TABLE OF CONTENTS

@Seismicisolation

@Seismicisolation

@Seismicisolation

@Seismicisolation

This page intentionally left blank.

@Seismicisolation

SECTION 12

BURIED STRUCTURES AND TUNNEL LINERS

12.1—SCOPE

This Section provides requirements for the selection of structural properties and dimensions of buried structures, e.g., culverts, and steel plate used to support tunnel excavations in soil.

Buried structure systems considered herein are metal pipe, structural plate pipe, long-span structural plate, deep corrugated plate, structural plate box, reinforced concrete pipe, reinforced concrete cast-in-place and precast arch, box and elliptical structures, thermoplastic pipe, and fiberglass pipe.

The type of liner plate considered is cold-formed steel panels.

C12.1

For buried structures, refer to Article 2.6.6 and the *AASHTO Drainage Manual* (2014) for hydraulic design considerations and for design methods related to location, length, and waterway openings.

Thermoplastic and fiberglass pipe are flexible plastic pipes that have similarities in installation and design; however, not all thermoplastic pipe design and installation specifications are applicable to fiberglass pipe. Fiberglass pipe is a smooth-walled thermoset resin pipe that relies on composite glass fiber within its wall for strength; thermoplastic pipe can have either solid or profile walls of homogenous material. The design specifications for fiberglass pipe are contained in Article 12.15 with reference to applicable sections from the thermoplastic pipe design specifications. Construction specifications for fiberglass pipe are included in the *AASHTO LRFD Bridge Construction Specifications*, Section 30, "Thermoplastic Pipe," with the provisions for thermoplastic pipe applicable to fiberglass pipe installations except as noted.

12.2—DEFINITIONS

Abrasion—Loss of section or coating of a culvert by the mechanical action of water conveying suspended bed load of sand, gravel, and cobble-size particles at high velocities with appreciable turbulence.

Buried Structure—A generic term for a structure built by embankment or trench methods.

Corrosion—Loss of section or coating of a buried structure by chemical and/or electrochemical processes.

Culvert—A curved or rectangular buried conduit for conveyance of water, vehicles, utilities, or pedestrians.

Culvert Material Service Life—The time duration for a material that can satisfy the structural loading, hydraulic loading, and joint performance for the culvert service life duration.

Culvert Service Life—The time duration during which a culvert is expected to provide the desired function with a specified level of maintenance established at the design or retrofit stage.

Deep Corrugated Plate—Structural Plate in AASHTO M 167M/M 167 with a corrugation depth greater than 5.0 in.

FEM—Finite Element Method.

Narrow Trench Width—The outside span of rigid pipe, plus 1.0 ft.

Projection Ratio—Ratio of the vertical distance between the outside top of the pipe and the ground or bedding surface to the outside vertical height of the pipe, applicable to reinforced concrete pipe only.

Side Radius—For deep corrugated plate structures, the side radius is the radius of the plate in the section adjacent to crown (top) section of the structure. In box-shaped structures, this is often called the haunch radius.

Soil Envelope—Zone of controlled soil backfill around culvert structure required to ensure anticipated performance based on soil–structure interaction considerations.

Soil–Structure Interaction System—A buried structure whose structural behavior is influenced by interaction with the soil envelope.

Tunnel—A horizontal or near horizontal opening in soil excavated to a predesigned geometry by tunneling methods exclusive of cut-and-cover methods.

12.3—NOTATION

A = wall area (in.^{2}/ft) (12.7.2.3)
A_{eff} = effective wall area (in.^{2}/in.) (12.12.3.10.1b)
A_g = gross wall area within a length of one period (in.^{2}/in.); area of fiber-reinforced pipe wall per unit length of pipe (12.12.3.5) (12.15.6.3) (12.12.3.10.1b)
A_L = axle load, taken as 50 percent of all axle loads that can be placed on the structure at one time (kip); sum of all axle loads in an axle group (kip); total axle load on single axle or tandem axles (kip) (12.8.4.2) (12.9.4.2) (12.9.4.5)
A_s = tension reinforcement area on cross-section width, b (in.^{2}/ft) (12.10.4.2.4a) (12.10.4.2.4b) (C12.11.4)
A_{smax} = minimum flexural reinforcement area without stirrups (in.^{2}/ft) (12.10.4.2.4c)
A_T = area of the top portion of the structure above the springline (ft^2) (12.8.4.2)
A_{vr} = stirrup reinforcement area to resist radial tension forces on cross-section width, b, in each line of stirrups at circumferential spacing, s_v (in.^{2}/ft) (12.10.4.2.6)
A_{vs} = required area of stirrups for shear reinforcement (in.^{2}/ft) (12.10.4.2.6)
B = width of culvert (ft) (C12.6.2.2.5) (12.11.2.2.1)
B_c = outside diameter or width of the structure (ft) (12.6.6.3) (12.11.2.2.1)
B'_c = out-to-out vertical rise of pipe (ft) (12.6.6.3)
B_d = horizontal width of trench at top of pipe (ft) (12.11.2.2)
B_{FE} = earth load bedding factor (12.10.4.3.1)
B_{FLL} = live load bedding factor (12.10.4.3.1)
B_1 = crack control coefficient for effect of cover and spacing of reinforcement (C12.10.4.2.4d)
b = width of section (12.10.4.2.4c) (12.10.4.2.4a)
b_e = element effective width (in.) (12.12.3.10.1b)
C_A = constant corresponding to the shape of the pipe (12.10.4.3.2a)
C_c = load coefficient for positive pipe projection (12.10.4.3.2a)
C_d = load coefficient for trench installation (12.11.2.2)
C_{dt} = load coefficient for tunnel installation (12.13.2.1)
C_H = adjustment factor for shallow cover heights over metal box culverts (12.9.4.4)
C_L = width of culvert on which live load is applied parallel to span (ft); live load coefficient as specified in Article 12.12.3.5 (12.7.2.2) (12.12.2.2)
C_n = calibration factor to account for nonlinear effects (12.12.3.10.1e)
$C_{\ell\ell}$ = live load adjusted for axle loads, tandem axles, and axles with other than four wheels; $C_1\, C_2\, A_L$ (kip) (12.9.4.2)
C_N = parameter that is a function of the vertical load and vertical reaction (12.10.4.3.2a)
C_n = scalar calibration factor to account for some nonlinear effects (12.8.9.6)
C_S = steel tunnel liner plate construction stiffness (12.13.3.5)
C_s = construction stiffness for tunnel liner plate (kip/in.) (12.5.6.4)
C_1 = 1.0 for single axles and $0.5 + S/50 \leq 1.0$ for tandem axles; adjustment coefficient for number of axles; crack control coefficient for various types of reinforcement (12.9.4.2) (12.9.4.3) (C12.10.4.2.4d)
C_2 = adjustment factor for number of wheels on a design axle as specified in Table 12.9.4.2-1; adjustment coefficient for number of wheels per axle (12.9.4.2) (12.9.4.3)
c = the larger of the distance from neutral axis of profile to the extreme innermost or outermost fiber (in.) (12.12.3.10.2b)
D = straight leg length of haunch (in.); pipe diameter (in.); required D-load capacity of reinforced concrete pipe (klf); diameter to centroid of pipe wall (in.) (C12.6.7) (12.9.4.1) (12.12.2.2)
D-load = resistance of pipe from three-edge bearing test load to produce a 0.01-in. crack (klf) (12.10.4.3.1)
D_f = shape factor (12.12.3.10.2b)
D_L = deflection lag factor (12.12.2.2)
D_o = outside diameter of pipe (in.) (12.12.2.2) (12.7.2.2) (12.10.2.1) (12.15.2)
d = required envelope width adjacent to the structure (ft); distance from compression face to centroid of tension reinforcement (in.) (12.8.5.3) (12.10.4.2.4a) (C12.11.4)
d' = width of warped embankment fill to provide adequate support for skewed installation (ft) (C12.6.8.2)
d_1 = distance from the structure (ft) (12.8.5.3)
E = modulus of elasticity of the plastic (ksi); initial modulus of elasticity (ksi) (12.12.3.3) (12.12.3.6)
E_{cf} = circumferential flexural modulus (ksi) (12.15.5.2) (12.15.3.1)
E_m = modulus of elasticity of metal (ksi) (12.7.2.4)

@Seismicisolation

E_p = short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi) (12.12.2.2) (12.8.9.6)
$E(x)$ = lateral unbalanced distributed load on culvert below sloping ground and skewed at end wall (lbs.) (C12.6.2.2.5)
F = concentrated load acting at the crown of a culvert (kip) (C12.6.2.2.5)
F_c = curvature correction factor (12.10.4.2.5)
F_{cr} = factor for adjusting crack control relative to average maximum crack width of 0.01 in. corresponding to F_{cr} = 1.0 (12.10.4.2.4d)
F_d = factor for crack depth effect resulting in increase in diagonal tension, shear, and strength with decreasing d (12.10.4.2.5)
F_e = soil–structure interaction factor for embankment installations (12.10.2.1)
FF = flexibility factor (in./kip) (12.5.6.3) (12.7.2.6)
F_n = coefficient for effect of thrust on shear strength (12.10.4.2.5)
F_{rp} = factor for process and local materials affecting radial tension strength of pipe (12.10.4.2.3)
F_{rt} = factor for pipe size effect on radial tension strength (12.10.4.2.4c)
F_t = soil–structure interaction factor for trench installations (12.11.2.2.1)
F_u = specified minimum tensile strength (ksi); material yield strength for design load duration (ksi) (12.7.2.4) (12.12.3.10.1b)
F_{vp} = process and material factors for shear strength for design of plant-made reinforced concrete pipe (12.10.4.2.3)
F_y = yield strength of metal (ksi) (12.7.2.3)
f'_c = compressive strength of concrete (ksi) (12.4.2.2) (12.10.4.2.4c)
f_{cr} = critical buckling stress (ksi) (12.7.2.4)
f_s = maximum stress in reinforcing steel at service limit state (ksi) (C12.11.4)
f_y = specified yield point for reinforcing steel (ksi) (12.10.4.2.4a)
H = rise of culvert (ft); height of cover from the box culvert rise to top of pavement (ft); height of cover over crown (ft); height of fill over top of pipe or culvert (ft) (C12.6.2.2.5) (12.9.4.2) (12.9.4.4) (12.9.4.5) (12.10.2.1) (12.12.3.7) (12.15.6.3)
HAF = horizontal arching factor (12.10.2.1)
H_D = vertical distance from mid-depth of corrugation to top grade (12.8.9.4)
H_{design} = design height of cover above top of culvert or above crown of arches or pipes (ft) (C12.6.2.2.5)
H_L = headwall strip reaction (kip) (C12.6.2.2.5)
H_{min} = Minimum allowable cover dimension (C12.6.6.3)
H_w = depth of water table above springline of pipe (ft); height of water over top of pipe (ft) (12.12.3.7) (12.15.6.3)
H_1 = depth of crown of culvert below ground surfaces (ft); height of cover above the footing to traffic surface (ft) (C12.6.2.2.5) (12.8.4.2)
H_2 = actual height of cover above top of culvert or above crown of arches or pipes (ft); height of cover from the structure springline to traffic surface (ft) (C12.6.2.2.5) (12.8.4.2)
h = vertical distance from the top of cover for design height to point of horizontal load application (ft); wall thickness of pipe or box culvert (in.); height of ground surface above top of pipe (ft) (C12.6.2.2.5) (12.10.4.2.4a) (C12.11.4)
I = moment of inertia (in.^{4}/in.) (12.7.2.6) (12.13.3.5)
ID = inside diameter (in.) (12.6.6.3)
I_p = moment of inertia of pipe profile per unit length of pipe (in.^{4}); moment of inertia of the fiber-reinforced pipe wall per unit length (in.^{4}/in.) (12.8.9.6) (12.12.2.2) (12.15.5.2)
i = coefficient for effect of axial force at service limit state, f_s (12.10.4.2.4d) (C12.11.4)
j = coefficient for moment arm at service limit state, f_s (12.10.4.2.4d) (C12.11.4)
K = ratio of the unit lateral effective soil pressure to unit vertical effective soil pressure, i.e., Rankine coefficient of active earth pressure (12.9.4.2)
K_a = factor for uncertainty in level of groundwater table (12.12.3.8)
K_b = bedding coefficient (12.12.2.2)
K_h = lateral earth pressure for culvert under sloping ground (psf/lf) (C12.6.2.2.5)
K_{h1} = lateral earth pressure distribution acting on upslope surface of culvert (psf/lf) (C12.6.2.2.5)
K_{h2} = lateral earth pressure distribution acting on downslope surface of culvert (psf/lf) (C12.6.2.2.5)
K_t = time factor as specified in Table 12.12.3.10.1b-1 (12.12.3.10.1b)
K_{wa} = factor for uncertainty in level of ground water table (C12.12.3.8)
$K_{\gamma E}$ = installation factor (12.12.3.5)
k = soil stiffness factor; edge support coefficient; plate buckling coefficient (12.7.2.4) (C12.13.3.3) (12.12.3.10.1b)

@Seismicisolation

L = distance along length of culvert from expansion joint to the centerline of the headwall (ft); length of stiffening rib on leg (in.) (C12.6.2.2.5) (12.9.4.1)
$LLDF$ = live load distribution factor as specified in Article 3.6.1.2.6 (12.7.2.2)
L_0 = length of live load surface contact area parallel to pipe diameter as specified in Article 3.6.1.2.5 (in.) (12.6.1)
L_w = lane width (ft); horizontal live load distribution width in the circumferential direction, at the elevation of the crown (in.) (12.8.4.2) (12.12.3.5)
ℓ_d = development length (12.10.4.4.3)
ℓ_{hd} = basic development length of welded deformed wire reinforcement (in) (12.10.4.4.4)
ℓ_w = live load patch length at depth H as specified in Article 3.6.1.2.6 (12.7.2.2)
$M_{d\ell}$ = dead load moment (kip-ft/ft); sum of the nominal crown and haunch dead load moments (kip-ft/ft) (12.9.4.2)
$M_{d\ell u}$ = factored dead load moment as specified in Article 12.9.4.2 (kip-ft) (12.9.4.3)
$M_{\ell\ell}$ = live load moment (kip-ft/ft); sum of the nominal crown and haunch live load moments (kip-ft/ft) (12.9.4.2)
$M_{\ell\ell u}$ = factored live load moment as specified in Article 12.9.4.2 (kip-ft) (12.9.4.3)
M_n = factored moment resistance (12.8.9.5)
M_{nu} = factored moment acting on cross-section width, b, as modified for effects of compressive or tensile thrust (kip-in./ft) (12.10.4.2.6)
M_P = plastic moment capacity of deep corrugated structure (k-ft/ft) (12.8.9.4)
M_p = plastic moment capacity of section (12.8.9.5)
M_{pc} = crown plastic moment capacity (kip-ft/ft) (12.9.4.3)
M_{ph} = haunch plastic moment capacity (kip-ft/ft) (12.9.4.3)
M_s = flexural moment at service limit state (kip-in./ft); moment acting on a cross-section of width, b, at service limit state taken as an absolute value in design equations (kip-in./ft); secant constrained soil modulus as specified in Table 12.12.3.5-1 or Table 12.12.3.5-3 (ksi) (12.8.9.6) (12.10.4.2.4d) (C12.11.4) (12.12.2.2) (12.12.3.5)
M_{thick} = adjusted moment (12.10.4.2.2)
M_{thin} = moment calculated using thin ring theory (12.10.4.2.2)
M_u = factored applied moment; ultimate moment acting on cross-section width, b (kip-in./ft) (12.8.9.5) (12.10.4.2.4a)
N_s = axial thrust acting on a cross-section width, b, at service limit state taken as positive when compressive and negative when tensile (kip/ft) (12.10.4.2.4d) (C12.11.4)
N_u = axial thrust acting on cross-section width, b, at strength limit state (kip/ft) (12.10.4.2.4a)
n = number of adjoining traffic lanes (12.8.4.2)
P_{Brg} = allowable bearing pressure to limit compressive strain in the trench wall or embankment (ksf) (12.8.5.3)
P_c = proportion of total moment carried by crown of metal box culvert (12.9.4.3)
P_{FD} = factored dead load vertical crown pressure as specified in Article 12.12.3.4 with VAF taken as 1.0 and D_o taken as S (ksf) (12.7.2.2)
P_{FL} = factored live load vertical crown pressure as specified in Article 12.12.3.4 (ksf) (12.7.2.2)
P_L = live load pressure as specified in Article 3.6.1.2.6 (psi) (12.12.2.2) (12.15.5.2)
P_{sp} = soil prism pressure evaluated at pipe springline as specified in Article 12.12.3.7 (psi); soil prism pressure (EV), evaluated at the top of the pipe determined as the unit weight of soil multiplied by the soil cover depth (psi); soil prism pressure (EV), evaluated at the crown of the pipe determined as the unit weight of soil multiplied by the soil cover depth (psi) (12.12.2.2) (12.15.5.2) (12.15.6.3)
P_{st} = stub compression capacity from T 341 (lb/in.) (12.12.3.10.1b)
P_u = design factored load (psi) (12.12.3.5)
P_w = hydrostatic water pressure (psi) (12.12.3.8)
P_1 = horizontal pressure from the structure at a distance, d_1 (ksf) (12.8.5.3)
p = positive projection ratio (12.10.4.3.2a)
q = ratio of the total lateral pressure to the total vertical pressure (12.10.4.3.2a)
R = rise of structure (ft); rise of box culvert or long-span structural plate structures (ft); average radius of the concrete pipe; radius to centroid of pipe wall profile (in.) (12.8.4.1) (12.9.4.1) (12.10.4.2.2) (12.12.2.2) (12.12.3.5)
R_{AL} = axle load correction factor (12.9.4.6)
R_b = nominal axial force in culvert wall to cause general buckling (12.8.9.6)
R_c = corner radius of the structure (ft); concrete strength correction factor (12.8.5.3) (12.9.4.6)
R_f = factor related to required relieving slab thickness, applicable for box structures where the span is less than 26.0 ft (12.9.4.6)
R_H = horizontal footing reaction component (kip/ft) (12.8.4.2) (12.9.4.3)

@Seismicisolation

R_h = haunch moment reduction factor; correction factor for backfill soil geometry (12.8.9.6) (12.9.4.3) (12.12.3.10.1e)
R_n = nominal resistance (klf) (12.5.1)
R_r = factored resistance (klf); factored resistance to thrust (kip/ft) (12.5.1)
R_T = top arc radius of long-span structural plate structures (ft) (12.8.3.2)
R_t = factored thrust resistance (12.8.9.5)
R_V = vertical footing reaction component (kip/ft) (12.8.4.2)
R_w = water buoyancy factor (12.15.6.3)
R_ϕ = (ϕ_r/ϕ_f); ratio of resistance factors for radial tension and moment specified in Article 12.5.5 (12.10.4.2.4c)
r = radius of gyration (in.); radius to centerline of concrete pipe wall (in.) (12.7.2.4) (12.10.4.2.5)
r_c = radius of crown (ft) (12.9.4.1)
r_h = radius of haunch (ft) (12.9.4.1)
r_s = radius of the inside reinforcement (in.) (12.10.4.2.4c)
r_{sd} = settlement ratio parameter (12.10.4.3.2a)
S = pipe, tunnel, or box diameter or span (in. or ft as indicated); culvert span (ft); span of structure between springlines of long-span structural plate structures (ft); box culvert span (ft) (12.6.6.3) (C12.6.7) (12.7.2.2) (12.7.2.4) (12.8.4.1) (12.8.4.2) (12.8.9.6) (12.9.4.1) (12.9.4.2) (12.9.4.5) (12.12.3.6)
S_b = long-term ring-bending strain (in./in.) (12.15.3.2)
S_H = hoop stiffness factor (12.12.3.5)
S_i = internal diameter or horizontal span of the pipe (in.) (12.10.4.2.4b)
S_ℓ = spacing of circumferential reinforcement (in.) (12.10.4.2.4d)
S_1, S_2 = shear forces acting along culvert bearing lines (lbs.) (C12.6.2.2.5)
s_v = spacing of stirrups (in.) (12.10.4.2.6)
T = total dead load and live load thrust in the structure (kip/ft) (12.8.5.3)
T_f = factored thrust (12.8.9.5)
T_L = factored thrust per unit length (kip/ft) (12.7.2.2)
T_s = service thrust per unit length (lb/in.) (12.12.3.5)
T_u = factored thrust per unit length (lb/in.) (12.12.3.5)
t = required thickness of cement concrete relieving slab (in.); thickness of element (in.) (12.9.4.6) (12.10.4.2.2) (12.12.3.10.1b)
t_b = basic thickness of cement concrete relieving slab (in.); clear cover over reinforcement (in.) (12.9.4.6) (12.10.4.2.4d)
V = unfactored footing reaction (kip/ft) (12.9.4.5)
VAF = vertical arching factor (12.7.2.2) (12.10.2.1)
V_c = factored shear force acting on cross-section width, b, which produces diagonal tension failure without stirrup reinforcement (kip/ft) (12.10.4.2.6)
V_{DL} = $[H_2(S) - A_T]\ \gamma_s/2$ (kip/ft) (12.8.4.2)
V_L = headwall strip reaction (kip) (C12.6.2.2.5)
V_{LL} = $n(A_L)/(8 + 2\ H_1)$ (kip/ft) (12.8.4.2)
V_n = nominal shear resistance of pipe section without radial stirrups per unit length of pipe (kip/ft) (12.10.4.2.5)
V_r = factored shear resistance per unit length (kip/ft) (12.10.4.2.5)
V_u = ultimate shear force acting on cross-section width, b (kip/ft) (12.10.4.2.5)
W_E = total unfactored earth load on pipe or liner (kip/ft) (12.10.2.1)
W_F = fluid load in the pipe (kip/ft) (12.10.4.3.1)
W_L = total unfactored live load on unit length pipe specified in Article 12.10.2.3 (kip/ft) (12.10.4.3.1)
W_T = total dead and live load on pipe or liner (kip/ft) (12.10.4.3.1)
w = unit weight of soil (pcf); total clear width of element between supporting elements (in.) (12.10.2.1) (12.12.3.10.1b)
w_t = tire patch width as specified in Article 3.6.1.2.5 (in.) (12.7.2.2)
x = parameter which is a function of the area of the vertical projection of the pipe over which active lateral pressure is effective (12.10.4.3.2a)
α = skew angle between the highway centerline or tangent thereto and the culvert headwall (degrees) (C12.6.2.2.5)
β = angle of fill slope measured from horizontal (degrees) (C12.6.2.2.5)
γ_b = unit weight of buoyant soil (lb/ft^3) (12.12.3.7)
γ_{EV} = load factor for vertical pressure from dead load of earth fill; load factor for vertical earth pressure (12.12.3.5) (12.12.3.10.2b) (12.15.6.3)
γ_{LL} = load factor for live load (12.12.3.5)

@Seismicisolation

γ_s = unit weight of backfill (kcf); soil unit weight (kcf); wet unit weight of soil (lb/ft^3) (12.8.4.2) (C12.9.2) (12.9.4.2) (12.9.4.5) (12.11.2.2) (12.12.3.7) (12.13.2.1)
γ_w = unit weight of water (lb/ft^3) (12.12.3.8) (12.15.6.3)
γ_{WA} = load factor for hydrostatic pressure (12.12.3.5)
Δ = calculated differential settlements across the structure taken from springline-to-springline (12.8.4.1); return angle of the structure (degrees) (12.8.4.2); haunch radius included angle (12.9.4.1)
Δ_A = total allowable deflection of pipe (in.) (12.12.2.2) (12.15.4)
Δ_f = deflection of pipe due to flexure (in.) (12.12.3.10.2b)
Δ_t = total deflection of pipe (in.) (12.12.2.2)
ε_{bck} = nominal strain capacity for general buckling (in./in.) (12.12.3.10.1e)
ε_{fb} = factored buckling strain demand (in./in.) (12.15.6.3)
ε_f = factored strain due to flexure (in./in.) (12.12.3.10.2b)
ε_{sc} = service compressive strain as specified in Article 12.12.3.10.1c (in./in.) (12.12.2.2)
ε_{uc} = factored compressive strain due to thrust (12.12.3.10.1c)
ε_{yc} = factored compressive strain limit as specified in Table 12.12.3.3-1 (12.12.3.10.1b)
ε_{yt} = service long-term strain limit as specified in Table 12.12.3.3-1 (12.12.3.10.2b)
η_{EV} = load modifier, specified in Article 1.3.2, as it applies to vertical earth loads on culverts (12.12.3.5)
η_{LL} = load modifier, specified in Article 1.3.2, as it applies to live loads on culverts (12.12.3.5)
θ = rotation of the structure (C12.8.4.1)
λ = slenderness factor (12.12.3.10.1b)
ν = Poisson's ratio of soil (12.12.3.10.1e)
ρ = effective width factor (12.10.4.2.5) (12.12.3.10.1b)
ϕ = resistance factor (12.5.1) (12.5.5) (12.7.2.3)
ϕ_b = resistance factor for general buckling (12.5.5) (12.8.9.6)
ϕ_{bck} = resistance factor for global buckling (12.5.5) (12.12.3.10.1e)
ϕ_f = resistance factor for flexure (12.5.5) (12.10.4.2.4c)
ϕ_h = resistance factor for plastic hinge (12.5.5)
ϕ_r = resistance factor for radial tension (12.5.5) (12.10.4.2.4c) (12.10.4.2.6)
ϕ_s = resistance factor for soil stiffness, $\phi_s = 0.9$; resistance factor for soil pressure (12.5.5) (12.8.9.6) (12.12.3.5) (12.12.3.10.1e)
ϕ_T = resistance factor for thrust effects (12.12.3.10.1d)
Ψ = central angle of pipe subtended by assumed distribution of external reactive force (degrees) (12.10.4.2.1)
ω = spacing of corrugation (in.) (12.12.3.10.1b)

12.3.1—Abbreviations

GC: clayey gravel
GM: silty gravel
GP: poorly-graded gravel
GW: well-graded gravel
PE: polyethylene
PL: prism load
PP: polypropylene
PVC: polyvinyl chloride
SC: clayey sand
SM: silty sand
SP: poorly-graded sand
SW: well-graded sand

12.4—SOIL AND MATERIAL PROPERTIES

12.4.1—Determination of Soil Properties

12.4.1.1—General

Subsurface exploration shall be carried out to determine the presence and influence of geologic and environmental conditions that may affect the performance of buried structures. For buried structures

C12.4.1.1

The following information may be useful for design:

- Strength and compressibility of foundation materials;

supported on footings and for pipe arches and large diameter pipes, a foundation investigation should be conducted to evaluate the capacity of foundation materials to resist the applied loads and to satisfy the movement requirements of the structure.

- Chemical characteristics of soil and surface waters, e.g., pH, resistivity, and chloride content of soil and pH, resistivity, and sulfate content of surface water;
- Stream hydrology, e.g., flow rate and velocity, maximum width, allowable headwater depth, and scour potential; and
- Performance and condition survey of culverts in the vicinity.

12.4.1.2—Foundation Soils

The type and anticipated behavior of the foundation soil shall be considered for stability of bedding and settlement under load.

C12.4.1.2

Refer to Article 10.4 for general guidance regarding foundation soil properties. The performance of rigid pipes is dependent on foundation and bedding stability.

12.4.1.3—Envelope Backfill Soils

The type, compacted density, and strength properties of the soil envelope adjacent to the buried structure shall be established. The backfill soils comprising the soil envelope shall conform to the requirements of AASHTO M 145 as follows:

- For standard flexible pipes and concrete structures: A-1, A-2, or A-3 (GW, GP, SW, SP, GM, SM, SC, GC),
- For metal box culverts and long-span structures with cover less than 12.0 ft: A-1, A-2-4, A-2-5, or A-3 (GW, GP, SW, SP, GM, SM, SC, GC),
- For long-span metal structures with cover not less than 12.0 ft: A-1 or A-3 (GW, GP, SW, SP, GM, SM),
- For structural plate culverts with deep corrugations: A-1, A-2-4, A-2-5, or A-3 (ASTM D2487) (GW, GP, SW, SP, GM, SM, SC, GC) and the culvert manufacturer's requirements, and
- For thermoplastic, fiberglass, and steel-reinforced thermoplastic culverts; bedding; and backfill materials: A-1, A-2-4, A-2-5, or A-3 soils. A maximum of 50 percent of the particle sizes may pass the No. 100 sieve and a maximum of 20 percent may pass the No. 200 sieve.

C12.4.1.3

Refer to Sections 26, 27, and 30 of the *AASHTO LRFD Bridge Construction Specifications* for compaction criteria of soil backfill for flexible and rigid culverts.

Refer to ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), for further information on equivalent soil classifications.

Wall stresses in buried structures are sensitive to the relative stiffness of the soil and pipe. Buckling stability of flexible culverts is dependent on soil stiffness.

In the selection of a type of backfill for the envelope, the quality of the material and its suitability for achieving the requirements of the design should be considered. The order of preference for selecting envelope backfill based on quality may be taken as follows:

- Angular, well-graded sand and gravel;
- Nonangular, well-graded sand and gravel;
- Flowable materials, e.g., cement–soil–fly ash mixtures, which result in low density/low strength backfill, for trench applications only;
- Uniform sand or gravel, provided that placement is confirmed to be dense and stable, but which may require a soil or geofabric filter to prevent the migration of fines;
- Clayey sand or gravel of low plasticity; and
- Stabilized soil, which should be used only under the supervision of an Engineer familiar with the behavior of the material.

The restriction on materials passing the No. 100 sieve and No. 200 sieve for thermoplastic, fiberglass, and steel-reinforced thermoplastic culverts is intended to eliminate uniform fine sands for use as pipe embedment. Such materials are difficult to work with, are sensitive to moisture content, and do not provide support comparable to coarser or more broadly graded materials at the same percentage of maximum density. The Engineer may permit exceptions to these restrictions in special cases. If so, a suitable plan should be submitted for control of moisture content and compaction procedures. These silty

@Seismicisolation

and clayey materials should never be used in a wet site. Increased inspection levels should be considered if such a plan is approved.

12.4.2—Materials

12.4.2.1—Aluminum Pipe and Structural Plate Structures

Aluminum for corrugated metal pipe and pipe-arches shall comply with the requirements of AASHTO M 196 (ASTM B745). Aluminum for structural plate pipe, pipe-arch, arch, and box structures shall meet the requirements of AASHTO M 219 (ASTM B746).

12.4.2.2—Concrete

Concrete shall conform to Article 5.4, except that f'_c may be based on cores.

12.4.2.3—Precast Concrete Pipe

Precast concrete pipe shall comply with the requirements of AASHTO M 170M and M 170 (ASTM C76M and C76), M 242M/M 242 (ASTM C655M and C655), or ASTM C1765. Design wall thickness other than the standard wall dimensions may be used, provided that the design complies with all applicable requirements of this Section.

12.4.2.4—Precast Concrete Structures

Precast concrete arch, elliptical, and box structures shall comply with the requirements of AASHTO M 206M/M 206 (ASTM C506M and C506), M 207M/M 207 (ASTM C507M and C507), M 259 (ASTM C789), and M 273 (ASTM C850).

12.4.2.5—Steel Pipe and Structural Plate Structures

Steel for corrugated metal pipe and pipe-arches shall comply with the requirements of AASHTO M 36 (ASTM A760/A760M). Steel for structural plate pipe, pipe-arch, arch, and box structures shall meet the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M).

12.4.2.6—Deep Corrugated Structures

Steel for deep corrugated structural plate shall comply with the requirements of AASHTO M 167M/M 167. Deep corrugated structural plate may be reinforced.

C12.4.2.6

Reinforcement for deep corrugated structures may consist of structural shapes, or deep corrugated structural plate meeting the requirements of AASHTO M 167M/M 167, with or without nonshrink grout, complete with shear studs.

12.4.2.7—Steel Reinforcement

Reinforcement shall comply with the requirements of Article 5.4.3, and shall conform to one of the following: AASHTO M 31M/M 31 (ASTM A615/A615M),

M 32M/M 32, M 55M/M 55, M 221M/M 221, M 336M/M 336, ASTM A1064/A1064M, or ASTM A706/A706M.

The nominal yield strength shall be the minimum as specified for the grade of steel selected, but shall not exceed 80 ksi.

12.4.2.8—Thermoplastic Pipe

Plastic pipe may be solid wall, corrugated, or profile wall and may be manufactured of polyethylene (PE) or polyvinyl chloride (PVC).

PE pipe shall comply with the requirements of ASTM F714 for solid wall pipe, AASHTO M 294 for corrugated pipe, and ASTM F894 for profile wall pipe.

PVC pipe shall comply with the requirements of AASHTO M 278 for solid wall pipe, ASTM F679 for solid wall pipe, and AASHTO M 304 for profile wall pipe.

12.4.2.9—Fiberglass Pipe

Glass-fiber-reinforced (fiberglass) pipe shall be solid wall and manufactured from glass-fiber-reinforced thermosetting resins, with or without aggregates or liners, and shall comply with the requirements of ASTM D3262. For pipe exposed to sunlight after installation, in addition to the requirements of ASTM D3262, the liner and surface layers shall provide protection from ultraviolet exposure for the design life of the product.

C12.4.2.9

ASTM D3262 specifies pipe for use in conveyance of sanitary sewer, storm water, and some industrial waste, primarily in buried applications. The standard includes pipes with stiffness from 9 to 72 psi. The lowest pipe stiffness allowed, 9 psi, should only be used for specific applications (e.g., concrete encasement), where the pipe is not subjected to typical installation forces such as backfill compaction.

12.4.2.10—Steel-Reinforced Thermoplastic Culverts

Steel-reinforced thermoplastic (SRPE) culverts have a profile wall which may be ribbed or corrugated. Ribbed profile wall steel-reinforced thermoplastic culverts shall meet the requirements of AASHTO M 335 and MP 42 for larger diameters (66.0 to 120 in.). Corrugated profile wall steel-reinforced thermoplastic culverts shall meet requirements of AASHTO MP 42.

12.5—LIMIT STATES AND RESISTANCE FACTORS

12.5.1—General

Buried structures and their foundations shall be designed by the appropriate methods specified in Articles 12.7 through 12.15 to resist the load combinations specified in Articles 12.5.2 and 12.5.3.

The factored resistance, R_r, shall be calculated for each applicable limit state as:

$$R_r = \phi R_n \tag{12.5.1-1}$$

where:

R_n = the nominal resistance

ϕ = the resistance factor specified in Table 12.5.5-1

C12.5.1

Procedures for determining nominal resistance are provided in Articles 12.7 through 12.15 for:

- Metal pipe, pipe arches, and arch structures;
- Long-span and deep corrugated structural plate structures;
- Structural plate box structures;
- Reinforced precast concrete pipe;
- Reinforced concrete cast-in-place and precast box structures and reinforced cast-in-place arches;
- Thermoplastic pipe;
- Steel tunnel liner plate;

- Precast reinforced concrete three-sided structures;
- Fiberglass pipe; and
- Steel-reinforced thermoplastic culverts.

12.5.2—Service Limit State

Buried structures shall be investigated at Service Load Combination I, as specified in Table 3.4.1-1.

- Deflection of metal structures, tunnel liner plate, thermoplastic pipe, and fiberglass pipe, and
- Crack width in reinforced concrete structures.

C12.5.2

Deflection of a tunnel liner depends significantly on the amount of overexcavation of the bore and is affected by delay in backpacking or inadequate backpacking. The magnitude of deflection is not primarily a function of soil modulus or the liner plate properties, so it cannot be computed with usual deflection formulae.

Where the tunnel clearances are important, the designer should oversize the structure to allow for deflection.

12.5.3—Strength Limit State

Buried structures and tunnel liners shall be investigated for construction loads and at Strength Load Combinations I and II, as specified in Table 3.4.1-1, as follows:

- For metal structures:
  - Wall area
  - Buckling
  - Seam failure
  - Flexibility limit for construction
  - Flexure of box and deep corrugated structures only
- For concrete structures:
  - Flexure
  - Shear
  - Thrust
  - Radial tension
- For thermoplastic pipe:
  - Wall area
  - Buckling
  - Flexibility limit
- For tunnel liner plate:
  - Wall area
  - Buckling
  - Seam strength
  - Construction stiffness
- For fiberglass pipe:
  - Flexure
  - Buckling
  - Flexibility limit

C12.5.3

Strength Load Combinations III and IV and the extreme event limit state do not control due to the relative magnitude of loads applicable to buried structures as indicated in Article 12.6.1. Buried structures have been shown not to be controlled by fatigue.

Flexibility limit requirement is waived for some metal structures. See design provisions in Article 12.8.

Thermoplastic pipe has many profile wall geometries and some of these are made up of thin sections that may be limited based on local buckling. The strength limit state for wall area includes evaluating the section capacity for local buckling.

@Seismicisolation

- For steel-reinforced thermoplastic culverts:
  - Wall area
  - Buckling
  - Flexibility limit for construction

Steel-reinforced thermoplastic culverts are comprised of thin sections that may be limited due to local buckling. The strength limit state for wall area includes evaluating the section capacity for local buckling.

12.5.4—Load Modifiers and Load Factors

Load modifiers shall be applied to buried structures and tunnel liners as specified in Article 1.3, except that the load modifiers for construction loads shall be taken as 1.0. For strength limit states, buried structures shall be considered redundant. Operational classification shall be determined on the basis of continued function and/or safety of the roadway.

12.5.5—Resistance Factors

Resistance factors for buried structures shall be taken as specified in Table 12.5.5-1. Values of resistance factors for the geotechnical design of foundations for buried structures shall be taken as specified in Section 10.

C12.5.5

The standard installations for direct design of concrete pipe were developed based on extensive parameter studies using the soil–structure interaction program, SPIDA. Although past research validates that SPIDA soil–structure models correlate well with field measurements, variability in culvert installation methods and materials suggests that the design for Type I installations be modified. This revision reduces soil–structure interaction for Type I installations by ten percent until additional performance documentation on installation in the field is obtained.

The new thermoplastic design method evaluates more load conditions than prior specifications. Separate resistance factors are provided for each mode of behavior. The resistance factor for buckling is set at 0.7 and preserves the same level of safety as prior editions of these Specifications with the inclusion of the installation factor of Article 12.12.3.5. Buckling is an undesirable failure mode for culverts. Buckling can result in near total collapse of the culvert and blockage of the waterway.

@Seismicisolation

Table 12.5.5-1—Resistance Factors for Buried Structures

Structure Type	Resistance Factor
Metal Pipe, Arch, and Pipe Arch Structures	
Helical pipe with lock seam or fully welded seam:	
• Minimum wall area and buckling	1.00
Annular pipe with spot-welded, riveted, or bolted seam:	
• Minimum wall area and buckling	1.00
• Minimum longitudinal seam strength	0.67
• Bearing resistance to pipe arch foundations	Refer to Section 10
Structural plate pipe:	
• Minimum wall area and buckling	1.00
• Minimum longitudinal seam strength	0.67
• Bearing resistance to pipe arch foundations	Refer to Section 10
Long-Span Structural Plate and Tunnel Liner Plate Structures	
• Minimum wall area	0.67
• Minimum seam strength	0.67
• Bearing resistance of pipe arch foundations	Refer to Section 10
Structural Plate Box Structures	
• Plastic moment strength	1.00
• Bearing resistance of pipe arch foundations	Refer to Section 10
Reinforced Concrete Pipe	
Direct design method:	
Type 1 installation:	
• Flexure	0.90
• Shear	0.82
• Radial tension	0.82
Other type installations:	
• Flexure	1.00
• Shear	0.90
• Radial tension	0.90
Reinforced Concrete Cast-in-Place Box Structures	
• Flexure	0.90
• Shear	0.85
Reinforced Concrete Precast Box Structures	
• Flexure	1.00
• Shear	0.90
Reinforced Concrete Precast Three-Sided Structures	
• Flexure	0.95
• Shear	0.90
Thermoplastic Pipe	
PE and PVC pipe:	
• Thrust, ϕ_T	1.00
• Soil stiffness, ϕ_s	0.90
• Global buckling, ϕ_{bck}	0.70
• Flexure, ϕ_f	1.00
Fiberglass Pipe	
• Flexure, ϕ_f	0.9
• Global Buckling, ϕ_{bck}	0.63
Deep Corrugated Structural Plate Structures	
• Minimum wall area and general buckling, ϕ_b	0.70
• Plastic hinge, φ_h	0.90
• Soil, ϕ_s	0.90
Steel-Reinforced Thermoplastic Culverts	
• Minimum wall area and buckling	1.00

@Seismicisolation

12.5.6—Flexibility Limits and Construction Stiffness

12.5.6.1—Corrugated Metal Pipe and Structural Plate Structures

Flexibility factors for corrugated metal pipe, structural plate structures, and steel-reinforced thermoplastic culverts shall not exceed the values specified in Table 12.5.6.1-1.

C12.5.6.1

Limits on construction stiffness and plate flexibility are construction requirements that do not represent any limit state in service.

Table 12.5.6.1-1—Flexibility Factor Limit

Type of Construction Material	Corrugation Size (in.)	Flexibility Factor (in./kip)
Steel Pipe	0.25	43
	0.5	43
	1.0	33
Aluminum Pipe	0.25 and 0.50	
	0.060 Material Thk.	31
	0.075 Material Thk.	61
	All Others	92
	1.0	60
Steel Plate	6.0 × 2.0	
	Pipe	20
	Pipe-Arch	30
	Arch	30
Aluminum Plate	9.0 × 2.5	
	Pipe	25
	Pipe-Arch	36
	Arch	36
Steel-Reinforced Thermoplastic Pipe		68

12.5.6.2—Spiral Rib Metal Pipe and Pipe Arches

Flexibility factors for spiral rib metal pipe and pipe arches shall not exceed the values, specified in Table 12.5.6.2-1, for embankment installations conforming to the provisions of Articles 12.6.6.2 and 12.6.6.3 and for trench installations conforming to the provisions of Articles 12.6.6.1 and 12.6.6.3.

Table 12.5.6.2-1—Flexibility Factor Limits

Material	Condition	Corrugation Size (in.)	Flexibility Factor (in./kip)
Steel	Embankment	0.75 × 0.75 × 7.5	$217I^{1/3}$
		0.75 × 1.0 × 11.5	$140I^{1/3}$
	Trench	0.75 × 0.75 × 7.5	$263I^{1/3}$
		0.75 × 1.0 × 11.5	$163I^{1/3}$
Aluminum	Embankment	0.75 × 0.75 × 7.5	$340I^{1/3}$
		0.75 × 1.0 × 11.5	$175I^{1/3}$
	Trench	0.75 × 0.75 × 7.5	$420I^{1/3}$
		0.75 × 1.0 × 11.5	$215I^{1/3}$

@Seismicisolation

Values of inertia, I, for steel and aluminum pipes and pipe arches shall be taken as tabulated in Tables A12-2 and A12-5.

12.5.6.3—Thermoplastic and Fiberglass Pipe

Flexibility factor, FF, of thermoplastic and fiberglass pipe shall not exceed 95.0 in./kip.

C12.5.6.3

PE and PVC are thermoplastic materials that exhibit higher flexibility factors at high temperatures and lower flexibility factors at low temperatures. The specified flexibility factor limits are defined in relation to pipe stiffness values in accordance with ASTM D2412 at 73.4 degrees F.

12.5.6.4—Steel Tunnel Liner Plate

Construction stiffness, C_s, in kip/in., shall not be less than the following:

- Two-flange liner plate

  $C_s \geq 0.050$ (kip/in.)

- Four-flange liner plate

  $C_s \geq 0.111$ (kip/in.)

C12.5.6.4

Assembled liner using two- and four-flange liner plates does not provide the same construction stiffness as a full steel ring with equal stiffness.

12.6—GENERAL DESIGN FEATURES

12.6.1—Loading

Buried structures shall be designed for force effects resulting from horizontal and vertical earth pressure, pavement load, live load, and vehicular dynamic load allowance. Earth surcharge, live load surcharge, downdrag loads, and external hydrostatic pressure shall be evaluated where construction or site conditions warrant. Water buoyancy loads shall be evaluated for buried structures with inverts below the water table to control flotation, as indicated in Article 3.7.2. Earthquake loads should be considered only where buried structures cross active faults.

For vertical earth pressure, the maximum load factor from Table 3.4.1-2 shall apply.

Wheel loads shall be distributed through earth fills according to the provisions of Article 3.6.1.2.6.

C12.6.1

Buried structures benefit from both earth shelter and support that reduce or eliminate from concern many of the loads and load combinations of Article 3.4. Wind, temperature, vehicle braking, and centrifugal forces typically have little effect due to earth protection. Structure dead load, pedestrian live load, and ice loads are insignificant in comparison with force effects due to earth fill loading. External hydrostatic pressure, if present, can add significantly to the total thrust in a buried pipe.

Vehicular collision forces are applicable to appurtenances such as headwalls and railings only. Water, other than buoyancy and vessel collision loads, can act only in the noncritical longitudinal direction of the culvert.

Due to the absence or low magnitude of these loadings, Service Load Combination I, Strength Load Combinations I and II, or construction loads control the design.

The finite element analyses used in the preparation of these metal box structure provisions are based on conservative soil properties of low plasticity clay (CL) compacted to 90 percent density as specified in AASHTO T 99. Although low plasticity clay is not considered an acceptable backfill material, as indicated in Article 12.4.1.3, the FEM results have been shown to yield conservative, upperbound moments.

The loading conditions that cause the maximum flexural moment and thrust are not necessarily the same, nor are they necessarily the conditions that will exist at the final configuration.

@Seismicisolation

12.6.2—Service Limit State

12.6.2.1—Tolerable Movement

Tolerable movement criteria for buried structures shall be developed based on the function and type of structure, anticipated service life, and consequences of unacceptable movements.

12.6.2.2—Settlement

12.6.2.2.1—General

Settlement shall be determined as specified in Article 10.6.2. Consideration shall be given to potential movements resulting from:

- Longitudinal differential settlement along the length of the pipe,
- Differential settlement between the pipe and backfill, and
- Settlement of footings and unbalanced loading of skewed structures extending through embankment slopes.

12.6.2.2.2—Longitudinal Differential Settlement

Differential settlement along the length of buried structures shall be determined in accordance with Article 10.6.2.4. Pipes and culverts subjected to longitudinal differential settlements shall be fitted with positive joints to resist disjointing forces meeting the requirements of Sections 26, "Metal Culverts," and 27, "Concrete Culverts," of the *AASHTO LRFD Bridge Construction Specifications*.

Camber may be specified for an installation to ensure hydraulic flow during the service life of the structure.

12.6.2.2.3—Differential Settlement between Structure and Backfill

Where differential settlement of arch structures is expected between the structure and the side fill, the foundation should be designed to settle with respect to the backfill.

Pipes with inverts shall not be placed on foundations that will settle much less than the adjacent side fill, and a uniform bedding of loosely compacted granular material should be provided.

C12.6.2.2.3

The purpose of this provision is to minimize downdrag loads.

12.6.2.2.4—Footing Settlement

Footings shall be designed to provide uniform longitudinal and transverse settlement. The settlement of footings shall be large enough to provide protection against possible downdrag forces caused by settlement of adjacent fill. If poor foundation materials are encountered, consideration shall be given to excavation

C12.6.2.2.4

Metal pipe arch structures, long-span arch structures, and box culvert structures should not be supported on foundation materials that are relatively unyielding compared with the adjacent sidefill. The use of massive footings or piles to prevent settlement of such structures is not recommended.

@Seismicisolation

of all or some of the unacceptable material and its replacement with compacted acceptable material.

In general, provisions to accommodate uniform settlement between the footings are desirable, provided that the resulting total settlement is not detrimental to the function of the structure.

Footing design shall comply with the provisions of Article 10.6.

Footing reactions for metal box culvert structures shall be determined as specified in Article 12.9.4.5.

The effects of footing depth shall be considered in the design of arch footings. Footing reactions shall be taken as acting tangential to the arch at the point of connection to the footing and to be equal to the thrust in the arch at the footing.

12.6.2.2.5—Unbalanced Loading

Buried structures skewed to the roadway alignment and extending through an embankment fill shall be designed in consideration of the influence of unsymmetrical loading on the structure section.

C12.6.2.2.5

Disregard of the effect of lateral unbalanced forces in the headwall design can result in failure of the headwall and adjacent culvert sections.

Due to the complexity of determining the actual load distribution on a structure subjected to unbalanced loading, the problem can be modeled using numerical methods or the following approximate method. The approximate method consists of analyzing 1.0-ft wide culvert strips for the unbalanced soil pressures wherein the strips are limited by planes perpendicular to the culvert centerline. Refer to Figure C12.6.2.2.5-1 for this method of analysis for derivation of force F. For semicomplete culvert strips, the strips may be assumed to be supported as shown in the lower part of the plan. The headwall shall be designed as a frame carrying the strip reactions, V_L and $H_L\cos\alpha$, in addition to the concentrated force, F, assumed to be acting on the crown. Force F is determined using the equations given herein.

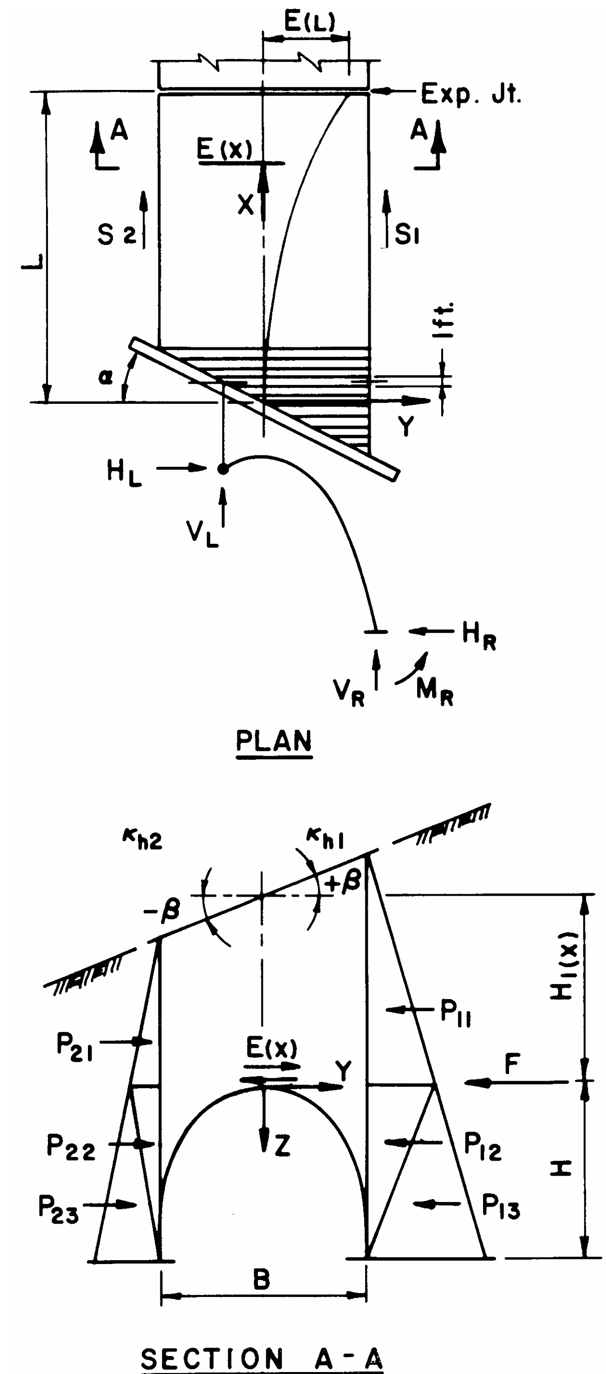


Figure C12.6.2.2.5-1—Forces on Culvert—Approximate Analysis

The unbalanced distributed load may be estimated by the following relationships:

$$E(x) = (P_{11} - P_{21}) + \frac{2}{3}(P_{12} - P_{22}) + \frac{1}{3}(P_{13} - P_{23}) \tag{C12.6.2.2.5-1}$$

in which:

$$P_{11} = \frac{1}{2} K_{h1} \left(H_{1(x)} + \frac{B}{2} \tan \beta \right)^2$$

$$P_{21} = \frac{1}{2} K_{h2} \left(H_{1(x)} - \frac{B}{2} \tan \beta \right)^2$$

$$P_{12} = \frac{1}{2} K_{h1} H \left(H_{1(x)} + \frac{B}{2} \tan \beta \right)$$

$$P_{22} = \frac{1}{2} K_{h2} H \left(H_{1(x)} - \frac{B}{2} \tan \beta \right) \qquad \text{(C12.6.2.2.5-2)}$$

$$P_{13} = \frac{1}{2} K_{h1} H \left(H + H_{1(x)} + \frac{B}{2} \tan \beta \right)$$

$$P_{23} = \frac{1}{2} K_{h2} H \left(H + H_{1(x)} - \frac{B}{2} \tan \beta \right)$$

When the pressures are substituted into Eq. C12.6.2.2.5-1, the following results:

$$E(x) = A_2 x^2 + A_1 x + A_0 \qquad \text{(C12.6.2.2.5-3)}$$

in which:

$$A_2 = \frac{1}{2} \left(\frac{H_{1(L)}}{L} \right)^2 (K_{h1} - K_{h2})$$

$$A_1 = \frac{1}{2} \left(\frac{H_{1(L)}}{L} \right) [B(K_{h1} + K_{h2}) \tan \beta + H(K_{h1} - K_{h2})]$$

$$A_0 = \frac{1}{24} [(3B^2 \tan^2 \beta + 4H^2)(K_{h1} - K_{h2}) + 6HB(K_{h1} + K_{h2}) \tan \beta]$$

(C12.6.2.2.5-4)

The support forces for the unbalanced distribution load, $E(x)$, are:

$$F = \frac{1}{6} L \sec \alpha (2A_2 L^2 + 3A_1 L + 6A_0)$$

$$S_1 = -\frac{1}{12} \frac{L}{B} [A_2 L^2 (3L - 2B \tan \alpha) + A_1 L (4L - 3B \tan \alpha) + 6A_0 (L - B \tan \alpha)]$$

$$S_2 = \frac{1}{12} \frac{L}{B} [A_2 L^2 (3L + 2B \tan \alpha) + A_1 L (4L + 3B \tan \alpha) + 6A_0 (L + B \tan \alpha)]$$

(C12.6.2.2.5-5)

For values of K_h, see Figure C12.6.2.2.5-2.

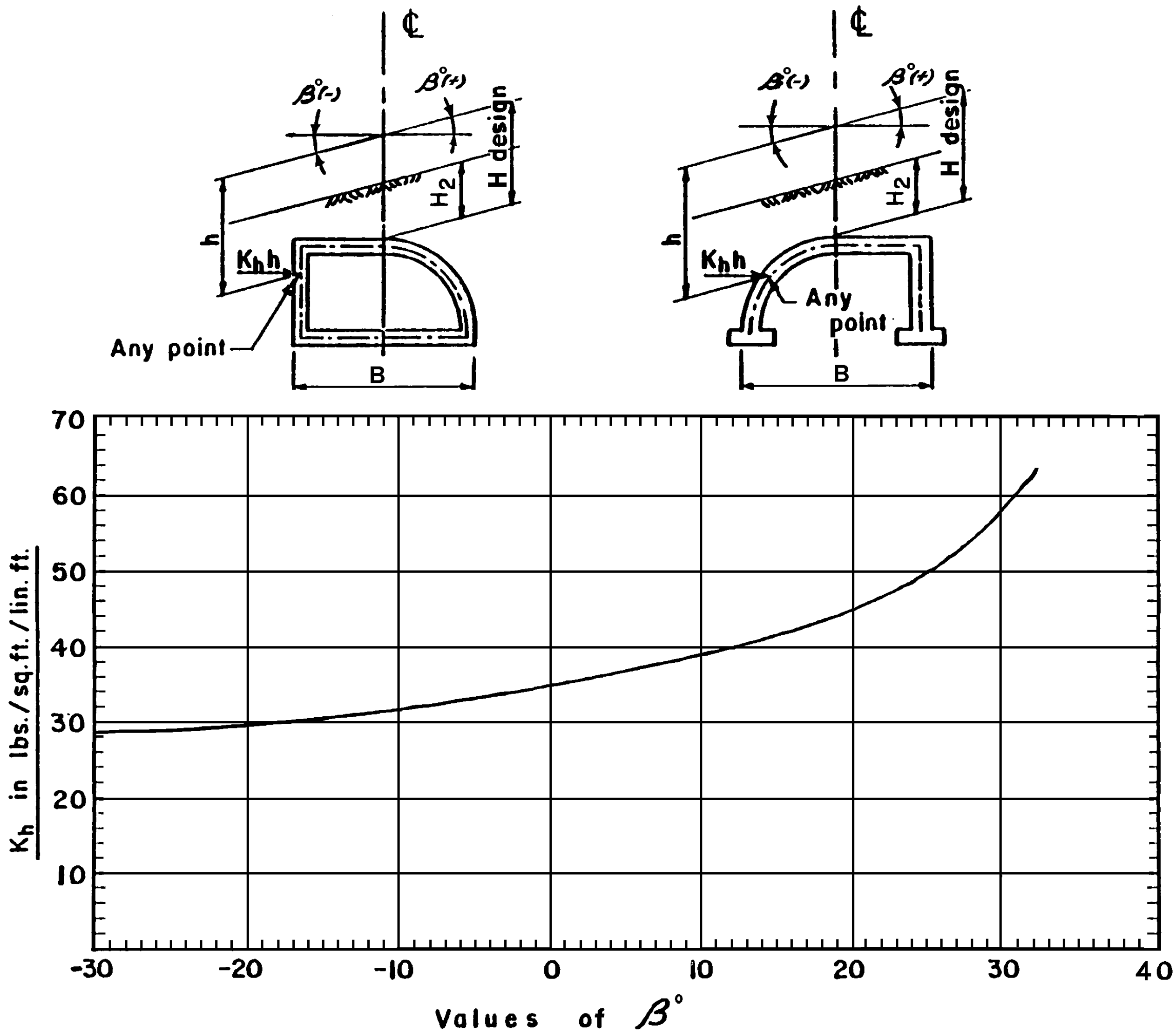


Figure C12.6.2.2.5-2—Lateral Earth Pressure as a Function of Ground Slope

12.6.2.3—Uplift

Uplift shall be considered where structures are installed below the highest anticipated groundwater level.

C12.6.2.3

To satisfy this provision, the dead load on the crown of the structure should exceed the buoyancy of the culvert, using load factors as appropriate.

12.6.3—Safety Against Soil Failure

12.6.3.1—Bearing Resistance and Stability

Pipe structures and footings for buried structures shall be investigated for bearing capacity failure and erosion of soil backfill by hydraulic gradients.

12.6.3.2—Corner Backfill for Metal Pipe Arches

The corner backfill for metal pipe arches shall be designed to account for corner pressure taken as the arch thrust divided by the radius of the pipe-arch corner. The soil envelope around the corners of pipe arches shall resist this pressure. Placement of select structural backfill compacted to unit weights higher than normal may be specified.

12.6.4—Hydraulic Design

Design criteria as specified in Article 2.6 and in the *AASHTO Drainage Manual* (2014) shall apply.

12.6.5—Scour

Buried structures shall be designed so that no movement of any part of the structure will occur as a result of scour.

In areas where scour is a concern, the wingwalls shall be extended far enough from the structure to protect the structural portion of the soil envelope surrounding the structure. For structures placed over erodible deposits, a cut-off wall or scour curtain, extending below the maximum anticipated depth of scour or a paved invert, shall be used. The footings of structures shall be placed not less than 2.0 ft below the maximum anticipated depth of scour.

12.6.6—Soil Envelope

12.6.6.1—Trench Installations

The minimum trench width shall provide sufficient space between the pipe and the trench wall to ensure sufficient working room to properly and safely place and compact backfill material.

The contract documents shall require that stability of the trench be ensured by either sloping the trench walls or providing support of steeper trench walls in conformance with OSHA or other regulatory requirements.

C12.6.6.1

As a guide, the minimum trench width should not be less than the greater of the pipe diameter plus 16.0 in. or the pipe diameter times 1.5 plus 12.0 in. The use of specially designed equipment may enable satisfactory installation and embedment even in narrower trenches. If the use of such equipment provides an installation meeting the requirements of this Article, narrower trench widths may be used as approved by the Engineer.

For trenches excavated in rock or high-bearing soils, decreased trench widths may be used up to the limits required for compaction. For these conditions, the use of a flowable backfill material, as specified in Article 12.4.1.3, allows the envelope to be decreased to within 6.0 in. along each side of the pipe.

@Seismicisolation

12.6.6.2—Embankment Installations

The minimum width of the soil envelope shall be sufficient to ensure lateral restraint for the buried structure. The combined width of the soil envelope and embankment beyond shall be adequate to support all the loads on the culvert and to comply with the movement requirements specified in Article 12.6.2.

C12.6.6.2

As a guide, the minimum width of the soil envelope on each side of the buried structure should not be less than the widths specified in Table C12.6.6.2-1:

Table C12.6.6.2-1—Minimum Width of Soil Envelope

Diameter, S (in.)	Minimum Envelope Width (ft)
<24	$S/12$
24–144	2.0
>144	5.0

12.6.6.3—Minimum Cover

The minimum cover, including a well-compacted granular subbase and base course, shall not be less than that specified in Table 12.6.6.3-1, where:

S = diameter of pipe (in.)
B_c = outside diameter or width of the structure (ft)
B'_c = out-to-out vertical rise of pipe (ft)
ID = inside diameter (in.)

C12.6.6.3

McGrath et al. (2005) has shown that the significant thermal expansion in thermoplastic pipe can affect pavement performance under shallow fills. Depending on the pipe material and the pavement type above it, the minimum cover may include the pavement thickness and base course, along with the sub-base.

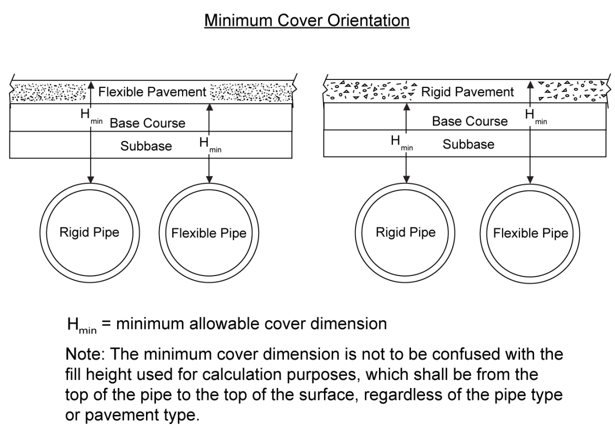


Figure 12.6.6.3-1—Minimum Cover Orientation

If the minimum cover provided in Table 12.6.6.3-1 is not sufficient to avoid placement of the pipe within the pavement layer, then the minimum cover should be increased to a minimum of the pavement thickness, unless an analysis is performed to determine the effect on both the pipe and the pavement.

@Seismicisolation

Table 12.6.6.3-1—Minimum Cover

Type	Condition	Minimum Cover*
Corrugated Metal Pipe	—	$S/8 \geq 12.0$ in.
Spiral Rib Metal Pipe	Steel Conduit	$S/4 \geq 12.0$ in.
	Aluminum Conduit where $S \leq 48.0$ in.	$S/2 \geq 12.0$ in.
	Aluminum Conduit where $S > 48.0$ in.	$S/2.75 \geq 24.0$ in.
Structural Plate Pipe Structures	—	$S/8 \geq 12.0$ in.
Long-Span Structural Plate Pipe Structures	—	Refer to Table 12.8.3.1.1-1
Structural Plate Box Structures	—	1.4 ft. as specified in Article 12.9.1
Deep Corrugated Structural Plate Structures	—	See Article 12.8.9.4
Fiberglass Pipe	—	12.0 in.
Thermoplastic Pipe	Under unpaved areas	$ID/8 \geq 12.0$ in.
	Under paved roads	$ID/2 \geq 24.0$ in.
Steel-Reinforced Thermoplastic Culverts	—	$S/5 \geq 12.0$ in.
Reinforced Concrete Pipe	Under unpaved areas or top of flexible pavement	$B_c/8$ or $B'_c/8$, whichever is greater, $\geq$ 12.0 in.
Reinforced Concrete Pipe	Under bottom of rigid pavement	9.0 in.
* Minimum cover taken from top of rigid pavement or bottom of flexible pavement		

If soil cover is not provided, the top of precast or cast-in-place reinforced concrete box structures shall be designed for direct application of vehicular loads.

Additional cover requirements during construction shall be taken as specified in Article 30.5.5 of the *AASHTO LRFD Bridge Construction Specifications.*

12.6.7—Minimum Spacing between Multiple Lines of Pipe

The spacing between multiple lines of pipe shall be sufficient to permit the proper placement and compaction of backfill below the haunch and between the structures.

Contract documents should require that backfilling be coordinated to minimize unbalanced loading between multiple, closely spaced structures. Backfill should be kept level over the series of structures when possible. The effects of significant roadway grades across a series of structures shall be investigated for the stability of flexible structures subjected to unbalanced loading.

C12.6.7

As a guide, the minimum spacing between pipes should not be less than that shown in Table C12.6.7-1.

Table C12.6.7-1—Minimum Pipe Spacing

Type of Structure	Minimum Distance Between Pipes (ft)
Round Pipes Diameter, *D* (ft)	
<2.0	1.0
2.0–6.0	$D/2$
>6.0	3.0
Pipe Arches Span, *S* (ft)	
<3.0	1.0
3.0–9.0	$S/3$
9.0–16.0	3.0
Arches Span, *S* (ft)	
All Spans	2.0

The minimum spacing can be reduced if a flowable backfill material, as specified in Article 12.4.1.3, is placed between the structures.

12.6.8—End Treatment

12.6.8.1—General

Protection of end slopes shall be given special consideration where backwater conditions occur or where erosion or uplift could be expected. Traffic safety treatments, such as a structurally adequate grating that conforms to the embankment slope, extension of the culvert length beyond the point of hazard, or provision of guide rail, should be considered.

C12.6.8.1

Culvert ends may represent a major traffic hazard.

When backwater conditions occur, pressure flow at the outlet end of culverts can result in uplift of pipe sections having inadequate cover and scour of erosive soils due to high velocities of water flow. Measures to control these problems include anchoring the pipe end in a concrete headwall or burying it in riprap having sufficient mass to resist uplift forces as well as lining outlet areas with riprap or concrete to prevent scour.

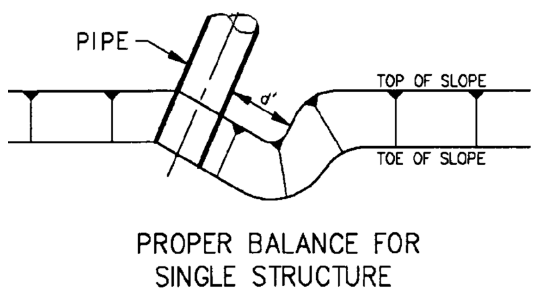
12.6.8.2—Flexible Culverts Constructed on Skew

The end treatment of flexible culverts skewed to the roadway alignment and extending through embankment fill shall be warped to ensure symmetrical loading along either side of the pipe or the headwall shall be designed to support the full thrust force of the cut end.

C12.6.8.2

For flexible structures, additional reinforcement of the end is recommended to secure the metal edges at inlet and outlet against hydraulic forces. Reinforcement methods include reinforced concrete or structural steel collars, tension tiebacks or anchors in soil, partial headwalls, and cut-off walls below invert elevation.

As a guide in Figure C12.6.8.2-1, limits are suggested for skews to embankments unless the embankment is warped. It also shows examples of warping an embankment cross-section to achieve a square-ended pipe for single and multiple flexible pipe installations where the minimum width of the warped embankment, d', is taken as 1.50 times the sum of the rise of the culvert and the cover or three times the span of the culvert, whichever is less.



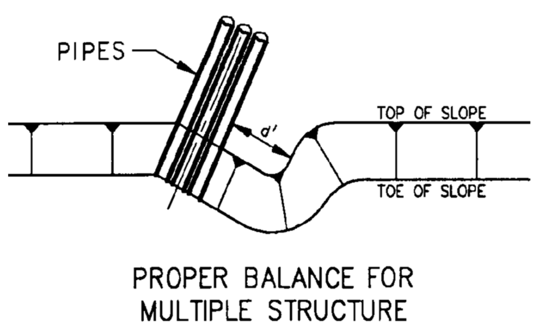


Figure C12.6.8.2-1—End Treatment of Skewed Flexible Culvert

12.6.9—Corrosive and Abrasive Conditions

The degradation of culvert material due to corrosion and abrasion is a consideration when selecting material types. The invert of culverts receives the largest impact due to corrosion and abrasion; however, the surrounding soil properties and groundwater may impact other portions of the culvert barrel.

Culvert service life is determined by the owner and is established based on the importance of the culvert. Considerations are given to the roadway classification, average daily traffic, risk of premature failure, and costs to replace.

Ensure the culvert material service life meets or exceeds the culvert service life. Use of different metals, protective linings, increased gauge thickness, or a combination of these methods are commonly used for metal culverts. Additional concrete cover over reinforcing steel or protective coatings are common approaches used in concrete culverts.

C12.6.9

Several long-term tests of the field performance of buried structures have resulted in development of empirical guidelines for estimating culvert material service life due to the effects of corrosion and abrasion. A representative listing includes Bellair and Ewing (1984), Koepf and Ryan (1986), Hurd (1984), Meacham et al. (1982), Potter (1988), Funahashi and Bushman (1991), Sagues (2001), NCHRP Synthesis No. 474 (2015), and Sargand (2016).

Commonly-used culvert service life requirements range from 25 to 75 years of service.

The National Corrugated Steel Pipe Association offers an online Service Life Calculator for corrugated steel pipe that requires input of soil resistivity, water pH, FHWA abrasion level (1–4), and a determination if the water contains soft water with $CaCO_3$. FHWA abrasion levels are as follows:

- Level 1—Nonabrasive conditions exist in areas of no bed load and very low velocities. This is the condition assumed for the soil side of drainage pipes.
- Level 2—Low abrasive conditions exist in areas of minor bed loads of sand and velocities of 5.0 ft/s or less.
- Level 3—Moderate abrasive conditions exist in areas of moderate bed loads of sand and gravel and velocities between 5.0 ft/s and 15.0 ft/s.
- Level 4—Severe abrasive conditions exist in areas of heavy bed loads of sand, gravel, and rock and velocities exceeding 15.0 ft/s.

12.7—METAL PIPE, PIPE ARCH, ARCH STRUCTURES, AND STEEL-REINFORCED THERMOPLASTIC CULVERTS

12.7.1—General

The provisions herein shall apply to the design of buried corrugated and spiral rib metal pipe and structural plate pipe structures.

Corrugated metal pipe and pipe-arches may be of riveted, welded, or lockseam fabrication with annular or helical corrugations. Structural plate pipe, pipe-arches, and arches shall be bolted with annular corrugations only.

The rise-to-span ratio of structural plate arches shall not be less than 0.3.

The provisions of Article 12.8 shall apply to structures with a radius exceeding 13.0 ft.

C12.7.1

These structures become part of a composite system comprised of the metal pipe section and the soil envelope, both of which contribute to the structural behavior of the system.

For information regarding the manufacture of structures and structural components referred to herein, AASHTO M 196 (ASTM B745) for aluminum, M 36 (ASTM A760/A760M) for steel corrugated metal pipe and pipe-arches, and M 167M/M 167 (ASTM A761/A761M) for steel and M 219 (ASTM B746) for aluminum structural plate pipe may be consulted.

12.7.2—Safety Against Structural Failure

Corrugated and spiral rib metal pipe and pipe arches and structural plate pipe shall be investigated at the strength limit state for:

@Seismicisolation

- wall area of pipe,
- buckling strength, and
- seam resistance for structures with longitudinal seams.

12.7.2.1—Section Properties

Dimensions and properties of pipe cross-sections; minimum seam strength; mechanical and chemical requirements for aluminum corrugated and steel corrugated pipe and pipe-arch sections; and aluminum and steel corrugated structural plate pipe, pipe-arch, and arch sections, may be taken as given in Appendix A12.

Dimensions, properties of pipe cross-sections, and material properties for steel-reinforced thermoplastic culverts shall be provided by the pipe manufacturer.

C12.7.2.1

Steel-reinforced thermoplastic culverts are pipe sections in which the main load-carrying members are steel ribs or corrugations encapsulated by thermoplastic material that may brace the ribs or corrugations from distortion and buckling. This composite system should be evaluated independently for each manufacturer's pipe system. Designers should obtain the required mechanical properties directly from the pipe manufacturer to determine fill heights.

12.7.2.2—Thrust

The factored thrust, T_L, per unit length of wall shall be taken as:

$$T_L = \frac{P_{FD}(S)}{2} + \frac{P_{FL}(C_L)F_1}{2} \tag{12.7.2.2-1}$$

in which:

$$C_L = \ell_w \leq S \tag{12.7.2.2-2}$$

- for corrugated metal pipe:

$$F_1 = \frac{0.75S}{\ell_w} \geq F_{\min} \tag{12.7.2.2-3}$$

$$F_{\min} = \frac{15}{12(S)} \geq 1 \tag{12.7.2.2-4}$$

- for long-span corrugated metal structures:

$$F_1 = \frac{0.54(S)}{\frac{w_t}{12} + LLDF(H) + 0.03(S)} \tag{12.7.2.2-5}$$

where:

C_L = width of culvert on which live load is applied parallel to span (ft)

$LLDF$ = live load distribution factor as specified in Article 3.6.1.2.6

ℓ_w = live load patch length at depth H as specified in Article 3.6.1.2.6

P_{FD} = factored dead load vertical crown pressure as specified in Article 12.12.3.4 with VAF taken as 1.0 and D_o taken as S (ksf)

C12.7.2.2

Factored vertical crown pressure is calculated as the factored free-field soil pressure at the elevation of the top of the structure, plus the factored live load pressure distributed through the soil cover to the top of the structure.

@Seismicisolation

P_{FL} = factored live load vertical crown pressure as specified in Article 12.12.3.4 (ksf)
S = culvert span (ft)
T_L = factored thrust per unit length (kip/ft)
w_t = tire patch width as specified in Article 3.6.1.2.5 (in.)

12.7.2.3—Wall Resistance

The factored axial resistance, R_n, per unit length of wall, without consideration of buckling, shall be taken as:

$$R_n = \phi F_y A \qquad (12.7.2.3\text{-}1)$$

where:

A = wall area (in.^{2}/ft)
F_y = yield strength of metal (ksi)
ϕ = resistance factor as specified in Article 12.5.5

12.7.2.4—Resistance to Buckling

The wall area, calculated using Eq. 12.7.2.3-1, shall be investigated for buckling. If $f_{cr} < F_y$, A shall be recalculated using f_{cr} in lieu of F_y.

$$\text{If } S < \left(\frac{r}{k}\right)\sqrt{\frac{24E_m}{F_u}}\text{, then } f_{cr} = F_u - \frac{\left(\frac{F_u kS}{r}\right)^2}{48E_m} \qquad (12.7.2.4\text{-}1)$$

$$\text{If } S > \left(\frac{r}{k}\right)\sqrt{\frac{24E_m}{F_u}}\text{, then } f_{cr} = \frac{12E_m}{\left(\frac{kS}{r}\right)^2} \qquad (12.7.2.4\text{-}2)$$

where:

S = diameter of pipe or span of plate structure (in.)
E_m = modulus of elasticity of metal (ksi)
F_u = tensile strength of metal (ksi)
f_{cr} = critical buckling stress (ksi)
r = radius of gyration of corrugation (in.)
k = soil stiffness factor taken as 0.22

C12.7.2.4

The use of 0.22 for the soil stiffness is thought to be conservative for the types of backfill material allowed for pipe and arch structures. This lower bound on soil stiffness has a long history of use in previous editions of the Standard Specifications.

12.7.2.4.1—Critical Compressive Stress

For steel-reinforced thermoplastic culvert, stub compression test (AASHTO T 341) data shall be provided to establish the f_{cr} of the wall profile being evaluated. If f_{cr} value is less than the value established by Article 12.7.2.4, it shall be used as the limiting critical compressive stress for the pipe wall.

@Seismicisolation

12.7.2.5—Seam Resistance

For pipe fabricated with longitudinal seams, the factored resistance of the seam shall be sufficient to develop the factored thrust in the pipe wall, T_L.

12.7.2.6—Handling and Installation Requirements

Handling flexibility shall be indicated by a flexibility factor determined as:

$$FF = \frac{S^2}{E_m I} \tag{12.7.2.6-1}$$

Values of the flexibility factors for handling and installation shall not exceed the values for steel and aluminum pipe and plate pipe structures as specified in Article 12.5.6.

C12.7.2.6

Transverse stiffeners may be used to assist corrugated structural plate structures to meet flexibility factor requirements.

12.7.2.7—Profile Evaluation for Steel-Reinforced Thermoplastic Culverts

To assure the adequacy of the thermoplastic liner, the culvert manufacturer shall provide the results of a three-dimensional finite element analysis of the profile that has been calibrated against results of full-scale tests. A minimum of two full-scale tests are required to properly calibrate the results of the finite element analysis. In particular, the measurement and prediction of the maximum strains within the profile shall be identified. The predicted long-term tensile strains within the profile shall be within the allowable limits for the HDPE material being used in the profile. The strain limits for polyethylene materials shall be taken as specified in Table 12.12.3.3-1.

Additionally, in order to establish the relative long-term interaction of the steel reinforcement with the HDPE profile, the stub compression test (AASHTO T 341) shall be performed utilizing stroke rates of 0.05 in./minute, 0.005 in./minute, and 0.0005 in./minute and the results compiled to determine if a reduction in the HDPE modulus results in a reduced result from the stub compression tests. If a reduction factor is deemed to be appropriate, the reduced value shall be used as described in Article 12.7.2.4.1.

C12.7.2.7

The full-scale tests should follow the loading conditions, measurements, and test methods consistent with those utilized in the research report *DuroMaxx Pipe Assessment* by I. D. Moore (24 in. – February 2009, 60 in. – August 2009) or similar approved method.

While the steel ribs or corrugations are the main load-carrying member of the culvert, the thermoplastic profile braces the steel ribs or corrugations from distortion or buckling under load and is critical to the pipe performance. The liner also serves to distribute the load between ribs or corrugations. A structural evaluation of the profile alone is not required. However, an evaluation of the composite system of thermoplastic liner and steel rib or corrugation is necessary. It is important to assure that the tensile strains within the profile do not exceed the long-term strain capacity for the thermoplastic material used in the construction of the pipe.

12.7.3—Smooth Lined Pipe

Corrugated metal pipe composed of a smooth liner and corrugated shell attached integrally at helical seams, spaced not more than 30.0 in. apart, may be designed on the same basis as a standard corrugated metal pipe having the same corrugations as the shell and a weight per ft not less than the sum of the weights per ft of liner and helically corrugated shell.

The pitch of corrugations shall not exceed 3.0 in., and the thickness of the shell shall not be less than 60 percent of the total thickness of the equivalent standard pipe.

@Seismicisolation

12.7.4—Stiffening Elements for Structural Plate Structures

The stiffness and flexural resistance of structural plate structures may be increased by adding circumferential stiffening elements to the crown. Stiffening elements shall be symmetrical and shall span from a point below the quarter-point on one side of the structure, across the crown, and to the corresponding point on the opposite side of the structure.

C12.7.4

Acceptable stiffening elements are:

- Continuous longitudinal structural stiffeners connected to the corrugated plates at each side of the top arc: metal or reinforced concrete, either singly or in combination; and
- Reinforcing ribs formed from structural shapes, curved to conform to the curvature of the plates, fastened to the structure to ensure integral action with the corrugated plates, and spaced at such intervals as necessary.

12.7.5—Construction and Installation

The contract documents shall require that construction and installation conform to Section 26, "Metal Culverts," of the *AASHTO LRFD Bridge Construction Specifications*.

12.8—LONG-SPAN STRUCTURAL PLATE STRUCTURES

12.8.1—General

The provisions herein and in Article 12.7 shall apply to the structural design of buried long-span structural plate corrugated metal structures.

The following shapes, illustrated in Figure 12.8.1-1, shall be considered long-span structural plate structures:

- Structural plate pipe and arch shaped structures that require the use of special features specified in Article 12.8.3.5, and
- Special shapes of any size having a radius of curvature greater than 13.0 ft in the crown or side plates. Metal box culverts are not considered long-span structures and are covered in Article 12.9.

C12.8.1

These structures become part of a composite system comprised of the metal structure section and the soil envelope, both of which contribute to the behavior of the system.

@Seismicisolation

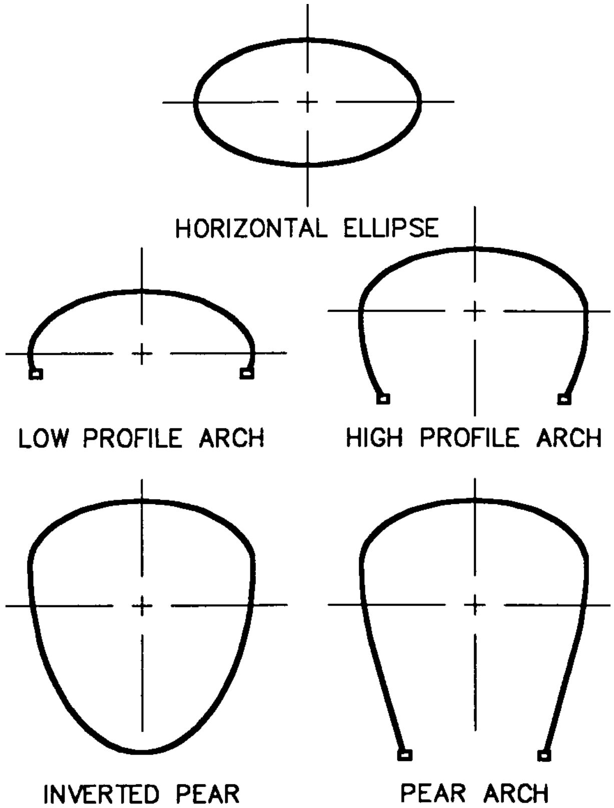


Figure 12.8.1-1—Long-Span Shapes

12.8.2—Service Limit State

No service limit state criteria need be required.

C12.8.2

Soil design and placement requirements for long-span structures are intended to limit structure deflections. The contract documents should require that construction procedures be monitored to ensure that severe deformations do not occur during backfill placement and compaction.

12.8.3—Safety Against Structural Failure

With the exception of the requirements for buckling and flexibility, the provisions of Article 12.7 shall apply, except as described herein.

Dimensions and properties of structure cross-sections, minimum seam strength, mechanical and chemical requirements, and bolt properties for long-span structural plate sections shall be taken as specified in Appendix A12 or as described herein.

C12.8.3

Most long-span culverts are designed for a larger load factor; however, the limit states of flexure and buckling are ignored for those structures. Considering these limit states reduces the uncertainty in the final design and permits use of a lower load factor. This is the same approach used for metal box culverts.

12.8.3.1—Section Properties

12.8.3.1.1—Cross-Section

The provisions of Article 12.7 shall apply, except as specified.

Structures not described herein shall be regarded as special designs.

Table A12-3 shall apply. Minimum requirements for section properties shall be taken as specified in Table 12.8.3.1.1-1. Covers that are less than that shown in Table 12.8.3.1.1-1 and that correspond to the minimum plate thickness for a given radius may be used if ribs are used to stiffen the plate. If ribs are used, the plate thickness may not be reduced below the minimum shown for that radius, and the moment of inertia of the rib and plate section shall not be less than that of the thicker unstiffened plate corresponding to the fill height. Use of soil cover less than the minimum values shown for a given radius shall require a special design.

Designs not covered in Table 12.8.3.1.1-1 should not be permitted unless substantiated by documentation acceptable to the Owner.

C12.8.3.1.1

Sharp radii generate high soil pressures. Avoid high ratios when significant heights of fill are involved.

Table 12.8.3.1.1-1—Minimum Requirements for Long-Span Structures with Acceptable Special Features

Top Arc Minimum Thickness (in.)					
Top Radius (ft)	≤15.0	15.0–17.0	17.0–20.0	20.0–23.0	23.0–25.0
6 in. × 2 in. Corrugated Steel Plate—Top Arc Minimum Thickness (in.)	0.111	0.140	0.170	0.218	0.249
Geometric Limits					
The following geometric limits shall apply: • Maximum plate radius—25.0 ft • Maximum central angle of top arc—80.0 degrees • Minimum ratio, top arc radius to side arc radius—2 • Maximum ratio, top arc radius to side arc radius—5					
Minimum Cover (ft)					
Top Radius (ft)	≤ 15.0	15.0–17.0	17.0–20.0	20.0–23.0	23.0–25.0
Steel thickness without ribs (in.)					
0.111	2.5	—	—	—	—
0.140	2.5	3.0	—	—	—
0.170	2.5	3.0	3.0	—	—
0.188	2.5	3.0	3.0	—	—
0.218	2.0	2.5	2.5	3.0	—
0.249	2.0	2.0	2.5	3.0	4.0
0.280	2.0	2.0	2.5	3.0	4.0

@Seismicisolation

12.8.3.1.2—Shape Control

The requirements of Articles 12.7.2.4 and 12.7.2.6 shall not apply for the design of long-span structural plate structures.

12.8.3.1.3—Mechanical and Chemical Requirements

Tables A12-3, A12-8, and A12-10 shall apply.

12.8.3.2—Thrust

The factored thrust in the wall shall be determined by Eq. 12.7.2.2-1, except the value of S in the Equation shall be replaced by twice the value of the top arc radius, R_T.

12.8.3.3—Wall Area

The provisions of Article 12.7.2.3 shall apply.

12.8.3.4—Seam Strength

The provisions of Article 12.7.2.5 shall apply.

12.8.3.5—Acceptable Special Features

12.8.3.5.1—Continuous Longitudinal Stiffeners

Continuous longitudinal stiffeners shall be connected to the corrugated plates at each side of the top arc. Stiffeners may be metal or reinforced concrete either singly or in combination.

12.8.3.5.2—Reinforcing Ribs

Reinforcing ribs formed from structural shapes may be used to stiffen plate structures. Where used, they should be:

- curved to conform to the curvature of the plates,
- fastened to the structure as required to ensure integral action with the corrugated plates, and
- spaced at such intervals as necessary to increase the moment of inertia of the section to that required for design.

12.8.4—Safety Against Structural Failure—Foundation Design

12.8.4.1—Settlement Limits

A geotechnical survey of the site shall be made to determine that site conditions will satisfy the requirement that both the structure and the critical backfill zone on each side of the structure be properly supported. Design shall satisfy the requirements of Article 12.6.2.2, with the following factors to be considered when establishing settlement criteria:

C12.8.4.1

@Seismicisolation

- Once the structure has been backfilled over the crown, settlement of the supporting backfill relative to the structure must be limited to control downdrag forces. If the sidefill will settle more than the structure, a detailed analysis may be required.
- Settlement along the longitudinal centerline of arch structures must be limited to maintain slope and preclude footing cracks in arches.

Calculated differential settlement across the structure taken from springline-to-springline, Δ, shall satisfy:

$$\Delta \leq \frac{0.01S^2}{R} \quad (12.8.4.1\text{-}1)$$

where:

S = span of structure between springlines of long-span structural plate structures (ft)
R = rise of structure (ft)

More restrictive settlement limits may be required where needed to protect pavements or to limit longitudinal differential deflections.

Once the top arc of the structure has been backfilled, downdrag forces may occur if the structure backfill settles into the foundation more than the structure. This results in the structure carrying more soil load than the overburden directly above it. If undertaken prior to erecting the structure, site improvements such as surcharging, foundation compacting, etc., often adequately correct these conditions.

Where the structure will settle uniformly with the adjacent soils, long-spans with full inverts can be built on a camber to achieve a proper final grade.

For design, differential settlement between the footings taken across the structure is limited to avoid excessive eccentricity. The limit on any settlement-induced rotation of the structure maintains the top arc centerline within one percent of span, as shown in Figure C12.8.4.1-1.

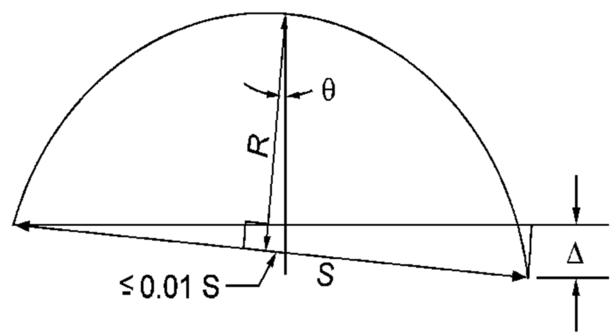


Figure C12.8.4.1-1—Differential Settlement

The rotation of the structure, θ, may be determined as:

$$\theta = \tan^{-1}\left(\frac{\Delta}{S}\right) \quad (C12.8.4.1\text{-}1)$$

12.8.4.2—Footing Reactions in Arch Structures

Footing reactions may be taken as:

$$R_V = \left(V_{DL} + V_{LL}\right)\cos\Delta \quad (12.8.4.2\text{-}1)$$

$$R_H = \left(V_{DL} + V_{LL}\right)\sin\Delta \quad (12.8.4.2\text{-}2)$$

in which:

V_{DL} = $[H_2(S) - A_T]\,\gamma_s/2$
V_{LL} = $n(A_L)/(8 + 2\,H_1)$
n = integer $(2H_1/L_w + 2) \leq$ number of adjoining traffic lanes

where:

R_V = vertical footing reaction component (kip/ft)
R_H = horizontal footing reaction component (kip/ft)
Δ = return angle of the structure (degrees)

C12.8.4.2

Footing reactions are calculated by simple statics to support the vertical loads. Soil load footing reactions (V_{DL}) are taken as the weight of the fill and pavement above the springline of the structure. Where footings extend out beyond the springline and the foundation has not previously carried the design overburden, this additional soil load (E_v) may need to be added to V_{DL} in an embankment installation.

Live loads that provide relatively limited pressure zones acting on the crown of the structure may be distributed to the footings as indicated in Figure C12.8.4.2-1.

@Seismicisolation

A_L = axle load (kip), taken as 50 percent of all axle loads that can be placed on the structure at one time, i.e.:

- 32.0 kip for the design truck axle
- 50.0 kip for the design tandem axle pair
- 160.0 kip for E80 railroad loading

A_T = area of the top portion of the structure above the springline (ft^2)

H_1 = height of cover above the footing to traffic surface (ft)

H_2 = height of cover from the springline of the structure to traffic surface (ft)

L_w = lane width (ft)

γ_s = unit weight of soil (kcf)

S = span (ft)

The distribution of live load through the fill shall be based on any accepted methods of analysis.

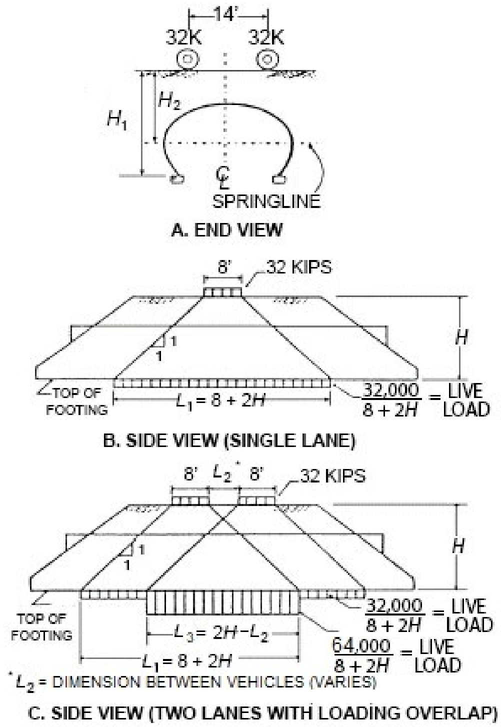


Figure C12.8.4.2-1—Live-Load Footing Reaction Due to Axles of the Design Truck, per Footing

12.8.4.3—Footing Design

Reinforced concrete footings shall be designed in accordance with Article 10.6 and shall be proportioned to satisfy settlement requirements of Article 12.8.4.1.

12.8.5—Safety Against Structural Failure—Soil Envelope Design

12.8.5.1—General

Structural backfill material in the envelope around the structure shall satisfy the requirements of Article 12.4.1.3 for long-span structures. The width of the envelope on each side of the structure shall be proportioned to limit shape change during construction activities outside the envelope and to control deflections at the service limit state.

C12.8.5.1

Structure erection, backfill, and construction shall meet all the requirements of Section 26 of the *AASHTO LRFD Bridge Construction Specifications*. The performance of the structure depends upon the in-situ embankment or other fill materials beyond the structural backfill. Design must consider the performance of all materials within the zone affected by the structure.

12.8.5.2—Construction Requirements

The structural backfill envelope shall either extend to the trench wall and be compacted against it or extend a distance adequate to protect the shape of the structure from construction loads. The remaining trench width may be filled with suitable backfill material compacted to satisfy the requirements of Article 12.8.5.3. In embankment conditions, the minimum structural backfill width shall be taken as 6.0 ft. Where dissimilar materials not meeting geotechnical filter criteria are used adjacent to each other, a suitable geotextile shall be provided to avoid migration.

C12.8.5.2

The purpose of this provision is to control shape change from construction activities outside the envelope in trench conditions.

12.8.5.3—Service Requirements

The width of the envelope on each side of the structure shall be adequate to limit horizontal compression strain to one percent of the structure's span on each side of the structure.

Determination of the horizontal compressive strain shall be based on an evaluation of the width and quality of the structural backfill material selected as well as the in-situ embankment or other fill materials within the zone on each side of the structure taken to extend to a distance equal to the rise of the structure, plus its cover height as indicated in Figure 12.8.5.3-1.

Forces acting radially off the small radius corner arc of the structure at a distance, d_1, from the structure may be taken as:

$$P_1 = \frac{T}{R_c + d_1} \tag{12.8.5.3-1}$$

where:

P_1 = horizontal pressure from the structure at a distance, d_1 (ksf)
d_1 = distance from the structure (ft)
T = total dead load and live load thrust in the structure (Article 12.8.3.2) (kip/ft)
R_c = corner radius of the structure (ft)

The required envelope width adjacent to the pipe, d, may be taken as:

$$d = \frac{T}{P_{Brg}} - R_c \tag{12.8.5.3-2}$$

where:

d = required envelope width adjacent to the structure (ft)
P_{Brg} = allowable bearing pressure to limit compressive strain in the trench wall or embankment (ksf)

C12.8.5.3

The purpose of this provision is to limit deflections under service loads. The limit on soil compression limits the theoretical design increase in span to two percent. This is a design limit, not a performance limit. Any span increase that occurs is principally due to the consolidation of the side support materials as the structure is loaded during backfilling. These are construction movements that attenuate when full cover is reached.

Eqs. 12.8.5.3-1 and 12.8.5.3-2 conservatively assume that the pressure from the structure acts radially outward from the corner arc without further dissipation. Figure C12.8.5.3-1 provides the geometric basis of these Equations.

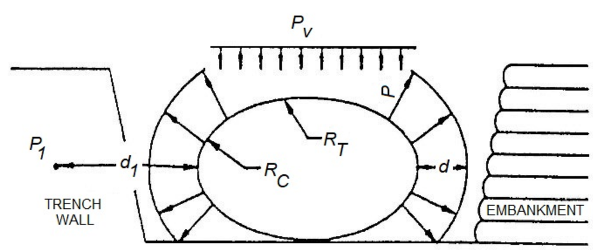


Figure C12.8.5.3-1—Radial Pressure Diagram

The structural backfill envelope shall be taken to continue above the crown to the lesser of:

- the minimum cover level specified for that structure,
- the bottom of the pavement or granular base course where a base course is present below the pavement, or
- the bottom of any relief slab or similar construction where one is present.

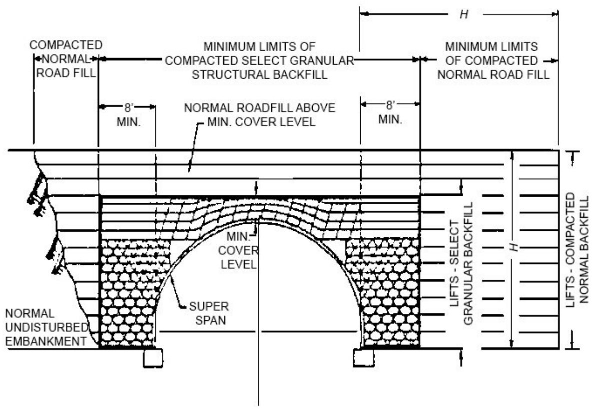


Figure 12.8.5.3-1—Typical Structural Backfill Envelope and Zone of Structure Influence

12.8.6—Safety Against Structural Failure—End Treatment Design

12.8.6.1—General

End treatment selection and design shall be considered as an integral part of the structural design.

C12.8.6.1

Proper end treatment design ensures proper support of the ends of the structure while providing protection from scour, hydraulic uplift, and loss of backfill due to erosion forces.

12.8.6.2—Standard Shell End Types

The standard end types for the corrugated plate shell shall be taken to be those shown in Figure 12.8.6.2-1.

C12.8.6.2

Standard end types refer to the way the structural plate structure's ends are cut to match the fill slope, stream banks, etc. While the type of end selected may have aesthetic or hydraulic considerations, the structural design must ensure adequate structural strength and protection from erosion. Hydraulic considerations may require wingwalls, etc.

Step bevel, full bevel, and skewed ends all involve cutting the plates within a ring. Each has its own structural considerations.

The square end is the simplest arrangement. No plates are cut and the barrel retains its integrity.

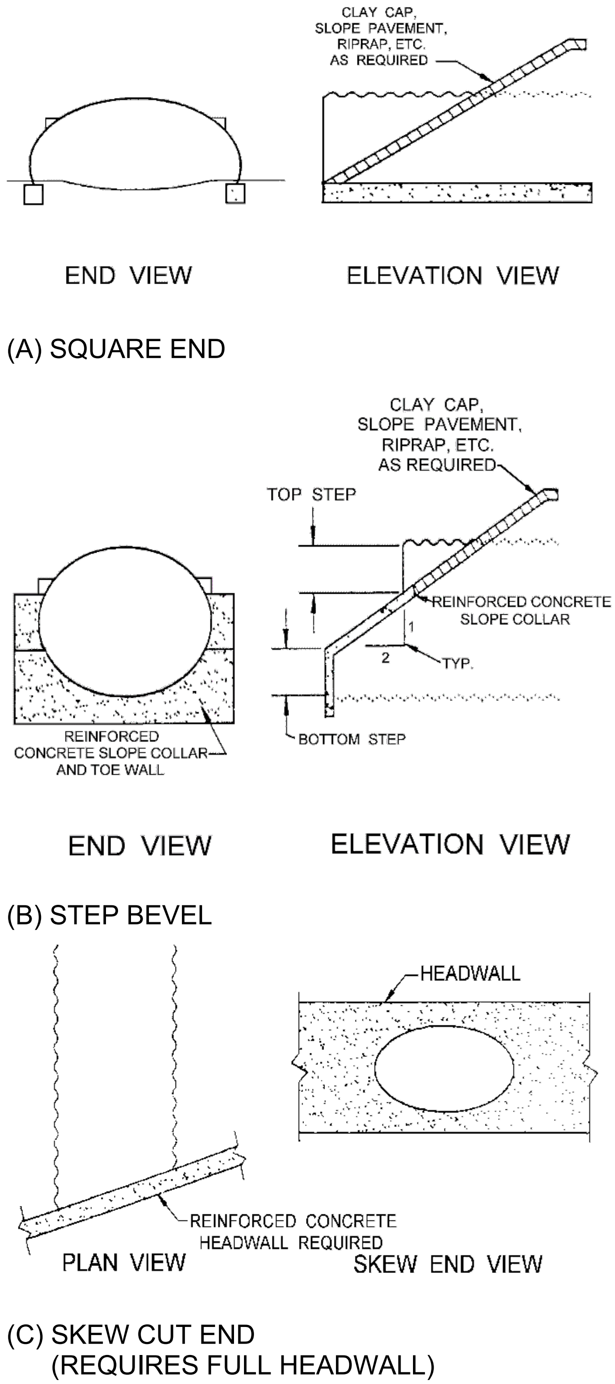


Figure 12.8.6.2-1—Standard Structure End Types

The following considerations shall apply to step bevels:

- The rise of the top step shall be equal to or greater than the rise of the top arc, i.e., plates in the top arc are left uncut.

Step bevels cut the corner (and side on pear and high profile arch shapes) plates on a diagonal (bevel) to match the fill slope.

Step bevels are widely used. The plates in the large radius top arc are left uncut to support the sides of the structure near each end.

- For structures with inverts, the bottom step shall satisfy the requirements for a top step.
- For arches, the bottom step shall be a minimum of 6.0 in. high.
- The slope of the cut plates generally should be no flatter than 3:1.
- The upper edge of the cut plates shall be bolted to and supported by a structural concrete slope collar, slope pavement, or similar device.

Full bevel ends shall be used in special designs only. Structures with full inverts shall have a bottom step conforming to the requirements for step bevel ends.

The bevel cut edge of all plates shall be supported by a suitable, rigid concrete slope collar.

Skew cut ends shall be fully connected to and supported by a headwall of reinforced concrete or other rigid construction. The headwall shall extend an adequate distance above the crown of the structure to be capable of reacting the ring compression thrust forces from the cut plates. In addition to normal active earth and live load pressures, the headwall shall be designed to react a component of the radial pressure exerted by the structure as specified in Article 12.8.5.

Invert plates must be left uncut to avoid leaving the invert as triangular shaped elements, when viewed in plan, running upstream and downstream.

Diagonally-cut corner and side plates become a retaining wall, supporting the fill slope beside them. They must be provided with suitable, rigid support at the top that acts as a top wale beam and be limited in length. These plates have limited longitudinal strength and inadequate bending strength or fixity to act as a cantilevered retaining wall.

When a full bevel cuts the top plates, additional support is necessary to backfill the structure. Typically, the top step is left in place and field cut only after a suitable rigid concrete slope collar has been poured and adequately cured.

Ring compressive thrust forces act circumferentially around the structure following the corrugations. At the skew cut ends of the plate, these forces act tangentially to the plate and must be resisted by a headwall. Additionally, because a skew cut structure is not perpendicular to the headwall, a portion of the radial pressure from the structure acts normal to the back of the headwall.

12.8.6.3—Balanced Support

Designs and details shall provide soil support that is relatively balanced from side to side, perpendicularly across the structure. In lieu of a special design, slopes running perpendicularly across the structure shall not exceed ten percent for cover heights of 10.0 ft or less and 15 percent for higher covers.

When a structure is skewed to an embankment, the fill shall be detailed to be warped to maintain balanced support and to provide an adequate width of backfill and embankment soil to support the ends.

C12.8.6.3

Flexible structures have relatively low bending strength. If the earth support is not balanced, the structure in effect becomes a retaining wall. An excessive imbalance causes shape distortion and ultimately failure.

When a structure is skewed to an embankment, two diagonally opposite areas at the ends of the structure are not adequately supported. This must be corrected by extending the embankment an adequate distance out beside the structure.

In lieu of a special design, details provided in Article C12.6.8.2 may be considered.

A properly warped embankment is characterized by equal elevation topographical lines crossing the structure perpendicularly and extending beyond it a suitable distance so that the volume of earth included in the warp provides a gravity retaining wall capable of supporting the radial pressures from the structure with adequate safety.

12.8.6.4—Hydraulic Protection

12.8.6.4.1—General

In hydraulic applications, provisions shall be made to protect the structure, taken to include the shell, footings, structural backfill envelope, and other fill materials within the zone influenced by the structure.

@Seismicisolation

12.8.6.4.2—Backfill Protection

Design or selection of backfill gradation shall include consideration of loss of backfill integrity due to piping. If materials prone to piping are used, the structure and ends of the backfill envelope shall be adequately sealed to control soil migration, infiltration, or both.

C12.8.6.4.2

Backfill piping and migration is always a major consideration in selecting its specific gradation. The ends of the backfill envelope may be sealed using one or a combination of a compacted clay cap, concrete slope pavements, grouted riprap, headwalls to the design storm elevation, and similar details.

12.8.6.4.3—Cut-Off (Toe) Walls

All hydraulic structures with full inverts shall be designed and detailed with upstream and downstream cut-off walls. Invert plates shall be bolted to cut-off walls at a maximum 20.0-in. center-to-center spacing using 0.75-in. bolts.

The cut-off wall shall extend to an adequate depth to limit hydraulic percolation to control uplift forces as specified in Article 12.8.6.4.4 and scour as specified in Article 12.8.6.4.5.

12.8.6.4.4—Hydraulic Uplift

Hydraulic uplift shall be considered for hydraulic structures with full inverts where the design flow level in the pipe can drop quickly. The design shall provide means to limit the resulting hydraulic gradients, with the water level higher in the backfill than in the pipe, so that the invert will not buckle and the structure will not float. Buckling may be evaluated as specified in Article 12.7.2.4, with the span of the structure taken as twice the invert radius.

C12.8.6.4.4

Structural plate structures are not watertight and allow for both infiltration and exfiltration through the structure's seams, bolt holes, and other discontinuities. Where uplift can be a concern, designs typically employ adequate cut-off walls and other means to seal off water flow into the structural backfill.

12.8.6.4.5—Scour

Scour design shall satisfy the requirements of Article 12.6.5. Where erodible soils are encountered, conventional means of scour protection may be employed to satisfy these requirements.

Deep foundations such as piles or caissons should not be used unless a special design is provided to consider differential settlement and the inability of intermittent supports to retain the structural backfill if scour proceeds below the pile cap.

C12.8.6.4.5

Structures with full inverts eliminate footing scour considerations when adequate cut-off walls are used. For arches, reinforced concrete invert pavements, riprap, grouted riprap, etc., can be employed to provide scour protection.

12.8.7—Concrete Relieving Slabs

Concrete relieving slabs may be used to reduce moments in long-span structures.

The length of the concrete relieving slab shall be at least 2.0 ft greater than the span of the structure. The relieving slab shall extend across the width subject to vehicular loading, and its depth shall be determined as specified in Article 12.9.4.6.

C12.8.7

Application of a typical concrete relieving slab is shown in Figure 12.9.4.6-1.

@Seismicisolation

12.8.8—Construction and Installation

The construction documents shall require that construction and installation conform to Section 26 of the *AASHTO LRFD Bridge Construction Specifications*.

12.8.9—Deep Corrugated Structural Plate Structures

12.8.9.1—General

The provisions of this Section shall apply to the structural design of buried, deep corrugated structural plate. These structures are designed as long-span culverts but must also meet provisions for flexure and general buckling. These structures may be manufactured in multiple shapes. Flexibility criteria and special features are not applicable to deep corrugated structures. The rise to span limit of 0.3 in Article 12.7.1 does not apply.

C12.8.9.1

The design of long-span metal structures in these Specifications is currently completed with empirical procedures that limit the shapes and plate thicknesses for the structures and require special features. If the provisions are met, then no design is required for flexure or buckling. NCHRP Report 473 recommended updating design provisions for long-span structures and included provisions to allow structures outside the current limits for long-span structures but included limit states for flexure and general buckling. Article 12.8.9 provides a design procedure for such structures. The provisions of Article 12.8.9 apply to structures fabricated from deep corrugated plate, defined in Article 12.2 as corrugated plate with a corrugation depth greater than 5.0 in.

12.8.9.2—Width of Structural Backfill

12.8.9.2.1—Deep Corrugated Structures with Ratio of Crown Radius to Haunch Radius ≤5

The structural backfill zone around deep corrugated structures with ratio of crown radius to haunch radius ≤5 shall extend to at least the minimum cover height above the crown. At the sides of the structure, the minimum extent of the structural backfill from the outside of the structure springline shall meet one of the following:

- Structure constructed in a trench in which the natural soil is at least as stiff as the engineered soil: 8.0 ft or
- Structure constructed in an embankment or in a trench in which the natural soil is less stiff than the engineered soil: one-third of the structure span but not less than 10.0 ft or more than 17.0 ft

but not less than required by culvert–soil interaction analysis.

12.8.9.2.2—Deep Corrugated Structures with Ratio of Crown Radius to Haunch Radius >5

The structural backfill zone around deep corrugated structures with ratio of crown radius to haunch radius >5 shall extend to at least the minimum cover height above the crown. At the sides of the structure, the minimum extent of structural backfill shall meet one of the following:

- for structures with spans up to and including 25.0 ft 5.0 in. and less than 5.0 ft of cover: a minimum of 3.5 ft beyond the widest part of the structure, or
- for structures with spans up to and including 25.0 ft 5.0 in. and greater than 5.0 ft of cover and for structures with spans greater than 25.0 ft 5.0 in. at all depths of fill: a minimum of one-fifth of the structure span beyond the widest part of the structure but not less than 5.0 ft nor more than 17.0 ft.

but not less than required by culvert–soil interaction analysis.

12.8.9.3—Safety Against Structural Failure

Deep corrugated structures shall be designed in accordance with the provisions of Articles 12.8.1 to 12.8.8, except for modified or additional provisions as follow.

12.8.9.3.1—Structural Plate Requirements

Deep corrugated structural plate used to manufacture structures designed under this section shall meet the requirements of AASHTO M 167M/M 167.

Sections may be stiffened. If stiffening is provided by ribs, the ribs shall be bolted to the structural plate corrugation prior to backfilling using a bolt spacing of not more than 16.0 in. for 15.0 by 5.5 in. and 16.0 by 6.0 in. corrugations, or 20.0 in. for 20.0 by 9.5 in. corrugations. The cross-section properties in Table A12-14 shall apply.

C12.8.9.3.1

It is acceptable to measure bolt spacing either at the centroid or crest of the structural plate corrugation.

12.8.9.3.2—Structural Analysis

Structures designed under the provisions of this Article shall be analyzed by accepted finite element analysis methods that consider both the strength and stiffness properties of the structural plate and the soil. The analysis shall produce thrust and moments for use in design. The analysis must consider all applicable combinations of construction, earth, live, and other applicable load conditions. Springline thrust due to earth load used in wall resistance, buckling, and seam resistance design shall not be less than 1.3 times the earth load thrust computed in accordance with Article 12.7.2.2.

C12.8.9.3.2

The computer program CANDE was developed by the FHWA specifically for the design of buried culverts and has the necessary soil and culvert material models to complete designs.

Because distribution of live loads to all long-span culverts does not consider arching, application of the 1.3 factor should be limited to just the earth load component of Article 12.7.2.2.

12.8.9.4—Minimum Depth of Fill

For deep corrugated structural plate structures, the minimum depth of cover (H_D) shall be the smaller of 3.0 ft or the limits for long-span structural plate structures based on top radius and plate thickness in Table 12.8.3.1.1-1. For deep corrugated structures with the ratio of crown radius to haunch radius >5, minimum cover shall be 1.5 ft for spans ≤25.0 ft 5.0 in. and 2.0 ft for spans >25.0 ft 5.0 in. The minimum depth of cover in all cases shall not be less than that required by culvert–soil interaction analysis.

12.8.9.5—Combined Thrust and Moment

The combined effects of moment and thrust at all stages of construction shall meet the following requirement:

$$\left(\frac{T_f}{R_t}\right)^2 + \left|\frac{M_u}{M_n}\right| \leq 1.00 \qquad (12.8.9.5\text{-}1)$$

where:

T_f = factored thrust
R_t = factored thrust resistance = $\phi_h F_y A$
M_u = factored applied moment
M_n = factored moment resistance = $\phi_h M_p$
M_p = plastic moment capacity of section

C12.8.9.5

The equation for combined moment and thrust is taken from the provision for buried structures in the Canadian Highway Bridge Design Code CSA-S6-06. The equation is more liberal than the AASHTO equations for combined moment and thrust (axial force) for steel structures in Article 12.8.9.6. However, the provisions in Article 12.8.9.6 are based on strong axis bending of wide flange sections. The equation for combined moment and thrust is taken from the provision for buried structures in the Canadian Highway Bridge Design Code CSA-S6-06. The equation is more liberal than the AASHTO equations for combined moment and thrust (axial force) for steel structures in Article 6.9.2.2. However, the provisions in Article 6.9.2.2 are based on strong axis bending of wide flange sections.

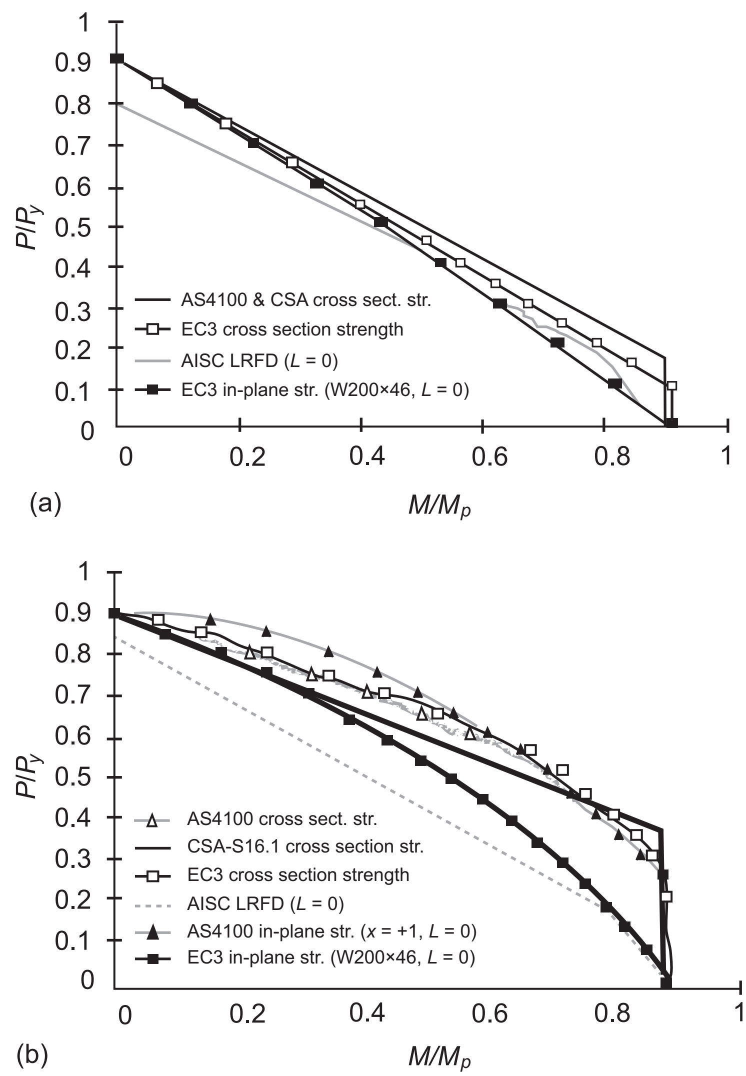


Figure C12.8.9.5-1—Strength Curves for Member of Zero Length: (a) Strong-Axis; (b) Weak-Axis from White and Clark (1997)

12.8.9.6—Global Buckling

The factored thrust in the culvert wall under the final installed condition shall not exceed the nominal resistance to general buckling capacity of the culvert, computed as:

$$R_b = 1.2\,\varphi_b\,C_n\,(E_p\,I_p)^{\frac{1}{3}}\,(\varphi_s\,M_s\,K_b)^{\frac{2}{3}}\,R_h \qquad (12.8.9.6\text{-}1)$$

where:

R_b = nominal axial force in culvert wall to cause general buckling
ϕ_b = resistance factor for general buckling
C_n = scalar calibration factor to account for some nonlinear effects = 0.55
E_p = modulus of elasticity of pipe wall material (ksi)
I_p = moment of inertia of stiffened culvert wall per unit length, (in.^{4})
ϕ_s = resistance factor for soil
M_s = constrained modulus of embedment computed based on the free field vertical stress at a depth halfway between the top and springline of the structure (Table 12.12.3.5-1)
K_b = $(1-2\nu)/(1-\nu^2)$
ν = Poisson's ratio of soil
R_h = correction factor for backfill geometry
= $11.4/(11+S/H)$
S = culvert span
H = depth of fill over top of culvert

C12.8.9.6

The proposed buckling equations are taken from the recommendations of NCHRP Report 473, *Recommended Specifications for Large-Span Culverts*.

Global buckling of deep corrugated structures occurs over a long wavelength; thus, the soil modulus computed at a depth halfway between the top and springline of the structure is representative of the overall soil resistance to buckling.

12.8.9.7—Connections

The factored moment resistance of longitudinal connections shall be at least equal to the factored applied moment but not less than the greater of:

- 75 percent of the factored moment resistance of the member or
- the average of the factored applied moment and the factored moment resistance of the member.

Moment resistance of connections may be obtained from qualified tests or published standards.

C12.8.9.7

The *AASHTO LRFD Bridge Construction Specifications* require longitudinal joints to be staggered to avoid a continuous line of bolts on a structure.

12.9—STRUCTURAL PLATE BOX STRUCTURES

12.9.1—General

The design method specified herein shall be limited to depth of cover from 1.4 to 5.0 ft.

The provisions of this Article shall apply to the design of structural plate box structures, hereinafter called "metal box culverts." The provisions of Articles 12.7 and 12.8 shall not apply to metal box culvert designs, except as noted.

If rib stiffeners are used to increase the flexural resistance and moment capacity of the plate, the transverse

C12.9.1

These Specifications are based on three types of data:

- finite element soil–structure interaction analyses,
- field loading tests on instrumented structures, and
- extensive field experience.

Structural plate box culverts are composite-reinforced rib plate structures of approximately stiffeners shall consist of structural steel or aluminum sections curved to fit the structural plates. Ribs shall be bolted to the plates to develop the plastic flexural resistance of the composite section. Spacing between ribs shall not exceed 2.0 ft on the crown and 4.5 ft on the haunch. Rib splices shall develop the plastic flexural resistance required at the location of the splice.

rectangular shape. They are intended for shallow covers and low, wide waterway openings. The shallow covers and extreme shapes of box culverts require special design procedures.

Metal box culverts differ greatly from conventional metal culvert shapes. Metal box culverts are relatively flat at the top and require a large flexural capacity due to extreme geometry and shallow depths of cover of 5.0 ft or less. Analyses over the range of sizes permitted under these Specifications indicate that flexural requirements govern the choice of section in all cases. The effects of thrust are negligible in comparison with those of flexure. This difference in behavior requires a different approach in design.

For information regarding the manufacture of structures and structural components referred to herein, see AASHTO M 167M/M 167 (ASTM A761/A761M) for steel and M 219 (ASTM B746) for aluminum.

12.9.2—Loading

For live loads, the provisions of Article 3.6.1 shall apply.

Unit weights for soil backfill, other than 0.12 kcf, may be considered as specified in Article 12.9.4.2.

C12.9.2

The earth loads for the design procedure described herein are based upon soil backfill having a standard unit weight, γ_s, of 0.12 kcf.

12.9.3—Service Limit State

No service limit state criteria need be applied in the design of box culvert structures.

C12.9.3

Soil design and placement requirements for box culvert structures can limit structure deflections satisfactorily. The contract documents should require that construction procedures be monitored to ensure that severe deformations do not occur during backfill placement and compaction, in which case no deflection limits should be imposed on the completed structure.

12.9.4—Safety Against Structural Failure

12.9.4.1—General

The resistance of corrugated box culverts shall be determined at the strength limit state in accordance with Articles 12.5.3, 12.5.4, and 12.5.5 and the requirements specified herein.

Box culvert sections for which these Articles apply are defined in Figure 12.9.4.1-1 and Table 12.9.4.1-1. Table A12-10 shall apply.

C12.9.4.1

Finite element analyses covering the range of metal box culvert shapes described in this Article have shown that flexural requirements govern the design in all cases. Effects of thrust are negligible when combined with flexure.

The structural requirements for metal box culverts are based on the results of finite element analyses and field measurements of in-service box culverts.

@Seismicisolation

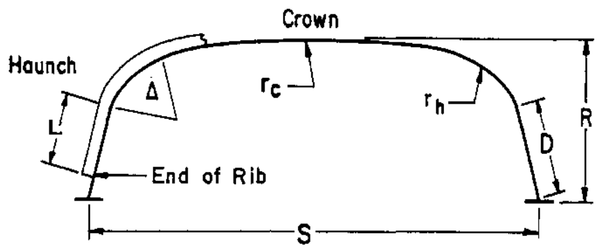


Figure 12.9.4.1-1—Geometry of Box Culverts

Table 12.9.4.1-1—Geometric Requirements for Box Culverts with Spans from 8 ft 9 in. to 25 ft 5 in.

Span, S: 8 ft 9 in. to 25 ft 5 in.
Rise, R: 2 ft 6 in. to 10 ft 6 in.
Radius of crown, $r_c \leq$ 24 ft 9 $^1/_2$ in.
Radius of haunch, $r_h \geq$ 2 ft 6 in.
Haunch radius included angle, Δ, 50° to 70°
Length of leg, D, measured to the bottom of the plate, may vary from 4 $^3/_4$ to 71 in.
Minimum length of rib on leg, L, least of 19.0 in., (D − 3.0) in., or to within 3.0 in. of the top of a concrete footing

Table 12.9.4.1-2—Geometric Requirements for Box Culverts with Spans from 25 ft 6 in. to 36 ft 0 in.

Span, S: 25 ft 6 in. to 36 ft 0 in.
Rise, R: 5 ft 7 in. to 14 ft 0 in.
Radius of crown, $r_c \leq$ 26 ft 4 in.
Radius of haunch, $r_h \geq$ 3 ft 8 in.
Haunch radius included angle, Δ: 48° to 68°
Length of leg, D, measured to the bottom of the plate, may vary from 4 $^3/_4$ to 71 in.
Minimum length of rib on leg, L, least of 28.0 in., (D – 3.0) in., or to within 3.0 in. of the top of a concrete footing

The flexural resistance of corrugated plate box structures shall be determined using the specified yield strength of the corrugated plate.

The flexural resistance of plate box structures with ribbed sections shall be determined using specified yield strength values for both rib and corrugated shell. Computed values may be used for design only after confirmation by representative flexural testing. Rib splices shall develop the plastic moment capacity required at the location of the splice.

12.9.4.2—Moments Due to Factored Loads

Unfactored crown and haunch dead and live load moments, $M_{d\ell}$ and $M_{\ell\ell}$, may be taken as:

For spans ≤25.0 ft 5.0 in.:

C12.9.4.2

The number of "wheels per notional axle group" determines the value of C_2 in Table 12.9.4.2-1. The following guidelines are consistent with the development of Table 12.9.4.2-1:

@Seismicisolation

$$M_{d\ell} = \gamma_s \left\{ S^3 \left[0.0053 - 0.00024 (S-12) \right] + 0.053 (H-1.4) S^2 \right\}$$ (12.9.4.2-1)

For spans from 25.0 ft 6.0 in. through 36.0 ft 0 in. with a geometry profile that meets r_c = 26.0 ft, r_h = 3.0 ft 8 $^7/_8$ in., and Δ = 49.16 degrees:

$$M_{d\ell} = \lambda_s \left\{ \begin{matrix} S^3 \left[0.00194 - 0.0002 (S-26)(H-1.1) \right] \\ + (H-1.4) \left[0.053 S^2 + 0.6 (S-26)^2 \right] \end{matrix} \right\}$$ (12.9.4.2-2)

$$M_{\ell\ell} = C_{\ell\ell} K_1 \frac{S}{K_2}$$ (12.9.4.2-3)

where:

$M_{d\ell}$ = sum of the nominal crown and haunch dead load moments (kip-ft/ft)
$M_{\ell\ell}$ = sum of the nominal crown and haunch live load moments (kip-ft/ft)
S = box culvert span (ft)
γ_s = soil unit weight (kcf)
H = height of cover from the box culvert rise to top of pavement (ft)
$C_{\ell\ell}$ = adjusted live load
= $C_1 \, C_2 \, A_L$ (kip)
A_L = sum of all axle loads in an axle group (kip)
C_1 = 1.0 for single axles and 0.5 + S/50 $\leq$ 1.0 for tandem axles
C_2 = adjustment factor for number of wheels on a design axle as specified in Table 12.9.4.2-1

in which:

$$K_1 = \frac{0.08}{\left(\frac{H}{S}\right)^{0.2}}, \text{ for } 8 \leq S < 20$$ (12.9.4.2-4)

$$K_1 = \frac{0.08 - 0.002\,(S-20)}{\left(\frac{H}{S}\right)^{0.2}}, \text{ for } 20 \leq S \leq 36$$ (12.9.4.2-5)

$$K_2 = 0.54H^2 - 0.4H + 5.05, \text{ for } 1.4 \leq H < 3.0$$ (12.9.4.2-6)

$$K_2 = 1.90H + 3, \text{ for } 3.0 \leq H \leq 5.0$$ (12.9.4.2-7)

- Use "2" as the number of wheels when the design is based on an axle with two wheels, e.g., two 16.0-kip wheels on one 32.0-kip axle.
- Use "4" as the number of wheels where the design is based on either an axle with four wheels, e.g., two 8.0-kip wheels on each end of a 32.0-kip axle; or two axles with two wheels each, e.g., two 12.5-kip wheels on each of two tandem 25.0-kip axles.
- Use "8" as the number of wheels when the design is based on two axles, each with a pair of wheels at each end of each axle.

For spans from 25.0 ft 6.0 in. through 36.0 ft 0 in. with profiles that do not meet the requirements of Eq. 12.9.4.2-2, finite element modeling that employs soil–structure interaction may be performed to obtain the nominal crown and haunch moments.

Table 12.9.4.2-1—Adjustment Coefficient Values (C_2) for Number of Wheels per Axle

Wheels per Notional Axle Group	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

Unless otherwise specified, the design truck specified in Article 3.6.1.2.2 should be assumed to have four wheels on an axle. The design tandem specified in Article 3.6.1.2.3 should be assumed to be an axle group consisting of two axles with four wheels on each axle.

The factored moments, $M_{d\ell u}$ and $M_{\ell\ell u}$ as referred to in Article 12.9.4.3, shall be determined as specified in Table 3.4.1-1, except that the live load factor used to compute $M_{\ell\ell u}$ shall be 2.0. The factored reactions shall be determined by factoring the reactions specified in Article 12.9.4.5.

12.9.4.3—Plastic Moment Resistance

The plastic moment resistance of the crown, M_{pc}, and the plastic moment resistance of the haunch, M_{ph}, shall not be less than the proportioned sum of adjusted dead and live load moments. The values of M_{pc} and M_{ph} shall be determined as follows:

$$M_{pc} \geq C_H P_c \left(M_{d\ell u} + M_{\ell\ell u} \right) \quad (12.9.4.3\text{-}1)$$

$$M_{ph} \geq C_H \left(1.0 - P_c \right) \left(M_{d\ell u} + R_h M_{\ell\ell u} \right) \quad (12.9.4.3\text{-}2)$$

where:

C_H = crown soil cover factor specified in Article 12.9.4.4

P_c = allowable range of the ratio of total moment carried by the crown as specified in Table 12.9.4.3-1

R_H = acceptable values of the haunch moment reduction factor as specified in Table 12.9.4.3-2

$M_{d\ell u}$ = factored dead load moment as specified in Article 12.9.4.2 (kip-ft)

$M_{\ell\ell u}$ = factored live load moment as specified in Article 12.9.4.2 (kip-ft)

C12.9.4.3

Some discretion is allowed relative to the total flexural capacity assigned to the crown and haunches of box culverts.

The distribution of moment between the crown and haunch, described in Article C12.9.4.2, is accomplished in the design using the crown moment proportioning factor, P_c, which represents the proportion of the total moment that can be carried by the crown of the box culvert and that varies with the relative flexural capacities of the crown and haunch components.

The requirements given herein can be used to investigate products for compliance with these Specifications. Using the actual crown flexural capacity, M_{pc}, provided by the metal box structure under consideration and the loading requirements of the application, Eq. 12.9.4.3-1 can be solved for the factor P_c, which should fall within the allowable range of Table 12.9.4.3-1. Knowing P_c, Eq. 12.9.4.3-2 can be solved for M_{ph}, which should not exceed the actual haunch flexural resistance provided by the structure section. If Eq. 12.9.4.3-1 indicates a higher value of P_c than permitted by the allowable ranges in Table 12.9.4.3-1, the actual crown is over designed, which is acceptable. However, in this case, only the maximum value of P_c, allowed by Table 12.9.4.3-1, should be used to calculate the required M_{ph} from Eq. 12.9.4.3-2.

Table 12.9.4.3-1—Crown Moment Proportioning Values, P_c, for Spans ≤ 25.0 ft 5.0 in.

Span (ft)	Allowable Range of P_c
<10.0	0.55–0.70
10.0–15.0	0.50–0.70
15.0–20.0	0.45–0.70
20.0–25.4	0.45–0.60

Table 12.9.4.3-2—Crown Moment Positioning Values, P_c, for Spans from 25.0 ft 6.0 in. to 36.0 ft 0.0 in.

Depth of Fill (ft)	Allowable Range of P_c
1.4–2.5	0.55–0.65
2.5–4.0	0.45–0.55
4.0–5.0	0.35–0.55

Table 12.9.4.3-3—Haunch Moment Reduction Values, R_H, for Spans ≤ 25.0 ft 5.0 in.

	Cover Depth (ft)			
	1.4	2.0	3.0	4.0 to 5.0
R_H	0.66	0.74	0.87	1.00

For spans from 25.0 ft 6.0 in. to 36.0 ft 0.0 in., R_H = 1.0 for all cover depths.

12.9.4.4—Crown Soil Cover Factor, C_H

For depths of soil cover greater than or equal to 3.5 ft, the crown soil cover factor, C_H, shall be taken as 1.0.

For crown cover depth between 1.4 and 3.5 ft, the crown soil cover factor shall be taken as:

$$C_H = 1.15 - \left(\frac{H - 1.4}{14}\right) \tag{12.9.4.4-1}$$

where:

H = depth of cover over crown (ft)

C12.9.4.4

The results of finite element analyses and field monitoring studies to evaluate the effects of load-induced deformations and in-plane deformed geometries indicate that the design moments should be increased where the cover is less than 3.5 ft.

Eq. 12.9.4.4-1 is discussed in Boulanger et al. (1989).

12.9.4.5—Footing Reactions

Reactions at the box culvert footing shall be determined as:

$$V = \gamma_s \left(\frac{HS}{2.0} + \frac{S^2}{40.0}\right) + \frac{A_L}{8 + 2(H + R)} \tag{12.9.4.5-1}$$

where:

V = unfactored footing reaction (kip/ft)
γ_s = unit weight of backfill (kcf)
H = depth of cover over crown (ft)
R = rise of culvert (ft)

S = span (ft)
A_L = total axle load (kip)

12.9.4.6—Concrete Relieving Slabs

Relieving slabs may be used to reduce flexural moments in box culverts. Relieving slabs shall not be in contact with the crown as shown in Figure 12.9.4.6-1.

The length of the concrete relieving slab shall be at least 2.0 ft greater than the culvert span and sufficient to project 1.0 ft beyond the haunch on each side of the culvert. The relieving slab shall extend across the width subject to vehicular loading.

The depth of reinforced concrete relieving slabs shall be determined as:

$$t = t_b R_{AL} R_c R_f \tag{12.9.4.6-1}$$

where:

t = minimum depth of slab (in.)
t_b = basic slab depth as specified in Table 12.9.4.6-1 (in.)
R_{AL} = axle load correction factor specified in Table 12.9.4.6-2
R_c = concrete strength correction factor specified in Table 12.9.4.6-3
R_f = factor taken as 1.2 for box structures having spans less than 26.0 ft

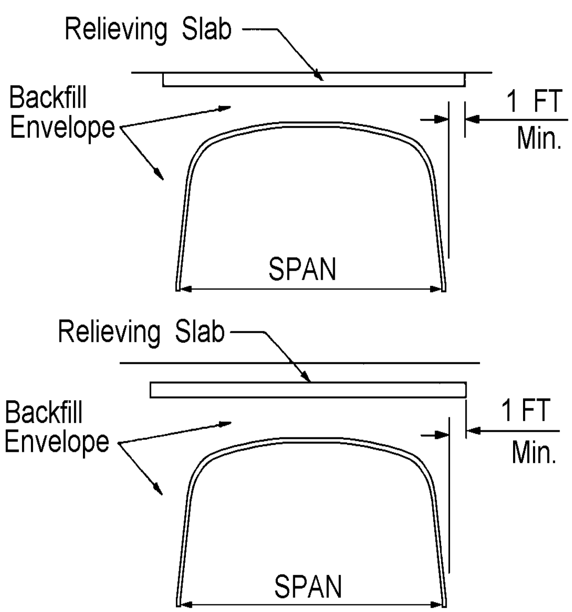


Figure 12.9.4.6-1—Metal Box Culverts with Concrete Relieving Slab

C12.9.4.6

The box culvert design procedure described herein does not directly incorporate consideration of concrete relieving slabs on the influence of concrete pavement. Therefore, the procedures described in Duncan et al. (1985) should be used instead. At this time, the beneficial effect of a relieving slab can only be determined by refined soil–structure interaction analyses. The provisions given herein are applicable only for box structures having spans under 26.0 ft. The purpose of avoiding contact between the relieving slab and the culvert is to avoid concentration of the load applied through the slab to the crown of the culvert. As little as 1.0- to 3.0-in. clearance is thought to be sufficient to distribute the load.

Where an Owner requires design for an axle other than the standard 32.0-kip axle, the factor R_{AL} may be used to adjust the depth of a concrete relieving slab as specified in Eq. 12.9.4.6-1.

@Seismicisolation

Table 12.9.4.6-1—Basic Slab Depth, t_b (in.) (Duncan et al., 1985)

Unified Classification of Subgrade Beneath Slab	Relative Compaction—% of Standard AASHTO Maximum Dry Unit Weight		
	100	95	90
	Basic Slab Depth (in.)		
GW, GP, SW, SP, or SM	7.5	8.0	8.5
SM–SC or SC	8.0	8.5	9.0
ML or CL	8.5	9.0	9.5

Table 12.9.4.6-2—Axle Load Correction Factor, R_{AL} (Duncan et al., 1985)

Single Axle Load (kip)	R_{AL}
10.0	0.60
20.0	0.80
30.0	0.97
32.0	1.00
40.0	1.05
45.0	1.10
50.0	1.15

Table 12.9.4.6-3—Concrete Strength Correction Factor, R_c (Duncan et al., 1985)

Concrete Compressive Strength, f'_c (ksi)	R_c
3.0	1.19
3.5	1.15
4.0	1.10
4.5	1.05
5.0	1.01
5.5	0.97
6.0	0.94

12.9.5—Construction and Installation

The contract documents shall require that construction and installation conform to Section 26, "Metal Culverts," of the *AASHTO LRFD Bridge Construction Specifications*.

12.10—REINFORCED CONCRETE PIPE

12.10.1—General

The provisions herein shall apply to the structural design of buried precast reinforced concrete pipes of circular, elliptical, and arch shapes.

C12.10.1

These structures become part of a composite system comprised of the reinforced concrete buried section and the soil envelope.

@Seismicisolation

The structural design of the types of pipes indicated above may proceed by either of two methods:

- The direct design method at the strength limit state as specified in Article 12.10.4.2, or
- The indirect design method at the service limit state as specified in Article 12.10.4.3.

Standard dimensions for these units are shown in AASHTO M 170 and M 170M (ASTM C76 and C76M), M 206M/M 206 (ASTM C506M and C506), M 207M/M 207 (ASTM C507M and C507), M 242M/M 242 (ASTM C655M and C655), and ASTM C1765.

NCHRP 20-07 Task 316 established that while direct design for small diameter pipes could lead to more steel reinforcement than when using indirect design, which results from simplifying conservative assumptions during direct design, either design procedure can be employed.

12.10.2 Loading

12.10.2.1 Standard Installations

The contract documents shall specify that the foundation bedding and backfill comply with the provisions of Article 27.5.2 of the *AASHTO LRFD Bridge Construction Specifications*.

Minimum compaction requirements and bedding thickness for standard embankment installations and standard trench installations shall be as specified in Tables 12.10.2.1-1 and 12.10.2.1-2, respectively.

C12.10.2.1

The four standard installations replace the historic bedding classes. A comprehensive soil–structure interaction analysis and design program (SPIDA) was developed and used to perform soil–structure interaction analyses for the various soil and installation parameters encompassed in the provisions. The SPIDA studies used to develop the standard installations were conducted for positive projection embankment conditions to provide conservative results for other embankment and trench conditions. These studies also conservatively assume a hard foundation and bedding existing beneath the invert of the pipe, plus void and/or poorly compacted material in the haunch areas, 15 degrees to 40 degrees each side of the invert, resulting in a load concentration such that calculated moments, thrusts, and shears are increased.

Table 12.10.2.1-1—Standard Embankment Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	For soil foundation, use D_o/24.0 in. minimum, not less than 3.0 in. For rock foundation, use D_o/12.0 in. minimum, not less than 6.0 in.	95% Category I	90% Category I, 95% Category II, or 100% Category III
Type 2—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use D_o/24.0 in. minimum, not less than 3.0 in. For rock foundation, use D_o/12.0 in. minimum, not less than 6.0 in.	90% Category I or 95% Category II	85% Category I, 90% Category II, or 95% Category III
Type 3—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use D_o/24.0 in. minimum, not less than 3.0 in. For rock foundation, use D_o/12.0 in. minimum, not less than 6.0 in.	85% Category I, 90% Category II, or 95% Category III	85% Category I, 90% Category II, or 95% Category III
Type 4	For soil foundation, no bedding required. For rock foundation, use D_o/12.0 in. minimum, not less than 6.0 in.	No compaction required, except if Category III, use 85% Category III	No compaction required, except if Category III, use 85% Category III

The following interpretations apply to Table 12.10.2.1-1:

@Seismicisolation

- Compaction and soil symbols (that is, 95 percent Category I) shall be taken to refer to a soil material category with a minimum standard proctor compaction of 95 percent. Equivalent modified proctor values shall be as given in Table 27.5.2.2-3 of the *AASHTO LRFD Bridge Construction Specifications*.
- Soil in the outer bedding, haunch, and lower side zones, except within $B_c/3$ from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in the overfill zone.
- The minimum width of a subtrench shall be $1.33B_c$, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.
- For subtrenches with walls of natural soil, any portion of the lower side zone in the subtrench wall shall be at least as firm as an equivalent soil placed to the compaction requirements specified for the lower side zone and as firm as the majority of soil in the overfill zone. Otherwise, it shall be removed and replaced with soil compacted to the specified level.

- A subtrench is defined as a trench in the natural material under an embankment used to retain bedding material with its top below finished grade by more than ten percent of the depth of soil cover on the top of the culvert or pipe. For roadways, the top of a subtrench is at an elevation lower than 1.0 ft below the bottom of the pavement base material.

Table 12.10.2.1-2—Standard Trench Installation Soils and Minimum Compaction Requirements

Installation Type	Bedding Thickness	Haunch and Outer Bedding	Lower Side
Type 1	For soil foundation, use $D_o/24.0$ in. minimum, not less than 3.0 in. For rock foundation, use $D_o/12.0$ in. minimum, not less than 6.0 in.	95% Category I	90% Category I, 95% Category II, or 100% Category III, or natural soils of equal firmness
Type 2—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use $D_o/24.0$ in. minimum, not less than 3.0 in. For rock foundation, use $D_o/12.0$ in. minimum, not less than 6.0 in.	90% Category I or 95% Category II	85% Category I, 90% Category II, 95% Category III, or natural soils of equal firmness
Type 3—Installations are available for horizontal elliptical, vertical elliptical, and arch pipe	For soil foundation, use $D_o/24.0$ in. minimum, not less than 3.0 in. For rock foundation, use $D_o/12.0$ in. minimum, not less than 6.0 in.	85% Category I, 90% Category II or 95% Category III	85% Category I, 90% Category II, 95% Category III, or natural soils of equal firmness
Type 4	For soil foundation, no bedding required. For rock foundation, use $D_o/12.0$ in. minimum, not less than 6.0 in.	No compaction required, except if Category III, use 85% Category III	No compaction required except if Category III, use 85% Category III, or natural soil of equal firmness

The following interpretations apply to Table 12.10.2.1-2:

- Compaction and soil symbols (that is, 95 percent Category I) shall be taken to refer to a soil material category with a minimum standard proctor compaction of 95 percent. Equivalent modified proctor values shall be as given in Table 27.5.2.2-3 of the *AASHTO LRFD Bridge Construction Specifications*.

- The trench top elevation shall be no lower than $0.1H$ below finish grade; for roadways, its top shall be no lower than an elevation of 1.0 ft below the bottom of the pavement base material.
- Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone.

If required for adequate space to attain the specified compaction in the haunch and bedding zones the trench width shall be wider than that shown in Figures 27.5.2.2-1 and 27.5.2.2-2 of the *AASHTO LRFD Bridge Construction Specifications*.

- For trench walls that are within 10 degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need not be considered.
- For trench walls with greater than 10 degrees slopes that consist of embankment, the lower side shall be compacted to at least the same compaction as specified for the soil in the backfill zone.

The unfactored earth load, W_E, shall be determined as:

$$W_E = F_e w B_c H \quad (12.10.2.1\text{-}1)$$

where:

W_E = unfactored earth load (kip/ft)
F_e = soil–structure interaction factor for the specified installation as defined herein
B_c = out-to-out horizontal dimension of pipe (ft)
H = height of fill above top of pipe (ft)
w = unit weight of soil (pcf)

The unit weight of soil used to calculate earth load shall be the estimated unit weight for the soils specified for the pipe soil installation but shall not be taken to be less than 110 lb/ft^3.

Standard installations for both embankments and trenches shall be designed for positive projection, embankment loading conditions where F_e shall be taken as the vertical arching factor, VAF, specified in Table 12.10.2.1-3 for each type of standard installation.

For standard installations, the earth pressure distribution shall be the Heger pressure distribution shown in Figure 12.10.2.1-1 and Table 12.10.2.1-3 for each type of standard installation.

The product wB_cH is sometimes referred to as the prism load, PL, the weight of the column of earth over the outside diameter of the pipe.

The earth load for designing pipe using a standard installation is obtained by multiplying the weight of the column of earth above the outside diameter of the pipe by the soil–structure interaction factor, F_e, for the design installation type. F_e accounts for the transfer of some of the overburden soil above the regions at the sides of the pipe because the pipe is more rigid than the soil at the side of the pipe for pipe in embankment and wide trench installations. Because of the difficulty of controlling maximum trench width in the field with the widespread use of trench boxes or sloped walls for construction safety, the potential reduction in earth load for pipe in trenches of moderate to narrow width is not taken into account in the determination of earth load and earth pressure distribution on the pipe. Both trench and embankment installations are to be designed for embankment (positive projecting) loads and pressure distribution in direct design or bedding factors in indirect design.

The earth pressure distribution and lateral earth force for a unit vertical load is the Heger pressure distribution and horizontal arching factor, HAF. The normalized pressure distribution and HAF values were obtained for each standard installation type from the results of soil–structure interaction analyses using SPIDA, together with the minimum soil properties for the soil types and compaction levels specified for the installations.

When nonstandard installations are used, the earth load and pressure distribution should be determined by an appropriate soil–structure interaction analysis.

@Seismicisolation

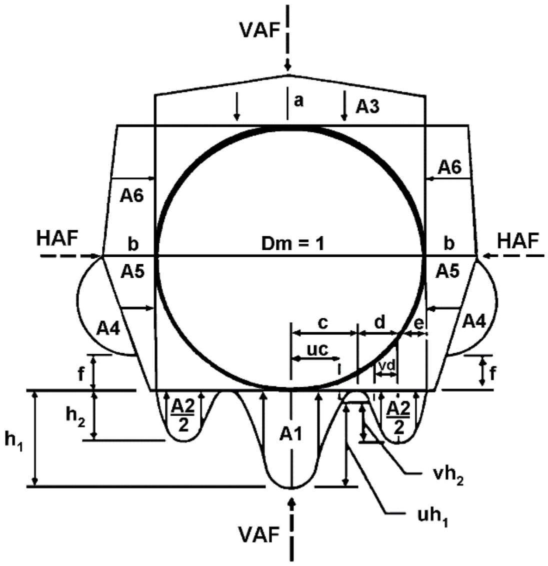


Figure 12.10.2.1-1—Heger Pressure Distribution and Arching Factors

Table 12.10.2.1-3—Coefficients for Use with Figure 12.10.2.1-1

	Installation Type			
	1	2	3	4
VAF	1.35	1.40	1.40	1.45
HAF	0.45	0.40	0.37	0.30
*A*1	0.62	0.85	1.05	1.45
*A*2	0.73	0.55	0.35	0.00
*A*3	1.35	1.40	1.40	1.45
*A*4	0.19	0.15	0.10	0.00
*A*5	0.08	0.08	0.10	0.11
*A*6	0.18	0.17	0.17	0.19
a	1.40	1.45	1.45	1.45
b	0.40	0.40	0.36	0.30
c	0.18	0.19	0.20	0.25
e	0.08	0.10	0.12	0.00
f	0.05	0.05	0.05	—
u	0.80	0.82	0.85	0.90
v	0.80	0.70	0.60	—

The following shall apply to Table 12.10.2.1-3:

- *VAF* and *HAF* are vertical and horizontal arching factors. These coefficients represent nondimensional total vertical and earth loads on the pipe, respectively. The actual total vertical and horizontal loads are (*VAF*) × (*PL*) and (*HAF*) × (*PL*), respectively, where *PL* is the prism load.

@Seismicisolation

- Coefficients $A1$ through $A6$ represent the integration of nondimensional vertical and horizontal components of soil pressure under the indicated portions of the component pressure diagrams, i.e., the area under the component pressure diagrams.
- The pressures are assumed to vary either parabolically or linearly, as shown in Figure 12.10.2.1-1, with the nondimensional magnitudes at governing points represented by h_1, h_2, uh_1, vh_2, a, and b.
- Nondimensional horizontal and vertical dimensions of component pressure regions are defined by c, d, e, uc, vd, and f coefficients,

where:

$$d = (0.5 - c - e) \quad (12.10.2.1\text{-}2)$$

$$h_1 = (1.5A1)/(c)(1+u) \quad (12.10.2.1\text{-}3)$$

$$h_2 = (1.5A2)/\left[(d)(1+v)+(2e)\right] \quad (12.10.2.1\text{-}4)$$

12.10.2.2 Pipe Fluid Weight

The unfactored weight of fluid, W_F, in the pipe shall be considered in design based on a fluid weight of 62.4 lb/ft^3, unless otherwise specified. For standard installations, the fluid weight shall be supported by vertical earth pressure that is assumed to have the same distribution over the lower part of the pipe as given in Figure 12.10.2.1-1 for earth load.

12.10.2.3—Live Loads

Live loads shall be as specified in Article 3.6 and shall be distributed through the earth cover as specified in Article 3.6.1.2.6. For standard installations, the live load on the pipe shall be assumed to have a uniform vertical distribution across the top of the pipe and the same distribution across the bottom of the pipe as given in Figure 12.10.2.1-1.

12.10.3—Service Limit State

The width of cracks in the wall shall be investigated at the service limit state for moment and thrust. Generally, the crack width should not exceed 0.01 in.

12.10.4—Safety Against Structural Failure

12.10.4.1—General

The resistance of buried reinforced concrete pipe structures against structural failure shall be determined at the strength limit state for:

- Flexure,
- Thrust,
- Shear, and
- Radial tension.

The dimensions of pipe sections shall be determined using either the analytically-based direct design method or the empirically-based indirect design method.

When quadrant mats, stirrups, and/or elliptical cages are specified in the contract documents, the orientation of the pipe installation shall be specified, and the design shall account for the possibility of an angular misorientation of 10 degrees during the pipe installation.

C12.10.4.1

The direct design method uses a pressure distribution on the pipe from applied loads and bedding reactions based on a soil–structure interaction analysis or an elastic approximation. The indirect design method uses empirically-determined bedding factors that relate the total factored earth load to the concentrated loads and reactions applied in three-edge bearing tests.

12.10.4.2—Direct Design Method

12.10.4.2.1—Loads and Pressure Distribution

The total vertical load acting on the pipe shall be determined as specified in Article 12.10.2.1.

The pressure distribution on the pipe from applied loads and bedding reaction shall be determined from either a soil–structure analysis or from a rational approximation, either of which shall permit the development of a pressure diagram, shown schematically in Figure 12.10.4.2.1-1, and the analysis of the pipe.

C12.10.4.2.1

The direct design method was accepted in 1993 by ASCE and is published in ASCE 93-15, *Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations* (SIDD). The design method was developed along with the research performed on the standard installations. However, the design equations are applied after the required flexural moments, thrusts, and shear forces at all critical sections have been determined using any one of the acceptable pressure distributions. Therefore, the use of the design equations herein is not limited to the standard installations or any one assumed pressure distribution.

Direct design requires:

- The determination of earth loads and live load pressure distributions on the structure for the bedding and installation conditions selected by the Engineer;
- Analysis to determine thrust, moments, and shears; and
- Design to determine circumferential reinforcement.

The procedures for analysis and design are similar to those used for other reinforced concrete structures.

@Seismicisolation

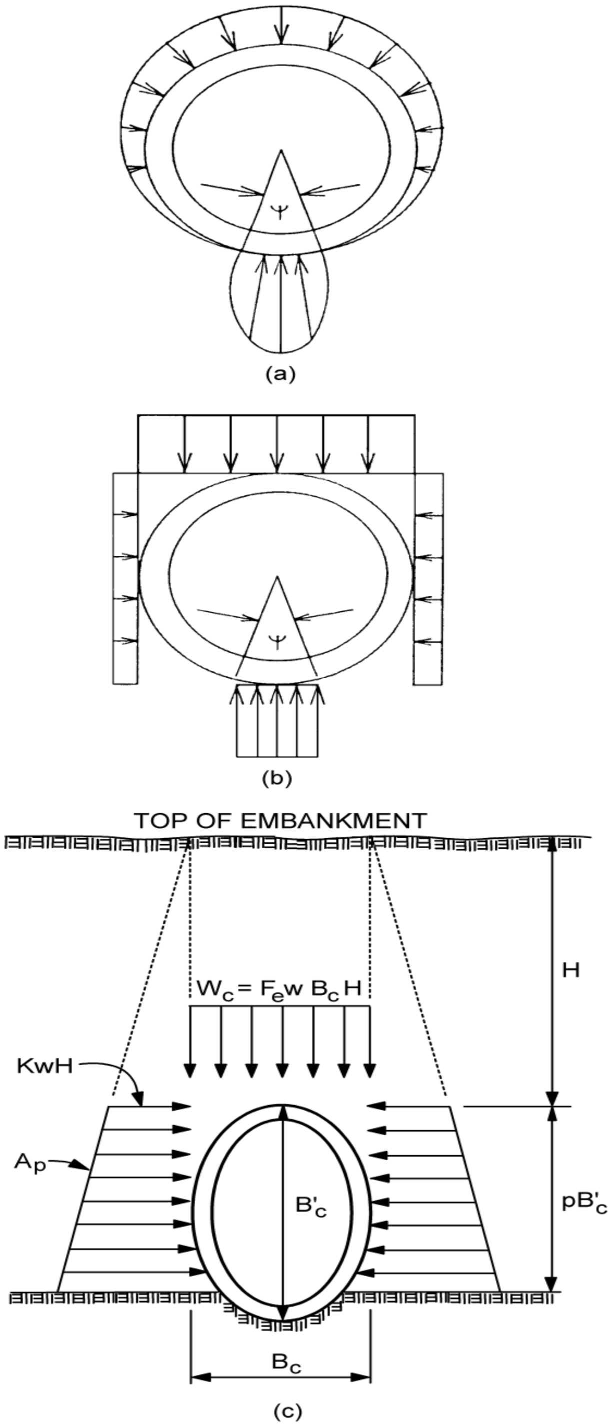


Figure 12.10.4.2.1-1—Suggested Design Pressure Distribution around a Buried Concrete Pipe for Analysis by Direct Design

12.10.4.2.2—Analysis for Force Effects with the Pipe Ring

Force effects in the pipe shall be determined by an elastic analysis of the pipe ring under the assumed pressure distribution or a soil–structure analysis.

If thin ring theory is employed, the expected moments at crown and invert can be adjusted to account for ring thickness by:

$$M_{thick} = M_{thin}\left(1 - 0.373\ \frac{t}{R}\right) \qquad (12.10.4.2.2\text{-}1)$$

where:

M_{thick} = adjusted moment
M_{thin} = moment calculated using thin ring theory
t = thickness of the concrete pipe
R = average radius of the concrete pipe

C12.10.4.2.2

NCHRP 20-07 Task 316 established that the discrepancies in quantities of flexural reinforcement resulting from direct and indirect design are partly due to the use of thin ring theory. Adjustment to account for thick ring theory resolves those discrepancies in part, though much of the conservatism of direct design remains (for example, the estimation of expected moment and consideration of ultimate limit state based on first plastic moment).

12.10.4.2.3—Process and Material Factors

Process and material factors, F_{rp} for radial tension and F_{vp} for shear strength, for design of plant-made reinforced concrete pipe should be taken as 1.0. Higher values of these factors may be used if substantiated by sufficient testing in accordance with AASHTO M 242M/M 242 (ASTM C655M and C655).

12.10.4.2.4—Flexural Resistance at the Strength Limit State

12.10.4.2.4a—Circumferential Reinforcement

Reinforcement for flexural resistance provided in a length, b, usually taken as 12.0 in., shall satisfy:

$$A_s \geq \frac{g\phi d - N_u - \sqrt{g\left[g\left(\phi d\right)^2 - N_u\left(2\phi d - h\right) - 2M_u\right]}}{f_y} \qquad (12.10.4.2.4a\text{-}1)$$

in which:

$$g = 0.85 b f_c' \qquad (12.10.4.2.4a\text{-}2)$$

where:

A_s = area of reinforcement per length of pipe, b (in.^{2}/ft)
f_y = specified yield strength of reinforcing less than or equal to 80 ksi
d = distance from compression face to centroid of tension reinforcement (in.)
h = wall thickness of pipe (in.)
M_u = moment due to factored loads (kip-in./ft)

C12.10.4.2.4a

NCHRP 20-07 Task 316 established that more sophisticated procedures can be employed for estimating the flexural and shear resistance at the strength limit state, such as Modified Compression Field Theory, to enforce compatibility between steel and concrete components and account for multiple layers of steel reinforcement and nonlinear concrete and steel behavior.

The required area of steel, A_s, as determined by Eq. 12.10.4.2.4a-1, should be distributed over a unit length of the pipe, b, which is typically taken as 12.0 in.

The factored actions should also be consistent with the selected unit width.

@Seismicisolation

N_u = thrust due to factored load, taken to be positive for compression (kip/ft)
ϕ = resistance factor for flexure specified in Article 12.5.5

12.10.4.2.4b—Minimum Reinforcement

The reinforcement, A_s, per ft of pipe shall not be less than:

- For inside face of pipe with two layers of reinforcement:

$$A_s \geq \frac{(S_i + h)^2}{1{,}000 f_y} \geq 0.07 \quad (12.10.4.2.4b\text{-}1)$$

- For outside face of pipe with two layers of reinforcement:

$$A_s \geq 0.60 \frac{(S_i + h)^2}{1{,}000 f_y} \geq 0.07 \quad (12.10.4.2.4b\text{-}2)$$

- For elliptical reinforcement in circular pipe and for 33.0-in. diameter and smaller pipe with a single cage of reinforcement in the middle third of the pipe wall:

$$A_s \geq 2 \frac{(S_i + h)^2}{1{,}000 f_y} \geq 0.07 \quad (12.10.4.2.4b\text{-}3)$$

where:

S_i = internal diameter or horizontal span of the pipe (in.)
h = wall thickness of pipe (in.)
f_y = yield strength of reinforcement less than or equal to 80 ksi

12.10.4.2.4c—Maximum Flexural Reinforcement without Stirrups

The flexural reinforcement per ft of pipe without stirrups shall satisfy:

- For inside steel in radial tension:

$$A_{smax} \leq \frac{0.506 r_s F_{rp} \sqrt{f'_c} (R_\phi) F_{rt}}{f_y} \quad (12.10.4.2.4c\text{-}1)$$

where:

A_{smax} = minimum flexural reinforcement area without stirrups (in.^{2}/ft)
r_s = radius of the inside reinforcement (in.)
f'_c = compressive strength of concrete (ksi)
f_y = specified yield strength of reinforcement less than or equal to 80 ksi

C12.10.4.2.4c

Research has shown that reinforcing steel with yield strengths up to 80 ksi can be used in the flexural design of concrete. Eqs. 12.10.4.2.4c-1 and 12.10.4.2.4c-2 utilize the reinforcing yield stress times the area of reinforcement to establish a tension force applied to the concrete. Higher reinforcing yield strengths will result in higher tension forces for an equivalent area of steel. Thus, the allowable maximum steel area is less when utilizing high strength reinforcement. Therefore, the same reinforcing yield strength value used in determining the flexural reinforcement area in Eq. 12.10.4.2.4a-1 should be used in Eqs. 12.10.4.2.4c-1 and 12.10.4.2.4c-2.

R_ϕ = (ϕ_r/ϕ_f); ratio of resistance factors for radial tension and moment specified in Article 12.5.5
F_{rp} = 1.0 unless a higher value substantiated by test data and approved by the Engineer

in which:

- For 12.0 in. $\leq S_i \leq$ 72.0 in.

  $F_{rt} = 1 + 0.00833\,(72 - S_i)$

- For 72.0 in. $< S_i \leq$ 144.0 in.:

$$F_{rt} = \frac{(144 - S_i)^2}{26{,}000} + 0.80$$

- For $S_i >$ 144.0 in.:

  $F_{rt} = 0.80$

- For reinforcing steel in compression:

$$A_{smax} \leq \frac{\left[\left(\frac{55g'\phi d}{87 + f_y}\right) - 0.75N_u\right]}{f_y} \tag{12.10.4.2.4c-2}$$

in which:

$$g' = b\,f'_c\,[0.85 - 0.05\,(f'_c - 4.0)] \tag{12.10.4.2.4c-3}$$

$$0.85\,b\,f'_c \geq g' \geq 0.65\,b\,f'_c \tag{12.10.4.2.4c-4}$$

where:

b = width of section taken as 12.0 in.
ϕ = resistance factor for flexure as specified in Article 5.5.4.2
f_y = specified yield strength of reinforcing less than or equal to 80 ksi

12.10.4.2.4d—Reinforcement for Crack Width Control

The crack width factor, F_{cr}, may be determined as:

- If N_s is compressive, it is taken as positive and:

$$F_{cr} = \frac{B_1}{30\phi dA_s}\left[\frac{M_s + N_s\left(d - \frac{h}{2}\right)}{ij} - 0.0316C_1bh^2\sqrt{f'_c}\right] \tag{12.10.4.2.4d-1}$$

- If N_s is tensile, it is taken as negative and:

$$F_{cr} = \frac{B_1}{30dA_s\phi}\left(1.1M_s - 0.6N_sd - 0.0316C_1bh^2\sqrt{f'_c}\right) \tag{12.10.4.2.4d-2}$$

C12.10.4.2.4d

The crack control coefficients, B_1 and C_1, are dependent on the type of reinforcement.

Crack control is assumed to be 1.0 in. from the closest tension reinforcement, even if the cover over the reinforcement is greater than or less than 1.0 in. The crack control factor, F_{cr}, in Eq. 12.10.4.2.4d-1 indicates the probability that a crack of a specified maximum width will occur.

If the ratio of e/d is less than 1.15, crack control will not govern.

@Seismicisolation

in which:

$$j = 0.74 + 0.1\frac{e}{d} \leq 0.9 \qquad (12.10.4.2.4d\text{-}3)$$

$$i = \frac{1}{\left(1 - \frac{jd}{e}\right)} \qquad (12.10.4.2.4d\text{-}4)$$

$$e = \frac{M_s}{N_s} + d - \frac{h}{2} \qquad (12.10.4.2.4d\text{-}5)$$

$$B_1 = \left(\frac{t_b S_\ell}{2n}\right)^{\frac{1}{3}} \qquad (12.10.4.2.4d\text{-}6)$$

where:

M_s = flexural moment at service limit state (kip-in./ft)
N_s = axial thrust at service limit state (kip/ft)
d = distance from compression face to centroid of tension reinforcement (in.)
h = wall thickness (in.)
f'_c = specified compressive strength of concrete (ksi)
C_1 = crack control coefficient for various types of reinforcement as specified in Table 12.10.4.2.4d-1
A_s = area of steel (in.^{2}/ft)
i = coefficient for effect of axial force at service limit state, f_s
j = coefficient for moment arm at service limit state, f_s
b = width of section taken as 12.0 in.
t_b = clear cover over reinforcement (in.)
S_ℓ = spacing of circumferential reinforcement (in.)
n = 1.0 when tension reinforcement is a single layer
n = 2.0 when tension reinforcement is made of multiple layers
ϕ = resistance factor for flexure as specified in Article 12.5.5

Table 12.10.4.2.4d-1—Crack Control Coefficients

Type	Reinforcement	C_1
1	Smooth wire or plain bars	1.0
2	Welded smooth wire fabric with 8.0-in. maximum spacing of longitudinals, welded deformed wire fabric, or deformed wire	1.5
3	Deformed bars or any reinforcement with stirrups anchored thereto	1.9

For Type 2 reinforcement in Table 12.10.4.2.4d-1 having $t_b^2 S_i/n > 3.0$, the crack width factor, F_{cr}, shall also be investigated using coefficients B_1 and C_1 specified for Type 3 reinforcement, and the larger value for F_{cr} shall be used.

Higher values for C_1 may be used if substantiated by test data and approved by the Engineer.

Where F_{cr} = 1.0, the specified reinforcement is expected to produce an average maximum crack width of 0.01 in. For F_{cr} < 1.0, the probability of a 0.01-in. crack is reduced, and for F_{cr} > 1.0, it is increased.

12.10.4.2.4e—Minimum Concrete Cover

The provisions of Article 5.10.1 shall apply to minimum concrete cover, except as follows:

- If the wall thickness is less than 2.5 in., the cover shall not be less than 0.75 in., and
- If the wall thickness is not less than 2.5 in., the cover shall not be less than 1.0 in.

12.10.4.2.5—Shear Resistance without Stirrups

C12.10.4.2.5

The section shall be investigated for shear at a critical section taken where $M_{nu}/(V_u d) = 3.0$. The factored shear resistance without radial stirrups, V_r, shall be taken as:

$$V_r = \phi V_n \quad (12.10.4.2.5\text{-}1)$$

in which:

$$V_n = 0.0316 b d F_{vp} \sqrt{f_c'}(1.1 + 63\rho)\left(\frac{F_d F_n}{F_c}\right) \quad (12.10.4.2.5\text{-}2)$$

$$\rho = \frac{A_s}{bd} \leq 0.02 \quad (12.10.4.2.5\text{-}3)$$

- For pipes with two cages or a single elliptical cage:

$$F_d = 0.8 + \frac{1.6}{d} \leq 1.3 \quad (12.10.4.2.5\text{-}4)$$

- For pipes not exceeding 36.0-in. diameter with a single circular cage:

$$F_d = 0.8 + \frac{1.6}{d} \leq 1.4 \quad (12.10.4.2.5\text{-}5)$$

For the purpose of this Article, a cage is considered to be a layer of reinforcement.

If N_u is compressive, it is taken as positive and:

$$F_n = 1 + \frac{N_u}{24h} \quad (12.10.4.2.5\text{-}6)$$

If N_u is tensile, it is taken as negative and:

$$F_n = 1 + \frac{N_u}{6h} \quad (12.10.4.2.5\text{-}7)$$

@Seismicisolation

$$F_c = 1 \pm \frac{d}{2r} \quad (12.10.4.2.5\text{-}8)$$

$$M_{nu} = M_u - N_u \left[\frac{(4h - d)}{8} \right] \quad (12.10.4.2.5\text{-}9)$$

The algebraic sign in Eq. 12.10.4.2.5-8 shall be taken as positive where tension is on the inside of the pipe and negative where tension is on the outside of the pipe.

where:

f'_{cmax} = 7.0 ksi
b = width of design section taken as 12.0 in.
d = distance from compression face to centroid of tension reinforcement (in.)
h = wall thickness (in.)
ϕ = resistance factor for shear as specified in Article 5.5.4.2
r = radius to centerline of concrete pipe wall (in.)
N_u = thrust due to factored loads (kip/ft)
V_u = shear due to factored loads (kip/ft)
F_{vp} = process and material factor specified in Article 12.10.4.2.3

If the factored shear resistance, as determined herein, is not adequate, radial stirrups shall be provided in accordance with Article 12.10.4.2.6.

12.10.4.2.6—Shear Resistance with Radial Stirrups

Radial tension and shear stirrup reinforcement shall not be less than:

- For radial tension:

$$A_{vr} = \frac{1.1 s_v (M_u - 0.45 N_u \phi_r d)}{f_y r_s \phi_r d} \quad (12.10.4.2.6\text{-}1)$$

$$s_v \leq 0.75 \phi_r d \quad (12.10.4.2.6\text{-}2)$$

- For shear:

$$A_{vs} = \frac{1.1 s_v}{f_y \phi_v d} (V_u F_c - V_c) + A_{vr} \quad (12.10.4.2.6\text{-}3)$$

$$s_v \leq 0.75 \phi_v d \quad (12.10.4.2.6\text{-}4)$$

in which:

$$V_c = \frac{4V_r}{\frac{M_{nu}}{V_u d} + 1} \leq 0.0633 \phi_v b d \sqrt{f'_c} \quad (12.10.4.2.6\text{-}5)$$

where:

M_u = flexural moment due to factored loads (kip-in./ft)
M_{nu} = factored moment acting on cross-section width, b, as modified for effects of compressive or tensile thrust (kip-in./ft)
N_u = thrust due to factored loads (kip/ft)
V_u = shear due to factored loads (kip/ft)
V_c = shear resistance of concrete section (kip/ft)
d = distance from compression face to centroid of tension reinforcement (in.)
f_y = specified yield strength for reinforcement; the value of f_y shall be taken as the lesser of the yield strength of the stirrup or its developed anchorage capacity (ksi)
r_s = radius of inside reinforcement (in.)
s_v = spacing of stirrups (in.)
V_r = factored shear resistance of pipe section without radial stirrups per unit length of pipe (kip/ft)
A_{vr} = stirrup reinforcement area to resist radial tension forces on cross-section width, b, in each line of stirrups at circumferential spacing, s_v (in.^{2}/ft)
A_{vs} = required area of stirrups for shear reinforcement (in.^{2}/ft)
f'_c = compressive strength of concrete (ksi)
ϕ_v = resistance factor for shear as specified in Article 12.5.5
ϕ_r = resistance factor for radial tension as specified in Article 12.5.5
F_c = curvature factor as determined by Eq. 12.10.4.2.5-8

12.10.4.2.7—Stirrup Reinforcement Anchorage

12.10.4.2.7a—Radial Tension Stirrup Anchorage

When stirrups are used to resist radial tension, they shall be anchored around each circumferential of the inside cage to develop the resistance of the stirrup, and they shall also be anchored around the outside cage or embedded sufficiently in the compression side to develop the required resistance of the stirrup.

C12.10.4.2.7a

Stirrup reinforcement anchorage development research by pipe manufacturers has demonstrated that the free ends of loop-type stirrups need only be anchored in the compression zone of the concrete cross-section to develop the full tensile strength of the stirrup wire. Stirrup loop lengths equivalent to 70 percent of the wall thickness may be considered to provide adequate anchorage.

12.10.4.2.7b—Shear Stirrup Anchorage

Except as specified herein, when stirrups are not required for radial tension but required for shear, their longitudinal spacing shall be such that they are anchored around each tension circumferential or every other tension circumferential. The spacing of such stirrups shall not exceed 6.0 in.

12.10.4.2.7c—Stirrup Embedment

Stirrups intended to resist forces in the invert and crown regions shall be anchored sufficiently in the opposite side of the pipe wall to develop the required resistance of the stirrup.

12.10.4.3—Indirect Design Method

12.10.4.3.1—Bearing Resistance

Earth and live loads on the pipe shall be determined in accordance with Article 12.10.2 and compared to three-edge bearing strength D-load for the pipe. The service limit state shall apply using the criterion of acceptable crack width specified herein.

The D-load for a particular class and size of pipe shall be determined in accordance with AASHTO M 170M and M 170 (ASTM C76M and C76), M 206M/M 206 (ASTM C506M and C506), M 207M/M 207 (ASTM C507M and C507), M 242M/M 242 (ASTM C655M and C655), or ASTM C1765.

The three-edge bearing resistance of the reinforced concrete pipe, corresponding to an experimentally tested service load, shall not be less than the design load determined for the pipe as installed, taken as:

$$D = \left(\frac{12}{S_i}\right)\left(\frac{W_E + W_F}{B_{FE}} + \frac{W_L}{B_{FLL}}\right) \qquad (12.10.4.3.1\text{-}1)$$

where:

B_{FE} = earth load bedding factor specified in Article 12.10.4.3.2a or Article 12.10.4.3.2b
B_{FLL} = live load bedding factor specified in Article 12.10.4.3.2c
S_i = internal diameter of pipe (in.)
W_E = total unfactored earth load specified in Article 12.10.2.1 (kip/ft)
W_F = total unfactored fluid load in the pipe as specified in Article 12.10.2.2 (kip/ft)
W_L = total unfactored live load on unit length pipe specified in Article 12.10.2.3 (kip/ft)

For Type 1 installations, *D* loads, as calculated above, shall be modified by multiplying by an installation factor of 1.10.

C12.10.4.3.1

The indirect design method has been the most commonly utilized method of design for buried reinforced concrete pipe. It is based on correlating the stress on the pipe in the field to the stress on the pipe when tested in the three-edge bearing apparatus.

The required D-load at which the pipe develops its ultimate strength in a three-edge bearing test is the design D-load at service, as calculated in Eq. 12.10.4.3.1-1, multiplied by a strength factor specified in the manufacturing standard. For conventionally reinforced concrete pipe produced to AASHTO M 170M and M 170 (ASTM C76M and C76), M 242M/M 242 (ASTM 655M and C655), M 206M/M 206 (ASTM C506M and C506) or M 207M/M 207 (ASTM C507M and C507) the service load criteria in the three-edge bearing test is a 0.01-in. crack. For fiber reinforced concrete pipe produced to ASTM C1765, a 0.01-in. crack is not reached at the design service load. Thus, the pipes are tested to ultimate strength in the three-edge bearing test.

12.10.4.3.2—Bedding Factor

The minimum compaction specified in Tables 12.10.2.1-1 and 12.10.2.1-2 shall be required by the contract document.

C12.10.4.3.2

The bedding factor is the ratio of the moment at service limit state to the moment applied in the three-edge bearing test. The standard supporting strength of buried pipe depends on the type of installation. The bedding factors given herein are based on the minimum levels of compaction indicated.

12.10.4.3.2a—Earth Load Bedding Factor for Circular Pipe

Earth load bedding factors, B_{FE}, for circular pipe are presented in Table 12.10.4.3.2a-1.

For pipe diameters, other than those listed in Table 12.10.4.3.2a-1, embankment condition bedding factors, B_{FE}, may be determined by interpolation.

C12.10.4.3.2a

The bedding factors for circular pipe were developed using the flexural moments produced by the Heger pressure distributions from Figure 12.10.2.1-1 for each of the standard embankment installations. The bedding factors for the embankment condition are conservative for each installation. This conservatism is based on assuming voids and poor compaction in the haunch areas and a hard bedding beneath the pipe in determining the moments, thrusts, and shears used to calculate the bedding factors. The modeling of the soil pressure distribution used to determine moments, thrusts, and shears is also conservative by 10–20 percent, compared with the actual SPIDA analysis.

Table 12.10.4.3.2a-1—Bedding Factors for Circular Pipe

Pipe Diameter, in.	Standard Installations			
	Type 1	Type 2	Type 3	Type 4
12	4.4	3.2	2.5	1.7
24	4.2	3.0	2.4	1.7
36	4.0	2.9	2.3	1.7
72	3.8	2.8	2.2	1.7
144	3.6	2.8	2.2	1.7

12.10.4.3.2b—Earth Load Bedding Factor for Arch and Elliptical Pipe

The bedding factor for installation of arch and elliptical pipe shall be taken as:

$$B_{FE} = \frac{C_A}{C_N - xq} \qquad (12.10.4.3.2b\text{-}1)$$

where:

C_A = constant corresponding to the shape of the pipe, as specified in Table 12.10.4.3.2b-1

C_N = parameter that is a function of the distribution of the vertical load and vertical reaction, as specified in Table 12.10.4.3.2b-1

x = parameter that is a function of the area of the vertical projection of the pipe over which lateral pressure is effective, as specified in Table 12.10.4.3.2b-1

q = ratio of the total lateral pressure to the total vertical fill load specified herein

Design values for C_A, C_N, and x are listed in Table 12.10.4.3.2b-1.

@Seismicisolation

Table 12.10.4.3.2b-1—Design Values of Parameters in Bedding Factor Equation

Pipe Shape	C_A	Installation Type	C_N	Projection Ratio, p	x
Horizontal Elliptical and Arch	1.337	2	0.630	0.9	0.421
				0.7	0.369
		3	0.763	0.5	0.268
				0.3	0.148
Vertical Elliptical	1.021	2	0.516	0.9	0.718
				0.7	0.639
		3	0.615	0.5	0.457
				0.3	0.238

The value of the parameter q is taken as:

- For arch and horizontal elliptical pipe:

$$q = 0.23\frac{p}{F_e}\left(1 + 0.35p\frac{B_c}{H}\right) \quad (12.10.4.3.2b\text{-}2)$$

- For vertical elliptical pipe:

$$q = 0.48\frac{p}{F_e}\left(1 + 0.73p\frac{B_c}{H}\right) \quad (12.10.4.3.2b\text{-}3)$$

where:

p = projection ratio, ratio of the vertical distance between the outside top of the pipe, and the ground of bedding surface to the outside vertical height of the pipe

12.10.4.3.2c—Live Load Bedding Factors

The bedding factor, B_{FLL}, for live load, W_L, for both circular pipe and arch and for elliptical pipe shall be taken as specified in Table 12.10.4.3.2c-1. For pipe diameters not listed in Table 12.10.4.3.2c-1, the bedding factor may be determined by interpolation.

Table 12.10.4.3.2c-1—Bedding Factors, B_{FLL}

Pipe diameter, in.	Fill Height, ft	
	< 2.0 ft	≥ 2.0 ft
12	3.2	2.4
18	3.2	2.4
24	3.2	2.4
30 and larger	2.2	2.2

C12.10.4.3.2c

The relatively large bending stiffness in the longitudinal direction of concrete pipe results in the distribution of the live load force along the length of the pipe. This ratio of distribution length to pipe diameter is higher in small diameter pipes designed by the Indirect Design Method. The bedding factor has been adjusted in Table 12.10.4.3.2c-1 to account for this higher distribution length.

12.10.4.4—Development of Quadrant Mat Reinforcement

12.10.4.4.1—Minimum Cage Reinforcement

In lieu of a detailed analysis, when quadrant mat reinforcement is used, the area of the main cage shall be

@Seismicisolation

no less than 25 percent of the area required at the point of maximum moment.

12.10.4.4.2—Development Length of Welded Wire Fabric

Unless modified herein, the provisions of Article 5.10.8.2.5 shall apply.

12.10.4.4.3—Development of Quadrant Mat Reinforcement Consisting of Welded Plain Wire Fabric

The embedment of the outermost longitudinals on each end of the circumferentials shall not be less than:

- the greater of 12 circumferential bar diameters or three-quarters of the wall thickness of the pipe beyond the point where the quadrant reinforcement is no longer required by the orientation angle, and
- a distance beyond the point of maximum flexural stress by the orientation angle plus the development length, ℓ_d, where ℓ_d is specified in Article 5.10.8.2.5.

The mat shall contain no less than two longitudinals at a distance 1.0 in. greater than that determined by the orientation angle from either side of the point requiring the maximum flexural reinforcement.

The point of embedment of the outermost longitudinals of the mat shall be at least a distance determined by the orientation angle past the point where the continuing reinforcement is no less than double the area required for flexure.

12.10.4.4.4—Development of Quadrant Mat Reinforcement Consisting of Deformed Bars, Deformed Wire, or Deformed Wire Fabric

When deformed bars, deformed wire, or deformed wire fabric is used, the circumferential bars in quadrant mat reinforcement shall satisfy the following requirements:

- Circumferentials shall extend past the point where they are no longer required by the orientation angle plus the greater of 12 wire or bar diameters or three-quarters of the wall thickness of the pipe,
- Circumferentials shall extend on either side of the point of maximum flexural stress not less than the orientation angle plus the development length, ℓ_{hd}, as required by Article 5.10.8.2.5a and modified by the applicable modification factor or factors, and
- Circumferentials shall extend at least a distance determined by the orientation angle past the point where the continuing reinforcement is no less than double the area required for flexure.

@Seismicisolation

12.10.5—Construction and Installation

The contract documents shall require that the construction and installation conform to Section 27, "Concrete Culverts," of the *AASHTO LRFD Bridge Construction Specifications*.

12.11—REINFORCED CONCRETE CAST-IN-PLACE AND PRECAST BOX CULVERTS AND REINFORCED CAST-IN-PLACE ARCHES

12.11.1—General

The provisions herein shall apply to the structural design of cast-in-place and precast reinforced concrete box culverts and cast-in-place reinforced concrete arches with the arch barrel monolithic with each footing.

Designs shall conform to applicable Articles of these Specifications, except as provided otherwise herein.

C12.11.1

These structures become part of a composite system comprised of the box or arch culvert structure and the soil envelope.

Precast reinforced concrete box culverts may be manufactured using conventional structural concrete and forms, or they may be machine made with dry concrete and vibrating form pipe making methods.

Standard dimensions for precast reinforced concrete box culverts are shown in AASHTO M 259 (ASTM C789) and M 273 (ASTM C850).

12.11.2—Loads and Live Load Distribution

12.11.2.1—General

Loads and load combinations specified in Table 3.4.1-1 shall apply. Live load shall be considered as specified in Article 3.6.1.3. Distribution of wheel loads and concentrated loads for culverts with less than 2.0 ft of fill shall be taken as specified in Article 4.6.2.10. For traffic traveling parallel to the span, box culverts shall be designed for a single loaded lane with the single lane multiple presence factor applied to the load. Requirements for bottom distribution reinforcement in top slabs of such culverts shall be as specified in Article 9.7.3.2 for mild steel reinforcement and Article 5.12.2.1 for prestressed reinforcement.

Distribution of wheel loads to culverts with 2.0 ft or more of cover shall be as specified in Article 3.6.1.2.6.

The dynamic load allowance for buried structures shall conform to Article 3.6.2.2.

For cast-in-place box culverts, and for precast box culverts with top slabs having span-to-thickness ratios (s/t) >18 or segment lengths <4.0 ft, edge beams shall be provided as specified in Article 4.6.2.1.4 as follows:

- At ends of culvert runs where wheel loads travel within 24.0 in. from the end of culvert,
- At expansion joints of cast-in-place culverts where wheel loads travel over or adjacent to the expansion joint.

C12.11.2.1

For the design of box culverts, three general load combinations envelope all controlling force effects for the Strength and Service limit states. These are:

- Maximum vertical, Maximum horizontal
- Maximum vertical, Minimum horizontal
- Minimum vertical, Maximum horizontal

Controlling force effects with maximum horizontal loads may occur with live load surcharge (LS) present or absent. Both situations should be investigated.

Research into live load distribution on box culverts (McGrath et al., 2004) has shown that design for a single loaded lane with a multiple presence factor of 1.2 on the live load and using the live load distribution widths in Article 4.6.2.10 will provide adequate design loading for multiple loaded lanes with multiple presence factors of 1.0 or less when the traffic direction is parallel to the span.

The edge beam provisions are only applicable for culverts with less than 2.0 ft of fill. Precast box culverts with span-to-thickness ratios (s/t) ≤18 have been shown to have significantly more strength than would be predicted by Article 5.7.3 (Garg et al., 2007) (Abolmaali and Garg, 2008). While the distribution of the load when it is applied to the edge of these structures would not be as large as would be predicted by Article 4.6.2.10, the residual strength in the structure more than compensates for the liberal load distribution.

12.11.2.2—Modification of Earth Loads for Soil–Structure Interaction

12.11.2.2.1—Embankment and Trench Conditions

In lieu of a more refined analysis, the total unfactored earth load, W_E, acting on the culvert may be taken as:

- For embankment installations:

$$W_E = F_e \gamma_s B_c H \qquad (12.11.2.2.1\text{-}1)$$

in which:

$$F_e = 1 + 0.20 \frac{H}{B_c} \qquad (12.11.2.2.1\text{-}2)$$

- For trench installations:

$$W_E = F_t \gamma_s B_c H \qquad (12.11.2.2.1\text{-}3)$$

in which:

$$F_t = \frac{C_d B_d^{\,2}}{H B_c} \leq F_e \qquad (12.11.2.2.1\text{-}4)$$

where:

W_E = total unfactored earth load (kip/ft)
B_c = outside width of culvert as specified in Figures 12.11.2.2.1-1 or 12.11.2.2.1-2, as appropriate (ft)
H = depth of backfill as specified in Figures 12.11.2.2.1-1 or 12.11.2.2.1-2 (ft)
F_e = soil–structure interaction factor for embankment installation specified herein
F_t = soil–structure interaction factor for trench installations specified herein
γ_s = unit weight of backfill (kcf)
B_d = horizontal width of trench as specified in Figure 12.11.2.2.1-2 (ft)
C_d = a coefficient specified in Figure 12.11.2.2.1-3

F_e shall not exceed 1.15 for installations with compacted fill along the sides of the box section, or 1.40 for installations with uncompacted fill along the sides of the box section.

For wide trench installations where the trench width exceeds the horizontal dimension of the culvert across the trench by more than 1.0 ft, F_t shall not exceed the value specified for an embankment installation.

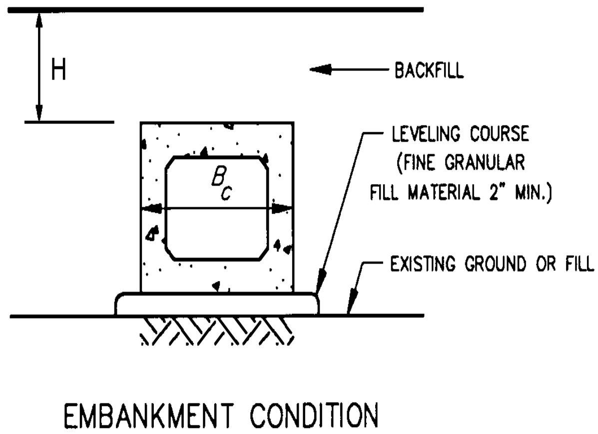


Figure 12.11.2.2.1-1—Embankment Condition—Precast Concrete Box Sections

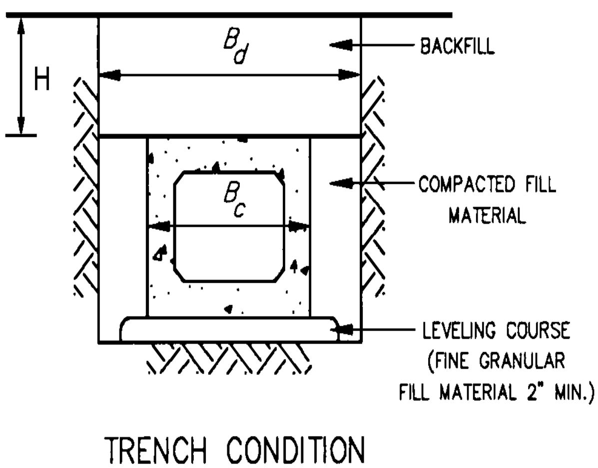


Figure 12.11.2.2.1-2—Trench Condition—Precast Concrete Box Sections

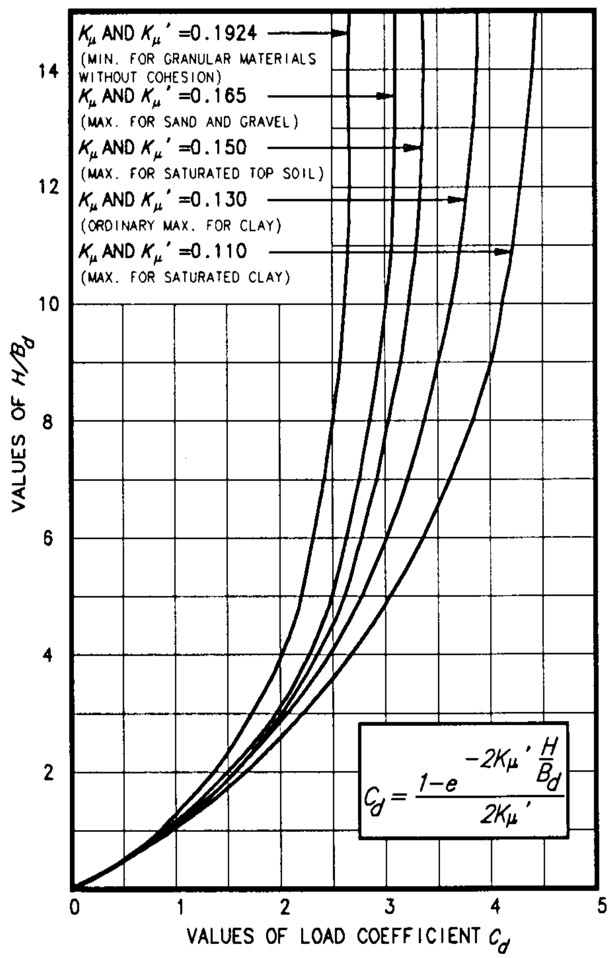


Figure 12.11.2.2.1-3—Coefficient, C_d, for Trench Installations

12.11.2.2.2—Other Installations

Methods of installation other than embankment or trench may be used to reduce the loads on the culvert, including partial positive projection, 0.0 projection, negative projection, induced trench, and jacked installations. The loads for such installations may be determined by accepted methods based on tests, soil–structure interaction analyses, or previous experience.

12.11.2.3—Distribution of Concentrated Loads to Bottom Slab of Box Culvert

The width of the top slab strip used for distribution of concentrated wheel loads, specified in Article 12.11.2, shall also be used for the determination of moments, shears, and thrusts in the side walls and the bottom slab.

C12.11.2.3

Restricting the live load distribution width for the bottom slab to the same width used for the top slab provides designs suitable for multiple loaded lanes, even though analysis is only completed for a single loaded lane (as discussed in Article C12.11.2.1).

While typical designs assume a uniform pressure distribution across the bottom slab, a refined analysis that considers the actual soil stiffness under box sections will result in pressure distributions that reduce bottom slab shear and moment forces (McGrath et al., 2004).

@Seismicisolation

Such an analysis requires knowledge of in-situ soil properties to select the appropriate stiffness for the supporting soil. A refined analysis taking this into account may be beneficial when analyzing existing culverts.

12.11.2.4—Distribution of Concentrated Loads in Skewed Box Culverts

Wheel distribution specified in Article 12.11.2.3 need not be corrected for skew effects.

12.11.3—Strength Limit State

The provisions of Article 12.10.4.2.4a may be applied to the flexural strength design of slabs and walls of reinforced concrete cast-in-place and precast box culverts and reinforced cast-in-place arches.

C12.11.3

Buried structures may be subject to high compressive thrust forces compared to most flexural members, and this thrust can result in a reduction in the required steel area. While influential in the reinforcement design, these thrust forces are not significant enough to warrant an individual analysis of compressive forces and flexural moments separately. Thus, Eq. 12.10.4.2.4a-1 may be used as a simplified, yet direct, method of determining the reinforcement areas for these structures.

12.11.4—Service Limit State

The provisions of Article 5.6.7 shall apply to crack width control in reinforced concrete cast-in-place and precast box culverts and reinforced cast-in-place arches.

C12.11.4

Although high compressive thrust forces are usually considered in strength design, they are often ignored in service limit state design. The following equations, derived from ACI SP-3 can be used to consider the effect of thrust on stresses at the service limit state:

$$f_s = \frac{M_s + N_s\left(d - \frac{h}{2}\right)}{(A_s jid)} \qquad \text{(C12.11.4-1)}$$

in which:

$$e = M_s / N_s + d - h/2$$

$$i = 1/\left(1 - jd/e\right)$$

$$j = 0.74 + 0.1\left(e/d\right) \leq 0.9$$

where:

M_s = flexural moment at service limit state (kip-in./ft)
N_s = axial thrust at service limit state (kip/ft)
d = distance from compression face to centroid of tension reinforcement (in.)
h = wall thickness (in.)
A_s = area of reinforcement per unit length (in.^{2}/ft)
f_s = reinforcement stress under service load condition (ksi)
e/d min = 1.15 (dim.)

i = coefficient for effect of axial force at service limit state, f_s
j = coefficient for moment arm at service limit state, f_s

12.11.5—Safety Against Structural Failure

12.11.5.1—General

All sections shall be designed for the applicable factored loads specified in Table 3.4.1-1 at the strength limit state, except as modified herein. Shear in culverts shall be investigated in conformance with Article 5.12.7.3.

12.11.5.2—Design Moment for Box Culverts

Where monolithic haunches inclined at 45 degrees are specified, negative reinforcement in walls and slabs may be proportioned based on the flexural moment at the intersection of the haunch and uniform depth member. Otherwise, the provisions of Section 5 shall apply.

12.11.5.3—Minimum Reinforcement

12.11.5.3.1—Cast-in-Place Structures

Reinforcement shall not be less than that specified in Article 5.6.3.3 at all cross-sections subject to flexural tension, including the inside face of walls. Shrinkage and temperature reinforcement shall be provided near the inside surfaces of walls and slabs in accordance with Article 5.10.6.

12.11.5.3.2—Precast Box Structures

At all box culvert cross-sections subjected to flexural tension, the minimum reinforcement area shall be not less than 0.002 times the gross concrete area.

For top slabs of box culverts having less than 2.0 ft of cover, the bottom longitudinal reinforcement area shall be the greater of the distribution reinforcement required per Article 9.7.3.2 or 0.002 times the gross concrete area.

For all other longitudinal reinforcement, the minimum longitudinal reinforcement area shall not be less than 0.03 in.^{2}/ft at each face.

Where the fabricated length exceeds 16.0 ft, the minimum reinforcement shall also meet the requirements of Article 5.10.6.

C12.11.5.3.2

The minimum requirement pertains to all reinforcement parallel to the span or rise on the inside and outside faces in walls and slabs of box culverts.

Designers suggested a minimum amount of reinforcement in consideration of handling, placement, backfilling, settlement, and long-term service life. For box culvert cross-sections subjected to flexural tension under typical in-service conditions, a ratio of reinforcement to gross concrete area of 0.002 is considered sufficient. Results from research performed on precast boxes at the University of Texas at Arlington showed no flexural tension in the direction perpendicular to the span of the box for wheel loads applied at the edge of a culvert segment. However, to protect the box culvert during the conditions listed above, some longitudinal steel is recommended. A minimum longitudinal reinforcement area of 0.03 in.^{2}/ft at each face is considered adequate to meet these needs. This minimum requirement may be at least partially satisfied by the transverse wires when wire mesh reinforcement is used.

@Seismicisolation

12.11.5.4—Minimum Cover for Precast Box Structures

The provisions of Article 5.10.1 shall apply.

12.11.6—Construction and Installation

The contract documents shall require that construction and installation conform to Section 27, "Concrete Culverts," of the *AASHTO LRFD Bridge Construction Specifications*.

12.12—THERMOPLASTIC PIPES

12.12.1—General

The provisions herein shall apply to the structural design of buried thermoplastic pipe with solid, corrugated, or profile wall, manufactured of PE, PP, or PVC.

C12.12.1

These structures become part of a composite system comprised of the plastic pipe and the soil envelope.

The following specifications are applicable:

For PE:

- Solid Wall—ASTM F714,
- Corrugated—AASHTO M 294, and
- Profile—ASTM F894.

For PP:

- Corrugated—AASHTO M 330

For PVC:

- Solid Wall—AASHTO M 278 and
- Profile—AASHTO M 304.

12.12.2—Service Limit States

12.12.2.1—General

The allowable maximum localized distortion of installed plastic pipe shall be limited based on the service requirements of the installation. The extreme fiber tensile strain shall not exceed the allowable long-term strain in Table 12.12.3.3-1. The net tension strain shall be the numerical difference between the bending tensile strain and ring compression strain.

C12.12.2.1

The allowable long-term strains should not be reached in pipes designed and constructed in accordance with this Specification. Deflections resulting from conditions imposed during pipe installation should also be considered in design.

12.12.2.2—Deflection Requirement

Total deflection, Δ_t, shall be less than the allowable deflection, Δ_A, as follows:

$$\Delta_t \leq \Delta_A \quad (12.12.2.2\text{-}1)$$

where:

Δ_t = total deflection of pipe expressed as a reduction of the vertical diameter taken as positive for

C12.12.2.2

Deflection is controlled through proper construction in the field, and construction contracts should place responsibility for control of deflections on the contractor. However, feasibility of a specified installation needs to be checked prior to writing the project specifications.

The construction specifications set the allowable deflection, Δ_A, for thermoplastic pipe at five percent as a generally appropriate limit. The Engineer may allow alternate deflection limits for specific projects if calculations using the design method in this section show reduction of the vertical diameter and expansion of horizontal diameter. (in.)

Δ_A = total allowable deflection of pipe, reduction of vertical diameter (in.)

Total deflection, calculated using Spangler's expression for predicting flexural deflection in combination with the expression for circumferential shortening, shall be determined as:

$$\Delta_t = \frac{K_B\left(D_L P_{sp} + C_L P_L\right) D_o}{1000\left(E_p I_p / R^3 + 0.061\, M_s\right)} + \varepsilon_{sc} D \qquad (12.12.2.2\text{-}2)$$

where:

ε_{sc} = service compressive strain as specified in Article 12.12.3.10.1c

D_L = deflection lag factor, a value of 1.5 is typical

K_B = bedding coefficient, a value of 0.10 is typical

P_{sp} = soil prism pressure evaluated at pipe springline as specified in Article 12.12.3.7 (psi)

C_L = live load coefficient as specified in Article 12.12.3.5

P_L = live load pressure as specified in Article 3.6.1.2.6

D_o = outside diameter of pipe (in.) as shown in Figure C12.12.2.2-1

E_p = short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi)

I_p = moment of inertia of pipe profile per unit length of pipe (in.^{4}/in.)

R = radius from center of pipe to centroid of pipe profile (in.) as shown in Figure C12.12.2.2-1

D = diameter to centroid of pipe profile (in.) as shown in Figure C12.12.2.2-1

M_s = secant constrained soil modulus, as specified in Article 12.12.3.5 (ksi)

that the pipe meets all of the strength-limit-state requirements.

Eq. 12.12.2.2-2 uses the constrained soil modulus, M_s, as the soil property. Note that the soil prism load is used as input, rather than the reduced load used to compute thrust.

This check should be completed to determine that the expected field deflection based on thrust and flexure is lower than the maximum allowable deflection for the project.

Thrust and hoop strain in the pipe wall are defined positive for compression.

There are no standard values for the deflection lag factor. Values from 1.0 to 6.0 have been recommended. The highest values are for installations with quality backfill and low initial deflections and do not generally control designs. A value of 1.5 provides some allowance for increase in deflection over time for installations with initial deflection levels of several percent.

The bedding coefficient, K_B, varies from 0.083 for full support to 0.110 for line support at the invert. Haunching is always specified to provide good support; however, it is still common to use a value of K_B equal to 0.10 to account for inconsistent haunch support.

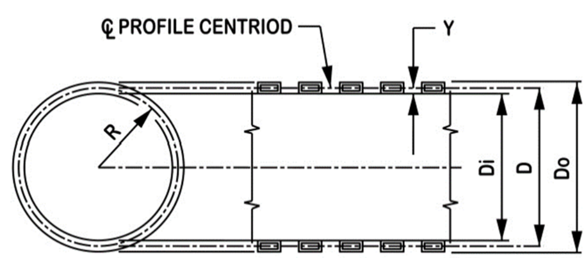


Figure C12.12.2.2-1—Schematic for Thermoplastic Pipe Terms

12.12.3—Safety Against Structural Failure

12.12.3.1—General

Buried thermoplastic culverts shall be investigated at the strength limit states for thrust, general and local buckling, and combined strain.

C12.12.3.1

Total compressive strain in a thermoplastic pipe can cause yielding or buckling, and total tensile strain can cause cracking.

12.12.3.2—Section Properties

Section properties for thermoplastic pipe, including wall area, moment of inertia, and profile geometry should

C12.12.3.2

Historically, AASHTO bridge specifications have contained minimum values for the moment of inertia and wall area of thermoplastic pipe; however, these values

@Seismicisolation

be determined from cut sections of pipe or obtained from the pipe manufacturer.

have been minimum values and are not meaningful for design. This is particularly so since provisions to evaluate local buckling were introduced in 2001. These provisions require detailed profile geometry that varies with manufacturer. Thus, there is no way to provide meaningful generic information on section properties. A convenient method for determining section properties for profile wall pipe is to make optical scans of pipe wall cross-sections and determine the properties with a computer drafting program.

12.12.3.3—Chemical and Mechanical Requirements

Mechanical properties for design shall be as specified in Table 12.12.3.3-1.

Except for buckling, the choice of either initial or long-term mechanical property requirements, as appropriate for a specific application, shall be determined by the Engineer. Investigation of general buckling shall be based on the value of modulus of elasticity that represents the design life of the project.

C12.12.3.3

Properties in Table 12.12.3.3-1 include "initial" and long-term values. No product standard requires determining the actual long-term properties; thus, there is some uncertainty in the actual values. However, pipe designed with the Table 12.12.3.3-1 values for 50-year modulus of elasticity have performed well, and the properties are assumed to be reasonably conservative. Estimated values for a modulus of elasticity for a 75-year design life have been estimated from relaxation tests on PVC, PP, and PE in parallel plate tests. The tests were conducted for over two years and show that the modulus of elasticity reduces approximately linearly with the logarithm of time. Further, with a log-linear extrapolation, the differences between 50-year and 75-year modulus values are very small. These values should be reasonably conservative, with the same reliability as the 50-year values. Pipe and thermoplastic resin suppliers should be asked to provide confirmation of long-term modulus values for any particular product. Values should meet or exceed those provided in Table 12.12.3.3-1. Where service life is in excess of 75 years, test data may be used for the desired life.

The service long-term tension strain limit and the factored compression strain limit in Table 12.12.3.3-1 need to be multiplied by the appropriate resistance factors to obtain the strain limits.

@Seismicisolation

Table 12.12.3.3-1—Mechanical Properties of Thermoplastic Pipe

Type of Pipe	Minimum Cell Class	Service Long-Term Tension Strain Limit, ε_{yt} (%)	Factored Compr. Strain Limit, ε_{yc} (%)	Initial F_u min (ksi)	Initial E min (ksi)	50-yr F_u min (ksi)	50-yr E min (ksi)	75-yr F_u min (ksi)	75-yr E min (ksi)
Solid Wall PE Pipe – ASTM F714	ASTM D3350, 335434C	5.0	4.1	3.0	110.0	1.44	22	1.40	21
Corrugated PE Pipe – AASHTO M 294	ASTM D3350, 435400C	5.0	4.1	3.0	110.0	0.90	22	0.90	21
Profile PE Pipe – ASTM F894	ASTM D3350, 334433C	5.0	4.1	3.0	80.0	1.12	20	1.10	19
	ASTM D3350, 335434C	5.0	4.1	3.0	110.0	1.44	22	1.40	21
Solid Wall PVC Pipe – AASHTO M 278, ASTM F679	ASTM D1784, 12454C	5.0	2.6	7.0	400.0	3.70	140	3.60	137
	ASTM D1784, 12364C	3.5	2.6	6.0	440.0	2.60	158	2.50	156
Profile PVC Pipe – AASHTO M 304	ASTM D1784, 12454C	5.0	2.6	7.0	400.0	3.70	140	3.60	137
	ASTM D1784, 12364C	3.5	2.6	6.0	440.0	2.60	158	2.50	156
Corrugated PP Pipe – AASHTO M 330	Requirements in M 330	2.5	3.7	3.5	175	1.0	29	1.0	28

12.12.3.4—Thrust

Loads on buried thermoplastic pipe shall be based on the soil prism load, modified as necessary to consider the effects of pipe–soil interaction. Calculations shall consider the duration of a load when selecting pipe properties to be used in design. Live loads need not be considered for the long-term loading condition.

C12.12.3.4

Because of the time-dependent nature of thermoplastic pipe properties, the load will vary with time.

Time of loading is an important consideration for some types of thermoplastic pipe. Live loads and occasional flood conditions are normally considered short-term loads. Earth loads or permanent high groundwater are normally considered long-term loads.

@Seismicisolation

12.12.3.5—Factored and Service Loads

The factored thrust shall be taken as:

$$T_u = \left[\eta_{EV}\left(\gamma_{EV} K_{\gamma E} K_2 VAF\, P_{sp} + \gamma_{WA} P_w\right) + \eta_{LL}\gamma_{LL} P_L\ C_L F_1 F_2\right]\frac{D_o}{2} \quad (12.12.3.5\text{-}1)$$

The service thrust shall be taken as:

$$T_s = \left[K_2 VAF\, P_{sp} + P_L\ C_L\ F_1 F_2 + P_w\right]\frac{D_o}{2} \quad (12.12.3.5\text{-}2)$$

in which:

$$VAF = 0.76 - 0.71\left(\frac{S_H - 1.17}{S_H + 2.92}\right) \quad (12.12.3.5\text{-}3)$$

$$S_H = \frac{\phi_s M_s R}{E_p A_g} \quad (12.12.3.5\text{-}4)$$

$$C_L = \frac{\ell_w}{D_o} \leq 1.0 \quad (12.12.3.5\text{-}5)$$

$$F_1 = \frac{0.75 D_o}{\ell_w} \geq F_{min} \quad (12.12.3.5\text{-}6)$$

$$F_{min} = \frac{15}{D_i} \geq 1 \quad (12.12.3.5\text{-}7)$$

$$F_2 = \frac{0.95}{1 + 0.6 S_H} \quad (12.12.3.5\text{-}8)$$

where:

T_u = factored thrust per unit length (lb/in.)

T_s = service thrust per unit length (lb/in.)

P_L = live load pressure as specified in Article 3.6.1.2.6

ℓ_w = live load distribution width in the circumferential direction as specified in Article 3.6.1.2.6

$K_{\gamma E}$ = installation factor typically taken as 1.5 to provide traditional safety. Use of a value less than 1.5 requires additional monitoring of the installation during construction and provisions for such monitoring shall be provided on the contract documents.

C12.12.3.5

For η factors, refer to Article 12.5.4 regarding assumptions about redundancy for earth loads and live loads.

The factor K_2 is introduced to consider variation in thrust around the circumference, which is necessary when combining thrust with moment or thrust due to earth and live load under shallow fill. K_2 is set at 1.0 to determine thrust at the springline and 0.6 to determine thrust at the crown. The term P_L is also modified for this reason in later sections.

Figure C3.11.3-1 shows the effect of groundwater on the earth pressure. P_{sp} does not include the hydrostatic pressure. P_{sp} is the pressure due to the weight of soil above the pipe and should be calculated based on the wet density for soil above the water table and based on the buoyant density for soil below the water table. See Table 3.5.1-1 for common unit weights.

In computing ℓ_w, add axle spacing (and increase total live load) if depth is sufficient for axle loads to interact.

The factor $K_{\gamma E}$ is introduced to provide the same safety level as traditionally used for thermoplastic culverts. Designers may consider using values of $K_{\gamma E}$ as low as 1.0 provided that procedures are implemented to ensure compliance with construction specifications. For culvert designs completed with an installation factor less than 1.5, the designer is required to specify additional minimum performance measures such as testing, monitoring, construction controls, and backfill requirements including active monitoring of the backfill gradation and compaction (see Article 30.7.4 of the *AASHTO LRFD Bridge Construction Specifications)*. The construction controls include deflection measurements and shall require the Contractor to submit and get approval from the Owner's Engineer for his/her construction plan to be used to achieve the more stringent

@Seismicisolation

performance measures which allowed for the use of a smaller installation factor in the design. Backfill placement and monitoring shall be done at levels along the side of the culvert and includes measurement of change in vertical pipe diameter when the backfill reaches the top of the pipe. As the backfill nears the top of pipe the vertical pipe diameter should be greater than the vertical diameter prior to backfilling, but not more than three percent greater than the vertical diameter prior to backfilling. When construction controls are implemented and the project specifications call for the total length of the pipe to be inspected and the vertical and horizontal deflection be measured no sooner than 30 days after backfill/embankment is completed, a value of $K_{\gamma E}$ =1.0 may be used for design.

K_2 = coefficient to account for variation of thrust around the circumference
= 1.0 for thrust at the springline
= 0.6 for thrust at the crown

VAF = vertical arching factor

The use of the vertical arching factor is based on the behavior, demonstrated by Burns and Richard (1964), that pipe with high hoop-stiffness ratios (S_H, ratio of soil stiffness to pipe hoop stiffness) carry substantially less load than the weight of the prism of soil directly over the pipe. This behavior was demonstrated experimentally by Hashash and Selig (1990) and analytically by Moore (1995). McGrath (1999) developed the simplified form of the equation presented in this Section.

The *VAF* approach is only developed for the embankment load case. No guidance is currently available to predict the reduced loads on pipe in trench conditions. The only trench load theory proposed for flexible pipe was that by Spangler, which does not have good guidance on selection of input parameters. It is conservative to use the *VAF* approach as presented for embankments.

S_H = hoop stiffness factor

If evaluating the short-term load condition, then use the initial modulus of elasticity to compute S_H. Similarly, if evaluating the long-term loading condition, then use the long-term modulus of elasticity to compute S_H.

P_w = hydrostatic water pressure at the springline of the pipe (psi)

C_L = live load distribution coefficient

η_{EV} = load modifier as specified in Article 1.3.2, as it applies to vertical earth loads on culverts

γ_{EV} = load factor for vertical pressure from dead load of earth fill, as specified in Article 3.4.1

P_{sp} = soil prism pressure (EV), evaluated at pipe springline (psi)

γ_{WA} = load factor for hydrostatic pressure, as specified in Article 3.4.1

η_{LL} = load modifier as specified in Article 1.3.2, as it applies to live loads on culverts

γ_{LL} = load factor for live load, as specified in Article 3.4.1

P_L = live load pressure (LL) with dynamic load allowance (psi)

ϕ_s = resistance factor for soil stiffness

M_s = secant constrained soil modulus as specified in Table 12.12.3.5-1 or Table 12.12.3.5-3 (ksi)

R = radius from center of pipe to centroid of pipe profile (in.)

E_p = short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi)

A_g = gross area of pipe wall per unit length of pipe (in.^{2}/in.)

D_o = outside diameter of pipe (in.)

$LLDF$ = live load distribution factor through earth fills in Article 3.6.1.2.6

In the absence of site-specific data, the secant constrained soil modulus, M_s, may be selected from Table 12.12.3.5-1 based on the backfill type and density and the geostatic earth pressure, P_{sp}. Linear interpolation between soil stress levels may be used for the determination of M_s. If crushed stone backfill is used, the secant constrained modulus, M_s, may be selected from Table 12.12.3.5-3.

For culverts in embankment or wide trench installations under depths of fill up to 10.0 ft, the secant constrained soil modulus, M_s, selected from Table 12.12.3.5-1 or Table 12.12.3.5-3 shall be based on the backfill type and density conditions for a width of one-half diameter on each side of the culvert, but never less than 18.0 in. on each side of the culvert. For culverts under depths of fill greater than 10.0 ft, the secant constrained soil modulus, M_s, selected shall be based on the backfill type and density conditions for a width of one diameter on each side of the culvert.

The term ϕ_s appears in Eq. 12.12.3.5-4 to account for variability in backfill compaction. A lower level of compaction increases the applied thrust force on the pipe.

For selecting values of the constrained soil modulus, M_s, prior editions of these Specifications contained the commentary "Suggested practice is to design for a standard Proctor backfill density five percent less than specified by the contract documents." This statement is not considered necessary with the addition of post-construction inspection guidelines to the *AASHTO LRFD Bridge Construction Specifications*, which should provide reasonable assurance that the design condition is achieved.

For culverts in trench installations under depths of fill greater than 10.0 ft, evaluation of the values of M_s for in-situ soil for a width one diameter either side of the pipe is not necessary, provided the in-situ soil has adequate vertical and lateral stiffness. Stable trench walls during the excavation process are predictive of adequate vertical and lateral stiffness.

Installation in narrow trenches reduces the vertical load, provided vertical stiffness of the soil is adequate to carry the load that is distributed around the pipe due to arching, as represented by the vertical arching factor (VAF) in the design method and adequate space is preserved at the side of the pipe to place and compact backfill. The minimum trench widths provided in the *AASHTO LRFD Bridge Construction Specifications* are set to provide adequate space. Narrow trenches yield a desirable level of conservatism, since the transfer of the load to in-situ trench wall is not considered in flexible pipe design.

If the structural backfill material is crushed stone, then the secant constrained soil modulus, M_s, values shown in Table 12.12.3.5-3 may be used. If an aggregate other than granite, limestone, or quartz is used, or if the aggregate type is unknown, the constrained soil modulus values for crushed limestone may be conservatively applied. These constrained modulus values are independent of burial depth. While it is not common practice to monitor density of crushed stone backfills, experience has found that a modest compaction effort improves culvert performance and allows the use of the compacted values.

The width of structural backfill is an important consideration when the in situ soil in the trench wall or the embankment fill at the side of the structural backfill is soft. Currently, only the American Water Works Association (AWWA)'s *Manual of Water Supply Practices—M45* addresses this issue.

The constrained modulus is the slope of the secant from the origin of the curve to a point on the curve corresponding to the soil prism pressure, P_{sp}, Figure C12.12.3.5-1.

@Seismicisolation

The constrained modulus may also be determined experimentally using the stress–strain curve resulting from a uniaxial strain test on a sample of soil compacted to the field-specified density.

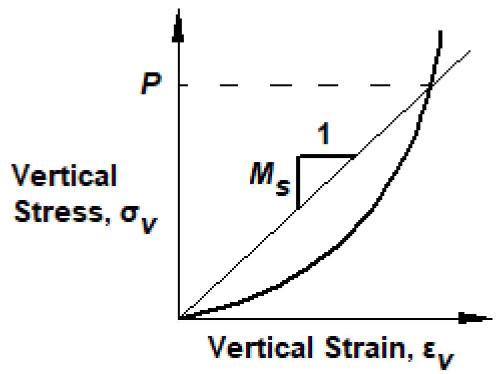


Figure C12.12.3.5-1—Schematic One-Dimensional Stress–Strain Curve of Soil Backfill

Table 12.12.3.5-1—M_s Based on Soil Type and Compaction Condition

P_{sp} Stress Level (psi)	Sn-100 (ksi)	Sn-95 (ksi)	Sn-90 (ksi)	Sn-85 (ksi)
1.0	2.350	2.000	1.275	0.470
5.0	3.450	2.600	1.500	0.520
10.0	4.200	3.000	1.625	0.570
20.0	5.500	3.450	1.800	0.650
40.0	7.500	4.250	2.100	0.825
60.0	9.300	5.000	2.500	1.000
P_{sp} Stress Level (psi)		Si-95 (ksi)	Si-90 (ksi)	Si-85 (ksi)
1.0		1.415	0.670	0.360
5.0		1.670	0.740	0.390
10.0		1.770	0.750	0.400
20.0		1.880	0.790	0.430
40.0		2.090	0.900	0.510
60.0				
P_{sp} Stress Level (psi)		Cl-95 (ksi)	Cl-90 (ksi)	Cl-85 (ksi)
1.0		0.530	0.255	0.130
5.0		0.625	0.320	0.175
10.0		0.690	0.355	0.200
20.0		0.740	0.395	0.230
40.0		0.815	0.460	0.285
60.0		0.895	0.525	0.345

1. The soil types are defined by a two-letter designation that indicates general soil classification, Sn for sands and gravels, Si for silts, and Cl for clays. Specific soil groups that fall into these categories, based on ASTM D2487 and AASHTO M 145, are listed in Table 12.12.3.5-2.
2. The numerical suffix to the soil type indicates the compaction level of the soil as a percentage of maximum dry density determined in accordance with AASHTO T 99.

@Seismicisolation

Table 12.12.3.5-2—Equivalent ASTM and AASHTO Soil Classifications

Basic Soil Type (1)	ASTM D2487	AASHTO M 145
Sn (Gravelly sand, SW)	SW, SP (2) GW, GP sands and gravels with 12% or less fines	A-1, A-3 (2)
Si (Sandy silt, ML)	GM, SM, ML also GC and SC with less than 20% passing a No. 200 sieve	A-2-4, A-2-5, A-4
Cl (Silty clay, CL)	CL, MH, GC, SC also GC and SC with more than 20% passing a No. 200 sieve	A-2-6, A-2-7, A-5, A-6

1. The soil classification listed in parentheses is the type that was tested to develop the constrained soil modulus values in Table 12.12.3.5-1. The correlations to other soil types are approximate.
2. Uniformly graded materials with an average particle size smaller than a No. 40 sieve shall not be used as backfill for thermoplastic culverts unless specifically allowed in the contract documents and special precautions are taken to control moisture content and monitor compaction levels.

Table 12.12.3.5-3—Constrained Modulus, M_s, Values for Crushed Stone Backfill Materials

Aggregate	Maximum Particle Size (in.)	Constrained Modulus, M_s (ksi)	
		Dumped	Compacted
Granite	0.75	7.000	8.500
	1.50	3.500	5.000
Limestone	0.75	3.500	5.500
Quartz	0.75	5.500	7.500

12.12.3.6—Handling and Installation Requirements

The flexibility factor, *FF*, in./kip shall be taken as:

$$FF = \frac{S^2}{EI} \quad (12.12.3.6\text{-}1)$$

where:

I = moment of inertia (in.^{4}/in.)
E = initial modulus of elasticity (ksi)
S = diameter of pipe (in.)

The flexibility factor, *FF*, shall be limited as specified in Article 12.5.6.3.

12.12.3.7—Soil Prism

The soil prism load shall be calculated as a pressure representing the weight of soil above the pipe springline. The pressure shall be calculated for three conditions:

C12.12.3.7

The soil prism load and vertical arching factor, *VAF*, serve as a common reference for the load on all types of pipe.

The soil prism calculation needs to consider the unit weight of the backfill over the pipe. Use the wet unit weight above the water table and the buoyant unit weight below the water table. In cases where the water table fluctuates, multiple conditions may need to be evaluated.

- If the water table is above the top of the pipe and at or above the ground surface:

$$P_{sp} = \frac{\left(H + 0.11\frac{D_o}{12}\right)\gamma_b}{144} \quad (12.12.3.7\text{-}1)$$

- If the water table is above the top of the pipe and below the ground surface:

$$P_{sp} = \frac{1}{144}\left[\begin{array}{c}\left[\left(H_W - \frac{D_o}{24}\right) + 0.11\frac{D_o}{12}\right]\gamma_b + \\ \left[H - \left(H_w - \frac{D_o}{24}\right)\right]\gamma_s\end{array}\right] \quad (12.12.3.7\text{-}2)$$

- If the water table is below the top of the pipe:

$$P_{sp} = \frac{\left(H + 0.11\frac{D_o}{12}\right)\gamma_s}{144} \quad (12.12.3.7\text{-}3)$$

where:

P_{sp} = soil prism pressure (EV), evaluated at pipe springline (psi)
D_o = outside diameter of pipe (in.)
γ_b = unit weight of buoyant soil (lb/ft^3)
H = depth of fill over top of pipe (ft)
H_w = depth of water table above springline of pipe (ft)
γ_s = wet unit weight of soil (lb/ft^3)

Figure C3.11.3-1 shows the effect of groundwater on the earth pressure. See Table 3.5.1-1 for common unit weights.

12.12.3.8—Hydrostatic Pressure

The pressure due to ground water shall be calculated as:

$$P_w = \frac{\gamma_w\, K_{wa}\, H_w}{144} \quad (12.12.3.8\text{-}1)$$

where:

P_w = hydrostatic water pressure at the springline of the pipe (psi)
γ_w = unit weight of water (lb/ft^3)
K_a = factor for uncertainty in level of groundwater table

C12.12.3.8

Hydrostatic loading due to external water pressure should be calculated in all cases where water table may be above the pipe springline at any time. This load contributes to hoop thrust but does not affect deflection.

There is often uncertainty in the level of the groundwater table and its annual variations. The designer may use the factor K_{wa} with values up to 1.3 to account for this uncertainty or may select conservative values of H_w with a lower value of K_{wa} but not less than 1.

12.12.3.9—Live Load

The provisions of Article 12.6.1 shall apply.

12.12.3.10—Wall Resistance

12.12.3.10.1—Resistance to Axial Thrust

12.12.3.10.1a—General

Elements of profile wall pipe shall be designed to resist local buckling. To determine local buckling resistance, profile-wall pipe geometry shall be idealized as specified herein and an effective area determined in accordance with the following provisions.

12.12.3.10.1b—Local Buckling Effective Area

For the determination of buckling resistance, profile wall pipe shall be idealized as straight elements. Each element shall be assigned a width based on the clear distance between the adjoining elements and a thickness based on the thickness at the center of the element. The idealization of a typical corrugated profile should be based on the approximation in Figure 12.12.3.10.1b-1.

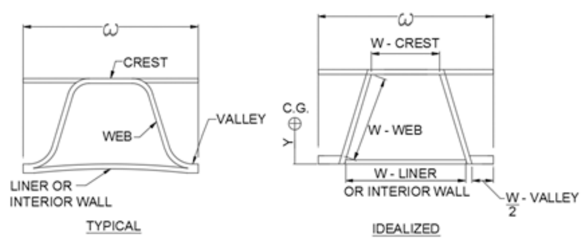


Figure 12.12.3.10.1b-1—Typical and Idealized Cross-section of Profile Wall Pipe

C12.12.3.10.1b

To complete the local buckling calculations, the profile is idealized into a group of rectangular elements. To complete the idealization, it should include:

- The actual total area.
- If the crest element is curved, it should be idealized at the centroid of the curvature. The idealized element need not touch the idealized webs.

See McGrath et al. (2009) for guidance on other profile types.

To evaluate the resistance to axial thrust, the area of the profile shall be reduced to an effective area, A_{eff}, for local buckling effects. The effective area of the profile shall be determined by subtracting the ineffective area of each element from the gross section area, as:

$$A_{eff} = A_g - \frac{\sum (w - b_e)t}{\omega} \qquad (12.12.3.10.1b\text{-}1)$$

in which:

$$b_e = \rho w \qquad (12.12.3.10.1b\text{-}2)$$

$$\rho = \frac{\left(1 - \frac{0.22}{\lambda}\right)}{\lambda} \qquad (12.12.3.10.1b\text{-}3)$$

The resistance to local buckling is based on the effective width concept used by the cold formed steel industry. This theory assumes that even though buckling is initiated in the center of a plate element, the element still has substantial post-buckling strength at the edges where the element is supported. This concept is demonstrated in Figure C12.12.3.10.1b-1.

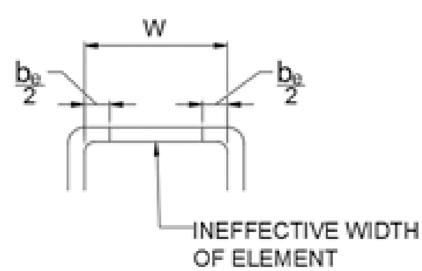


Figure C12.12.3.10.1b-1—Effective Width Concept

@Seismicisolation

$$\lambda = \left(\frac{w}{t}\right)\sqrt{\frac{\varepsilon_{yc}}{k}} \geq 0.673 \quad (12.12.3.10.1b\text{-}4)$$

The local buckling evaluation reduces the capacity of pipe wall sections with high ratios of width to thickness.

The calculations in Eqs. 12.12.3.10.1b-1 to 12.12.3.10.1b-4 must be repeated for each element in the idealized profile.

where:

A_{eff} = effective area of pipe wall per unit length of pipe (in.^{2}/in.)
b_e = element effective width (in.)
ρ = effective width factor
λ = slenderness factor
ω = spacing of corrugation (in.) as specified in Figure 12.12.3.10.1b-1
ε_{yc} = factored compressive strain limit as specified in Table 12.12.3.3-1
A_g = gross area of pipe wall per unit length of pipe (in.^{2}/in.)
t = thickness of element (in.)
w = total clear width of element between supporting elements (in.)
k = plate buckling coefficient, k=4 for supported elements, $k = 0.43$ for unsupported elements, such as free standing ribs

The plate buckling coefficient is analogous to the effective length factor, k, in column buckling.

As an alternate to determining the effective area by the calculation procedure presented above, the results of the stub compression test, AASHTO T 341, may be used, in which case the effective area, A_{eff}, shall satisfy:

The stub compression test has been incorporated as a requirement into AASHTO product standards M 294, M 330, and M 304. The test data should be readily available from manufacturers and quality control tests.

$$A_{eff} = \frac{P_{st}K_t}{F_u} \leq A_g \quad (12.12.3.10.1b\text{-}5)$$

in which:

P_{st} = stub compression capacity from T 341 (kip/in.)
K_t = time factor as specified in Table 12.12.3.10.1b-1
F_u = material yield strength for design load duration (ksi)

Table 12.12.3.10.1b-1—Time Factor

Time Period	PE	PP	PVC
Initial	0.9	0.9	0.95
50 yr	0.3	0.3	0.6
75 yr (est.)	0.25	0.25	0.5

12.12.3.10.1c—Compression Strain

The factored compressive strain due to factored thrust, ε_{uc}, and the service compressive strain due to service thrust, ε_{sc}, shall be taken as:

$$\varepsilon_{uc} = \frac{T_u}{1{,}000\left(A_{eff}E_p\right)} \quad (12.12.3.10.1c\text{-}1)$$

$$\varepsilon_{sc} = \frac{T_s}{1{,}000\left(A_{eff} E_p\right)} \quad (12.12.3.10.1c\text{-}2)$$

where:

ε_{uc} = factored compressive strain due to thrust
ε_{sc} = service compressive strain due to thrust
T_u = factored thrust per unit length (lb/in.)
T_s = service thrust per unit length (lb/in.)
A_{eff} = effective area of pipe wall per unit length of pipe (in.^{2}/in.)
E_p = short-term modulus for short-term loading or long-term modulus of pipe material for long-term loading as specified in Table 12.12.3.3-1 (ksi)

12.12.3.10.1d—Thrust Strain Limits

The factored compression strain due to thrust, ε_{uc}, shall satisfy:

$$\varepsilon_{uc} \leq \phi_T \varepsilon_{yc} \quad (12.12.3.10.1d\text{-}1)$$

where:

ε_{uc} = factored compressive strain due to thrust
ϕ_T = resistance factor for thrust effects
ε_{yc} = factored compression strain limit of the pipe wall material as specified in Table 12.12.3.3-1

12.12.3.10.1e—General Buckling Strain Limits

The factored compression strain due to thrust, incorporating local buckling effects, ε_{uc}, shall satisfy:

$$\varepsilon_{uc} \leq \phi_{bck} \varepsilon_{bck} \quad (12.12.3.10.1e\text{-}1)$$

The nominal strain capacity for general buckling of the pipe shall be determined as:

$$\varepsilon_{bck} = \frac{1.2 C_n \left(E_p I_p\right)^{\frac{1}{3}}}{A_{eff} E_p} \left[\frac{\phi_s M_s \left(1-2\nu\right)}{\left(1-\nu\right)^2}\right]^{\frac{2}{3}} R_h \quad (12.12.3.10.1e\text{-}2)$$

in which:

$$R_h = \frac{11.4}{11 + \frac{D}{12H}} \quad (12.12.3.10.1e\text{-}3)$$

where:

ε_{uc} = factored compressive strain due to thrust
ϕ_{bck} = resistance factor for global buckling
ε_{bck} = nominal strain capacity for general buckling

C12.12.3.10.1e

The equations for global resistance presented here are a conservative simplification of the continuum buckling theory presented by Moore (1990). Detailed analysis using the full theory may be applied in lieu of the calculations in this section.

The term ϕ_s appears in this expression for ε_{bck} to account for backfills compacted to levels below that specified in the design. Lower levels of compaction increases the thrust force in the pipe.

For designs meeting all other requirements of these specifications and the *AASHTO LRFD Bridge Construction Specifications*, the correction for backfill soil geometry, R_h, is equal to value at left.

R_h = correction factor for backfill soil geometry

The complete theory proposed by Moore (1990) provides variations in R_h that consider nonuniform backfill support. In the extreme case where the width of structural backfill at the side of the culvert is 0.1 times the span and the modulus of the soil outside of the structural backfill is 0.1 times the modulus of the backfill, then:

$$R_h = \frac{20}{56 + \frac{D}{12H}} \quad \text{(C12.12.3.10.1e-1)}$$

C_n = calibration factor to account for nonlinear effects = 0.55
E_p = short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi)
I_p = moment of inertia of pipe profile per unit length of pipe (in.^{4}/in.)
A_{eff} = effective area of pipe profile per unit length of pipe (in.^{2}/in.)
ϕ_s = resistance factor for soil pressure
M_s = secant constrained soil modulus as specified in Table 12.12.3.5-1 (ksi)
ν = Poisson's ratio of soil

Poisson's ratio is used to convert the constrained modulus of elasticity to the plane strain modulus. Values for Poisson's ratio of soils are provided in many geotechnical references. One reference is Selig (1990).

D = diameter to centroid of pipe profile (in.)
H = depth of fill over top of pipe (ft)

12.12.3.10.2—Bending and Thrust Strain Limits

12.12.3.10.2a—General

To ensure adequate flexural capacity the combined strain at the extreme fibers of the pipe profile must be evaluated at the allowable deflection limits against the limiting strain values.

12.12.3.10.2b—Combined Strain

If summation of axial strain, ε_{uc}, and bending strain, ε_f, produces tensile strain in the pipe wall, the combined strain at the extreme fiber where flexure causes tension shall satisfy:

$$\varepsilon_f - \varepsilon_{uc} < \phi_f \varepsilon_{yt} \quad \text{(12.12.3.10.2b-1)}$$

The combined strain at the extreme fiber where flexure causes compression shall satisfy:

$$\varepsilon_f + \varepsilon_{uc} < \phi_T \left(1.5\varepsilon_{yc}\right) \quad \text{(12.12.3.10.2b-2)}$$

C12.12.3.10.2b

The criteria for combined compressive strain are based on limiting local buckling. A higher strain limit is allowed for combined strains because under bending, the web elements have a low stress near the centroid of the element and are thus unlikely to buckle. Thus the unbuckled web elements increase the stability of the crest and valley elements.

The strain limit for combined compression strain is 50 percent higher than that for hoop compression alone because the web elements, which experience low strains due to bending, are not likely to buckle, thus increasing the stability of elements near the crest and valley. While this behavior would be more accurately modeled as an where:

ε_f = factored strain due to flexure
ε_{uc} = factored compressive strain due to thrust
ε_{yt} = service long-term tension strain limit of the pipe wall material as specified in Table 12.12.3.3-1
ϕ_f = resistance factor for flexure
ϕ_T = resistance factor for thrust
ε_{yc} = factored compression strain limit of the pipe wall material as specified in Table 12.12.3.3-1

In the absence of a more-detailed analysis, the flexural strain may be determined based on the empirical relationship between strain and deflection as:

$$\varepsilon_f = \gamma_{EV} D_f \left(\frac{c}{R}\right)\left(\frac{\Delta_f}{D}\right) \quad (12.12.3.10.2b\text{-}3)$$

in which:

$$\Delta_f = \Delta_A - \varepsilon_{sc} D \quad (12.12.3.10.2b\text{-}4)$$

where:

ε_f = factored strain due to flexure
ε_{sc} = service compression strain due to thrust
Δ_f = reduction of vertical diameter due to flexure (in.)
γ_{EV} = load factor for vertical pressure from dead load of earth fill, as specified in Article 3.4.1
D_f = shape factor as specified in Table 12.12.3.10.2b-1. The shape factors for corrugated PE and PP pipes can be reduced by 1.0 from the table values to account for the effect of the low hoop stiffness ratio.
c = the larger of the distance from neutral axis of profile to the extreme innermost or outermost fiber (in.)
R = radius from center of pipe to centroid of pipe profile (in.)
D = diameter to centroid of pipe profile (in.)
Δ_A = total allowable deflection of pipe, reduction of vertical internal diameter (in.)

increase in the k factor of Eq. 12.12.3.10.1b-4, the increase in the limiting strain is considered adequate for this simplified design method.

For thrust capacity, the section is limited by consideration of hoop compression capacity alone. The check of combined compression strain, hoop plus bending, is used to limit the allowable pipe deflection.

Elements subjected primarily to bending (such as a web element in Figure 12.12.3.10.1b-1 when the pipe is deflected) are not highly stressed near the centroid, where buckling initiates, and theoretical k factors for plates in bending are greater than 20. To simplify the analysis for combined bending and thrust, elements, such as the web whose centroid is within $c/3$ of the centroid of the entire profile wall, may be analyzed only for the effect of hoop compression strains. That is, increases in strain due to bending may be ignored.

Past practice has used tensile strain limits specified in Table 12.12.3.3-1, with no guidance on ultimate strain limits. For purposes of design calculations, assume that ultimate tensile strain capacity is 50 percent greater than the service capacities provided in Table 12.12.3.3-1.

A higher strain limit is allowed under combined bending and compression. This increase is permitted because the web element under flexure has a low stress at the center of the element, reducing the likelihood of buckling, and allowing it to provide more stability to the crest and valley elements.

Flexural strains are always taken as positive. The service compressive strain is used for determination of the factored strain due to flexure instead of the factored compressive strain. The use of the factored compressive strain would result in an unconservative flexural strain demand.

The empirical shape factor is used in the design of fiberglass pipe and is presented in AWWA *Manual of Water Supply Practices—M45*. It demonstrates that bending strains are highest in low stiffness pipe backfilled in soils that require substantial compactive effort (silts and clays), and is lowest in high stiffness pipe backfilled in soils that require little compactive effort (sands and gravels).

Table 12.12.3.10.2b-1 does not cover all possible backfills and density levels. Designers should interpolate or extrapolate the Table as necessary for specific projects.

More detailed analyses must consider the likelihood of inconsistent soil support to the pipe in the haunch zone, and of local deformations during placement and compaction of backfill.

Bending strains typically cannot be accurately predicted during design due to variations in backfill materials and compactive effort used during installation. Installation deflection limits are specified in the construction specifications to assure that design parameters are not exceeded.

@Seismicisolation

The deflection design limit is five percent reduction of the vertical diameter as specified in the construction specification. The pipe must be designed to permit this deflection, unless extraordinary measures are specified in contract documents to minimize compactive effort and to control deflections.

The *AASHTO LRFD Bridge Construction Specifications* currently restrict the allowable total vertical deflection to five percent.

Table 12.12.3.10.2b-1—Shape Factors, D_f, based on Pipe Stiffness, Backfill, and Compaction Level

Pipe Stiffness (F/Δ_y, ksi) $= EI / 0.149\ R^3$	Pipe Zone Embedment Material and Compaction Level			
	Gravel (1)		Sand (2)	
	Dumped to Slight (3)	Moderate to High (4)	Dumped to Slight (3)	Moderate to High (4)
0.009	5.5	7.0	6.0	8.0
0.018	4.5	5.5	5.0	6.5
0.036	3.8	4.5	4.0	5.5
0.072	3.3	3.8	3.5	4.5

1. GW, GP, GW–GC, GW–GM, GP–GC, and GP–GM per ASTM D2487 (includes crushed rock)
2. SW, SP, SM, SC, GM, and GC or mixtures per ASTM D2487
3. <85% of maximum dry density per AASHTO T 99, < 40% relative density (ASTM D4253 and D4254)
4. ≥85% of maximum dry density per AASHTO T 99, ≥ 40% relative density (ASTM D4253 and D4254)

12.12.4—Construction and Installation

The contract documents shall require that the construction and installation conform to Section 30, "Thermoplastic Culverts," of the *AASHTO LRFD Bridge Construction Specifications*.

12.13—STEEL TUNNEL LINER PLATE

12.13.1—General

The provisions of this Article shall apply to the structural design of steel tunnel liner plates. Construction shall conform to Section 25, "Steel and Concrete Tunnel Liners," of the *AASHTO LRFD Bridge Construction Specifications*.

The tunnel liner plate may be two-flange, fully corrugated with lapped longitudinal seams or four-flange, partially corrugated with flanged longitudinal seams. Both types shall be bolted together to form annular rings.

C12.13.1

The supporting capacity of a nonrigid tunnel lining, such as a steel liner plate, results from its ability to deflect under load, so that side restraint developed by the lateral resistance of the soil constrains further deflection. Thus, deflection tends to equalize radial pressures and to load the tunnel liner as a compression ring.

12.13.2—Loading

The provisions for earth loads given in Article 3.11.5 shall not apply to tunnels.

C12.13.2

The earth load to be carried by the tunnel liner is a function of the type of soil. In granular soil with little or no cohesion, the load is a function of the angle of internal friction of the soil and the diameter of the tunnel. In cohesive soils such as clays, the load to be carried by the tunnel liner is dependent on the shearing strength of the soil above the roof of the tunnel.

@Seismicisolation

12.13.2.1—Earth Loads

The provisions of Article 12.4.1 shall apply. When more refined methods of soil analysis are not employed, the earth pressure may be taken as:

$$W_E = C_{dt} \gamma_s S \qquad (12.13.2.1\text{-}1)$$

where:

C_{dt} = load coefficient for tunnel installation specified in Figure 12.13.2.1-1
γ_s = total unit weight of soil (kcf)
W_E = earth pressure at the crown (ksf)
S = tunnel diameter or span (ft)

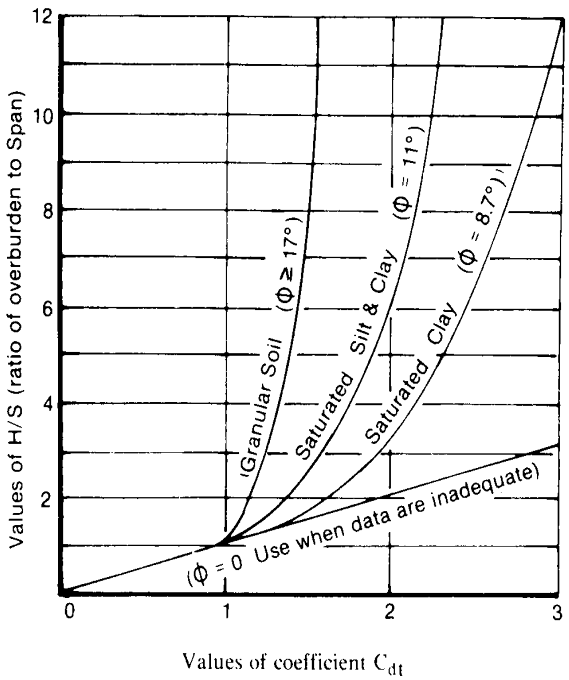


Figure 12.13.2.1-1—Diagram for Coefficient C_{dt} for Tunnel in Soil

in which:

H = height of soil over top of tunnel (ft)

C12.13.2.1

Eq. 12.13.2.1-1 is a form of the Marston formula. It proportions the amount of total overburden pressure acting on the tunnel based on the internal friction angle of the soil to be tunneled.

In the absence of adequate borings and soil tests, use $\phi_f = 0$ when calculating W_E.

12.13.2.2—Live Loads

The provisions of Article 12.6.1 shall apply.

12.13.2.3—Grouting Pressure

If the grouting pressure is greater than the computed design load, the design load, W_T, on the tunnel liner shall be the grouting pressure.

12.13.3—Safety Against Structural Failure

12.13.3.1—Section Properties

Steel tunnel liner plate shall meet the minimum requirements of Table 12.13.3.1-1 for cross-sectional properties, Table 12.13.3.1-2 for seam strength, and Table 12.13.3.1-3 for mechanical properties.

12.13.3.2—Wall Area

The requirements of Articles 12.7.2.2 and 12.7.2.3 shall apply using effective area from Table 12.13.3.1-1.

12.13.3.3—Buckling

The requirements of Article 12.7.2.4 shall apply, except that the soil stiffness factor, k, may vary from 0.22 to 0.44 depending upon the quality and extent of the backpacking material used.

C12.13.3.3

Wall buckling is a function of the stiffness, k, of the surrounding soil bearing on the plates. Where portland cement grouting or quality backpacking (meeting the requirements of Section 25, "Steel and Concrete Tunnel Liners," of the *AASHTO LRFD Bridge Construction Specifications*) material fill the void outside the plates, $k = 0.22$ is applicable. For other soils or in-situ backpacking material, $k = 0.44$ is suggested. Where tunneled soils slough or voids are left in the backpacking, additional consideration as to the value of k may be required.

12.13.3.4—Seam Strength

The requirements of Article 12.7.2.5 shall apply.

12.13.3.5—Construction Stiffness

Construction stiffness shall be indicated by a construction stiffness factor as:

$$C_S = \frac{EI}{S^2} \qquad (12.13.3.5\text{-}1)$$

where:

S = diameter or span (in.)
E = modulus of elasticity (ksi)
I = moment of inertia (in.^{4}/in.)

The value of C_S from Eq. 12.13.3.5-1 shall not be less than the values for steel tunnel liner plate as given in Article 12.5.6.4.

C12.13.3.5

The liner plate ring should have sufficient rigidity to resist the unbalanced loads of normal construction from grouting, local slough-ins, and miscellaneous concentrated loads.

The minimum construction stiffness required for these loads, C_S, can be expressed for convenience by the formula below. It must be recognized, however, that the limiting values given here are only recommended minimums. Actual job conditions may require greater effective stiffness. Final determination of this factor should be based on intimate knowledge of the project and on practical experience.

The construction stiffness, C_S, given by Eq. 12.13.3.5-1, considers the moment of inertia of an individual plate.

@Seismicisolation

Table 12.13.3.1-1—Cross-Sectional Properties—Steel Tunnel Liner Plate

2-Flange Tunnel Liner Plates				
Thickness (in.)		Effective Area (in.^{2}/in.)	Moment of Inertia (in.^{4}/in.)	Radius of Gyration (in.)
0.075		0.096	0.034	0.595
0.105		0.135	0.049	0.602
0.135		0.174	0.064	0.606
0.164		0.213	0.079	0.609
0.179		0.233	0.087	0.611
0.209		0.272	0.103	0.615
0.239		0.312	0.118	0.615
4-Flange Tunnel Liner Plates				
Thickness (in.)	Area (in.^{2}/in.)	Effective Area (in.^{2}/in.)	Moment of Inertia (in.^{4}/in.)	Radius of Gyration (in.)
0.1050	0.133	0.067	0.042	0.561
0.1196	0.152	0.076	0.049	0.567
0.1350	0.170	0.085	0.055	0.568
0.1640	0.209	0.105	0.070	0.578
0.1790	0.227	0.114	0.075	0.555
0.2090	0.264	0.132	0.087	0.574
0.2390	0.300	0.150	0.120	0.632
0.2500	0.309	0.155	0.101	0.571
0.3125	0.386	0.193	0.123	0.564
0.3750	0.460	0.230	0.143	0.557

Table 12.13.3.1-2—Minimum Longitudinal Seam Strength with Bolt and Nut Requirements for Steel Tunnel Plate Liner

	2-Flange Plate			4-Flange Plate		
	Longitudinal Seam Bolts		Ultimate Seam Strength (kip/ft)	Longitudinal Seam Bolts		Ultimate Seam Strength (kip/ft)
Plate Thickness (in.)	Diameter (in.)	Material ASTM		Diameter (in.)	Material ASTM	
0.075	0.625	A307	20	—	—	—
0.105	0.625	A307	30	0.500	A307	26
0.135	0.625	A307	47	0.500	A307	43
0.164	0.625	A307	55	0.500	A307	50
0.179	0.625	A307	62	0.625	A307	54
0.209	0.625	A449	87	0.625	A307	67
0.239	0.625	A449	92	0.625	A307	81
0.313	0.625	—	—	0.625	A307	115
0.375	0.625	—	—	0.625	A307	119

All nuts shall conform to ASTM A307, Grade A or better.
Circumferential seam bolts shall conform to ASTM A307 or better for all plate thicknesses.

Table 12.13.3.1-3—Mechanical Properties—Steel Tunnel Liner Plate (Plate before Cold Forming)

Minimum Tensile Strength	42.0 ksi
Minimum Yield Strength	28.0 ksi
Elongation, 2.0 in.	30%
Modulus of Elasticity	29,000 ksi

@Seismicisolation

12.14—PRECAST REINFORCED CONCRETE THREE-SIDED STRUCTURES

12.14.1—General

The provisions herein shall apply to the design of precast reinforced concrete three-sided structures supported on a concrete footing foundation.

C12.14.1

Units may be manufactured using conventional structural concrete and forms (formed) or machine made using dry concrete and vibrating forms.

12.14.2—Materials

12.14.2.1—Concrete

Concrete shall conform to Article 5.4.2, except that evaluation of f'_c may also be based on cores.

12.14.2.2—Reinforcement

Reinforcement shall meet the requirements of Article 5.4.3, except that for welded wire fabric a yield strength of 65,000 psi may be used. For wire fabric, the spacing of longitudinal wires shall be a maximum of 8.0 in. Circumferential welded wire fabric spacing shall not be greater than 4.0 in. or less than 2.0 in. Prestressing, if used, shall be in accordance with Article 5.9.

12.14.3—Concrete Cover for Reinforcement

The minimum concrete cover for reinforcement in precast three-sided structures reinforced with welded wire fabric shall be taken as three times the wire diameter, but not less than 1.0 in., except for the reinforcement in the top of the top slab of structures covered by less than 2.0 ft of fill, in which case the minimum cover shall be taken as 2.0 in.

12.14.4—Geometric Properties

Except as noted herein, the shape of the precast three-sided structures may vary in span, rise, wall thickness, haunch dimensions, and curvature. Specific geometric properties shall be specified by the manufacturer. Wall thicknesses shall be a minimum of 8.0 in. for spans under 24.0 ft and 10.0 in. for 24.0 ft and larger spans.

12.14.5—Design

12.14.5.1—General

Designs shall conform to applicable sections of these Specifications, except as provided otherwise herein. Analysis shall be based on a pinned connection at the footing and shall take into account anticipated footing movement.

@Seismicisolation

12.14.5.2—Distribution of Concentrated Load Effects in Top Slab and Sides

Distribution of wheel loads and concentrated loads for the top slab and sides of three-sided structures shall be taken as specified in Article 12.11.2.1.

12.14.5.3—Distribution of Concentrated Loads in Skewed Culverts

Wheel loads on skewed culverts shall be distributed using the same provisions as given for culverts with main reinforcement parallel to traffic. For culvert elements with skews greater than 15 degrees, the effect of the skew shall be considered in analysis.

12.14.5.4—Shear Transfer in Transverse Joints between Culvert Sections

The provisions of Article 4.6.2.10.4 shall apply.

In addition, except as provided herein, a means of shear transfer between adjacent units shall be provided in the top slab of structures having flat tops under less than 2.0 ft of fill and subjected to vehicular live loads. Shear transfer between adjacent units may be considered adequate where the thickness of the top slab is equal to or greater than:

- For prestressed slabs:

$$\frac{S}{28} \tag{12.14.5.4-1}$$

- For non-prestressed slabs:

$$\frac{(S+10)}{30} \tag{12.14.5.4-2}$$

where:

S = clear span (ft) measured parallel to the joint with the adjacent section

C12.14.5.4

Flat top structures with less than 2.0 ft of fill and with top slabs that are thinner than specified in this Article may experience differential deflection of adjacent units which can cause pavement cracking if a means of shear transfer is not utilized.

The specified minimum slab thickness and span-to-slab thickness ratios reflect years of experience in the design and construction of flat top three-sided structures and are influenced by Table 9.5(a) of ACI 318-08 and Table 8.9.2 of the *AASHTO Standard Specifications for Highway Bridges*, 17th Ed. Past performance of flat top three-sided structures designed in accordance with these provisions provides additional support for this exception.

For skewed sections, design is based on the span measured parallel to the joint with the adjacent section. This is a longer span than measured perpendicular to the end walls. However, designing for a longer span provides additional reinforcement to address the non-uniform stresses introduced by the skewed geometry which are not explicitly considered for modest skew angles.

Arch-top structures, because of their geometry and interaction with the surrounding soil, do not exhibit significant differential deflections that could cause pavement cracking for structures with less than 2.0 ft of fill. Thus, the requirements of this Article do not apply to arch-top structures.

The minimum thickness provision of this Section pertains only to addressing the need for shear transfer between adjacent three-sided sections. All other provisions of this Specification must be met.

12.14.5.5—Span Length

When monolithic haunches inclined at 45 degrees are taken into account, negative reinforcement in walls and slabs may be proportioned on the basis of flexural moment at the intersection of the haunch and uniform depth member.

@Seismicisolation

12.14.5.6—Resistance Factors

The provisions of Articles 12.5.5 and 1.3.1 shall apply as appropriate.

12.14.5.7—Crack Control

The provisions of Article 5.6.4 for buried structures shall apply.

12.14.5.8—Minimum Reinforcement

The provisions of Article 5.9.5.5 shall not be taken to apply to precast three-sided structures.

The primary flexural reinforcement in the direction of the span shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.002. Such minimum reinforcement shall be provided at all cross-sections subject to flexural tension, at the inside face of walls, and in each direction at the top of slabs of three-sided sections with less than 2.0 ft of fill.

12.14.5.9—Deflection Control at the Service Limit State

The deflection limits for concrete structures specified in Article 2.5.2.6.2 shall be taken as mandatory and pedestrian usage as limited to urban areas.

12.14.5.10—Footing Design

Design shall include consideration of differential horizontal and vertical movements and footing rotations. Footing design shall conform to the applicable Articles in Sections 5 and 10.

12.14.5.11—Structural Backfill

Specification of backfill requirements shall be consistent with the design assumptions used. The contract documents should require that a minimum backfill compaction of 90 percent Standard Proctor Density be achieved to prevent roadway settlement adjacent to the structure. A higher backfill compaction density may be required on structures utilizing a soil–structure interaction system.

12.14.5.12—Scour Protection and Waterway Considerations

The provisions of Article 2.6 shall apply as appropriate.

12.15—FIBERGLASS PIPE

12.15.1—General

The provisions of this Article shall apply to the structural design of solid wall buried fiberglass pipe.

C12.15.1

The provisions of this Article are based on Chapter 5 of AWWA's *Manual of Water Supply Practices—M45*, "Fiberglass Pipe Design." The internal pressure design requirements of M45 have been omitted here as culverts are typically designed for gravity flow.

Specific design requirements rely on the provisions for thermoplastic pipe as modified in this Article. Not all thermoplastic pipe design requirements are applicable to fiberglass pipe design.

12.15.2—Section Properties

Section properties for service and strength limit state calculations for solid wall fiberglass pipe shall be determined from the manufacturer's published nominal wall thickness, and from the nominal diameter requirements of ASTM D3262, Tables 2 and 3. Fiber-reinforced wall thickness shall be determined in accordance with ASTM D3567.

12.15.3—Mechanical Requirements

Solid wall fiberglass pipe mechanical properties shall be determined as specified in this Article.

C12.15.3

Because of the composite nature of fiberglass pipe, variety of manufacturing processes, and variables such as the amount, type, and orientation of fiber reinforcement, mechanical properties used in design must be actual properties from test results and measurements of the pipe in accordance with ASTM D3262 and ASTM D3567, as required in this Article.

12.15.3.1—Circumferential Flexural Modulus

The design value for circumferential flexural modulus of the pipe, E_{cf}, shall be calculated from pipe stiffness test results, conducted in accordance with ASTM D3262, using the pipe's fiber-reinforced wall thickness determined in accordance with Article 12.15.2.

C12.15.3.1

Appendix 2 of ASTM D2412 includes discussion on determining the circumferential flexural modulus from pipe stiffness test results.

12.15.3.2—Long-Term Ring-Bending Strain

The design value for long-term ring-bending strain, S_b, shall be determined from tests in accordance with ASTM D5365 using a water test solution with a pH between 5 and 9. Test data shall be statistically extrapolated to 75 years. Alternatively, the results from tests in accordance with ASTM D3681 may be used when tested in a solution of 1N H_2SO_4 with the results extrapolated to 75 years.

C12.15.3.2

As fiberglass pipe was originally designed for sanitary sewer applications, testing in acid is common and results are generally available from manufacturers. Testing in acid gives conservative results over water, and, if the results are already available, eliminates the need for the manufacturer to perform additional testing in water.

12.15.4—Total Allowable Deflection

The total allowable deflection, Δ_A (in.), shall be 5.0 percent of the inside diameter of the pipe or a lower deflection as specified by the Engineer. The total allowable deflection and the upper deflection limit where remediation or replacement is required shall be prominently displayed in the construction documents.

C12.15.4

Higher stiffness fiberglass pipe is easily produced for specific installations by increasing the quantity of glass fiber reinforcement or the wall thickness, or both. The allowable deflection for higher stiffness pipes may need to be reduced.

@Seismicisolation

12.15.5—Service Limit States

12.15.5.1—General

Similar to other types of buried flexible pipe, fiberglass pipe shall be installed in a manner that will ensure that external loads will not cause a long-term decrease in the vertical diameter of the pipe exceeding the total allowable deflection in Article 12.15.4.

12.15.5.2—Deflection Requirement

The deflection requirements for thermoplastic pipe in Article 12.12.2.2 shall apply with the total allowable deflection, Δ_A, as defined in Article 12.15.4, and total deflection, Δ_t, determined as:

$$\Delta_t = \frac{D_o K_B \left(D_L P_{sp} + C_L P_L\right)}{1{,}000\left(E_{cf} I_p / R^3 + 0.061 M_s\right)} \quad (12.15.5.2\text{-}1)$$

where:

Δ_t = total deflection of pipe, expressed as a reduction of the vertical diameter and taken as positive for a reduction in the vertical diameter (in.)
D_o = outside diameter of pipe, as specified in Article 12.15.2 (in.)
K_B = bedding coefficient, as specified in Article 12.12.2.2
D_L = deflection lag factor, as specified in Article 12.12.2.2
P_{sp} = soil prism pressure (EV), evaluated at the top of the pipe determined as the unit weight of soil multiplied by the soil cover depth (psi)
C_L = live load distribution coefficient, as specified in Article 12.12.3.5
P_L = service live load on pipe, as specified in Article 12.6.1 (psi)
E_{cf} = circumferential flexural modulus as defined in Article 12.15.3.1 (ksi)
I_p = moment of inertia of the fiber-reinforced pipe wall per unit length (in.^{4}/in.)
R = radius of pipe to centroid of pipe wall (in.)
M_s = constrained soil modulus as specified in Table 12.12.3.5-1 (ksi)

C12.15.5.2

This Article is similar to Article 12.12.2.2 for thermoplastic pipe; however, the critical loading for fiberglass pipe occurs due to flexural tension at the invert. Axial compressive thrust in fiberglass pipe is not significant due to the solid wall and high modulus and is therefore ignored. However, the solid wall and high modulus do increase the service load on the pipe to the total soil prism load. Thus, Article 12.12.2.2 is adapted to fiberglass pipe by eliminating the thrust component of deflection and calculating the load as the soil prism pressure at the crown of the pipe.

12.15.6—Safety Against Structural Failure

12.15.6.1—General

Buried fiberglass culverts shall be investigated at the strength limit states for flexure (ring-bending), global buckling, and flexibility limit.

@Seismicisolation

12.15.6.2—Flexure

Factored long-term strain due to flexure shall satisfy:

$$\varepsilon_f \leq \phi_f S_b \qquad (12.15.6.2\text{-}1)$$

where:

ε_f = factored long-term strain due to flexure, determined in accordance with Eq. 12.12.3.10.2b-3 as modified below (in./in.)
ϕ_f = resistance factor for flexure, as defined in Table 12.5.5-1
S_b = long-term ring-bending strain, as defined in Article 12.15.3.2

The variable Δ_f in Eq. 12.12.3.10.2b-3 shall be set to Δ_A, as specified in Article 12.15.4.

12.15.6.3—Global Buckling

Loads to be considered for global buckling of fiberglass culverts shall include hydrostatic groundwater, soil pressure, and live loads. Factored buckling strain demand shall satisfy:

$$\varepsilon_{fb} \leq \phi_{bck} \varepsilon_{bck} \qquad (12.15.6.3\text{-}1)$$

in which:

$$\varepsilon_{fb} = \frac{(\gamma_{WA}\eta_{EV} H_w \gamma_w / 144 + \gamma_{EV}\eta_{EV} P_{sp} R_w + \gamma_{LL}\eta_{LL} C_L P_L) R}{1{,}000 E_{cf} A_g} \qquad (12.15.6.3\text{-}2)$$

in which:

$$R_w = 1 - 0.33\frac{H_w}{H} \quad \text{for } 0 \leq H_w \leq H \qquad (12.15.6.3\text{-}3)$$

where:

ε_{fb} = factored buckling strain demand (in./in.)
ϕ_{bck} = resistance factor for global buckling as defined in Table 12.5.5-1
ε_{bck} = nominal strain capacity for general buckling, determined in accordance with Eq. 12.12.3.10.1e-2 as modified below (in./in.)
γ_{WA} = load factor for hydrostatic pressure
η_{EV} = load modifier for vertical earth pressure on buried structures, from Article 1.3.2 applied in accordance with Article 12.5.4
H_w = height of water over top of pipe (ft)
γ_w = unit weight of water (lb/ft^3)
γ_{EV} = load factor for vertical earth pressure

P_{sp} = soil prism pressure (EV), evaluated at the crown of the pipe determined as the unit weight of soil multiplied by the soil cover depth (psi)
R_w = water buoyancy factor
γ_{LL} = load factor for live load
η_{LL} = load modifier for live load on buried structures, from Article 1.3.2 applied in accordance with Article 12.5.4
C_L = live load distribution coefficient, as specified in Article 12.12.3.5
P_L = service live load on pipe, as specified in Article 12.6.1 (psi)
R = radius of pipe to centroid of pipe wall (in.)
E_{cf} = circumferential flexural modulus, as defined in Article 12.15.3.1 (ksi)
A_g = area of fiber-reinforced wall of the pipe per unit length, determined from reinforced wall thickness measurements (in.^{2}/in.)
H = height of fill above top of pipe (ft)

The variables E_p and A_{eff} in Eq. 12.12.3.10.1e-2 shall be replaced with E_{cf} and A_g, respectively.

12.15.6.4—Flexibility Limit

The flexibility factor for fiberglass pipe shall be determined in accordance with Article 12.12.3.6, but with the variable E in Eq. 12.12.3.6-1 set to E_{cf}, as defined in Article 12.15.3.1. The flexibility factor shall be limited as specified in Article 12.5.6.3.

12.15.7—Construction and Installation

The contract documents shall require that the construction and installation conform to Section 30, “Thermoplastic Culverts,” of the *AASHTO LRFD Bridge Construction Specifications.*

12.16—REFERENCES

AA. *Aluminum Drainage Products Manual*, 1st Ed. Aluminum Association, Washington, DC, 1983.

AASHTO. *Standard Specifications for Highway Bridges*, 17th ed. HB-17. American Association of State Highway and Transportation Officials, Washington, DC, 2002.

AASHTO. *AASHTO LRFD Bridge Construction Specifications*, 4th ed. with 2020 Interims, LRFDCONS-4. American Association of State Highway and Transportation Officials, Washington, DC, 2017.

AASHTO. *AASHTO Drainage Manual*, 1st Ed. American Association of State Highway and Transportation Officials, Washington, DC, 2014.

AASHTO. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, HM-WB. American Association of State Highway and Transportation Officials, Washington, DC.

Abolmaali, A. and A. Garg. Effect of Wheel Live Load on Shear Behavior of Precast Reinforced Concrete Box Culverts. *Journal of Bridge Engineering*, Vol. 13, Issue 1, January 2008, pp. 93–99. https://doi.org/10.1061/(ASCE)1084-0702(2008)13:1(93)

Abolmaali, A. A. and Garg. Shear Behavior and Mode of Failure for ASTM C1433 Precast Box Culverts. *Journal of Bridge Engineering*, Vol. 13, Issue 4, July 2008, pp. 331–338. https://doi.org/10.1061/(ASCE)1084-0702(2008)13:4(331)

ACI. *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. American Concrete Institute, Farmington Hills, MI, 2008.

ASTM. *Annual Book of ASTM Standards.* ASTM International, West Conshohocken, PA.

AWWA. Fiberglass Pipe Design. *AWWA Manual of Water Supply Practices—M45.* 2nd Ed. American Water Works Association, Denver, CO, 2005.

Bellair, P. J., and J. P. Ewing. *Metal Loss Rates of Uncoated Steel and Aluminum Culverts in New York*, Research Report 115. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, NY, 1984.

Boulanger, R. W., R. B. Seed, R. D. Baird, and J. C. Schluter. Measurements and Analyses of Deformed Flexible Box Culverts. In *Transportation Research Record 1231*. Transportation Research Board, National Research Council, Washington, DC, 1989.

Burns, J. Q., and R. M. Richard. Attenuation of Stresses for Buried Cylinders. *Proc., Conference on Soil Structure Interaction*. University of Arizona, Tucson, AZ, 1964.

CSA. *Canadian Highway Bridge Design Code*, CAN/CSA-S6-06. Canadian Standards Association, Rexdale, ON, Canada, 2006.

Duncan, J. M., R. B. Seed, and R. H. Drawsky. Design of Corrugated Metal Box Culverts. In *Transportation Research Record 1008.* Transportation Research Board, National Research Council, Washington, DC, 1985. pp. 33–41.

FHWA. *Hydraulic Design of Highway Culverts*, FHWA-IP-85-15. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 1985.

FHWA. *Project Development and Design Manual*. Federal Lands Highway Programs, Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2018. https://flh.fhwa.dot.gov/resources/design/pddm/

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

Funahashi, M., and J. B. Bushman. Technical Review of 100 mV Polarization Shift Criterion for Reinforcing Steel in Concrete. *Corrosion*, Vol. 47, No. 5, May 1991, pp. 376–386.

Garg, A., A. Abolmaali, and R. Fernandez. Experimental Investigation of Shear Capacity of Precast Reinforced Concrete Box Culverts. *Journal of Bridge Engineering*, Vol. 12, Issue 4, July 2007, pp. 511–517. https://doi.org/10.1061/(ASCE)1084-0702(2007)12:4(511)

Gemperline, Mark, Erica Gemperline and Claire Gemperline. Large Scale Constrained Modulus Test. *Plastics Pipe Institute*. MCG Geotechnical Engineering, Morrison, CO, 2010. https://plasticpipe.org/pdf/ms-study-report.pdf

Hashash, N., and E. T. Selig. Analysis of the Performance of a Buried High Density Polyethylene Pipe. *Proc., First National Conference on Flexible Pipes*, Columbus, OH, October 1990, pp. 95–103.

Hurd, J. O. Field Performance of Concrete Pipe and Corrugated Steel Pipe Culverts and Bituminous Protection of Corrugated and Steel Pipe Culverts. In *Transportation Research Record 1001*. Transportation Research Board, National Research Council, Washington, DC, 1984.

@Seismicisolation

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

Koepf, A. H., and P. H. Ryan. Abrasion Resistance of Aluminum Culvert Based on Long-Term Field Performance. In *Transportation Research Record 1087*. Transportation Research Board, National Research Council, Washington, DC, 1986. pp. 15–25.

Latona, R. W., F. J. Heger, M. Bealey. Computerized Design of Precast Reinforced Concrete Box Culverts. In *Highway Research Record 443*. Highway Research Board, National Research Council, Washington, DC, 1973.

MacDougall, K., N. Hoult, and I. D. Moore. 2015. Measured Load Capacity of Buried Reinforced Concrete Pipes, *ACI Structural Journal*, Vol. 113, Issue 1, January/February 2016, pp. 63–73. https://doi.org/10.14359/51688059

McGrath, T. J. *A Proposed Design Method for Calculating Loads and Hoop Compression Stresses for Buried Pipe*. Draft report submitted to the Polyethylene Pipe Design Task Group of the AASHTO Flexible Culvert Liaison Committee, 1996.

McGrath, T. J. Calculating Loads on Buried Culverts Based on Pipe Hoop Stiffness. In *Transportation Research Record 1656*. Transportation Research Board, National Research Council, Washington, DC, 1999.

McGrath, T. J. *Structural Investigation of Metal Box Section with Spans up to 36 ft* Prepared for CONTECH Construction Products, Inc. by Simpson Gumpertz & Heger Inc., Waltham, MA, 2005.

McGrath, T.J., A.A. Liepins, and J.L Beaver, Live Load Distribution Widths for Reinforced Concrete Box Sections. *Transportation Research Record:CD 11-S*, Transportation Research Board, National Research Council, Washington, DC, 2005, pp. 99–108.

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

McGrath, T. J., I. D. Moore, E. T. Selig, M. C. Webb, and B. Taleb. *National Cooperative Highway Research Program Report 473: Recommended Specifications for Large-Span Culverts*. National Cooperative Highway Research Program, Transportation Research Board, Washington, DC, 2002.

McGrath, T. J., I. D. Moore, and G. Y. Hsuan. *National Cooperative Highway Research Program Report 631: Updated Test and Design Methods for Thermoplastic Drainage Pipe*. National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC, 2009.

McGrath, T. J., and J. L. Beaver. *Performance of Thermoplastic Pipe under Highway Vehicle Loading*. Research Report to Minnesota Department of Transportation. Simpson Gumpertz & Heger Inc., Oakdale, MN, 2005.

McGrath, T. J., and V. E. Sagan. *National Cooperative Highway Research Program Report 438: LRFD Specifications for Plastic Pipe and Culverts*. National Cooperative Highway Research Program, Transportation Research Board, Washington, DC, 1999.

Meacham, D. G., J. O. Hurd, and W. W. Shislar. *Culvert Durability Study*, Report No. ODOT/LandD/82-1. Ohio Department of Transportation, Columbus, OH, 1982.

Moore, I. D. Three-Dimensional Response of Elastic Tubes. *International Journal of Solids and Structures*. Vol. 26, No. 4, Elsevier, Maryland Heights, MO, 1990.

Moore, I. D. Three-Dimensional Response of Deeply Buried Profiled Polyethylene Pipe. In *Transportation Research Record 1514*. Transportation Research Board, National Research Council, Washington, DC, 1995, pp. 49–58.

Moore, I. D. *DuroMaxx Pipe Assessment* (24 in. – February 2009, 60 in. – August 2009). Kingston, ON, Canada. 2009.

Moore, I. D. Effect of Time to Failure on Resistance to Local Buckling. DuroMaxx Pipe Assessment. Kingston, ON, Canada, 2010.

@Seismicisolation

Moore, I. D., N. A. Hoult, and K. MacDougall. *National Cooperative Highway Research Program Project 20-07 Task 316 Final Report: Establishment of Appropriate Guidelines for Use of the Direct and Indirect Design Methods for Reinforced Concrete Pipe*. National Cooperative Highway Research Program, Transportation Research Board, Washington, DC, March 2014. 126 pp.

NCHRP. *National Cooperative Highway Research Program Report 647: Recommended Design Specifications for Live Load Distribution to Buried Structures*National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC, 2010.

NCHRP. Service Life of Culverts. *NCHRP Synthesis of Highway Practice No. 474*. Transportation Research Board, National Research Council, Washington, DC, 2015.

NCHRP. *National Cooperative Highway Research Program Report 679: Design of Concrete Structures Using High-Strength Reinforcement*. National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC, 2011.

Park, Y., A. Abolmaali, E. Attiogbe, and S. Lee. Time-Dependent Behavior of Synthetic Fiber Reinforced Concrete Pipes Under Long-Term Sustained Loading *Journal of Transportation Research Board*, v2407, 2014, pp. 71–79.

Potter, J. C. *Life Cycle Cost for Drainage Structures*, Technical Report GL-88-2. Prepared for the Department of the Army by the Waterways Experiment Station, Vicksburg, MS, 1988.

Sagues, A. Alberto, J. Pena, C. Cotrim, M. Pech-Canul, and I. Urdaneta. Corrosion Resistance and Service Life of Drainage Culverts. State Job 99700-3335-119. Florida Department of Transportation, Tallahassee, FL, 2001.

Sargand, Shad, John Hurd, Kevin White, Teruhisa Masada, Johnnatan Garcia-Ruiz, and Gabriel Colorado-Urrea. Assessment of ODOT's Conduit Service Life Prediction Methodology. State Job 134725. Ohio Department of Treansportation, 2016.

Selig, E.T. Soil Properties for Plastic Pipe Installation. *Buried Plastic Pipe Technology*, ASTM STP 1093, George S. Buczala and Michael J. Cassady, eds. American Society for Testing and Materials, Philadelphia, PA, 1990.

White, D. W., and M. J. Clarke. Design of Beam-Columns in Steel Frames II: Comparison of Standards. *Journal of Structural Engineering*. Vol. 123, No. 12. American Society of Civil Engineers, Reston, VA, 1997..

Wilson, A., and A. Abolmaali. Comparison of Material Behavior of Steel and Synthetic Fibers in Dry Cast Application. *Transportation Research Record: Journal of the Transportation Research Board* Vol. 2332 Issue 1. Transportation Research Board, National Research Council, Washington, DC, 2013, pp. 23–28.

Wilson, A., and A. Abolmaali. Performance of Synthetic Fiber Concrete Pipes. *ASCE Journal of Pipeline Systems Engineering and Practice*, 5(3), 04014002. American Society of Civil Engineers, Reston, VA, 2013.

@Seismicisolation

APPENDIX A12—PLATE, PIPE, AND PIPE ARCH PROPERTIES

Table A12-1—Corrugated Steel Pipe—Cross-Section Properties

1 ½ × ¼ in. Corrugation			
Thickness (in.)	A (in.^{2}/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
0.052	0.608	0.0824	0.344
0.064	0.761	0.0832	0.439
0.079	0.950	0.0846	0.567
0.109	1.331	0.0879	0.857
0.138	1.712	0.0919	1.205
0.168	2.098	0.0967	1.635

2 $^2/_3$ × ½ in. Corrugation			
Thickness (in.)	A (in.^{2}/ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.040	0.465	0.1702	1.121
0.052	0.619	0.1707	1.500
0.064	0.775	0.1712	1.892
0.079	0.968	0.1721	2.392
0.109	1.356	0.1741	3.425
0.138	1.744	0.1766	4.533
0.168	2.133	0.1795	5.725

3 × 1 in. Corrugation			
Thickness (in.)	A (in.^{2}/ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.064	0.890	0.3417	8.659
0.079	1.113	0.3427	10.883
0.109	1.560	0.3448	15.459
0.138	2.008	0.3472	20.183
0.168	2.458	0.3499	25.091

5 × 1 in. Corrugation			
Thickness (in.)	A (in.^{2}/ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.064	0.794	0.3657	8.850
0.079	0.992	0.3663	11.092
0.109	1.390	0.3677	15.650
0.138	1.788	0.3693	20.317
0.168	2.186	0.3711	25.092

@Seismicisolation

Table A12-2—Spiral Rib Steel Pipe—Cross-Section Properties

¾ × ¾ × 7 ½ in. Corrugation			
Thickness (in.)	A (in.^{2}/ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.064	0.509	0.258	2.821
0.079	0.712	0.250	3.701
0.109	1.184	0.237	5.537
0.138	1.717	0.228	7.433

¾ × 1 × 11 ½ in. Corrugation			
Thickness (in.)	A (in.^{2}/ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.064	0.374	0.383	4.58
0.079	0.524	0.373	6.08
0.109	0.883	0.355	9.26

Note: Effective section properties are taken at full yield stress.

Table A12-3—Steel Structural Plate—Cross-Section Properties

6 × 2 in. Corrugations			
Thickness (in.)	A (in.^{2})	r (in.)	I (in.^{4}/in. × 10^{-3})
0.110	1.556	0.682	60.4
0.140	2.003	0.684	78.2
0.170	2.449	0.686	96.2
0.188	2.739	0.688	108.0
0.218	3.199	0.690	126.9
0.249	3.650	0.692	146.2
0.280	4.119	0.695	165.8
0.318	4.671	0.698	190.0
0.380	5.613	0.704	232.0

Table A12-4—Corrugated Aluminum Pipe—Cross-Section Properties

1 ½ × ¼ in. Corrugation			
Thickness (in.)	A (in.^{2}/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

2 ⅔ × ½ in. Corrugation			
Thickness (in.)	A (in.^{2}/ft)	r (in.)	$I \times 10^{-3}$ (in.^{4}/in.)
0.060	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

@Seismicisolation

Table A12-4—Corrugated Aluminum Pipe—Cross-Section Properties (continued)

3 × 1 in. Corrugation			
Thickness (in.)	A (in.²/ft)	r (in.)	$I \times 10^{-3}$ (in.⁴/in.)
0.060	0.890	0.3417	8.659
0.075	1.118	0.3427	10.883
0.105	1.560	0.3448	15.459
0.135	2.088	0.3472	20.183
0.164	2.458	0.3499	25.091

6 × 1 in. Corrugation			
Effective Thickness (in.)	A (in.²/ft)	Effective Area (in.²/ft)	r (in.)
0.060	0.775	0.387	0.3629
0.075	0.968	0.484	0.3630
0.105	1.356	0.678	0.3636
0.135	1.744	0.872	0.3646
0.164	2.133	1.066	0.3656

Table A12-5—Aluminum Spiral Rib Pipe—Cross-Section Properties

¾ × ¾ × 7 ½ in. Corrugation			
Thickness (in.)	A (in.²/ft)	r (in.)	$I \times 10^{-3}$ (in.⁴/in.)
0.060	0.415	0.272	2.558
0.075	0.569	0.267	3.372
0.105	0.914	0.258	5.073
0.135	1.290	0.252	6.826

¾ × 1 × 11 ½ in. Corrugation			
Thickness (in.)	A (in.²/ft)	r (in.)	$I \times 10^{-3}$ (in.⁴/in.)
0.060	0.312	0.396	4.08
0.075	0.427	0.391	5.45
0.105	0.697	0.380	8.39
0.135	1.009	0.369	11.48

Note: Effective section properties are taken at full yield stress.

Table A12-6—Corrugated Aluminum Structural Plate or Pipe Arch—Cross-Section Properties

9 × 2 ½ in. Corrugations			
Thickness (in.)	A (in.²/ft)	r (in.)	I (in.⁴/in. × 10^{-3})
0.100	1.404	0.8438	83.1
0.125	1.750	0.8444	104.0
0.150	2.100	0.8449	124.9
0.175	2.449	0.8454	145.9
0.200	2.799	0.8460	167.0
0.225	3.149	0.8468	188.2
0.250	3.501	0.8473	209.4

@Seismicisolation

Table A12-7—Minimum Longitudinal Seam Strength Corrugated Aluminum and Steel Pipe—Riveted or Spot Welded

2 × ½ and 2 $^2/_3$ × ½ in. Corrugated Aluminum Pipe			
Thickness (in.)	Rivet Size (in.)	Single Rivets (kip/ft)	Double Rivets (kip/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 in. Corrugated Aluminum Pipe		
Thickness (in.)	Rivet Size (in.)	Double Rivets (kip/ft)
0.060	$^3/_8$	16.5
0.075	$^3/_8$	20.5
0.105	½	28.0
0.135	½	42.0
0.164	½	54.5

6 × 1 in. Corrugated Aluminum Pipe		
Thickness (in.)	Rivet Size (in.)	Double Rivets (kip/ft)
0.060	½	16.0
0.075	½	19.9
0.105	½	27.9
0.135	½	35.9
0.167	½	43.5

2 × ½ and 2 $^2/_3$ × ½ in. Corrugated Steel Pipe			
Thickness (in.)	Rivet Size (in.)	Single Rivets (kip/ft)	Double Rivets (kip/ft)
0.064	$^5/_{16}$	16.7	21.6
0.079	$^5/_{16}$	18.2	29.8
0.109	$^3/_8$	23.4	46.8
0.138	$^3/_8$	24.5	49.0
0.168	$^3/_8$	25.6	51.3

3 × 1 in. Corrugated Steel Pipe		
Thickness (in.)	Rivet Size (in.)	Double Rivets (kip/ft)
0.064	$^3/_8$	28.7
0.079	$^3/_8$	35.7
0.109	$^7/_{16}$	53.0
0.138	$^7/_{16}$	63.7
0.168	$^7/_{16}$	70.7

@Seismicisolation

Table A12-8—Minimum Longitudinal Seam Strengths Steel and Aluminum Structural Plate Pipe—Bolted

6 × 2 in. Steel Structural Plate Pipe				
Bolt Thickness (in.)	Bolt Diameter (in.)	4 Bolts/ft (kip/ft)	6 Bolts/ft (kip/ft)	8 Bolts/ft (kip/ft)
0.109	¾	43.0	—	—
0.138	¾	62.0	—	—
0.168	¾	81.0	—	—
0.188	¾	93.0	—	—
0.218	¾	112.0	—	—
0.249	¾	132.0	—	—
0.280	¾	144.0	180.0	194.0
0.318	7/8	—	—	235.0
0.380	7/8	—	—	285.0

9 × 2 1/2 in. Aluminum Structural Plate Pipe			
Thickness (in.)	Bolt Diameter (in.)	Steel Bolts 5.5 Bolts per ft (kip/ft)	Aluminum Bolts 5.5 Bolts per ft (kip/ft)
0.100	¾	28.0	26.4
0.125	¾	41.0	34.8
0.150	¾	54.1	44.4
0.175	¾	63.7	52.8
0.200	¾	73.4	52.8
0.225	¾	83.2	52.8
0.250	¾	93.1	52.8

Table A12-9—Mechanical Properties for Spiral Rib and Corrugated Metal Pipe and Pipe Arch

Material	Minimum Tensile Strength, F_u (ksi)	Minimum Yield Stress, F_y (ksi)	Modulus of Elasticity, E_m (ksi)
Aluminum H34(1)&(4)	31.0	24.0	10,000
Aluminum H32(2)&(4)	27.0	20.0	10,000
Steel(3)	45.0	33.0	29,000

1. Shall meet the requirements of AASHTO M 197 (ASTM B744), for Alclad Alloy 3004-H34
2. Shall meet the requirements of AASHTO M 197 (ASTM B744), for Alclad Alloy 3004-H32
3. Shall meet the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M), M 218, and M 246 (ASTM A742)
4. H34 temper material shall be used with riveted pipe to achieve seam strength. Both H32 and H34 temper material may be used with helical pipe

@Seismicisolation

Table A12-10—Mechanical Properties—Corrugated Aluminum and Steel Plate

Material	Minimum Tensile Strength (ksi)	Minimum Yield Stress (ksi)	Modulus of Elasticity (ksi)
Aluminum[1] Plate Thickness (in.)			
0.100–0.175	35.0	24.0	10,000
0.176–0.250	34.0	24.0	10,000
Steel[2] Plate Thickness (in.)			
All	45.0	33.0	29,000
Steel Deep Corrugated Plate	55.0	44.0	29,000

1. Shall meet the requirements of AASHTO M 219 (ASTM B746), Alloy 5052
2. Shall meet the requirements of AASHTO M 167M/M 167 (ASTM A761/A761M)

Table A12-11—PE Corrugated Pipes (AASHTO M 294)

Nominal Size (in.)	Min. *ID* (in.)	Max. *OD* (in.)	Min. *A* (in.^{2}/ft)	Min. *c* (in.)	Min. *I* (in.^{4}/in.)
12	11.8	14.7	1.5	0.35	0.024
15	14.8	18.0	1.9	0.45	0.053
18	17.7	21.5	2.3	0.50	0.062
24	23.6	28.7	3.1	0.65	0.116
30	29.5	36.4	3.9	0.75	0.163
36	35.5	42.5	4.5	0.90	0.222
42*	41.5	48.0	4.69	1.11	0.543
48*	47.5	55.0	5.15	1.15	0.543

* For the 42.0-in. and 48.0-in. pipe, the wall thickness should be designed using the long-term tensile strength provision, i.e., 900 psi, until new design criteria are established in the AASHTO bridge and structures specifications.

Table A12-12—PE Ribbed Pipes (ASTM F894)

Nominal Size (in.)	Min. *ID* (in.)	Max. *OD* (in.)	Min. *A* (in.^{2}/ft)	Min. *c* (in.)	Min. *I* (in.^{4}/in.)	
					Cell Class 334433C	Cell Class 335434C
18	17.8	21.0	2.96	0.344	0.052	0.038
21	20.8	24.2	4.15	0.409	0.070	0.051
24	23.8	27.2	4.66	0.429	0.081	0.059
27	26.75	30.3	5.91	0.520	0.125	0.091
30	29.75	33.5	5.91	0.520	0.125	0.091
33	32.75	37.2	6.99	0.594	0.161	0.132
36	35.75	40.3	8.08	0.640	0.202	0.165
42	41.75	47.1	7.81	0.714	0.277	0.227
48	47.75	53.1	8.82	0.786	0.338	0.277

@Seismicisolation

Table A12-13—PVC Profile Wall Pipes (AASHTO M 304)

Nominal Size (in.)	Min. *I.D.* (in.)	Max. *O.D.* (in.)	Min. *A* (in.^{2}/ft)	Min. *c* (in.)	Min. *I* (in.^{4}/in.) Cell Class 12454C	Min. *I* (in.^{4}/in.) Cell Class 12364C
12	11.7	13.6	1.20	0.15	0.004	0.003
15	14.3	16.5	1.30	0.17	0.006	0.005
18	17.5	20.0	1.60	0.18	0.009	0.008
21	20.6	23.0	1.80	0.21	0.012	0.011
24	23.4	26.0	1.95	0.23	0.016	0.015
30	29.4	32.8	2.30	0.27	0.024	0.020
36	35.3	39.5	2.60	0.31	0.035	0.031
42	41.3	46.0	2.90	0.34	0.047	0.043
48	47.3	52.0	3.16	0.37	0.061	0.056

Table A12-14—Steel Structural Plate with Deep Corrugations—Cross-Section Properties

15 × 5 ½ in. Corrugations			
Coated Thickness (in.)	*A* (in.^{2}/ft)	*r* (in.)	*I* (in.^{4}/in.)
0.140	2.260	1.948	0.714
0.170	2.762	1.949	0.875
0.188	3.088	1.950	0.979
0.218	3.604	1.952	1.144
0.249	4.118	1.953	1.308
0.280	4.633	1.954	1.472
16 x 6 in. Corrugations			
Coated Thickness (in.)	*A* (in^2/ft)	*R* (in.)	*I* (in.^{4}/in.)
0.174	2.736	2.081	0.988
0.202	3.218	2.083	1.163
0.241	3.902	2.084	1.413
0.281	4.554	2.086	1.652
0.320	5.166	2.088	1.877
20 x 9 ½ in. Corrugations			
Coated Thickness (in.)	*A* (in.^{2}/ft)	*r* (in.)	*I* (in.^{4}/in.)
0.280	5.021	3.21	4.321
0.319	5.737	3.22	4.945
0.380	6.855	3.22	5.921

@Seismicisolation

Table A12-15—Minimum Longitudinal Seam Strengths, Deep Corrugated Structures—Bolted

15 × 5 ½ in. Corrugations		
Coated Thickness (in.)	Bolt Diameter (in.)	6 Bolts/Corrugation (lb/ft of seam)
0.140	¾	66,000
0.170	¾	87,000
0.188	¾	102,000
0.218	¾	127,000
0.249	¾	144,000
0.280	¾	144,000
0.249	7/8	159,000
0.280	7/8	177,000
16 × 6 in. Corrugations		
Coated Thickness (in.)	Bolt Diameter (in.)	6 Bolts/Corrugation (lb/ft of seam)
0.174	¾	85,000[b]
0.202	¾	124,000[b]
0.241	¾	163,000[b]
0.281	¾	163,000[b]
0.320	¾	163,000[b]
0.281	7/8	201,000[b]
0.320	7/8	209,000[b]
16 × 6 in. Corrugations		
Coated Thickness (in.)	Bolt Diameter (in.)	8 Bolts/Corrugation (lb/ft of seam)
0.281	7/8	200,000[b]
0.320	7/8	226,000[b]
20 × 9 ½ in. Corrugations		
Coated Thickness (in.)	Bolt Diameter (in.)	12 Bolts/Corrugation (lb/ft of seam)
0.280	7/8	197,000[a]
0.319	7/8	218,000[a]
0.380	7/8	267,000[a]

[a] The number of bolts per corrugation includes the bolts in the corrugation crest, tangent, and valley; the number of bolts within one pitch. The ultimate seam strengths listed are based on tests of staggered seams in assemblies fabricated from panels with a nominal width of 40.0 in. and include the contribution of additional bolts at the stagger. The ultimate seam strengths listed are only applicable for panels with a nominal width of 40.0 in. and staggered seams.

[b] Seam strengths determined based on tests on specimens with 3 corrugation widths.

Note: The listed ultimate seam strengths are only applicable for panels with a nominal width of 30.0 in. and with staggered seams. The number of bolts per corrugation includes bolts within one pitch: those in the corrugation crest and in the corrugation valley. The ultimate seam strengths listed are based on tests of staggered seams in assemblies fabricated from panels with a nominal width of 30.0 in. and include the contribution of additional bolts at the stagger.

@Seismicisolation