the crown and haunch components. Limits for P are given in Table 12.8.4B.

12.8.4.3.1 The sum of the factored crown and haunch dead load moments are

$$M_{d\ell} = \gamma \times 10^{-3}\,\{S^3[0.0053 - 0.00024(S - 12)] + 0.053\,(H - 1.4)S^2\} \times (\text{Dead load, load factor}) \qquad (12\text{-}12)$$

where

$M_{d\ell}$ = The sum of the factored crown and haunch dead load moments (kip-ft/ft)
S = Box culvert span in feet.
γ = Soil density (lbs/ft^3)
H = Height of cover from the box culvert rise to top of pavement (ft)

12.8.4.3.2 The sum of the factored crown and haunch live load moments are

$$M_{\ell\ell} = (C_{\ell\ell}K_1S/K_2) \times (\text{Live load, load factor}) \qquad (12\text{-}13)$$

where

$M_{\ell\ell}$ = The sum of the factored crown and haunch live load moments (kip-ft/ft)
$C_{\ell\ell}$ = Live load adjustment coefficient for axle loads, tandem axles, and axles with other than 4 wheels;

$$C_{\ell\ell} = C_1C_2AL \qquad (12\text{-}14)$$

AL = Total axle load on single axle or tandem axles in kips;
C_1 = Adjustment coefficient for number of axles;
C_1 = 1.0, for single axle;
C_1 = (0.5 + S/50), for tandem axles, ($C_1 \leq 1.0$);
S = Box culvert span in feet;
C_2 = Adjustment coefficient for number of wheels per axle. (Values for C2 are given in Table 12.8.4A.)
H = Height of cover from the box culvert rise to top of pavement (ft.)

$$K_1 = \frac{0.08}{(H/S)^{0.2}},\ \text{for } 8 \leq S < 20 \qquad (12\text{-}15)$$

$$K_1 = \frac{0.08 - 0.002(S - 20)}{(H/S)^{0.2}},\ \text{for } 20 \leq S \leq 26 \qquad (12\text{-}16)$$

$$K_2 = 0.54H^2 - 0.4H + 5.05, \text{ for } 1.4 \leq H < 3.0 \quad (12\text{-}17)$$

$$K_2 = 1.90H + 3, \text{ for } 3.0 \leq H \leq 5.0 \quad (12\text{-}18)$$

TABLE 12.8.4A C_2, Adjustment Coefficient Values for Number of Wheels Per Axle

Wheels per Axle	Cover Depth, ft			
	1.4	2.0	3.0	5.0
2	1.18	1.21	1.24	1.02
4	1.00	1.00	1.00	1.00
8	0.63	0.70	0.82	0.93

12.8.4.3.3 Crown plastic moment capacity (Mpc ϕ), and haunch plastic moment capacity (Mph ϕ), must be equal to or greater than the proportioned sum of load adjusted dead and live load moments.

$$\phi M_{pc} \geq P[(M_{d\ell}) + (M_{\ell\ell})] \quad (12\text{-}19)$$

$$\phi M_{ph} \geq (1.0 - P)[(M_{d\ell}) + (R_h M_{\ell\ell})] \quad (12\text{-}20)$$

where

- P = Proportion of total moment carried by the crown. Limits for P are given in Table 12.8.4D;
- R_h = Haunch moment reduction factor from Table 12.8.4E.

12.8.4.3.4 Article 12.8 can be used to check the adequacy of manufactured products for compliance with the requirements of this specification. Using the actual crown moment capacity provided by the box culvert under consideration and the loading requirements of the application, Equation (12-19) is solved for the factor P. This factor should fall within the allowable range of Table 12.8.4B. Knowing the factor P, Equation (12-20) is then solved for required haunch moment capacity, which should be less than or equal to the actual haunch moment capacity provided.

TABLE 12.8.4B P, Crown Moment Proportioning Values

Span ft	Allowable Range of P
Less Than 10	0.55 to 0.70
10–15	0.50 to 0.70
15–20	0.45 to 0.70
20–26	0.45 to 0.60

TABLE 12.8.4C R_h, Haunch Moment Reduction Values

Cover Depth, ft			
1.4	2	3	4 to 5
0.66	0.74	0.87	1.00

If Equation (12-19) indicates a higher P factor than permitted by the ranges of Table 12.8.4B, the actual crown is over designed, which is acceptable. However, in this case only the maximum value of P allowed by the table shall be used to calculate the required haunch moment capacity from Equation (12-20).

12.8.4.4 Footing Reactions

The reaction at the box culvert footing may be computed using the following equation

$$V = \gamma(HS/2{,}000 + S^2/40{,}000) + AL/[8 + 2(H + R)] \quad (12\text{-}21)$$

where

- V = Reaction in kips per foot acting in the direction of the box culvert straight side;
- γ = Backfill unit weight in pounds per cubic foot;
- H = Height of cover over the crown in feet;
- S = Span of box culvert in feet;
- AL = Axle load in kips;
- R = Rise of box culvert in feet.

12.8.5 Manufacturing and Installation

12.8.5.1 Manufacture and assembly of structural plates shall be in accordance with Division II, Articles 23.3.1.4, 26.3.2, 26.3.3, 26.3.4, and 26.4.1. Reinforcing ribs shall be attached as shown by the manufacturer. Bolts connecting plates, plates to ribs and rib splices shall be torqued to 150-foot pounds.

12.8.5.2 Sidefill and overfill per Article 12.8.3 shall be placed in uniform layers not exceeding 8 inches in compacted thickness at near optimum moisture with equipment and methods which do not damage or distort the box culvert.

12.8.5.3 Following completion of roadway paving, crown deflection due to live load may be checked. After a minimum of 10 loading cycles with the design live load, the change in rise loaded with the design live load relative to the rise unloaded, should not exceed $1/200$ of the box culvert span.

loads imposed on it, but shall not be less than 6.0% of the inside diameter of the pot, D_p, except at the rim.

The diameter of the piston rim shall be the inside diameter of the pot less a clearance, c. The clearance, c, shall be as small as possible in order to prevent escape of the elastomer, but not less than 0.02 inch. If the surface of the piston rim is cylindrical, the clearance shall satisfy

$$c \geq \theta_m \left(w - \frac{D_p \theta_m}{2} \right) \qquad (14.6.4.7\text{-}1)$$

where

D_p = internal diameter of pot (in)
w = height of piston rim (in)
θ_m = design rotation specified in Article 14.4.1 (rad)

14.6.4.8 Lateral Loads

Pot bearings which are subjected to lateral loads shall be proportioned so that the thickness, t, of the pot wall and the pot base shall satisfy

$$t > \sqrt{\frac{40 H_m \theta_m}{F_y}} \qquad (14.6.4.8\text{-}1)$$

For pot bearings which transfer lateral load through the piston

$$w \geq \frac{2.5 H_m}{D_p F_y} \qquad (14.6.4.8\text{-}2)$$

and $w \geq 1/8''$

where w is the rim thickness of the piston which is in contact with the pot wall.

14.6.5 Steel Reinforced Elastomeric Bearings—Method B

14.6.5.1 General

Steel reinforced elastomeric bearings shall consist of alternate layers of steel reinforcement and elastomer, bonded together. Tapered elastomer layers shall not be used. All internal layers of elastomer shall be of the same thickness. The top and bottom cover layers shall be no thicker than 70% of the internal layers. In addition to any internal reinforcement, bearings may have external steel load plates bonded to the upper or lower elastomer layers or both.

14.6.5.2 Material Properties

The elastomer shall have a shear modulus between 0.08 and 0.175 ksi and a nominal hardness between 50 and 60 on the Shore A scale.

The shear modulus of the elastomer at 73°F shall be used as the basis for design. If the elastomer is specified explicitly by its shear modulus, then that value shall be used in design and the other properties shall be obtained from Table 14.6.5.2-1. If the material is specified by its hardness, the shear modulus shall be taken as the least favorable value from the range for that hardness given in Table 14.6.5.2-1. Intermediate values shall in all cases be obtained by interpolation.

For the purposes of bearing design, all bridge sites shall be classified as being in temperature Zones A, B, C, D or E. Characteristics for each zone are given in Table 14.6.5.2-2. In the absence of more precise information, Figure 14.6.5.2-1 may be used as a guide in selecting the zone required for a given region.

Bearings shall be made from AASHTO low temperature grades of elastomer as defined in Section 18 of Division II. The minimum grade of elastomer required for each low temperature zone is specified in Table 14.6.5.2-2.

Any of the three design options listed below may be used:

- specify the elastomer with the minimum low temperature grade indicated in Table 14.6.5.2-2 and determine the shear force transmitted by the bearing as specified in Article 14.5.3.1.
- specify the elastomer with the minimum low temperature grade for use when special force provisions are incorporated in the design and provide a low friction sliding surface, in which case the special force provision is that the bridge components shall be designed to withstand twice the design shear force specified in Article 14.5.3.1, or
- specify the elastomer with the minimum low temperature grade for use when special force provisions are incorporated in the design, but do not provide a low friction sliding surface, in which case the components of the bridge shall be designed to resist four times the design shear force as specified in Article 14.5.3.1.

Table 14.6.5.2-1 Elastomer Properties At Different Hardnesses

Hardness (Shore 'A')	50	60	70
Shear modulus at 73°F (psi)	95–130	130–200	200–300
Creep deflection at 25 yrs / Instantaneous deflection	25%	35%	45%

Table 14.6.5.2-2 Low Temperature Zones and Elastomer Grades

Low Temperature Zone	A	B	C	D	E
50 year low temperature (°F)	0	−20	−30	−45	all others
Max. no. of days below 32°F	3	7	14	N/A	N/A
Low temp. elastomer grade without special provisions	0	2	3	4	5
Low temp. elastomer grade with special provisions	0	0	2	3	5

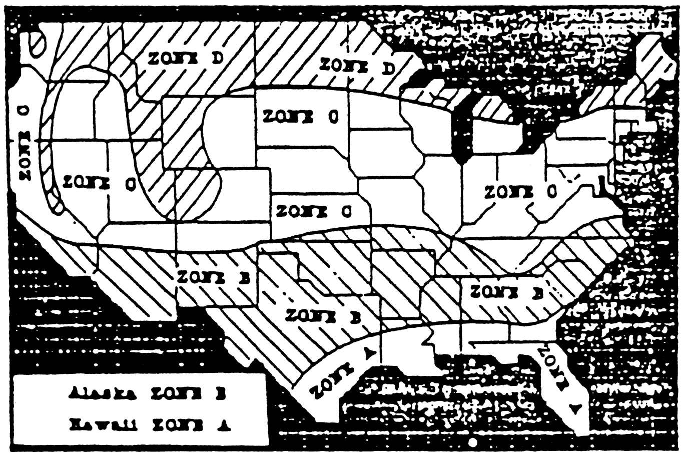


FIGURE 14.6.5.2-1 Map of Low Temperature Zones

14.6.5.3 Design Requirements

14.6.5.3.1 Scope

Bearings designed by the provisions of this section shall be subsequently tested in accordance with the requirements for steel reinforced elastomeric bearings of Article 18.7 of Division II of this Specification. Steel reinforced elastomeric bearings may also be designed under the provisions of Article 14.6.6.

14.6.5.3.2 Compressive Stress

In any bearing layer, the average compressive stress (ksi) shall satisfy the following:

- for bearings subject to shear deformation

$$\sigma_{TL} \leq 1.6 \text{ ksi}$$
$$\sigma_{TL} \leq 1.66\, GS \qquad (14.6.5.3.2\text{-}1)$$
$$\sigma_L \leq 0.66\, GS$$

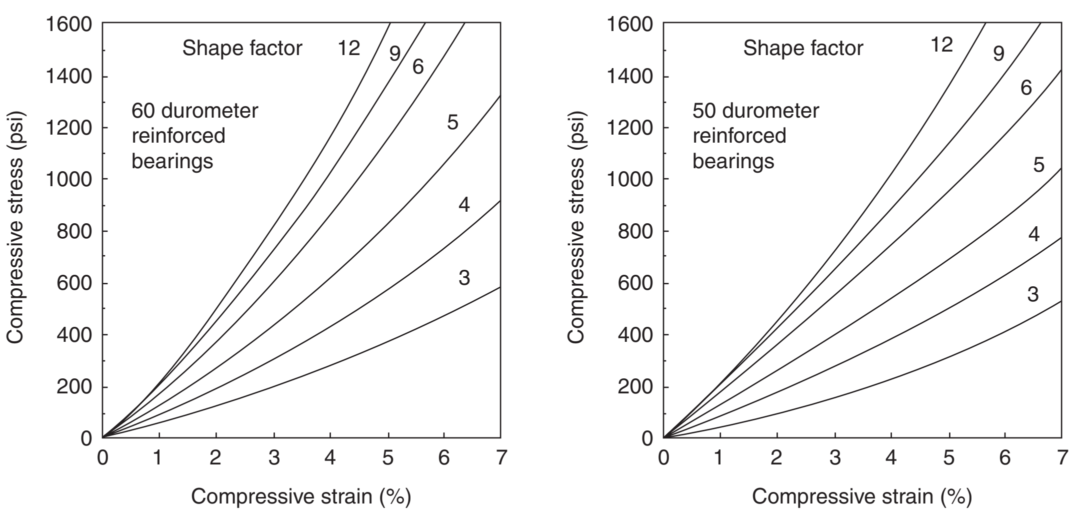


FIGURE 14.6.5.3.3-1 Load Deflection Behavior of Elastomeric Bearings

(a) Axial Forces—the maximum and minimum axial load is the dead load plus, or minus, the axial load determined from the final iteration of Step 3.
(b) Moments—the column overstrength plastic moments corresponding to the maximum compressive axial load specified in (a) above, with a strength reduction factor of 1.3 for reinforced concrete and 1.25 times the nominal yield strength for structural steel.
(c) Shear Force—the shear force corresponding to the column overstrength moments in (b) above, noting the provisions in Step 2 above.

7.2.3 Column and Pile Bent Design Forces

Design forces for columns and pile bents shall be the following:

(a) Axial Forces—the minimum and maximum design force shall either be the elastic design values determined in Article 3.9 added to the dead load, or the values corresponding to plastic hinging of the column and determined in Article 7.2.2. Generally, the values corresponding to column hinging will be smaller.
(b) Moments—the modified design moments determined in Article 7.2.1.
(c) Shear Force—either the elastic design value determined from Article 7.2.1 using an R-Factor of 1 for the column or the value corresponding to plastic hinging of the column as determined in Article 7.2.2. Generally, the value corresponding to column hinging will be significantly smaller.

7.2.4 Pier Design Forces

The design forces shall be those determined in Article 7.2.1 except if the pier is designed as a column in its weak direction. If the pier is designed as a column the design forces in the weak direction shall be as specified in Article 7.2.3 and all the design requirements for columns of Article 7.6 shall apply. (Note: When the forces due to plastic hinging are used in the weak direction the combination of forces specified in Article 3.9 is not applicable.)

7.2.5 Connection Design Forces

The design forces shall be those determined in Article 7.2.1 except that for superstructure connections to columns and column connections to cap beams or footings, the alternate forces specified in 7.2.5(C) below are recommended. Additional design forces at connections are as follows:

7.2.5(A) Longitudinal Linkage Forces

Positive horizontal linkage shall be provided between adjacent sections of the superstructure at supports and expansion joints within a span. The linkage shall be designed for a minimum force of the Acceleration Coefficient times the weight of the lighter of the two adjoining spans or parts of the structure. If the linkage is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motions, sufficient slack must be allowed in the linkage so that the linkage force does not start to act until the design displacement is exceeded. Where linkage is to be provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans. Positive linkage shall be provided by ties, cables, dampers, or an equivalent mechanism. Friction shall not be considered a positive linkage.

7.2.5(B) Hold-Down Devices

Hold-down devices shall be provided at all supports or hinges in continuous structures, where the vertical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50% but is less than 100% of the dead load reaction. In this case, the minimum net upward force for the hold-down device shall be 10% of the dead load downward force that would be exerted if the span were simply supported.

If the vertical seismic force (Q) due to the longitudinal horizontal seismic load opposes and exceeds 100 percent of the dead load reaction (DR), the net upwards force for the hold-down device shall be $1.2(Q - DR)$ but it shall not be less than that specified in the previous paragraph.

7.2.5(C) Column and Pier Connections to Cap Beams and Footings

The recommended connection design forces between the superstructure and columns, columns and cap beams, and columns and spread footings or pile caps are the forces developed at the top and bottom of the columns due to column hinging and determined in Article 7.2.2. The smaller of these or the values specified in Article 7.2.1 may be used. Note that these forces should be calculated after the column design is complete and the overstrength moment capacities have been obtained.

7.2.6 Foundation Design Forces

The design forces for foundations including footings, pile caps, and piles may be either those forces determined in Article 7.2.1(B) or the forces at the bottom of the columns corresponding to column plastic hinging as determined in Article 7.2.2. Generally, the values corresponding to column hinging will be significantly smaller.

When the columns of a bent have a common footing the final force distribution at the base of the columns from Step 4 of Article 7.2.2(B) may be used for the design of the footing in the plane of the bent. This force distribution produces lower shear forces and moments on the footing because one exterior column may be in tension and the other in compression due to the seismic overturning moment. This effectively increases the ultimate moments and shear forces on one column and reduces them on the other.

7.2.7 Abutment and Retaining Wall Design Forces

The components connecting the superstructure to an abutment (e.g., bearings and shear keys) shall be designed to resist the forces specified in Article 7.2.1.

Design requirements for abutments are given in Article 7.4.3 for SPC C and Article 7.4.5 for SPC D.

7.3 DESIGN DISPLACEMENT FOR SEISMIC PERFORMANCE CATEGORIES C AND D

The seismic design displacements shall be the maximum of those determined in accordance with Article 3.8 or those specified in Article 7.3.1.

7.3.1 Minimum Support Length Requirements for Seismic Performance Categories C and D

Bridges classified as SPC C or D shall meet the following requirement: Bearing seats supporting the expansion ends of girders, as shown in Figure 3.10, shall be designed to provide a minimum support length N (in. or mm), measured normal to the face of an abutment or pier, not less than that specified below.

$$N = (12 + 0.03L + 0.12H)(1 + 0.000125S^2) \text{ (in.)} \tag{7-3A}$$

or,

$$N = (305 + 2.5L + 10H)(1 + 0.000125S^2) \text{ (mm)} \tag{7-3B}$$

where,

L = length, in feet for Equation (7-3A) or meters for Equation (7-3B), of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, L shall be the sum of L_1 and L_2, the distances to either side of the hinge. For single span bridges L equals the length of the bridge deck. These lengths are shown in Figure 3.10.

S = angle of skew of support in degrees measured from a line normal to the span.

and H is given by one of the following:

for abutments, H is the average height, in feet for Equation (7-3A) or meters for Equation (7-3B), of columns supporting the bridge deck to the next expansion joint. H = 0 for single span bridges.

for columns and/or piers, H is the column or pier height in feet for Equation (7-3A) or meters for Equation (7-3B).

for hinges within a span, H is the average height of the adjacent two columns or piers in feet for Equation (7-3A) or meters for Equation (7-3B).

Positive horizontal linkages shall be provided at all superstructure expansion joints, including those joints within a span, as specified in Article 7.2.5.

Relative displacements between different segments of the bridge should be carefully considered in the evaluation of the results determined in accordance with Article 3.8. Relative displacements arise from effects that are not easily included in the analysis procedure but should be considered in determining the design displacements. They include the following:

(a) Torsional displacements of bridge decks on skewed supports.
(b) Rotation and/or lateral displacements of the foundations.
(c) Out-of-phase displacements of different segments of the bridge. This is especially important in determining seat widths at expansion joints.
(d) Out-of-phase rotation of abutments and columns induced by traveling seismic waves.

7.4 FOUNDATION AND ABUTMENT DESIGN REQUIREMENTS FOR SEISMIC PERFORMANCE CATEGORIES C AND D

7.4.1 General

This section includes only those foundation and abutment requirements that are specifically related to seismic resistant construction in SPC C and D. It assumes compliance with all requirements that are necessary to provide support for vertical and lateral loads other than those due to earthquake motions. These include, but are not limited to, provisions for the extent of foundation investigation, fills, slope stability, bearing and lateral soil pressures, drainage, settlement control, and pile requirements and capacities.

tom of the bond length. Centralizers are not required on tendons installed utilizing a hollow-stem auger if it is grouted through the auger and the drill hole is maintained full of a stiff grout (9-inch slump or less) during extraction of the auger. A combination centralizer-spacer may be used.

Centralizers are not required on tendons installed utilizing a pressure injection system in coarse-grained soils using grouting pressures greater than 150 psi.

6.4.1.2 Encapsulation Protected Ground Anchor Tendon

The tendon bond length shall be encapsulated by a grout-filled corrugated plastic or deformed steel tube, or by a fusion-bonded epoxy coating. The tendon can be grouted inside the encapsulation prior to inserting the tendon in the drill hole or after the tendon has been placed in the drill hole. Punching holes in the encapsulation and allowing the grout to flow from the encapsulation to the drill hole, or vice versa, will not be permitted. The tendon shall be centralized within the encapsulation and the tube sized to provide an average of 0.20 inches of grout cover for the prestressing steel. Spacers and centralizers shall be used to satisfy the same requirements specified in Article 6.4.1.1 for grout protected ground anchor tendons. The anchorage device of tendons protected with fusion-bonded epoxy shall be electrically isolated from the structure.

6.4.2 Unbonded Length

The unbonded length of the tendon shall be a minimum of 15 feet or as indicated on the plans or approved working drawings.

Corrosion protection shall be provided by a sheath completely filled with corrosion inhibiting grease or grout, or a heat shrinkable tube. If grease is used to fill the sheath, provisions shall be made to prevent it from escaping at the ends. The grease shall completely coat the tendon and fill the interstices between the wires of seven-wire strands. Continuity of corrosion protection shall be provided at the transition from the bonded length to unbonded length of the tendon.

If the sheath provided is not a smooth tube, then a separate bondbreaker must be provided to prevent the tendon from bonding to the anchor grout surrounding the unbonded length.

6.4.3 Anchorage and Trumpet

Nonrestressable anchorages may be used unless restressable anchorages are designated on the plans or specified in the special provisions.

Bearing plates shall be sized so that the bending stresses in the plate and average bearing stress on the concrete, if applicable, do not exceed the allowable stresses described in Division I, Article 9.21.7.2, “Bearing Strength.” The size of bearing plates shall not be less than that shown on the plans or on the approved working drawings.

The trumpet shall be welded to the bearing plate. The trumpet shall have an inside diameter at least 0.25 inches greater than the diameter of the tendon at the anchorage. The trumpet shall be long enough to accommodate movements of the structure during testing and stressing. For strand tendons with encapsulation over the unbonded length, the trumpet shall be long enough to enable the tendons to make a transition from the diameter of the tendon in the unbonded length to the diameter of the tendon at the anchorhead without damaging the encapsulation. Trumpets filled with corrosion-inhibiting grease shall have a permanent Buna-N rubber or approved equal seal provided between the trumpet and the unbonded length corrosion protection. Trumpets filled with grout shall have a temporary seal provided between the trumpet and the unbonded length corrosion protection.

6.4.4 Tendon Storage and Handling

Tendons shall be stored and handled in such a manner as to avoid damage or corrosion. Damage to tendon prestressing steel as a result of abrasions, cuts, nicks, welds and weld splatter will be cause for rejection by the Engineer. Grounding of welding leads to the prestressing steel is not permitted. A slight rusting, provided it is not sufficient to cause pits visible to the unaided eye, shall not be cause for rejection. Prior to inserting a tendon into the drilled hole, its corrosion protection elements shall be examined for damage. Any damage found shall be repaired in a manner approved by the Engineer.

6.5 INSTALLATION

The Contractor shall select the drilling method, the grouting procedure and grouting pressure to be used for the installation of the ground anchor as necessary to satisfy the load test requirements.

6.5.1 Drilling

The drilling method used may be core drilling, rotary drilling, percussion drilling, auger drilling or driven casing. The method of drilling used shall prevent loss of ground above the drilled hole that may be detrimental to the structure or existing structures. Casing for anchor holes, if used, shall be removed, unless permitted by the Engineer to be left in place. The location, inclination, and alignment of the drilled hole shall be as shown on the plans. Inclination and alignment shall be within plus or minus 3° of the planned angle at the bearing plate, and within plus or minus 12 inches of the planned location at the ground surface (point of entry).

6.5.2 Tendon Insertion

The tendon shall be inserted into the drilled hole to the desired depth without difficulty. When the tendon cannot be completely inserted it shall be removed and the drill hole cleaned or redrilled to permit insertion. Partially inserted tendons shall not be driven or forced into the hole.

6.5.3 Grouting

A neat cement grout or sand-cement grout conforming to Article 6.3.2 shall be used. Admixtures, if used, shall be mixed in quantities not to exceed the manufacturer's recommendations.

The grouting equipment shall produce a grout free of lumps and undispersed cement. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gauge to monitor grout pressures. The pressure gauge shall be capable of measuring pressures of at least 150 psi or twice the actual grout pressures used, whichever is greater. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

The grout shall be injected from the lowest point of the drill hole. The grout may be pumped through grout tubes, casing, hollow-stem augers or drill rods. The grout may be placed before or after insertion of the tendon. The quantity of the grout and the grout pressures shall be recorded. The grout pressures and grout takes shall be controlled to prevent excessive heave of the ground or fracturing of rock formations.

Except where indicated below, the grout above the top of the bond length may be placed at the same time as the bond length grout, but it shall not be placed under pressure. The grout at the top of the drill hole shall stop 6 inches from the back of the structure or from the bottom of the trumpet, whichever is lowest.

If the ground anchor is installed in a fine-grained soil using a drilled hole larger than 6 inches in diameter, then the grout above the top of the bond length shall be placed after the ground anchor has been load tested. The entire drill hole may be grouted at the same time if it can be demonstrated that the ground anchor system does not derive a significant portion of its load resistance from the soil above the bond length portion of the ground anchor.

If grout protected tendons are used for ground anchors anchored in rock, then pressure grouting techniques shall be utilized. Pressure grouting requires that the drill hole be sealed and that the grout be injected until a 50-psi grout pressure can be maintained on the grout within the bond length for a period of 5 minutes.

Upon completion of grouting, the grout tube may remain in the drill hole provided it is filled with grout.

After grouting, the tendon shall not be loaded for a minimum of 3 days.

6.5.4 Trumpet and Anchorage

The corrosion protection surrounding the unbonded length of the tendon shall extend into the trumpet a minimum of 6 inches beyond the bottom seal in the trumpet.

The corrosion protection surrounding the unbonded length of the tendon shall not contact the bearing plate or the anchorhead during load testing or stressing.

The bearing plate and anchorhead shall be placed perpendicular to the axis of the tendon.

The trumpet shall be completely filled with corrosion inhibiting grease or grout. The grease may be placed any time during construction. The grout shall be placed after the ground anchor has been load tested. The Contractor shall demonstrate that the procedures selected for placement of either grease or grout will produce a completely filled trumpet.

Anchorages not encased in concrete shall be covered with a corrosion inhibiting grease-filled or grout-filled steel enclosure.

6.5.5 Testing and Stressing

Each ground anchor shall be load tested by the Contractor. No load greater than 10% of the design load may be applied to the ground anchor prior to load testing. The test load shall be simultaneously applied to the entire tendon.

6.5.5.1 Testing Equipment

A dial gauge or vernier scale capable of measuring displacements to 0.001 inches shall be used to measure ground anchor movement. It shall have adequate travel so total ground anchor movement can be measured without resetting the device.

A hydraulic jack and pump shall be used to apply the test load. The jack and a calibrated pressure gauge shall be used to measure the applied load. The pressure gauge

Only oxygen flame or mechanical cutting devices shall be used to cut strand after installation in the member or after stressing. Electric arc welders shall not be used.

10.10.1.3 Sequence of Stressing

When the sequence of stressing individual tendons is not otherwise specified, the stressing of post-tensioning tendons and the release of pretensioned tendons shall be done in a sequence that produces a minimum of eccentric force in the member.

10.10.1.4 Measurement of Stress

A record of gauge pressures and tendon elongations for each tendon shall be provided by the Contractor for review and approval by the Engineer. Elongations shall be measured to an accuracy of $+\frac{1}{16}$ inch. Stressing tails of post-tensioned tendons shall not be cut off until the stressing records have been approved.

The stress in tendons during tensioning shall be determined by the gauge or load cell readings and shall be verified with the measured elongations. Calculations of anticipated elongations shall utilize the modulus of elasticity, based on nominal area, as furnished by the manufacturer for the lot of steel being tensioned, or as determined by a bench test of strands used in the work.

All tendons shall be tensioned to a preliminary force as necessary to eliminate any take-up in the tensioning system before elongation readings are started. This preliminary force shall be between 5% and 25% of the final jacking force. The initial force shall be measured by a dynamometer or by other approved method, so that its amount can be used as a check against elongation as computed and as measured. Each strand shall be marked prior to final stressing to permit measurement of elongation and to insure that all anchor wedges set properly.

It is anticipated that there may be discrepancy in indicated stress between jack gauge pressure and elongation. In such event, the load used as indicated by the gauge pressure, shall produce a slight over-stress rather than under-stress. When a discrepancy between gauge pressure and elongation of more than 5% in tendons over 50 feet long or 7% in tendons of 50 feet or less in length occurs, the entire operation shall be carefully checked and the source of error determined and corrected before proceeding further. When provisional ducts are provided for addition of prestressing force in event of an apparent force deficiency in tendons over 50 feet long, the discrepancy between the force indicated by gauge pressure and elongation may be increased to 7% before investigation into the source of the error.

10.10.2 Pretensioning Method Requirements

Stressing shall be accomplished by either single strand stressing or multiple strand stressing. The amount of stress to be given each strand shall be as shown in the plans or the approved working drawings.

All strand to be stressed in a group (multiple strand stressing) shall be brought to a uniform initial tension prior to being given their full pretensioning. The amount of the initial tensioning force shall be within the range specified in Article 10.5.1 and shall be the minimum required to eliminate all slack and to equalize the stresses in the tendons as determined by the Engineer. The amount of this force will be influenced by the length of the casting bed and the size and number of tendons in the group to be tensioned.

Draped pretensioned tendons shall either be tensioned partially by jacking at the end of the bed and partially by uplifting or depressing tendons, or they shall be tensioned entirely by jacking, with the tendons being held in their draped positions by means of rollers, pins, or other approved methods during the jacking operation.

Approved low-friction devices shall be used at all points of change in slope of tendon trajectory when tensioning draped pretensioned strands, regardless of the tensioning method used.

If the load for a draped strand, as determined by elongation measurements, is more than 5% less than that indicated by the jack gauges, the strand shall be tensioned from both ends of the bed and the load as computed from the sum of elongation at both ends shall agree within 5% of that indicated by the jack gauges.

When ordered by the Engineer, prestressing steel strands in pretensioned members, if tensioned individually, shall be checked by the Contractor for loss of prestress not more than 3 hours prior to placing concrete for the members. The method and equipment for checking the loss of prestress shall be subject to approval by the Engineer. All strands that show a loss of prestress in excess of 3% shall be retensioned to the original computed jacking stress.

Stress on all strands shall be maintained between anchorages until the concrete has reached the compressive strength required at time of transfer of stress to concrete.

When prestressing steel in pretensioned members is tensioned at a temperature more than 25°F lower than the estimated temperature of the concrete and the prestressing steel at the time of initial set of the concrete, the calculated elongation of the prestressing steel shall be increased to compensate for the loss in stress, due to the change in temperature, but in no case shall the jacking stress exceed 80% of the specified minimum ultimate tensile strength of the prestressing steel.

Strand splicing methods and devices shall be approved by the Engineer. When single strand jacking is used, only one splice per strand will be permitted. When multi-strand jacking is used, either all strands shall be spliced or no more than 10% of the strands shall be spliced. Spliced strands shall be similar in physical properties, from the same source, and shall have the same "twist" or "lay." All splices shall be located outside of the prestressed units.

Side and flange forms that restrain deflection shall be removed before release of pretensioning reinforcement.

Except when otherwise shown on the plans, all pretensioned-prestressing strands shall be cut off flush with the end of the member and the exposed ends of the strand and a 1-inch strip of adjoining concrete shall be cleaned and painted. Cleaning shall be by wire brushing or abrasive blast cleaning to remove all dirt and residue that is not firmly bonded to the metal or concrete surfaces. The surfaces shall be coated with one thick coat of zinc-rich paint conforming to the requirements of Federal Specification TT-P-641. The paint shall be thoroughly mixed at the time of application, and shall be worked into any voids in the strands.

10.10.3 Post-Tensioning Method Requirements

Prior to post-tensioning any member, the Contractor shall demonstrate to the satisfaction of the Engineer that the prestressing steel is free and unbonded in the duct.

All strands in each tendon, except for those in flat ducts with not more than four strands, shall be stressed simultaneously with a multi-strand jack.

Tensioning shall be accomplished so as to provide the forces and elongations specified in Article 10.5.1.

Except as provided herein or when shown on the plans or on the approved working drawings, tendons in continuous post-tensioned members shall be tensioned by jacking at each end of the tendon. For straight tendons and when one end stressing is shown on the plans, tensioning may be performed by jacking from one end or both ends of the tendon at the option of the Contractor.

10.11 GROUTING

10.11.1 General

When the post-tensioning method is used, the prestressing steel shall be provided with permanent protection and shall be bonded to the concrete by completely filling the void space between the duct and the tendon with grout.

10.11.2 Preparation of Ducts

All ducts shall be clean and free of deleterious materials that would impair bonding or interfere with grouting procedures.

Ducts with concrete walls (cored ducts) shall be flushed to ensure that the concrete is thoroughly wetted. Metal ducts shall be flushed if necessary to remove deleterious material.

Water used for flushing ducts may contain slack lime (calcium hydroxide) or quicklime (calcium oxide) in the amount of 0.1 lb per gallon.

After flushing, all water shall be blown out of the duct with oil-free compressed air.

10.11.3 Equipment

The grouting equipment shall include a mixer capable of continuous mechanical mixing which will produce a grout free of lumps and undispersed cement, a grout pump and standby flushing equipment with water supply. The equipment shall be able to pump the mixed grout in a manner which will comply with all requirements.

Accessory equipment which will provide for accurate solid and liquid measures shall be provided to batch all materials.

The pump shall be a positive displacement type and be able to produce an outlet pressure of at least 150 psi. The pump should have seals adequate to prevent introduction of oil, air, or other foreign substance into the grout, and to prevent loss of grout or water.

A pressure gauge having a full-scale reading of no greater than 300 psi shall be placed at some point in the grout line between the pump outlet and the duct inlet.

The grouting equipment shall contain a screen having clear openings of 0.125-inch maximum size to screen the grout prior to its introduction into the grout pump. If a grout with a thixotropic additive is used, a screen opening of $^{3}/_{16}$ inch is satisfactory. This screen shall be easily accessible for inspection and cleaning.

The grouting equipment shall utilize gravity feed to the pump inlet from a hopper attached to and directly over it. The hopper must be kept at least partially full of grout at all times during the pumping operation to prevent air from being drawn into the post-tensioning duct.

Under normal conditions, the grouting equipment shall be capable of continuously grouting the largest tendon on the project in no more than 20 minutes.

10.11.4 Mixing of Grout

Water shall be added to the mixer first, followed by Portland cement and admixture, or as required by the admixture manufacturer.

installation method. To perform the calibrated wrench verification test for short grip bolts, direct tension indicators (DTI) with solid plates may be used in lieu of a tension measuring device. The DTI lot shall be first verified with a longer grip bolt in the Skidmore-Wilhelm Calibrator or an acceptable equivalent device. The frequency of confirmation testing, the number of tests to be performed, and the test procedure shall be as specified in Articles 11.5.6.4.4 through 11.5.6.4.7, as applicable. The accuracy of the tension measuring device shall be confirmed by an approved testing agency at least annually.

Bolts and nuts together with washers of size and quality specified, located as required below, shall be installed in properly aligned holes and tensioned and inspected by any of the installation methods described in Articles 11.5.6.4.4 through 11.5.6.4.7 to at least the minimum tension specified in Table 11.5A. Tensioning may be done by turning the bolt while the nut is prevented from rotating when it is impractical to turn the nut. Impact wrenches, if used, shall be of adequate capacity and sufficiently supplied with air to tension each bolt in approximately 10 seconds.

AASHTO M 253 (ASTM A 490) fasteners and galvanized AASHTO M 164 (ASTM A 325) fasteners shall not be reused. Other AASHTO M 164 (ASTM A 325) bolts may be reused if approved by the Engineer. Touching up or retensioning previously tensioned bolts which may have been loosened by the tensioning of adjacent bolts shall not be considered as reuse provided the tensioning continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 11.5B.

Bolts shall be installed in all holes of the connection and the connection brought to a snug condition. Snug is defined as having all plies of the connection in firm contact.

Snugging shall progress systematically from the most rigid part of the connection to the free edges. The snugging sequence shall be repeated until the full connection is in a snug condition.

TABLE 11.5B Nut Rotation from the Snug-Tight Condition[a,b] Geometry of Outer Faces of Bolted Parts

Bolt Length Measured From Underside of Head to End of Bolt	Both Faces Normal to Bolt Axis	One Face Normal to Bolt Axis and Other Face Sloped Not More Than 1:20, Bevel Washer Not Used	Both Faces Sloped Not More Than 1:20 From Normal To Bolt Axis, Bevel Washers Not Used
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters[c]	2/3 turn	5/6 turn	1 turn

[a] Nut rotation is relative to bolt, regardless of the element (nut or bolt) being turned. For bolts installed by 1/2 turn and less, the tolerance should be plus or minus 30 degrees; for bolts installed by 2/3 turn and more, the tolerance should be plus or minus 45 degrees.
[b] Applicable only to connections in which all material within grip of the bolt is steel.
[c] No research work has been performed by the Research Council Riveted and Bolted Structural Joints to establish the turn-of-nut procedure when bolt lengths exceed 12 diameters. Therefore, the required rotation must be determined by actual tests in a suitable tension device simulating the actual conditions.

11.5.6.4.2 Rotational-Capacity Tests

Rotational-capacity testing is required for all fastener assemblies. Galvanized assemblies shall be tested galvanized. Washers are required as part of the test even though they may not be required as part of the installation procedure. The following shall apply:

(a) Except as modified herein, the rotational-capacity test shall be performed in accordance with the requirements of AASHTO M 164 (ASTM A 325).
(b) Each combination of bolt production lot, nut lot and washer lot shall be tested as an assembly. Where washers are not required by the installation procedures, they need not be included in the lot identification.
(c) A rotational-capacity lot number shall have been assigned to each combination of lots tested.
(d) The minimum frequency of testing shall be two assemblies per rotational-capacity lot.
(e) For bolts that are long enough to fit in a Skidmore-Wilhelm Calibrator, the bolt, nut and washer assembly shall be assembled in a Skidmore-Wilhelm Calibrator or an acceptable equivalent device.
(f) Bolts that are too short to be tested in a Skidmore-Wilhelm Calibrator may be tested in a steel joint. The tension requirement, in (g) below, need not apply. The maximum torque requirement, torque < 0.25 PD, shall be computed using a value of P equal to the turn test tension taken as 1.15 times the bolt tension in Table 11.5A.
(g) The tension reached at the below rotation (i.e., turn-test tension) shall be equal to or greater than 1.15 times the required fastener tension (i.e., installation tension) shown in Table 11.5A.

(h) The minimum rotation from an initial tension of 10% of the minimum required tension (snug condition) shall be two times the required number of turns indicated in Table 11.5B without stripping or failure.

(i) After the required installation tension listed above has been exceeded, one reading of tension and torque shall be taken and recorded. The torque value shall conform to the following:

$$\text{Torque} \leq 0.25\ \text{PD}$$

Where:

Torque = measured torque (foot-pounds)
P = measured bolt tension (pounds)
D = bolt diameter (feet).

11.5.6.4.3 Requirement for Washers

Where the outer face of the bolted parts has a slope greater than 1:20 with respect to a plane normal to the bolt axis, a hardened beveled washer shall be used to compensate for the lack of parallelism.

Hardened beveled washers for American Standard Beams and Channels shall be required and shall be square or rectangular, shall conform to the requirements of AASHTO M 293 (ASTM F 436), and shall taper in thickness.

Where necessary, washers may be clipped on one side to a point not closer than ⅞ of the bolt diameter from the center of the washer.

Hardened washers are not required for connections using AASHTO M 164 (ASTM A 325) and AASHTO M 253 (ASTM A 490) bolts except as follows:

- Hardened washers shall be used under the turned element when tensioning is to be performed by calibrated wrench method.
- Irrespective of the tensioning method, hardened washers shall be used under both the head and the nut when AASHTO M 253 (ASTM A 490) bolts are to be installed in material having a specified yield point less than 40 ksi. However, when DTIs are used they may replace a hardened washer provided a standard hole is used.
- Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in oversize or short-slotted holes in an outer ply, a hardened washer conforming to AASHTO M 293 (ASTM F 436) shall be used.
- When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be installed in an oversized or short-slotted hole in an outer ply, hardened washers conforming to AASHTO M 293 (ASTM F 436) except with 5⁄16 inch minimum thickness shall be used under both the head and the nut in lieu of standard thickness hardened washers. Multiple hardened washers with combined thickness equal to or greater than 5⁄16 inch do not satisfy this requirement.
- Where AASHTO M 164 (ASTM A 325) bolts of any diameter or AASHTO M 253 (ASTM A 490) bolts equal to or less than 1 inch in diameter are to be installed in a long slotted hole in an outer ply, a plate washer or continuous bar of at least 5⁄16 inch thickness with standard holes shall be provided. These washers or bars shall have a size sufficient to completely cover the slot after installation and shall be of structural grade material, but need not be hardened except as follows. When AASHTO M 253 (ASTM A 490) bolts over 1 inch in diameter are to be used in long slotted holes in external plies, a single hardened washer conforming to AASHTO M 293 (ASTM F 436) but with 5⁄16 inch minimum thickness shall be used in lieu of washers or bars of structural grade material. Multiple hardened washers with combined thickness equal to or greater than 5⁄16 inch do not satisfy this requirement.

Alternate design fasteners meeting the requirements of Article 11.3.2.6 with a geometry which provides a bearing circle on the head or nut with a diameter equal to or greater then the diameter of hardened washers meeting the requirements of AASHTO M 293 (ASTM F 436) satisfy the requirements for washers specified herein and may be used without washers.

11.5.6.4.4 Turn-of-Nut Installation Method

When the turn-of-nut installation method is used, hardened washers are not required except as may be specified in Article 11.5.6.4.3.

Verification testing using a representative sample of not less than three fastener assemblies of each diameter, length and grade to be used in the work shall be performed at the start of work in a device capable of indicating bolt tension. This verification test shall demonstrate that the method used to develop a snug condition and control the turns from snug by the bolting crew develops a tension not less than 5% greater than the tension required by Table 11.5A. Periodic retesting shall be performed when ordered by the Engineer.

After snugging, the applicable amount of rotation specified in Table 11.5B shall be achieved. During the tensioning operation there shall be no rotation of the part not turned by the wrench. Tensioning shall progress systematically from the most rigid part of the joint to its free edges.

11.5.6.4.5 Calibrated Wrench Installation Method

The calibrated wrench method may be used only when wrenches are calibrated on a daily basis and when a hardened washer is used under the turned element. Standard for grades 0 and 2 materials, unless especially requested by the Engineer.

For grade 3 material, in lieu of the low temperature crystallization test, the manufacturer may choose to provide certificates from low-temperature crystallization tests performed on identical material within the last year, unless otherwise specified by the Engineer.

Every finished bearing shall be visually inspected in accordance with Article 18.7.4.5.5.

Every steel reinforced bearing shall be subjected to the short-term load test described in Article 18.7.4.5.6.

From each lot of bearings either designed by the provisions of Article 14.6.5 of Division I of this specification or made from grade 4 or grade 5 elastomer, a random sample shall be subjected to the long-term load test described in Articles 18.7.2.7 and 18.7.4.5.7. The sample shall consist of at least one bearing chosen randomly from each size and material batch and shall comprise at least 10% of the lot. If one bearing of the sample fails, all the bearings of that lot shall be rejected, unless the manufacturer elects to test each bearing of the lot at own expense. In lieu of this procedure, the Engineer may require every bearing of the lot to be tested.

The Engineer may require shear stiffness tests on material from a random sample of the finished bearings in accordance with Article 18.7.4.5.8.

18.7.4.5.3 Ambient Temperature Tests on the Elastomer

The elastomer used shall at least satisfy the limits prescribed in the appropriate Table 18.4.5.1-1A or -1B for durometer hardness, tensile strength, ultimate elongation, heat resistance, compression set, and ozone resistance. The bond to the reinforcement, if any, shall also satisfy Article 18.4.5.3. The shear modulus of the material shall be tested at 73°F ± 2°F using the apparatus and procedure described in Annex A of ASTM D 4014, amended where necessary by the requirements of Table 18.4.5.1-1A or -1B. It shall fall within 15% of the specified value, or within the range of its hardness given in Article 14.6.5.2 of Division I if no shear modulus is specified.

18.7.4.5.4 Low Temperature Tests on the Elastomer

The tests shall be performed in accordance with the requirements of Tables 18.4.5.1-1A and -1B and the compound shall satisfy all limits for its grade. The testing frequency shall be in accordance with Article 18.7.4.5.2.

18.7.4.5.5 Visual Inspection of the Finished Bearing

Each finished bearing shall be inspected for compliance with dimensional tolerances and for overall quality of manufacture. In steel reinforced bearings, the edges of the steel shall be protected everywhere from corrosion.

18.7.4.5.6 Short-Duration Compression Tests on Bearings

Each finished bearing shall be subjected to a short-term compression test as described in Article 18.7.2.5. If the bulging pattern suggests laminate parallelism or a layer thickness that is outside the specified tolerances, or poor laminate bond, the bearing shall be rejected. If there are three or more separate surface cracks that are greater than 0.08 inches wide and 0.08 inches deep, the bearing shall be rejected.

18.7.4.5.7 Long-Duration Compression Tests on Bearings

The bearing shall be subject to a long-term compression test as described in Article 18.7.2.6. The bearing shall be examined visually at the end of the test while it is still under load. If the bulging pattern suggests laminate parallelism or a layer thickness that is outside the specified tolerances, or poor laminate bond, the bearing shall be rejected. If there are three or more separate surface cracks that are greater than 0.08 inches wide and 0.08 inches deep, the bearing shall be rejected.

18.7.4.5.8 Shear Modulus Tests on Materials from Bearings

The shear modulus of the material in the finished bearing shall be evaluated by testing a specimen cut from it using the apparatus and procedure described in Annex A of ASTM D 1014, amended where necessary by the requirements of Table 18.4.5.1-1A or -1B, or, at the discretion of the Engineer, a comparable nondestructive stiffness test may be conducted on a pair of finished bearings. The shear modulus shall fall within 15% of the specified value, or within the range for its hardness given in Table 14.6.5.2.1 of Division I if no shear modulus is specified. If the test is conducted on finished bearings, the material shear modulus shall be computed from the measured shear stiffness of the bearings, taking due account of the influence on shear stiffness of bearing geometry and compressive load.

18.7.4.7 Test Requirements for Bronze and Copper Alloy Bearings

Material certification tests for the bronze or copper shall be performed to verify the properties of the metal.

Bearing friction tests as defined in Article 18.7.2.7 or material friction tests as defined in Article 18.7.2.2 may be required by the Engineer.

18.7.4.8 Test Requirements for Disc Bearings

18.7.4.8.1 Material Certification Tests

The manufacturer shall select, at random, samples for material certification tests as defined in Article 18.7.2.1. The tests shall be performed, and certifications shall be delivered to the Engineer.

Certification shall be provided for all polyether urethane elements. Their material properties shall satisfy the requirements of the design documents and the tests described in Article 18.4.8.1. Additional tests may be required by the Engineer.

18.7.4.8.2 Testing by the Engineer

When quality assurance testing is called for by the special provisions, the manufacturer shall furnish to the Engineer the required number of complete bearings and component samples to perform quality assurance testing. At least one set of material property tests in accordance with Article 18.4.8.1 shall be conducted per lot of bearings. All exterior surfaces of sampled production bearings shall be smooth and free from irregularities or protrusions that might interfere with testing procedures.

For quality assurance testing, the Engineer may select at random the required sample bearing(s) and the material samples from completed lots of bearings.

A minimum of 30 days shall be allowed for inspection, sampling, and quality assurance testing of production bearings and component materials.

18.7.4.8.3 Bearing Tests

Critical dimensions shall include the clearance between the upper and lower parts of the steel housing, and shall be verified by the Clearance Test described in Article 18.7.2.4.

A Long-term Deterioration Test as described in Article 18.7.2.8 shall be performed on one disc bearing of each lot. The test shall be performed at the maximum design rotation combined with a maximum dead plus live load. If size limitations prevent testing of the full size bearing, a special bearing with the same rotational capacity and no less than 200 kips compressive load capacity may be tested in its place.

A Long-term Compression Proof Load Test as described in Article 18.7.2.6 may be required by the Engineer.

18.7.5 Cost of Transporting

The Contractor shall assume the cost of transporting all samples from the place of manufacture to the test site and back, or if applicable, to the project site.

18.7.6 Use of Tested Bearings in the Structure

Bearings which have been satisfactorily tested in accordance with the requirements of this section may be used in the structure provided that they are equipped with new deformable elements, sliding elements and seals, as required by the Engineer.

18.8 PACKING, SHIPPING AND STORING

For transportation and storage, bearings shall be packaged in a way that prevents relative movement of their components and damage by handling, weather, dust, or other normal hazards. They shall be stored only in a clean, protected environment. When installed, bearings shall be clean and free from all foreign substances.

Bearings shall not be opened or dismantled at the site except under the direct supervision of, or with the written approval of, the manufacturer or its assigned agents.

18.9 INSTALLATION

18.9.1 General Installation Requirements

Bearings shall be installed by qualified personnel at the locations shown on the plans. Bearings shall be set to the dimensions and offsets prescribed by the manufacturer, the Engineer, and the plans and shall be adjusted as necessary to take into account the temperature and future movements of the bridge due to temperature changes, release of falsework, shortening due to prestressing and other bridge movements.

Each bridge bearing shall be located within ± ⅛ inch of its correct position in the horizontal plane and oriented to within an angular tolerance of 0.02 radians. Guided Bearings shall also satisfy the requirements of Article 18.9.2.3. All bearings except those which are placed in opposing pairs shall be set horizontal to within an angular tolerance of 0.005 radians, and must have full and even contact with load plates, where these exist. The superstructure supported by the bearing shall be set on it so that, under full dead load, its slope lies within an angular tolerance of 0.005 radians of the design value. Any departure from this tolerance shall be corrected by means of a tapered plate or by other means approved by the Engineer. If shim stacks are needed to level the bearing they shall be removed after grouting and before the weight of the superstructure acts on the bearing.

Metallic bearing assemblies not embedded in the concrete shall be bedded on the concrete with a filler or fabric material conforming to Article 18.4.9.

Bearings seated directly on steel work require the supporting surface to be machined so as to provide a level and planar surface upon which the bearing is placed.

Preshaping may consist of a simple "V" graded into the soil as shown in Figure 26.5.3.

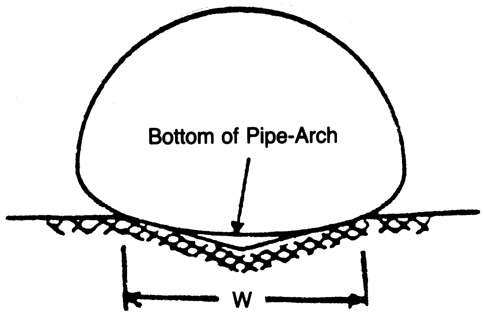


FIGURE 26.5.3 "V" Shaped Bed (Foundation) for Larger Pipe Arch, Horizontal Ellipse and Underpass Structures

26.5.4 Structural Backfill

26.5.4.1 General

Correct placement of materials of the proper quality and moisture content is essential. Sufficient field testing must be used to verify procedures, but is no substitute for inspection that ensures that the proper procedures are followed. This is of extreme importance because the structural integrity of the corrugated metal structure is vitally affected by the quality of construction in the field.

Backfill material shall meet the requirements of Article 26.3.8 and shall be placed as shown in Figure 26.5.2D in layers not exceeding 8-inch loose lift thickness to a minimum 90% standard density per AASHTO T 99. Equipment used to compact backfill within 3 feet from sides of pipe or from edge of footing for arches and box culverts shall be approved by the Engineer prior to use. Except as provided below for long-span structures, the equipment used for compacting backfill beyond these limits may be the same as used for compacting embankment.

The backfill shall be placed and compacted with care under the haunches of the pipe and shall be brought up evenly on both sides of the pipe by working backfill operations from side to side. The side to side backfill differential shall not exceed 24 inches or ⅓ of the size of the structure, whichever is less. Backfill shall continue to not less than 1 foot above the top for the full length of the pipe. Fill above this elevation may be material for embankment fill or other materials as specified to support the pavement. The width of trench shall be kept to the minimum width required for placing pipe, placing adequate bedding and sidefill, and safe working conditions. Ponding or jetting of backfill will not be permitted except upon written permission by the Engineer.

Where single or multiple structures are installed at a skew to the embankment (i.e. cross the embankment at other than 90°), proper support for the pipe must be provided. This may be done with a rigid, reinforced concrete head wall or by warping the embankment fill to provide the necessary balanced side support. Figure 26.5.4 provides guidelines for warping the embankment.

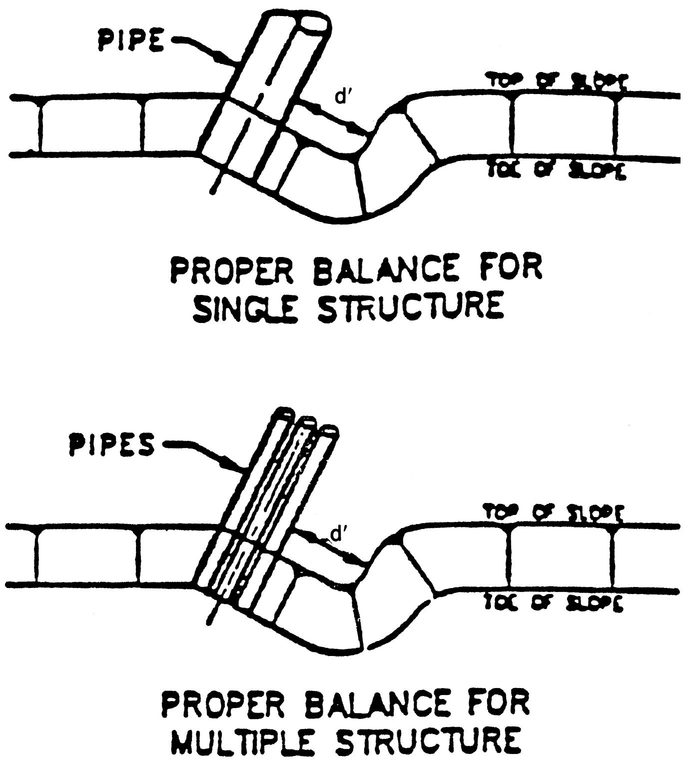
26.5.4.2 Arches

Arches may require special shape control considerations during the placement and compaction of structure backfill. Pin connections at the footing restrict uniform shape change. Arches may peak excessively and experience curvature flattening in their upper quadrants. Using lighter compaction equipment, more easily compacted structure backfill, or top loading (placing a small load of structure backfill on the crown) will aid installation.

26.5.4.3 Long-Span Structures

Backfill requirements for long-span structural-plate structures are similar to those for smaller structures. Their size and flexibility require special control of backfill and continuous monitoring of structure shape. Prior to beginning construction, the manufacturer shall provide a preconstruction conference to advise the Contractor(s) and Engineer of the more critical functions to be performed.

Equipment and construction procedures used to backfill long-span structural plate structures shall be such that excessive structure distortion will not occur. Structure shape shall be checked regularly during backfilling to verify acceptability of the construction methods used. Magnitude of allowable shape changes will be specified by the manufacturer (fabricator of long-span structures). The manufacturer shall provide a qualified shape control inspector to aid the Engineer during the placement of all structural backfill to the minimum cover level over the structure (as required by the design to carry full highway loads). The Inspector shall advise the Engineer on the acceptability of all backfill material and construction methods and the proper monitoring of the shape. Structure backfill material shall be placed in horizontal uniform layers not exceeding an 8-inch loose lift thickness and shall be brought up uniformly on both sides of the structure. Each layer shall be compacted to a density not less than 90% per AASHTO T 180. The structure backfill shall be constructed to the minimum lines and grades shown on the plans, keeping it at or below the level of adjacent soil or embankment. Permissible exceptions to required structure backfill density are: the area under the invert, the 12-inch to 18-inch width of soil immediately adjacent to the large radius side plates of high-profile



d′ = 1.5 (Rise + Cover)

FIGURE 26.5.4 End Treatment of Skewed Flexible Culvert

arches and inverted pear shapes, and the lower portion of the first horizontal lift of overfill carried ahead of and under the small, tracked vehicle initially crossing the structure.

26.5.4.4 Box Culverts

Metal box culverts are not long-span structures in that they are relatively stiff, semi-rigid frames. They do not require a preconstruction conference or shape control considerations beyond those of a standard metal culvert.

Structural backfill material shall be placed in uniform horizontal layers not exceeding an 8-inch maximum loose lift thickness and compacted to a density not less than 90% per AASHTO T 180. The structural backfill shall be constructed to the minimum lines and grades shown on the plans, keeping it at or below the level of the adjacent soil or embankment.

26.5.4.5 Bracing

When required, temporary bracing shall be installed and shall remain in place as long as necessary to protect workmen and to maintain structure shape during erection.

For long-span structures which require temporary bracing or cabling to hold the structure in shape, the supports shall not be removed until backfill is placed to an adequate elevation to provide the necessary support. In no case shall internal braces be left in place when backfilling reaches the top quadrant of the pipe or the top radius arc portion of a long span.

26.5.5 Arch Substructures and Headwalls

Substructures and headwalls shall be designed in accordance with the requirements of Division I.

The ends of the corrugated metal arch shall rest in a keyway formed into continuous concrete footings, or shall rest on a metal bearing surface, usually an angle or channel shape, which is securely anchored to or embedded in the concrete footing.

The metal bearing when specified may be a hot-rolled or cold-formed galvanized steel angle or channel, or an extruded aluminum angle or channel. These shapes shall be not less than $\frac{3}{16}$ inch in thickness and shall be securely anchored to the footing at a maximum spacing of 24 inches. When the metal bearing member is not completely embedded in a groove in the footing, one vertical

Section 30
THERMOPLASTIC PIPE

30.1 GENERAL

30.1.1 Description

This work shall consist of furnishing and installing thermoplastic pipe in conformance with these Specifications, any special provisions, and the details shown on the plans. As used in this specification, thermoplastic pipe is defined in Division I, Section 17, "Soil-Thermoplastic Pipe Interaction Systems."

30.1.2 Workmanship and Inspection

All thermoplastic pipe materials shall conform to the workmanship and inspection requirements of AASHTO M 278, M 294, or M 304; or ASTM F 679, F 714, F 794, or F 894 as applicable.

30.2 WORKING DRAWINGS

Whenever specified or requested by the Engineer, the Contractor shall provide manufacturer's installation instructions or working drawings with supporting data in sufficient detail to permit a structural review. Sufficient copies shall be furnished to meet the needs of the Engineer and other entities with review authority. The working drawings shall be submitted sufficiently in advance of proposed installation and use to allow for their review, revision, if needed, and approval without delay of the work. The Contractor shall not start construction of any thermoplastic pipe installations for which working drawings are required until the drawings have been approved by the Engineer. Such approval will not relieve the Contractor of responsibility for results obtained by use of these drawings or any of the other responsibilities under the contract.

30.3 MATERIALS

30.3.1 Thermoplastic Pipe

Polyethylene pipe shall conform to the requirements of AASHTO M 294, or ASTM F 714, or ASTM F 894.

Poly(Vinyl Chloride) (PVC) pipe shall conform to the requirements of AASHTO M 278 or M 304; or ASTM F 679 or F 794.

30.3.2 Bedding Material and Structural Backfill

Bedding and structural backfill shall meet the requirements of AASHTO M 145, A-1, A-2-4, A-2-5, or A-3. Bedding material shall have a maximum particle size of 1.25 inch. Backfill for thermoplastic pipe shall be free of organic material, stones larger than 1½ inch in greatest dimension, or frozen lumps. Moisture content shall be in the range of optimum (typically −3% to +2%) permitting thorough compaction. Consideration should be given to the potential for migration of fines from adjacent materials into open-graded backfill and bedding materials.

For pipe types that are not smooth on the outside (corrugated or profile walls), backfill gradations should be selected that will permit the filling of the corrugation or profile valleys.

Flowable fills, such as controlled low strength mortar (CLSM) or controlled density fill (CDF), may be used for backfill and bedding provided adequate flotation resistance can be achieved by restraints, weighting, or placement technique. With CLSM backfill, trench width can be reduced to a minimum of the outside diameter plus 12 inches. When CLSM is used all joints shall have gaskets.

30.4 ASSEMBLY

30.4.1 General

Thermoplastic pipe shall be assembled in accordance with the manufacturer's instructions. All pipe shall be unloaded and handled with reasonable care. Pipe shall not be rolled or dragged over gravel or rock and shall be prevented from striking rock or other hard objects during placement in trench or on bedding.

Thermoplastic pipe shall be placed in the bed starting at the downstream end.

30.4.2 Joints

Joints for thermoplastic pipe shall meet the performance requirements for soiltightness unless watertightness is specified.

30.4.2.1 Field Joints

Joints shall be so installed that the connection of pipe sections will form a continuous line free from irregularities in the flow line. Suitable field joints can be obtained with the following types of connections:

(a) Corrugated bands (with or without gaskets)
(b) Bell and spigot pipe ends (with or without gaskets)
(c) Double bell couplings (with or without gaskets)

30.5 INSTALLATION

30.5.1 General Installation Requirements

Trenches must be excavated in such a manner as to insure that the sides will be stable under all working conditions. Trench walls shall be sloped or supported in conformance with all standards of safety. Only as much trench as can be safely maintained shall be opened. All trenches shall be backfilled as soon as practicable, but not later than the end of each working day.

Trench details, including foundation, bedding, haunching, initial backfill, final backfill, pipe zone, and trench width are shown in Figure 30.5.1.

30.5.2 Trench Widths

Trench width shall be sufficient to ensure working room to properly and safely place and compact haunching and other backfill materials. The space between the pipe

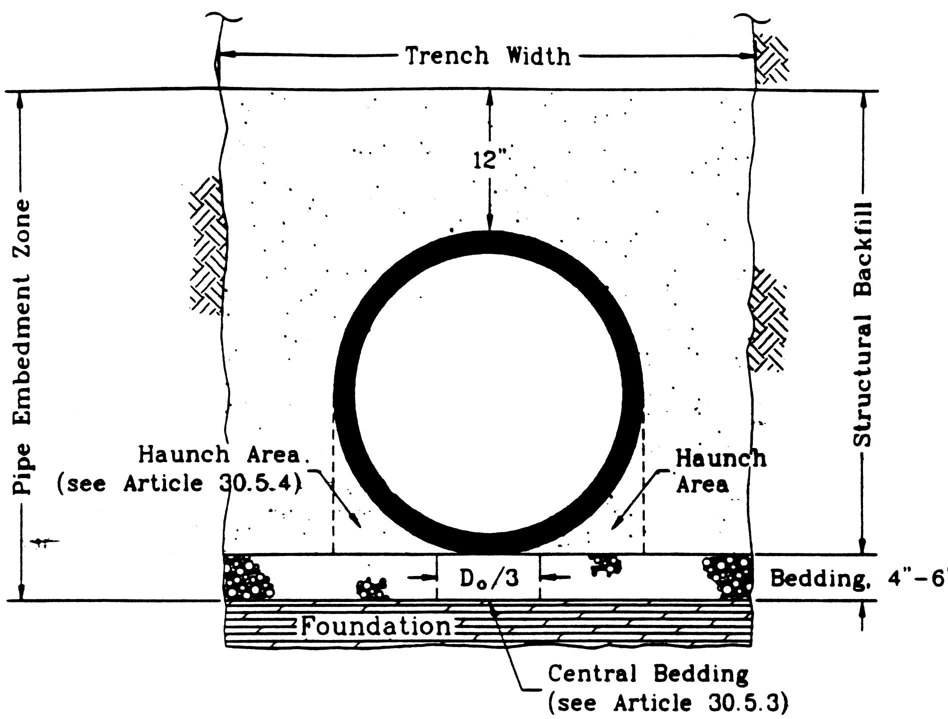


FIGURE 30.5.1