SECTION 10

FOUNDATIONS

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FOUNDATIONS

10.1—SCOPE

Provisions of this Section shall apply for the design of spread footings, driven piles, drilled shaft, and micropile foundations.

The probabilistic LRFD basis of these Specifications, which produces an interrelated combination of load, load factor resistance, resistance factor, and statistical reliability, shall be considered when selecting procedures for calculating resistance other than that specified herein. Other methods, especially when locally recognized and considered suitable for regional conditions, may be used if resistance factors are developed in a manner that is consistent with the development of the resistance factors for the method(s) provided in these Specifications, and are approved by the Owner.

C10.1

The development of the resistance factors provided in this Section are summarized in Allen (2005), with additional details provided in Appendix A of Barker et al. (1991), in Paikowsky et al. (2004), in Allen (2005), and in D'Appolonia (2006).

The specification of methods of analysis and calculation of resistance for foundations herein is not intended to imply that field verification and/or reaction to conditions actually encountered in the field are no longer needed. These traditional features of foundation design and construction are still practical considerations when designing in accordance with these Specifications.

10.2—DEFINITIONS

Battered Pile—A pile or micropile installed at an angle inclined to the vertical to provide higher resistance to lateral loads.

Bearing Pile—A pile or micropile whose purpose is to carry axial load through friction or point bearing.

Bent—A type of pier comprised of multiple columns or piles supporting a single cap and in some cases connected with bracing.

Bent Cap—A flexural substructure element supported by columns or piles that receives loads from the superstructure.

Bond Length—The length of a micropile that is bonded to the ground and which is conceptually used to transfer the applied axial loads to the surrounding soil or rock. Also known as the load transfer length.

Casing—Steel pipe introduced during the drilling process to temporarily stabilize the drill hole. Depending on the details of micropile construction and composition, this casing may be fully extracted during or after grouting, or may remain partially or completely in place as part of the final micropile pile configuration.

Centralizer—A device to centrally locate the core steel within a borehole.

Column Bent—A type of bent that uses two or more columns to support a cap. Columns may be drilled shafts or other independent units supported by individual footings or a combined footing; and may employ bracing or struts for lateral support above ground level.

Combination Point Bearing and Friction Pile—Pile that derives its capacity from contributions of both point bearing developed at the pile tip and resistance mobilized along the embedded shaft.

Combined Footing—A footing that supports more than one column.

Core Steel—Reinforcing bars or pipes used to strengthen or stiffen a micropile, excluding any left-in casing.

CPT—Cone Penetration Test.

CU—Consolidated Undrained.

Deep Foundation—A foundation that derives its support by transferring loads to soil or rock at some depth below the structure by end bearing, adhesion or friction, or both.

DMT—Flat Plate Dilatometer Test.

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Drilled Shaft—A deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacity from the surrounding soil and/or from the soil or rock strata below its tip. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles, or drilled piers.

Effective Stress—The net stress across points of contact of soil particles, generally considered as equivalent to the total stress minus the pore water pressure.

ER—Hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used in a Standard Penetration Test.

Free (Unbonded) Length—The designed length of a micropile that is not bonded to the surrounding ground or grout.

Friction Pile—A pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile.

Geomechanics Rock Mass Rating System—Rating system developed to characterize the engineering behavior of rock masses (Bieniawski, 1984).

Geotechnical Bond Strength—The nominal grout-to-ground bond strength.

GSI—Geological Strength Index

IGM—Intermediate Geomaterial, a material that is transitional between soil and rock in terms of strength and compressibility, such as residual soils, glacial tills, or very weak rock.

Isolated Footing—Individual support for the various parts of a substructure unit; the foundation is called a footing foundation.

Length of Foundation—Maximum plan dimension of a foundation element.

Load Test—Incremental loading of a foundation element, recording the total movement at each increment.

Micropile—A small-diameter drilled and grouted non-displacement pile (normally less than 12.0 in.) that is typically reinforced.

OCR—Over Consolidation Ratio, the ratio of the preconsolidation pressure to the current vertical effective stress.

Pile—A slender deep foundation unit, wholly or partly embedded in the ground, that is installed by driving, drilling, auguring, jetting, or otherwise and that derives its capacity from the surrounding soil and/or from the soil or rock strata below its tip.

Pile Bent—A type of bent using pile units, driven or placed, as the column members supporting a cap.

Pile Cap—A flexural substructure element located above or below the finished ground line that receives loads from substructure columns and is supported by shafts or piles.

Pile Shoe—A metal piece fixed to the penetration end of a pile to protect it from damage during driving and to facilitate penetration through very dense material.

Piping—Progressive erosion of soil by seeping water that produces an open pipe through the soil through which water flows in an uncontrolled and dangerous manner.

Plunge Length—The length of casing inserted into the bond zone to effect a transition between the upper cased portion to the lower uncased portion of a micropile.

Plunging—A mode of behavior observed in some pile load tests, wherein the settlement of the pile continues to increase with no increase in load.

PMT—Pressuremeter Test.

Point-Bearing Pile—A pile whose support capacity is derived principally from the resistance of the foundation material on which the pile tip bears.

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Post Grouting—The injection of additional grout into the load bond length of a micropile after the primary grout has set. Also known as regrouting or secondary grouting.

Primary Grout—Portland cement-based grout that is injected into a micropile hole, prior to or after the installation of the reinforcement to provide the load transfer to the surrounding ground along the micropile and afford a degree of corrosion protection for a micropile loaded in compression.

Reinforcement—The steel component of a micropile which accepts and/or resists applied loadings.

RMR—Rock Mass Rating.

RQD—Rock Quality Designation.

Shallow Foundation—A foundation that derives its support by transferring load directly to the soil or rock at shallow depth.

Slickensides—Polished and grooved surfaces in clayey soils or rocks resulting from shearing displacements along planes.

SPT—Standard Penetration Test.

Total Stress—Total pressure exerted in any direction by both soil and water.

UU—Unconsolidated undrained.

VST—Vane Shear Test (performed in the field).

Width of Foundation—Minimum plan dimension of a foundation element.

10.3—NOTATION

A = steel pile cross-sectional area (ft^2) (10.7.3.8.2)
A_b = cross-sectional area of steel reinforcing bar (in.^{2}) (10.9.3.10.2a)
A_c = cross-sectional area of steel casing (in.^{2}) (10.9.3.10.2a)
A_{ct} = cross-sectional area of steel casing considering reduction for threads (in.^{2}) (10.9.3.10.3a)
A_g = cross-sectional area of grout within micropile (in.^{2}) (10.9.3.10.3a)
A_p = area of pile or micropile tip or base of drilled shaft (ft^2) (10.7.3.8.6a) (10.8.3.5) (10.9.3.5.1)
A_s = acceleration coefficient; surface area of pile shaft; area of grout to ground bond surface of micropile through bond length (ft^2) (10.5.4.2) (10.7.3.8.6a) (10.8.3.5) (10.9.3.5.1)
A_u = uplift area of a belled drilled shaft (ft^2) (10.8.3.7.2)
a_{si} = pile perimeter at the point considered (ft) (10.7.3.8.6g)
B = footing width; pile group width; pile diameter (ft) (10.6.1.3) (10.7.2.3.2) (10.7.2.4)
B_f = least width of footing (ft) (10.6.2.4.2c)
B' = effective footing width (ft) (10.6.1.3)
C = correction factor for concrete-soil interference (10.6.3.4)
C_α = secondary compression index, void ratio definition (dim) (10.4.6.3)
$C_{\alpha\varepsilon}$ = secondary compression index, strain definition (dim) (10.6.2.4.3)
C_c = compression index, void ratio definition (dim) (10.4.6.3)
$C_{c\varepsilon}$ = compression index, strain definition (dim) (10.6.2.4.3)
C_F = correction factor for K_δ when δ is not equal to ϕ_f (dim) (10.7.3.8.6f)
C_N = overburden stress correction factor for N (dim) (10.4.6.2.4)
C_r = recompression index, void ratio definition (dim) (10.4.6.3)
$C_{r\varepsilon}$ = recompression index, strain definition (dim) (10.6.2.4.3)
C_{wq}, $C_{w\gamma}$ = correction factors for groundwater effect (dim) (10.6.3.1.2a)
C' = bearing capacity index (dim) (10.6.2.4.2)
C_1 = correction factor to incorporate the effect of strain relief due to embedment (10.6.2.4.2c)
C_2 = correction factor to incorporate time-dependent (creep) increase in settlement for t (years) after construction (10.6.2.4.2c)
c = cohesion of soil taken as undrained shear strength (ksf) (10.6.3.1.2a)
c_v = coefficient of consolidation (ft^2/yr) (10.4.6.3)
c_1 = undrained shear strength of the top layer of soil as depicted in Figure 10.6.3.1.2e-1 (ksf) (10.6.3.1.2e)

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c_2 = undrained shear strength of the lower layer of soil as depicted in Figure 10.6.3.1.2e-1 (ksf) (10.6.3.1.2e)
c'_1 = drained shear strength of the top layer of soil in a two-layer system such as depicted in Figure 10.6.3.1.2e-1 (10.6.3.1.2f)
c^* = reduced effective stress soil cohesion for punching shear (ksf) (10.6.3.1.2b)
c' = effective stress cohesion intercept (ksf) (10.4.6.2.3)
c'_i = instantaneous cohesion at a discrete value of normal stress (ksf) (C10.4.6.4)
D = depth of pile embedment; pile width or diameter; diameter of drilled shaft (ft) (10.7.2.3) (10.7.3.8.6g) (10.8.3.5.1c)
DD = downdrag load per pile (kips) (C10.7.3.7)
D' = effective depth of pile or micropile group (ft) (10.7.2.3.2) (10.9.2.3.2)
D_b = depth of penetration into bearing strata (ft) (10.7.2.3.1) (10.7.3.8.6b) (10.7.3.8.6g)
$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile (ft) (C10.7.3.7)
D_f = footing embedment depth (ft) (10.6.3.1.2a) (10.6.3.1.2c)
D_i = pile width or diameter at the point considered (ft) (10.7.3.8.6g)
D_p = diameter of the bell (ft) (10.8.3.7.2)
D_r = relative density of sand (C10.6.3.1.2b)
D_w = depth to water surface taken from the ground surface (ft) (10.6.3.1.2a)
d_b = grouted bond zone diameter (ft) (10.9.3.5.2)
d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation (dim) (10.6.3.1.2a)
E = elastic modulus of layer i based on guidance provided in Table C10.4.6.3-1 or as specified in Table 10.6.2.4.2c-1 in Article 10.6.2.4.2c; modulus of elasticity of pile material (ksi) (C10.4.6.3) (10.6.2.4.2c) (10.7.3.8.2)
E_d = developed hammer energy (ft-lb) (10.7.3.8.5)
E_i = modulus of elasticity of intact rock (ksi) (10.4.6.5)
E_m = rock mass modulus (ksi) (10.4.6.5)
E_p = modulus of elasticity of pile (ksi) (10.7.3.13.4)
ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used (dim) (10.4.6.2.4)
E_s = soil (Young's) modulus (ksi) (C10.4.6.3)
e = void ratio (dim) (10.6.2.4.3)
e_B = eccentricity of load parallel to the width of the footing (ft) (10.6.1.3)
e_L = eccentricity of load parallel to the length of the footing (ft) (10.6.1.3)
e_o = void ratio at initial vertical effective stress (dim) (10.6.2.4.3)
F_{co} = base resistance of wood in compression parallel to the grain (ksi) (10.7.8)
f'_c = 28-day compressive strength of concrete or grout, unless another age is specified (ksi) (10.6.2.5.2) (10.7.8) (10.9.3.10.2a)
f_{pe} = effective prestressing stress in concrete (ksi) (10.7.8)
f_s = approximate constant sleeve friction resistance measured from a CPT at depths below $8D$ (ksf) (C10.7.3.8.6g)
f_{si} = unit local sleeve friction resistance from CPT at the point considered (ksf) (10.7.3.8.6g)
f_y = specified minimum yield strength of steel (ksi) (10.7.8) (10.9.3.10.2a)
H = horizontal component of inclined loads (kips) (10.6.3.1.2a)
H_c = initial height of layer i (ft); height of compressible soil layer i (ft) (10.6.2.4.2b) (10.6.2.4.2c) (10.6.2.4.3)
H_{crit} = minimum distance below a spread footing to a second separate layer of soil with different properties that will affect shear strength of the foundation (ft) (10.6.3.1.2d)
H_d = length of longest drainage path in compressible soil layer (ft) (10.6.2.4.3)
H_s = height of sloping ground mass below bottom of footing (ft) (10.6.3.1.2c)
H_{s2} = distance from bottom of footing to top of the second soil layer (ft) (10.6.3.1.2e)
h_i = length interval at the point considered (ft) (10.7.3.8.6g)
I = influence factor of the effective group embedment (dim) (10.7.2.3.2)
I_p = influence coefficient to account for rigidity and dimensions of footing (dim) (10.6.2.4.4)
I_w = weak axis moment of inertia for a pile (ft^4) (C10.7.3.13.4)
I_z = strain influence factor from Figure 10.6.2.4.2c-1a
i_c, i_q, i_γ = load inclination factors (dim) (10.6.3.1.2a)
K_c = correction factor for side friction in clay (dim) (10.7.3.8.6g)
K_s = correction factor for side friction in sand (dim) (10.7.3.8.6g)
K_δ = coefficient of lateral earth pressure at midpoint of soil layer under consideration (dim) (10.7.3.8.6f)
L = length of foundation; pile length (ft) (10.6.1.3) (10.6.3.1.2a) (10.7.3.8.2)

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L_b = micropile bonded length (ft) (10.9.3.5.2)
L_f = length of footing (ft) (10.6.2.4.2c)
L_i = depth to middle of length interval at the point considered (ft) (10.7.3.8.6g)
L_p = micropile casing plunge length (ft) (10.9.3.10.4)
L' = effective footing length (ft) (10.6.1.3)
LL = liquid limit of soil (percent) (10.4.6.3)
m_b, s, a = fractured rock mass parameters (10.4.6.4)
N = uncorrected Standard Penetration Test (*SPT*) blow count (blows/ft) (10.4.6.2.4)
$\overline{N1}_{60}$ = average corrected *SPT* blow count along pile side (blows/ft) (10.7.3.8.6g)
$N1$ = *SPT* blow count corrected for overburden pressure σ'_v (blows/ft) (10.4.6.2.4)
$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/ft) (10.4.6.2.4) (10.7.3.8.6g)
N_b = number of hammer blows for 1.0 in. of pile permanent set (blows/in.) (10.7.3.8.5)
N_c = cohesion term (undrained loading) bearing capacity factor (dim) (10.6.3.1.2a)
$N_{cm}, N_{qm}, N_{\gamma m}$ = modified bearing capacity factors (dim) (10.6.3.1.2a)
N_m = modified bearing capacity factor (dim) (10.6.3.1.2e)
N_s = slope stability factor (dim) (10.6.3.1.2c)
N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor (dim) (10.6.3.1.2a)
N_u = uplift adhesion factor for bell (dim) (10.8.3.7.2)
N'_q = pile bearing capacity factor from Figure 10.7.3.8.6f-8 (dim) (10.7.3.8.6f)
N_γ = unit weight (footing width) term (drained loading) bearing capacity factor (dim) (10.6.3.1.2a)
N_1 = number of intervals between the ground surface and a point 8*D* below the ground surface (dim) (10.7.3.8.6g)
N_2 = number of intervals between 8*D* below the ground surface and the tip of the pile (dim) (10.7.3.8.6g)
N_{60} = *SPT* blow count corrected for hammer efficiency (blows/ft) (10.4.6.2.4)
n = porosity (dim); number of soil layers within zone of stress influence of the footing (dim) (10.4.6.2.4) (10.6.2.4.2)
n_h = rate of increase of soil modulus with depth (ksi/ft) (10.4.6.3) (C10.7.3.13.4)
PL = plastic limit of soil (percent) (10.4.6.3)
P_f = probability of failure (dim) (C10.5.5.2.1)
P_m = *P*-multipliers from Table 10.7.2.4-1 (dim) (10.7.2.4)
P_t = factored axial load transferred to the ground through the plunge length of the cased portion of a micropile (kips) (10.9.3.10.4)
P_u = factored axial load on uncased micropile segment adjusted for plunge length load transfer (kips) (10.9.3.10.4)
p_a = atmospheric pressure (ksf) (Sea level value equivalent to 2.12 ksf or 1 atm or 14.7 psi) (10.8.3.5.1b) (10.8.3.5.2b)
p_o = effective in-situ overburden stress at the foundation depth (ksf) (10.6.2.4.2c)
p_{op} = effective in-situ overburden stress at the depth to peak strain influence factor, I_{zp}, as shown in Figure 10.6.2.4.2c-1b (ksf) (10.6.2.4.2c)
Q = load applied to top of footing, shaft, or micropile (kips); load test load (kips) (C10.6.3.1.2b) (10.7.3.8.2) (10.9.3.10.4)
Q_f = load at failure during load test (kips) (10.7.3.8.2)
Q_g = bearing capacity for block failure (kips) (C10.7.3.9)
Q_p = factored load per pile, excluding downdrag load (kips) (C10.7.3.7)
q = net foundation pressure applied at $2D_b/3$; this pressure is equal to applied load at top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (ksf) (10.7.2.3.2)
q_c = static cone tip resistance (ksf) (C10.4.6.3)
$\overline{q_c}$ = average static cone tip resistance over a depth *B* below the equivalent footing (ksf) (10.6.3.1.3)
q_{c1} = average q_c over a distance of *yD* below the pile tip (path a-b-c) (ksf) (10.7.3.8.6g)
q_{c2} = average q_c over a distance of 8*D* above the pile tip (path c-e) (ksf) (10.7.3.8.6g)
q_L = limiting unit tip resistance from Figure 10.7.3.8.6f-9 (ksf) (10.7.3.8.6f)
q_ℓ = limiting pile tip resistance(ksf) (10.7.3.8.6g)
q_n = nominal bearing resistance (ksf) (10.6.3.1.1)

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$q_{n\text{-sloping ground}}$ = nominalbearing resistance of footings constructed on or adjacent to slopes (ksf) (10.6.3.1.2c)
q_o = applied vertical stress at base of loaded area (ksf) (10.6.2.4.4)
q_p = nominal unit tip resistance of pile or micropile (ksf) (10.7.3.8.6a) (10.9.3.5.1)
q_R = factored bearing resistance (ksf) (10.6.3.1.1)
q_s = unit shear resistance (ksf); unit side resistance of pile or micropile (ksf); unit grout-to-ground bond resistance (ksf) (10.6.3.4) (10.7.3.8.6a) (10.7.3.8.6g) (10.9.3.5.1)
q_{sbell} = nominal unit uplift resistance of a belled drilled shaft (ksf) (10.8.3.7.2)
q_u = average unconfined compressive strength of rock core (ksf) (10.4.6.4)
q_{ult} = nominal bearing resistance (ksf) (10.6.3.1.2e)
q_1 = nominal bearing resistance of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (ksf) (10.6.3.1.2d)
q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (ksf) (10.6.3.1.2d) (10.6.3.1.2f)
R_C = factored structural resistance of a micropile to axial compression loading (kips) (10.9.3.10.2)
RC_{BC} = reduction coefficient for bearing resistance of footings due to slope effects (dim) (10.6.3.1.2c)
R_{CC} = factored structural axial compression resistance of cased micropile segments (kips) (10.9.3.10.2a)
R_{CU} = factored structural axial compression resistance of uncased micropile segments (kips) (10.9.3.10.2b)
R_{ep} = nominal passive resistance of soil available throughout the design life of the structure (kips) (10.6.3.4)
R_n = nominal resistance of footing, pile, shaft, or micropile (kips) (10.6.3.4) (10.9.3.5.1)
R_{ndr} = nominal pile driving resistance including downdrag (kips) (C10.7.3.3) (10.7.3.8.5)
R_{nstat} = predicted nominal resistance of pile from static analysis method (kips) (C10.7.3.3)
R_p = nominal pile or micropile tip resistance (kips) (10.7.3.8.6a) (10.8.3.5) (10.9.3.5.1)
R_R = factored nominal resistance of a footing, pile, micropile, or shaft (kips) (10.6.3.4) (10.9.3.5.1)
R_s = pile side resistance (kips); nominal uplift resistance due to side resistance (kips); nominal micropile grout-to-ground bond resistance (kips) (10.7.3.8.6a) (10.7.3.10) (10.9.3.5.1)
R_{sbell} = nominal uplift resistance of a belled drilled shaft (kips) (10.8.3.7.2)
R_{sdd} = side resistance which must be overcome during driving through downdrag zone (kips) (C10.7.3.7)
R_T = factored structural axial tension resistance (kips) (10.9.3.10.3)
R_{TC} = factored structural axial tension resistance of cased micropile segments (kips) (10.9.3.10.3a)
R_{TU} = factored structural axial tension resistance of uncased micropile segments (kips) (10.9.3.10.3b)
R_{ug} = nominal uplift resistance of a pile group (kips) (10.7.3.11)
R_τ = nominal sliding resistance between the footing and the soil (kips) (10.6.3.4)
r = radius of circular footing or B/2 for square footing (ft) (10.6.2.4.4)
S_c = primary consolidation settlement (ft) (10.6.2.4.1)
$S_{c(1\text{-}D)}$ = single dimensional consolidation settlement (ft) (10.6.2.4.3)
S_e = elastic (or immediate) settlement (ft) (10.6.2.4.1)
S_i = elastic (or immediate) settlement based on Schmertmann method (ft) (10.6.2.4.2c)
S_s = secondary settlement (ft) (10.6.2.4.1)
S_t = total settlement (ft) (10.6.2.4.1)
S_u = undrained shear strength (ksf) (C10.4.6.2.2) (C10.7.3.13.4)
$\overline{S_u}$ = average undrained shear strength along pile side (ksf) (10.7.3.9)
s = pile permanent set (in.) (10.7.3.8.5)
s_c, s_γ,s_q = shape factors (dim) (10.6.3.1.2a)
s_f = pile top movement during load test (in.) (10.7.3.8.2)
T = time factor (dim) (10.6.2.4.3)
t = time from completion of construction to date under consideration for evaluation of C_2 (years); time for a given percentage of one-dimensional consolidation settlement to occur (years) (10.6.2.4.2c) (10.6.2.4.3)
t_1, t_2 = arbitrary time intervals for determination of secondary settlement, S_s (years) (10.6.2.4.3)
U = percentage of consolidation (10.6.4.2.3)
V = total vertical force applied by a footing (kips); pile displacement volume (ft^3/ft) (10.6.3.1.2a) (10.6.3.4) (10.7.3.8.6f)
v = Poisson's ratio (C10.4.6.5)
W_g = weight of block of soil, piles and pile cap (kips) (10.7.3.11)
X = a factor used to determine the value of elastic modulus; width or smallest dimension of pile group (ft) (10.6.2.4.2c) (10.7.3.11)
Y = length of pile group (ft) (10.7.3.11)
Z = total embedded pile length; penetration of shaft (ft) (C10.7.3.8.6g) (10.7.3.11) (10.8.3.5.1c)
z = depth below ground surface (ft) (C10.4.6.3)

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α = adhesion factor applied to s_u (dim) (10.7.3.8.6b)
α_b = nominal micropile grout-to-ground bond stress (ksf) (10.9.3.5.2)
α_E = reduction factor to account for jointing in rock (dim) (10.8.3.5.4b)
α_t = coefficient from Figure 10.7.3.8.6f-7 (dim) (10.7.3.8.6f)
β = reliability index; coefficient relating the vertical effective stress and the unit skin friction of a pile or drilled shaft; load transfer coefficient (dim) (C10.5.5.2.1) (10.7.3.8.6c) (10.8.3.5.2b)
β_m = punching index (dim) (10.6.3.1.2e)
β_z = factor to account for footing shape and rigidity (dim) (10.6.2.4.4)
γ = unit density of soil (kcf) (10.6.3.1.2c)
γ_f = total (moist) unit weight of soil below the bearing depth of the footing (kcf) (10.6.3.1.2a)
γ_p = load factor for downdrag (C10.7.3.7)
γ_q = total (moist) unit weight of soil above the bearing depth of the footing (kcf) (10.6.3.1.2a)
γ_{SE} = load factor for settlement (C10.5.2.2.2) (C10.6.2.4.2b)
ΔH_i = elastic settlement of layer i (ft) (10.6.2.4.2)
ΔJ_i = elastic spring stiffness of layer i (ft/ksf) (10.6.2.4.2c)
Δp = net uniform applied stress (load intensity) at the foundation depth as shown in Figure 10.6.2.4.2c-1b in which p is equal to the uniform applied footing stress, σ_v, as specified in Article 11.6.3.2 (ksf) (10.6.2.4.2c)
δ = elastic deformation of pile (in.); friction angle between foundation and soil (degrees) (C10.7.3.8.2) (10.7.3.8.6f)
ε_v = vertical strain of over consolidated soil (in./in.) (10.6.2.4.3)
η = shaft efficiency reduction factor for axial resistance of a drilled shaft or micropile group (dim) (10.7.3.9)
θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees) (10.6.3.1.2a)
λ = empirical coefficient relating the passive lateral earth pressure and the unit skin friction of a pile (dim) (10.7.3.8.6d)
μ_c = reduction factor for consolidation settlements to account for three-dimensional effects (dim) (10.6.2.4.3)
σ'_p = maximum past vertical effective stress in soil at midpoint of soil layer under consideration (ksf) (10.6.2.4.3)
σ'_o = initial vertical effective stress in soil at midpoint of soil layer under consideration (ksf) (10.6.2.4.3)
ϕ_f = angle of internal friction of drained soil (degrees) (C10.4.6.2.4)
ϕ'_f = drained (long term) effective angle of internal friction of clays (degrees) (10.4.6.2.3)
ϕ'_i = instantaneous friction angle of the rock mass (degrees) (10.4.6.4)
ϕ'_1 = effective stress angle of internal friction of the top layer of soil (degrees) (10.6.3.1.2f)
ϕ'_s = secant friction angle (degrees) (10.4.6.2.4)
ϕ^* = reduced effective stress soil friction angle for punching shear (degrees) (10.6.3.1.2b)
φ = resistance factor (dim) (10.5.5.2.3) (10.6.3.4) (10.9.3.5.1)
φ_b = resistance factor for bearing of shallow foundations (dim) (10.5.5.2.2)
φ_{bl} = resistance factor for driven piles or shafts, block failure in clay (dim) (10.5.5.2.3)
φ_C = structural resistance factor for micropiles in axial compression (dim) (10.9.3.10.2)
φ_{CC} = structural resistance factor for cased micropiles segments in axial compression (dim) (10.5.5.2.5) (10.9.3.10.2a)
φ_{CU} = structural resistance factor for uncased micropiles segments in axial compression (dim) (10.5.5.2.5) (10.9.3.10.2b)
φ_{da} = resistance factor for driven piles, drivability analysis (dim) (10.5.5.2.3)
φ_{dyn} = resistance factor for driven piles, dynamic analysis and static load test methods (dim) (10.5.5.2.3) (C10.7.3.7)
φ_{ep} = resistance factor for passive soil resistance (dim) (10.5.5.2.2)
φ_{load} = resistance factor for shafts, static load test (dim) (10.5.5.2.4)
φ_{qp} = resistance factor for tip resistance (dim) (10.8.3.5) (10.9.3.5.1)
φ_{qs} = resistance factor for shaft side resistance (dim) (10.8.3.5)
φ_{stat} = resistance factor for driven piles or shafts, static analysis methods (dim) (10.5.5.2.3)
φ_T = structural resistance factor for micropiles in axial tension (dim) (10.9.3.10.3)
φ_{TC} = structural resistance factor for cased micropiles segments in axial tension (dim) (10.5.5.2.5) (10.9.3.10.3a)
φ_{TU} = structural resistance factor for uncased micropiles segments in axial tension (dim) (10.5.5.2.5) (10.9.3.10.3b)

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φ_{ug} = resistance factor for group uplift (dim) (10.5.5.2.3)
φ_{up} = resistance factor for uplift resistance of a single pile or drilled shaft (dim) (10.5.5.2.3)
φ_{upload} = resistance factor for shafts, static uplift load test (dim) (10.5.5.2.4) (10.9.3.5.1)
φ_{τ} = resistance factor for sliding resistance between soil and footing (dim) (10.5.5.2.2)

10.4—SOIL AND ROCK PROPERTIES

10.4.1—Informational Needs

The expected project requirements shall be analyzed to determine the type and quantity of information to be developed during the geotechnical exploration. This analysis should consist of the following:

- Identify design and constructability requirements, e.g., provide grade separation, support loads from bridge superstructure, provide for dry excavation, and their effect on the geotechnical information needed.
- Identify performance criteria, e.g., limiting settlements, right of way restrictions, proximity of adjacent structures, and schedule constraints.
- Identify areas of geologic concern on the site and potential variability of local geology.
- Identify areas of hydrologic concern on the site, e.g., potential erosion or scour locations.
- Develop likely sequence and phases of construction and their effect on the geotechnical information needed.
- Identify engineering analyses to be performed, e.g., bearing capacity, settlement, global stability.
- Identify engineering properties and parameters required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type and construction methods.
- Determine the number of tests/samples needed and appropriate locations for them.

C10.4.1

The first phase of an exploration and testing program requires that the Engineer understand the project requirements and the site conditions and/or restrictions. The ultimate goal of this phase is to identify geotechnical data needs for the project and potential methods available to assess these needs.

Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties (Sabatini et al., 2002) provides a summary of information needs and testing considerations for various geotechnical applications.

10.4.2—Subsurface Exploration

Subsurface explorations shall be performed to provide the information needed for the design and construction of foundations. The extent of exploration shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction. The exploration program should be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the potential for liquefaction, and the groundwater conditions. The exploration program should be sufficient to identify and delineate problematic subsurface conditions such as karstic formations, mined out areas, swelling/collapsing soils, existing fill or waste areas, etc.

Borings should be sufficient in number and depth to establish a reliable longitudinal and transverse substrata

C10.4.2

The performance of a subsurface exploration program is part of the process of obtaining information relevant for the design and construction of substructure elements. The elements of the process that should precede the actual exploration program include a search and review of published and unpublished information at and near the site, a visual site inspection, and design of the subsurface exploration program. Refer to Mayne et al. (2001) and Sabatini et al. (2002) for guidance regarding the planning and conduct of subsurface exploration programs.

The suggested minimum number and depth of borings are provided in Table 10.4.2-1. While engineering judgment will need to be applied by a licensed and experienced geotechnical professional to adapt the exploration program to the foundation types and depths needed and to the variability in the subsurface

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profile at areas of concern such as at structure foundation locations and adjacent earthwork locations, and to investigate any adjacent geologic hazards that could affect the structure performance.

As a minimum, the subsurface exploration and testing program shall obtain information adequate to analyze foundation stability and settlement with respect to:

- geological formation(s) present,
- location and thickness of soil and rock units,
- engineering properties of soil and rock units, such as unit weight, shear strength and compressibility,
- groundwater conditions,
- ground surface topography, and
- local considerations, e.g., liquefiable, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential.

Table 10.4.2-1 shall be used as a starting point for determining the locations of borings. The final exploration program should be adjusted based on the variability of the anticipated subsurface conditions as well as the variability observed during the exploration program. If conditions are determined to be variable, the exploration program should be increased relative to the requirements in Table 10.4.2-1 such that the objective of establishing a reliable longitudinal and transverse substrata profile is achieved. If conditions are observed to be homogeneous or otherwise are likely to have minimal impact on the foundation performance, and previous local geotechnical and construction experience has indicated that subsurface conditions are homogeneous or otherwise are likely to have minimal impact on the foundation performance, a reduced exploration program relative to what is specified in Table 10.4.2-1 may be considered.

If requested by the Owner or as required by law, boring and penetration test holes shall be plugged.

Laboratory tests, in-situ tests, or both shall be performed to determine the strength, deformation, and permeability characteristics of soils and/or rocks and their suitability for the foundation proposed.

conditions observed, the intent of Table 10.4.2-1 regarding the minimum level of exploration needed should be carried out. The depth of borings indicated in Table 10.4.2-1 performed before or during design should take into account the potential for changes in the type, size and depth of the planned foundation elements.

This Table should be used only as a first step in estimating the number of borings for a particular design, as actual boring spacings will depend upon the project type and geologic environment. In areas underlain by heterogeneous soil deposits and/or rock formations, it will probably be necessary to drill more frequently and/or deeper than the minimum guidelines in Table 10.4.2-1 to capture variations in soil and/or rock type and to assess consistency across the site area. For situations where large diameter rock socketed shafts will be used or where drilled shafts are being installed in formations known to have large boulders, or voids such as in karstic or mined areas, it may be necessary to advance a boring at the location of each shaft. Even the best and most detailed subsurface exploration programs may not identify every important subsurface problem condition if conditions are highly variable. The goal of the subsurface exploration program, however, is to reduce the risk of such problems to an acceptable minimum.

In a laterally homogeneous area, drilling or advancing a large number of borings may be redundant, since each sample tested would exhibit similar engineering properties. Furthermore, in areas where soil or rock conditions are known to be very favorable to the construction and performance of the foundation type likely to be used, e.g., footings on very dense soil, and groundwater is deep enough to not be a factor, obtaining fewer borings than provided in Table 10.4.2-1 may be justified. In all cases, it is necessary to understand how the design and construction of the geotechnical feature will be affected by the soil and/or rock mass conditions in order to optimize the exploration.

Borings may need to be plugged due to requirements by regulatory agencies having jurisdiction and/or to prevent water contamination and/or surface hazards.

Parameters derived from field tests, e.g., driven pile resistance based on cone penetrometer testing, may also be used directly in design calculations based on empirical relationships. These are sometimes found to be more

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reliable than analytical calculations, especially in familiar ground conditions for which the empirical relationships are well-established.

Table 10.4.2-1—Minimum Number of Exploration Points and Depth of Exploration (modified after Sabatini et al., 2002)

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining Walls	A minimum of one exploration point for each retaining wall. For retaining walls more than 100 ft in length, exploration points spaced every 100 to 200 ft with locations alternating from in front of the wall to behind the wall. For anchored walls, additional exploration points in the anchorage zone spaced at 100 to 200 ft. For soil-nailed walls, additional exploration points at a distance of 1.0 to 1.5 times the height of the wall behind the wall spaced at 100 to 200 ft.	Investigate to a depth below bottom of wall at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth and between one and two times the wall height. Exploration depth should be great enough to fully penetrate soft highly compressible soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing capacity, e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock.
Shallow Foundations	For substructure, e.g., piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered.	Depth of exploration should be: • great enough to fully penetrate unsuitable foundation soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing resistance, e.g., stiff to hard cohesive soil, or compact to dense cohesionless soil or bedrock ; • at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth; and • if bedrock is encountered before the depth required by the second criterion above is achieved, exploration depth should be great enough to penetrate a minimum of 10 ft into the bedrock, but rock exploration should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.
Deep Foundations	For substructure, e.g., bridge piers or abutments, widths less than or equal to 100 ft, a minimum of one exploration point per substructure. For substructure widths greater than 100 ft, a minimum of two exploration points per substructure. Additional exploration points should be provided if erratic subsurface conditions are encountered, especially for the case of shafts socketed into bedrock. To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft in diameter), especially when shafts are socketed into bedrock.	In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft, or a minimum of two times the minimum pile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. For piles bearing on rock, a minimum of 10 ft of rock core shall be obtained at each exploration point location to verify that the boring has not terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 ft of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the minimum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft or rock core may be required to verify that adequate quality bedrock is present.

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10.4.3—Laboratory Tests

10.4.3.1—Soil Tests

Laboratory testing should be conducted to provide the basic data with which to classify soils and to measure their engineering properties.

When performed, laboratory tests shall be conducted in accordance with the AASHTO, ASTM, or Owner-supplied procedures applicable to the design properties needed.

C10.4.3.1

Laboratory tests of soils may be grouped broadly into two general classes:

- Classification or index tests. These may be performed on either disturbed or undisturbed samples.
- Quantitative or performance tests for permeability, compressibility and shear strength. These tests are generally performed on undisturbed samples, except for materials to be placed as controlled fill or materials that do not have a stable soil-structure, e.g., cohesionless materials. In these cases, tests should be performed on specimens prepared in the laboratory.

Detailed information regarding the types of tests needed for foundation design is provided in Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties (Sabatini et al., 2002).

10.4.3.2—Rock Tests

If laboratory strength tests are conducted on intact rock samples for classification purposes, they should be considered as upper bound values. If laboratory compressibility tests are conducted, they should be considered as lower bound values. Additionally, laboratory tests should be used in conjunction with field tests and field characterization of the rock mass to give estimates of rock mass behavioral characteristics. When performed, laboratory tests shall be conducted in accordance with the ASTM or Owner-supplied procedures applicable to the design properties needed.

C10.4.3.2

Rock samples small enough to be tested in the laboratory are usually not representative of the entire rock mass. Laboratory testing of rock is used primarily for classification of intact rock samples, and, if performed properly, serves a useful function in this regard.

Detailed information regarding the types of tests needed and their use for foundation design is provided in Geotechnical Engineering Circular #5—Evaluation of Soil and Rock Properties, April 2002 (Sabatini et al., 2002).

10.4.4—In-Situ Tests

In-situ tests may be performed to obtain deformation and strength parameters of foundation soils or rock for the purposes of design and/or analysis. In-situ tests should be conducted in soils that do not lend themselves to undisturbed sampling as a means to estimate soil design parameters. When performed, in-situ tests shall be conducted in accordance with the appropriate ASTM or AASHTO standards.

Where in-situ test results are used to estimate design properties through correlations, such correlations should be well established through long-term widespread use or through detailed measurements that illustrate the accuracy of the correlation.

C10.4.4

Detailed information on in-situ testing of soils and rock and their application to geotechnical design can be found in Sabatini et al. (2002) and Wyllie (1999).

Correlations are in some cases specific to a geological formation. While this fact does not preclude the correlation from being useful in other geologic formations, the applicability of the correlation to those other formations should be evaluated.

For further discussion, see Article 10.4.6.

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10.4.5—Geophysical Tests

Geophysical testing should be used only in combination with information from direct methods of exploration, such as *SPT, CPT*, or other direct methods of exploration approved by the owner, to establish:

- stratification of the subsurface materials,
- the profile of the top of bedrock and bedrock quality,
- depth to groundwater,
- limits of types of soil deposits,
- the presence of voids,
- anomalous deposits,
- buried pipes, and
- depths of existing foundations.

Geophysical tests shall be selected and conducted in accordance with available ASTM standards. For those cases where ASTM standards are not available, other widely-accepted detailed guidelines, such as Sabatini et al. (2002), AASHTO *Manual on Subsurface Investigations* (1988), Arman et al. (1997) and Campanella (1994), should be used.

C10.4.5

Geophysical testing offers some notable advantages and some disadvantages that should be considered before the technique is recommended for a specific application. The advantages are summarized as follows:

- Many geophysical tests are noninvasive and thus, offer, significant benefits in cases where conventional drilling, testing and sampling are difficult, e.g., deposits of gravel, talus deposits, or where potentially contaminated subsurface soils may occur.
- In general, geophysical testing covers a relatively large area, thus providing the opportunity to generally characterize large areas in order to optimize the locations and types of in-situ testing and sampling. Geophysical methods are particularly well suited to projects that have large longitudinal extent compared to lateral extent, e.g., new highway construction.
- Geophysical measurement assesses the characteristics of soil and rock at very small strains, typically on the order of 0.001 percent, thus providing information on truly elastic properties, which are used to evaluate service limit states.
- For the purpose of obtaining subsurface information, geophysical methods are relatively inexpensive when considering cost relative to the large areas over which information can be obtained.

Some of the disadvantages of geophysical methods include:

- Most methods work best for situations in which there is a large difference in stiffness or conductivity between adjacent subsurface units.
- It is difficult to develop good stratigraphic profiling if the general stratigraphy consists of hard material over soft material or resistive material over conductive material.
- Results are generally interpreted qualitatively and, therefore, only an experienced engineer or geologist familiar with the particular testing method can obtain useful results.
- Specialized equipment is required (compared to more conventional subsurface exploration tools).
- Since evaluation is performed at very low strains, or no strain at all, information regarding ultimate strength for evaluation of strength limit states is only obtained by correlation.

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There are a number of different geophysical in-situ tests that can be used for stratigraphic information and determination of engineering properties. These methods can be combined with each other and/or combined with the in-situ tests presented in Article 10.4.4 to provide additional resolution and accuracy. ASTM D6429, Standard Guide for Selecting Surface Geophysical Methods, provides additional guidance on selection of suitable methods.

10.4.6—Selection of Design Properties

10.4.6.1—General

Subsurface soil or rock properties shall be determined using one or more of the following methods:

- In-situ testing during the field exploration program, including consideration of any geophysical testing conducted,
- Laboratory testing, and
- Back analysis of design parameters based on site performance data.

Local experience, local geologic formation specific property correlations, and knowledge of local geology, in addition to broader-based experience and relevant published data, should also be considered in the final selection of design parameters. If published correlations are used in combination with one of the methods listed above, the applicability of the correlation to the specific geologic formation shall be considered through the use of local experience, local test results, and/or long-term experience.

The focus of geotechnical design property assessment and final selection shall be on the individual geologic strata identified at the project site.

The design values selected for the parameters should be appropriate to the particular limit state and its correspondent calculation model under consideration.

The determination of design parameters for rock shall take into consideration that rock mass properties are generally controlled by the discontinuities within the rock mass and not the properties of the intact material. Therefore, engineering properties for rock should account for the properties of the intact pieces and for the properties of the rock mass as a whole, specifically considering the discontinuities within the rock mass. A combination of laboratory testing of small samples, empirical analysis, and field observations should be employed to determine the engineering properties of rock masses, with greater emphasis placed on visual observations and quantitative descriptions of the rock mass.

C10.4.6.1

A geologic stratum is characterized as having the same geologic depositional history and stress history, and generally has similarities throughout the stratum in terms of density, source material, stress history, and hydrogeology. The properties of a given geologic stratum at a project site are likely to vary significantly from point to point within the stratum. In some cases, a measured property value may be closer in magnitude to the measured property value in an adjacent geologic stratum than to the measured properties at another point within the same stratum. However, soil and rock properties for design should not be averaged across multiple strata.

It should also be recognized that some properties, e.g., undrained shear strength in normally consolidated clays, may vary as a predictable function of a stratum dimension, e.g., depth below the top of the stratum. Where the property within the stratum varies in this manner, the design parameters should be developed taking this variation into account, which may result in multiple values of the property within the stratum as a function of a stratum dimension such as depth.

The observational method, or use of back analysis, to determine engineering properties of soil or rock is often used with slope failures, embankment settlement or excessive settlement of existing structures. With landslides or slope failures, the process generally starts with determining the geometry of the failure and then determining the soil/rock parameters or subsurface conditions that result from a combination of load and resistance factors that approach 1.0. Often the determination of the properties is aided by correlations with index tests or experience on other projects. For embankment settlement, a range of soil properties is generally determined based on laboratory performance testing on undisturbed samples. Monitoring of fill settlement and pore pressure in the soil during construction allows the soil properties and prediction of the rate of future settlement to be refined. For structures such as bridges that experience unacceptable settlement or retaining walls that have excessive deflection, the engineering properties of the soils can sometimes be determined if the magnitudes of the loads are known. As with slope stability analysis, the subsurface stratigraphy must be adequately known, including the history of the groundwater level at the site.

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Local geologic formation-specific correlations may be used if well-established by data comparing the prediction from the correlation to measured high quality laboratory performance data, or back-analysis from full scale performance of geotechnical elements affected by the geologic formation in question.

The Engineer should assess the variability of relevant data to determine if the observed variability is a result of inherent variability of subsurface materials and testing methods or if the variability is a result of significant variations across the site. Methods to compare soil parameter variability for a particular project to published values of variability based on database information of common soil parameters are presented in Sabatini (2002) and Duncan (2000). Where the variability is deemed to exceed the inherent variability of the material and testing methods, or where sufficient relevant data is not available to determine an average value and variability, the Engineer may perform a sensitivity analysis using average parameters and a parameter reduced by one standard deviation, i.e., "mean minus 1 sigma," or a lower bound value. By conducting analyses at these two potential values, an assessment is made of the sensitivity of the analysis results to a range of potential design values. If these analyses indicate that acceptable results are provided and that the analyses are not particularly sensitive to the selected parameters, the Engineer may be comfortable with concluding the analyses. If, on the other hand, the Engineer determines that the calculation results are marginal or that the results are sensitive to the selected parameter, additional data collection/review and parameter selection are warranted.

When evaluating service limit states, it is often appropriate to determine both upper and lower bound values from the relevant data, since the difference in displacement of substructure units is often more critical to overall performance than the actual value of the displacement for the individual substructure unit.

For strength limit states, average measured values of relevant laboratory test data, in-situ test data, or both were used to calibrate the resistance factors provided in Article 10.5, at least for those resistance factors developed using reliability theory, rather than a lower bound value. It should be recognized that to be consistent with how the resistance factors presented in Article 10.5.5.2 were calibrated, i.e., to average property values, accounting for the typical variability in the property, average property values for a given geologic unit should be selected. However, depending on the availability of soil or rock property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the Engineer may have no choice but to use a more conservative selection of design parameters to mitigate the additional risks created by potential variability or the paucity of relevant data. Note that for those resistance factors that were determined based on calibration by fitting to allowable

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stress design, this property selection issue is not relevant, and property selection should be based on past practice.

10.4.6.2—Soil Strength

10.4.6.2.1—General

The selection of soil shear strength for design should consider, at a minimum, the following:

- the rate of construction loading relative to the hydraulic conductivity of the soil, i.e., drained or undrained strengths;
- the effect of applied load direction on the measured shear strengths during testing;
- the effect of expected levels of deformation for the geotechnical structure; and
- the effect of the construction sequence.

C10.4.6.2.1

Refer to Sabatini et al. (2002) for additional guidance on determining which soil strength parameters are appropriate for evaluating a particular soil type and loading condition. In general, where loading is rapid enough and/or the hydraulic conductivity of the soil is low enough such that excess pore pressure induced by the loading does not dissipate, undrained (total) stress parameters should be used. Where loading is slow enough and/or the hydraulic conductivity of the soil is great enough such that excess pore pressures induced by the applied load dissipate as the load is applied, drained (effective) soil parameters should be used. Drained (effective) soil parameters should also be used to evaluate long term conditions where excess pore pressures have been allowed to dissipate or where the designer has explicit knowledge of the expected magnitude and distribution of the excess pore pressure.

10.4.6.2.2—Undrained Strength of Cohesive Soils

Where possible, laboratory consolidated undrained (CU) and unconsolidated undrained (UU) testing should be used to estimate the undrained shear strength, S_u, supplemented as needed with values determined from in-situ testing. Where collection of undisturbed samples for laboratory testing is difficult, values obtained from in-situ testing methods may be used. For relatively thick deposits of cohesive soil, profiles of S_u as a function of depth should be obtained so that the deposit stress history and properties can be ascertained.

C10.4.6.2.2

For design analyses of short-term conditions in normally to lightly overconsolidated cohesive soils, the undrained shear strength, S_u, is commonly evaluated. Since undrained strength is not a unique property, profiles of undrained strength developed using different testing methods will vary. Typical transportation project practice entails determination of S_u based on laboratory CU and UU testing and, for cases where undisturbed sampling is very difficult, field vane testing. Other in-situ methods can also be used to estimate the value of S_u.

Specific issues that should be considered when estimating the undrained shear strength are described below:

- Strength measurements from hand torvanes, pocket penetrometers, or unconfined compression tests should not be solely used to evaluate undrained shear strength for design analyses. Consolidated undrained (CU) triaxial tests and in-situ tests should be used.
- For relatively deep deposits of cohesive soil, e.g., approximately 20.0 ft depth or more, all available undrained strength data should be plotted with depth. The type of test used to evaluate each undrained strength value should be clearly identified. Known soil layering should be used so that trends in undrained strength data can be developed for each soil layer.
- Review data summaries for each laboratory strength test method. Moisture contents of specimens for strength testing should be compared to moisture contents of other samples at similar depths. Significant changes in moisture content will affect measured undrained strengths. Review boring logs, Atterberg

limits, grain size, and unit weight measurements to confirm soil layering.

- CU tests on normally to slightly over consolidated samples that exhibit disturbance should contain at least one specimen consolidated to at least $4\sigma_p'$ to permit extrapolation of the undrained shear strength at σ_p'.
- Undrained strengths from CU tests correspond to the effective consolidation pressure used in the test. This effective stress needs to be converted to the equivalent depth in the ground.
- A profile of σ_p' (or OCR) should be developed and used in evaluating undrained shear strength.
- Correlations for S_u based on in-situ test measurements should not be used for final design unless they have been calibrated to the specific soil profile under consideration. Correlations for S_u based on *SPT* tests should be avoided.

10.4.6.2.3—Drained Strength of Cohesive Soils

Long-term effective stress strength parameters, c' and ϕ'_f, of clays should be evaluated by slow consolidated drained direct shear box tests, consolidated drained (CD) triaxial tests, or consolidated undrained (CU) triaxial tests with pore pressure measurements. In laboratory tests, the rate of shearing should be sufficiently slow to ensure substantially complete dissipation of excess pore pressure in the drained tests or, in undrained tests, complete equalization of pore pressure throughout the specimen.

C10.4.6.2.3

The selection of peak, fully softened, or residual strength for design analyses should be based on a review of the expected or tolerable displacements of the soil mass.

The use of a nonzero cohesion intercept, c', for long-term analyses in natural materials must be carefully assessed. With continuing displacements, it is likely that the cohesion intercept value will decrease to zero for long-term conditions, especially for highly plastic clays.

10.4.6.2.4—Drained Strength of Granular Soils

The drained friction angle of granular deposits should be evaluated by correlation to the results of *SPT* testing, *CPT* testing, or other relevant in-situ tests. Laboratory shear strength tests on undisturbed samples, if feasible to obtain, or reconstituted disturbed samples, may also be used to determine the shear strength of granular soils.

If *SPT N* values are used, unless otherwise specified for the design method or correlation being used, they shall be corrected for the effects of overburden pressure determined as:

$$N1 = C_N N \qquad (10.4.6.2.4\text{-}1)$$

$N1$ = *SPT* blow count corrected for overburden pressure, σ'_v (blows/ft)
C_N = $[0.77 \log_{10}(40/\sigma'_v)]$, and $C_N < 2.0$
σ'_v = vertical effective stress (ksf)
N = uncorrected *SPT* blow count (blows/ft)

C10.4.6.2.4

Because obtaining undisturbed samples of granular deposits for laboratory testing is extremely difficult, the results of in-situ tests are commonly used to develop estimates of the drained friction angle, ϕ_f. If reconstituted disturbed soil samples and laboratory tests are used to estimate the drained friction angle, the reconstituted samples should be compacted to the same relative density estimated from the available in-situ data. The test specimen should be large enough to allow the full grain size range of the soil to be included in the specimen. This may not always be possible, and if not possible, it should be recognized that the shear strength measured would likely be conservative.

A method using the results of *SPT* testing is presented. Other in-situ tests, such as *CPT* and *DMT*, may be used. For details on determinatfion of ϕ_f from these tests, refer to Sabatini et al. (2002).

SPT N values should also be corrected for hammer efficiency, if applicable to the design method or correlation being used, determined as:

The use of automatic trip hammers is increasing. In order to use correlations based on standard rope and cathead hammers, the *SPT N* values must be corrected to

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$$N_{60} = \left(\frac{ER}{60\%}\right)N \quad (10.4.6.2.4\text{-}2)$$

where:

N_{60} = *SPT* blow count corrected for hammer efficiency (blows/ft)
ER = hammer efficiency expressed as percent of theoretical free fall energy delivered by the hammer system actually used (dim)
N = uncorrected *SPT* blow count (blows/ft)

When *SPT* blow counts have been corrected for both overburden effects and hammer efficiency effects, the resulting corrected blow count shall be denoted as $N1_{60}$, determined as:

$$N1_{60} = C_N N_{60} \quad (10.4.6.2.4\text{-}3)$$

The drained friction angle of granular deposits should be determined based on the following correlation.

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

For gravels and rock fill materials where *SPT* testing is not reliable, Figure 10.4.6.2.4-1 should be used to estimate the drained friction angle.

Rock Fill Grade	Particle Unconfined Compressive Strength (ksf)
A	>4,610
B	3,460–4,610
C	2,590– 3,460
D	1,730– 2,590
E	≤1,730

reflect the greater energy delivered to the sampler by these systems.

Hammer efficiency (ER) for specific hammer systems used in local practice may be used in lieu of the values provided. If used, specific hammer system efficiencies shall be developed in general accordance with ASTM D4945 for dynamic analysis of driven piles or other accepted procedure.

The following values for ER may be assumed if hammer specific data are not available, e.g., from older boring logs:

ER = 60 percent for conventional drop hammer using rope and cathead
ER = 80 percent for automatic trip hammer

Corrections for rod length, hole size, and use of a liner may also be made if appropriate. In general, these are only significant in unusual cases or where there is significant variation from standard procedures. These corrections may be significant for evaluation of liquefaction. Information on these additional corrections may be found in Youd and Idriss (1997).

The $N1_{60}$-ϕ_f correlation used is modified after Bowles (1977). The correlation of Peck, Hanson, and Thornburn (1974) falls within the ranges specified. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant silt-sized material will fall in the lower portion of the range. Coarser materials with less than five percent fines will fall in the upper portion of the ranges. The geologic history and angularity of the particles may also need to be considered when selecting a value for ϕ_f.

Care should be exercised when using other correlations of *SPT* results to soil parameters. Some published correlations are based on corrected values ($N1_{60}$) and some are based on uncorrected values (N).

The designer should ascertain the basis of the correlation and use either $N1_{60}$ or N as appropriate.

Care should also be exercised when using *SPT* blow counts to estimate soil shear strength if in soils with coarse gravel, cobbles, or boulders. Large gravels, cobbles, or boulders could cause the *SPT* blow counts to be unrealistically high.

The secant friction angle derived from the procedure to estimate the drained friction angle of gravels and rock fill materials depicted in Figure 10.4.6.2.4-1 is based on a straight line from the origin of a Mohr diagram to the intersection with the strength envelope at the effective normal stress. Thus, the angle derived is applicable only to analysis of field conditions subject to similar normal stresses. See Terzaghi, Peck, and Mesri (1996) for additional details regarding this procedure.

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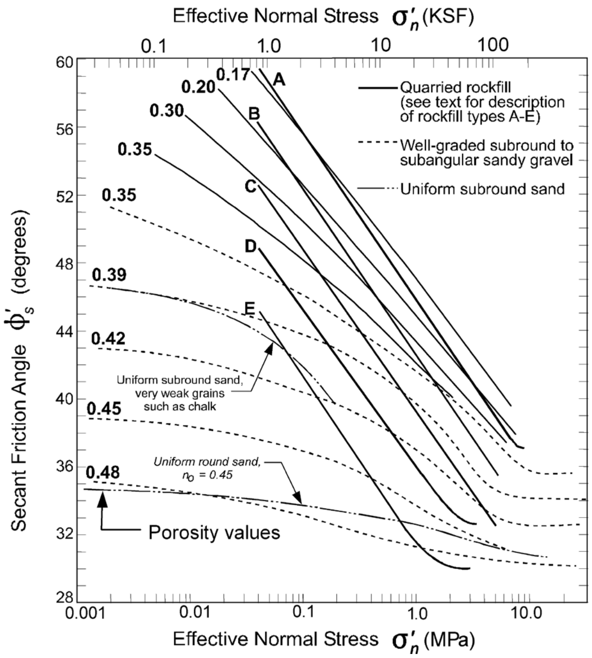

Figure 10.4.6.2.4-1—Estimation of Drained Friction Angle of Gravels and Rock Fills (modified after Terzaghi, Peck, and Mesri, 1996)

10.4.6.3—Soil Deformation

Consolidation parameters C_c, C_r, and C_α should be determined from the results of one-dimensional consolidation tests. To assess the potential variability in the settlement estimate, the average, upper and lower bound values obtained from testing should be considered.

Preconsolidation stress may be determined from one-dimensional consolidation tests and in-situ tests. Knowledge of the stress history of the soil should be used to supplement data from laboratory and/or in-situ tests, if available.

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It is important to understand whether the values obtained are computed based on a void ratio definition or a strain definition. Computational methods vary for each definition.

For preliminary analyses or where accurate prediction of settlement is not critical, values obtained from correlations to index properties may be used. Refer to Sabatini et al. (2002) for discussion of the various correlations available. If correlations for prediction of settlement are used, their applicability to the specific geologic formation under consideration should be evaluated.

A profile of σ_p', or OCR = σ_p'/σ_o', with depth should be developed for the site for design applications where the stress history could have a significant impact on the design properties selected and the performance of the foundation. As with consolidation properties, an upper and lower bound profile should be developed based on laboratory tests and plotted with a profile based on particular in-situ test(s), if used. It is particularly important to accurately compute preconsolidation stress values for relatively shallow depths where in-situ effective stresses are low. An underestimation of the preconsolidation stress at shallow depths will result in overly conservative estimates of settlement for shallow soil layers.

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The coefficient of consolidation, c_v, should be determined from the results of one-dimensional consolidation tests. The variability in laboratory determination of c_v results should be considered in the final selection of the value of c_v to be used for design.

Due to the numerous simplifying assumptions associated with conventional consolidation theory, on which the coefficient of consolidation is based, it is unlikely that even the best estimates of c_v from high-quality laboratory tests will result in predictions of time rate of settlement in the field that are significantly better than a prediction within one order of magnitude. In general, the in-situ value of c_v is larger than the value measured in the laboratory test. Therefore, a rational approach is to select average, upper, and lower bound values for the appropriate stress range of concern for the design application. These values should be compared to values obtained from previous work performed in the same soil deposit. Under the best-case conditions, these values should be compared to values computed from measurements of excess pore pressures or settlement rates during construction of other structures.

CPTu tests in which the pore pressure dissipation rate is measured may be used to estimate the field coefficient of consolidation.

For preliminary analyses or where accurate prediction of settlement is not critical, values obtained from correlations to index properties presented in Sabatini et al. (2002) may be used.

Where evaluation of elastic settlement is critical to the design of the foundation or selection of the foundation type, in-situ methods such as *PMT* or *DMT* for evaluating the modulus of the stratum should be used.

For preliminary design or for final design where the prediction of deformation is not critical to structure performance, i.e., the structure design can tolerate the potential inaccuracies inherent in the correlations. The elastic properties (E_s, ν) of a soil may be estimated from empirical relationships presented in Table C10.4.6.3-1.

The specific definition of E_s is not always consistent for the various correlations and methods of in-situ measurement. See Sabatini et al. (2002) for additional details regarding the definition and determination of E_s.

An alternative method of evaluating the equivalent elastic modulus using measured shear wave velocities is presented in Sabatini et al. (2002).

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Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim)
Clay:		
Soft sensitive	0.347–2.08	0.4–0.5 (undrained)
Medium stiff to stiff	2.08–6.94	
Very stiff	6.94–13.89	
Loess	2.08–8.33	0.1–0.3
Silt	0.278–2.78	0.3–0.35
Fine Sand:		
Loose	1.11–1.67	0.25
Medium dense	1.67–2.78	
Dense	2.78–4.17	
Sand:		
Loose	1.39–4.17	0.20–0.36
Medium dense	4.17–6.94	
Dense	6.94–11.11	0.30–0.40
Gravel:		
Loose	4.17–11.11	0.20–0.35
Medium dense	11.11–13.89	
Dense	13.89–27.78	0.30–0.40

Estimating E_s from *SPT N* Value	
Soil Type	E_s (ksi)
Silts, sandy silts, slightly cohesive mixtures	$0.056\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.097\ N1_{60}$
Coarse sands and sands with little gravel	$0.139\ N1_{60}$
Sandy gravel and gravels	$0.167\ N1_{60}$

Estimating E_s from q_c (static cone resistance)	
Sandy soils	$0.028q_c$

The modulus of elasticity for normally consolidated granular soils tends to increase with depth. An alternative method of defining the soil modulus for granular soils is to assume that it increases linearly with depth starting at zero at the ground surface in accordance with the following equation:

$$E_s = n_h \times z \qquad \text{(C10.4.6.3-1)}$$

where:

E_s = the soil modulus at depth z (ksi)
n_h = rate of increase of soil modulus with depth as defined in Table C10.4.6.3-2 (ksi/ft)
z = depth below the ground surface (ft)

Table C10.4.6.3-2—Rate of Increase of Soil Modulus with Depth n_h (ksi/ft) for Sand

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

The potential for soil swell that may result in uplift on deep foundations or heave of shallow foundations should be evaluated based on Table 10.4.6.3-1.

Table 10.4.6.3-1—Method for Identifying Potentially Expansive Soils (Reese and O'Neill, 1988)

Liquid Limit *LL* (%)	Plastic Limit *PL* (%)	Soil Suction (ksf)	Potential Swell (%)	Potential Swell Classification
>60	>35	>8	>1.5	High
50–60	25–35	3–8	0.5–1.5	Marginal
<50	<25	<3	<0.5	Low

The formulation provided in Eq. C10.4.6.3-1 is used primarily for analysis of lateral response or buckling of deep foundations.

10.4.6.4—Rock Mass Strength

The strength of intact rock material should be determined using the results of unconfined compression tests on intact rock cores. If point load strength index tests are used to assess intact rock compressive strength in lieu of a full suite of unconfined compression tests on intact rock cores, the point load test results should be calibrated to unconfined compression strength tests obtained from the same geologic unit.

Except as noted for design of spread footings in rock, for a rock mass of “randomly” oriented discontinuities such that it that contains a sufficient number behaves as an isotropic mass, and thus, its behavior is largely independent of the direction of the applied loads, the strength of the rock mass should first be classified using its geological strength index (GSI) as described in Figures 10.4.6.4-1 and 10.4.6.4-2, and then assessed using the Hoek-Brown failure criterion.

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Point load strength index tests rely on empirical correlations to intact rock compressive strength. The correlation provided in the ASTM point load test procedure (ASTM D5731) is empirically based and may not be valid for the specific rock type under consideration. Therefore, a site-specific correlation with uniaxial compressive strength test results from the same geologic unit is recommended. Point load strength index tests should not be used for weak to very weak rocks (< 2,200 psi/15 MPa).

Because the engineering behavior of rock is strongly influenced by the presence and characteristics of discontinuities, emphasis is placed on visual assessment of the rock and the rock mass. The application of a rock mass classification system essentially assumes that the rock mass contains a sufficient number of “randomly” oriented discontinuities such that it behaves as an isotropic mass, and thus its behavior is largely independent of the direction of the applied loads. It is generally not appropriate to use such classification systems for rock masses with well defined, dominant structural fabrics or where the orientation of discrete, persistent discontinuities controls behavior to loading.

The GSI was introduced by Hoek et al. (1995) and Hoek and Brown (1997), and updated by Hoek et al.

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(1998) to classify jointed rock masses. Marinos et al. (2005) provides a comprehensive summary of the applications and limitations of the GSI for jointed rock masses (Figure 10.4.6.4-1) and for heterogeneous rock masses that have been tectonically disturbed (Figure 10.4.6.4-2). Hoek et al. (2005) further distinguish heterogeneous sedimentary rocks that are not tectonically disturbed and provide several diagrams for determining GSI values for various rock mass conditions. In combination with rock type and uniaxial compressive strength of intact rock, q_u, GSI provides a practical means to assess rock mass strength and rock mass modulus for foundation design using the Hoek-Brown failure criterion (Hoek et al., 2002).

The design procedures for spread footings in rock provided in Article 10.6.3.2 have been developed using the rock mass rating (RMR) system. For design of foundations in rock in Articles 10.6.2.4 and 10.6.3.2, classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

Other methods for assessing rock mass strength, including in-situ tests or other visual systems that have proven to yield accurate results may be used in lieu of the specified method.

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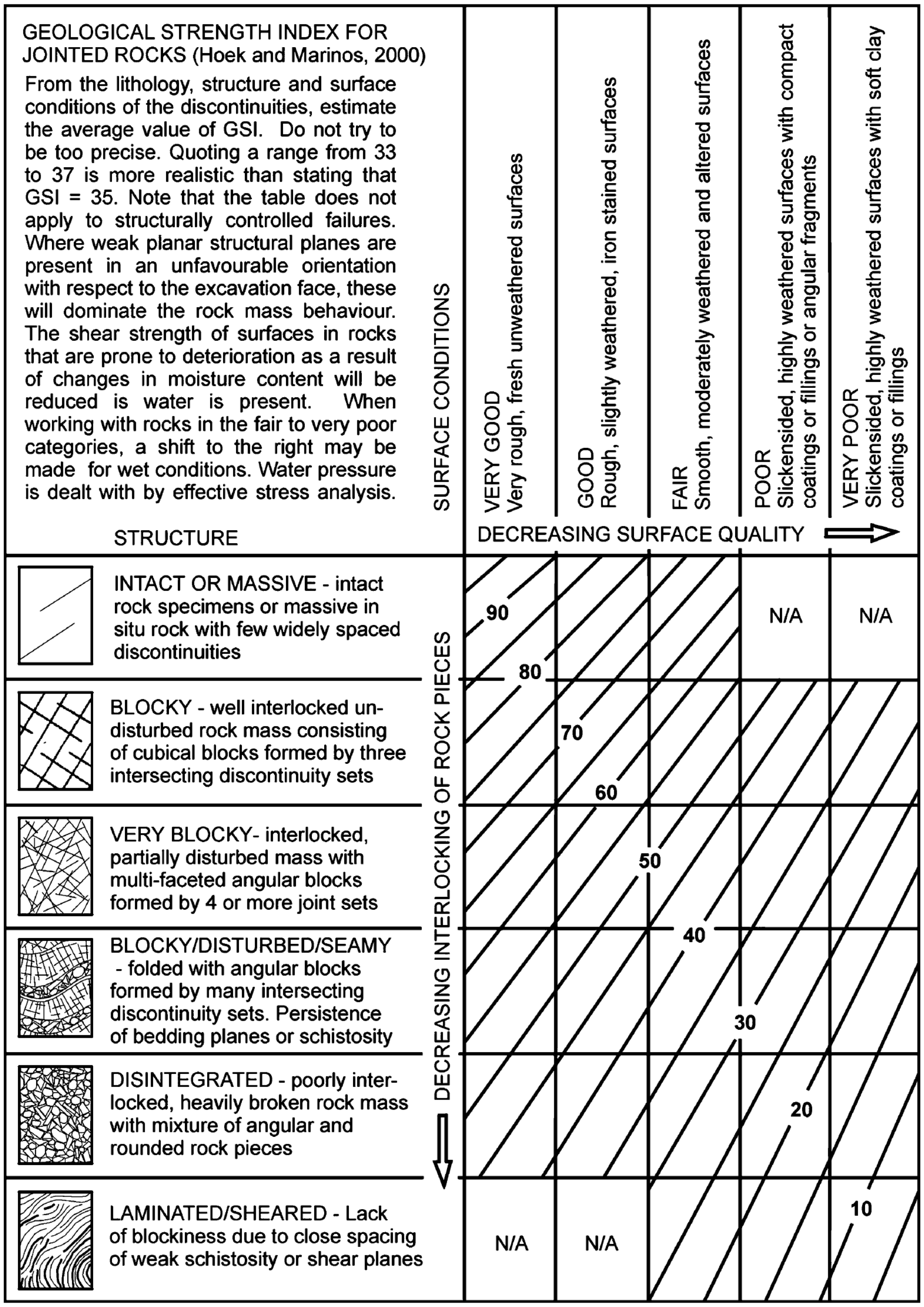

Figure 10.4.6.4-1—Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000)

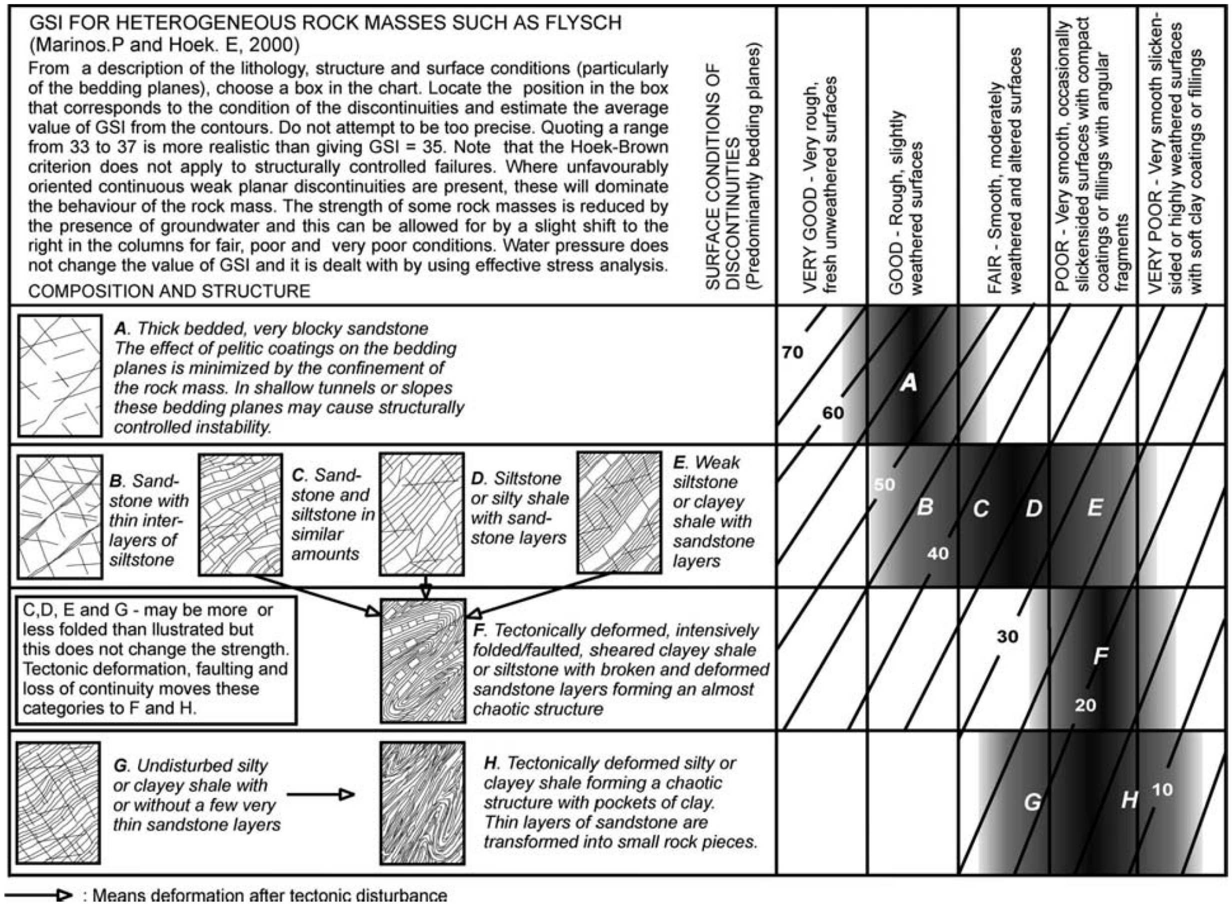

Figure 10.4.6.4-2—Determination of GSI for Tectonically Deformed Heterogeneous Rock Masses (Marinos and Hoek, 2000)

The strength of jointed rock masses should be evaluated using the Hoek-Brown failure criterion (Hoek et al., 2002). This nonlinear strength criterion is expressed in its general form as:

$$\sigma_1' = \sigma_3' + q_u\left(m_b\frac{\sigma_3'}{q_u} + s\right)^a \qquad (10.4.6.4\text{-}1)$$

in which:

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)} \qquad (10.4.6.4\text{-}2)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right) \qquad (10.4.6.4\text{-}3)$$

where:

e	=	2.718 (natural or Naperian log base)
D	=	disturbance factor (dim)
σ'_1 and σ'_3	=	principal effective stresses (ksf)
q_u	=	average unconfined compressive strength of rock core (ksf)
m_b, s, and a	=	empirically determined parameters

The value of the constant m_i should be estimated from Table 10.4.6.4-1, based on lithology. Relationships between GSI and the parameters m_b, s, and a, according to Hoek et al. (2002) are as follows:

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \tag{10.4.6.4-4}$$

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

<table>
<tr><th rowspan="2">Rock type</th><th rowspan="2">Class</th><th rowspan="2">Group</th><th colspan="4">Texture</th></tr>
<tr><th>Coarse</th><th>Medium</th><th>Fine</th><th>Very fine</th></tr>
<tr><td rowspan="4">SEDIMENTARY</td><td colspan="2">Clastic</td><td>Conglomerate
(21 ± 3)
Breccia
(19 ± 5)</td><td>Sandstone
17 ± 4</td><td>Siltstone
7 ± 2
Greywacke
(18 ± 3)</td><td>Claystone
4 ± 2
Shale
(6 ± 2)
Marl
(7 ± 2)</td></tr>
<tr><td rowspan="3">Non-Clastic</td><td>Carbonates</td><td>Crystalline
Limestone
(12 ± 3)</td><td>Sparitic
Limestone
(10 ± 5)</td><td>Micritic
Limestone
(8 ± 3)</td><td>Dolomite
(9 ± 3)</td></tr>
<tr><td>Evaporites</td><td></td><td>Gypsum
10 ± 2</td><td>Anhydrite
12 ± 2</td><td></td></tr>
<tr><td>Organic</td><td></td><td></td><td></td><td>Chalk
7 ± 2</td></tr>
<tr><td rowspan="3">METAMORPHIC</td><td colspan="2">Non Foliated</td><td>Marble
9 ± 3</td><td>Hornfels
(19 ± 4))
Metasandstone
(19 ± 3)</td><td>Quartzite
20 ± 3</td><td></td></tr>
<tr><td colspan="2">Slightly foliated</td><td>Migmatite
(29 ± 3)</td><td>Amphibolite
26 ± 6</td><td>Gneiss
28 ± 5</td><td></td></tr>
<tr><td colspan="2">Foliated*</td><td></td><td>Schist
(10 ± 3)</td><td>Phyllite
(7 ± 3)</td><td>Slate
7 ± 4</td></tr>
<tr><td rowspan="5">IGNEOUS</td><td rowspan="2">Plutonic</td><td>Light</td><td>Granite
32 ± 3</td><td>Diorite
25 ± 5</td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="2">Granodiorite
(29 ± 3)</td><td></td><td></td></tr>
<tr><td></td><td>Dark</td><td>Gabbro
27 ± 3</td><td>Dolerite
(16 ± 5)</td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="2">Norite
20 ± 5</td><td></td><td></td></tr>
<tr><td colspan="2">Hypabyssal</td><td colspan="2">Porphyries
(20 ± 5)</td><td>Diabase
(15 ± 5)</td><td>Peridotite
(25 ± 5)</td></tr>
<tr><td></td><td rowspan="2">Volcanic</td><td>Lava</td><td></td><td>Rhyolite
(25 ± 5)
Andesite
25 ± 5</td><td>Dacite
(25 ± 3))
Basalt
(25 ± 5)</td><td></td></tr>
<tr><td></td><td>Pyroclastic</td><td>Agglomerate
(19 ± 3)</td><td>Volcanic breccia
(19 ± 5)</td><td>Tuff
(13 ± 5)</td><td></td></tr>
</table>

* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

Disturbance to the foundation excavation caused by the rock removal methodology should be considered through the disturbance factor, D, in Eqs. 10.4.6.4-2 through 10.4.6.4-4.

The disturbance factor, D, ranges from 0.0 (undisturbed) to 1.0 (highly disturbed), and is an adjustment for the rock mass disturbance induced by the excavation method. Suggested values for various tunnel and slope excavations can be found in Hoek et al. (2002).

However, these values may not directly applicable to foundations. If using blasting techniques to remove the rock in a shaft foundation, due to its confined state, a disturbance factor approaching 1.0 should be considered, as the blast energy will tend to radiate laterally into the intact rock, potentially disturbing the rock. If using rock coring techniques, much less disturbance is likely and a disturbance factor approaching zero may be considered. If using a down the hole hammer to break up the rock, the disturbance factor is likely between these two extremes.

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core or, if possible, on actual discontinuities using an oriented shear box.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied or, if possible, direct shear tests should be performed on actual discontinuities using an oriented shear box.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

Table C10.4.6.4-1—Typical Ranges of Friction Angles for Smooth Joints in a Variety of Rock Types (modified after Barton, 1976 and Jaeger and Cook, 1976)

Rock Class	Friction Angle Range	Typical Rock Types
Low Friction	20–27°	Schists (high mica content), shale, marl
Medium Friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High Friction	34–40°	Basalt, granite, limestone, conglomerate

Note: Values assume no infilling and little relative movement between joint faces.

When a major discontinuity with a significant thickness of infilling is to be investigated, the shear strength will be governed by the strength of the infilling material and the past and expected future displacement of the discontinuity. Refer to Sabatini et al. (2002) for detailed procedures to evaluate infilled discontinuities.

10.4.6.5—Rock Mass Deformation

The elastic modulus of a rock mass, E_m, shall be taken as the lesser of the intact modulus of a sample of rock core, E_R, or the modulus determined from Table 10.4.6.5-1.

For critical or large structures, determination of rock mass modulus, E_m, using in-situ tests should be considered. Refer to Sabatini et al. (2002) for descriptions of suitable in-situ tests.

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Methods for establishing design values of E_m include:

- Empirical correlations that relate E_m to strength or modulus values of intact rock (q_u or E_R) and GSI,
- Estimates based on previous experience in similar rocks or back-calculated from load tests, and
- In-situ testing, such as pressuremeter test.

Empirical correlations that predict rock mass modulus, E_m, from GSI and properties of intact rock, either uniaxial compressive strength, q_u, or intact modulus, E_R, are presented in Table 10.4.6.5-1. The recommended approach is to measure uniaxial compressive strength and modulus of intact rock in

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laboratory tests on specimens prepared from rock core. Values of GSI should be determined for representative zones of rock for the particular foundation design being considered. The correlation equations in Table 10.4.6.5-1 should then be used to evaluate modulus and its variation with depth. If pressuremeter tests are conducted, it is recommended that measured modulus values be calibrated to the values calculated using the relationships in Table 10.4.6.5-1.

Preliminary estimates of the elastic modulus of intact rock may be made from Table C10.4.6.5-1. Note that some of the rock types identified in the Table are not present in the U.S.

It is extremely important to use the elastic modulus of the rock mass for computation of displacements of rock materials under applied loads. Use of the intact modulus will result in unrealistic and unconservative estimates.

Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m\left(GPa\right)=\sqrt{\frac{q_u}{100}}10^{\frac{GSI-10}{40}}$ for $q_u \leq 100$ MPa $E_m\left(GPa\right)=10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa	Accounts for rocks with $q_u < 100$ MPa; notes q_u in MPa	Hoek and Brown (1997); Hoek et al. (2002)
$E_m=\frac{E_R}{100}e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)
Notes: E_r = modulus of intact rock, E_m = equivalent rock mass modulus, GSI = geological strength index, q_u = uniaxial compressive strength, and 1 MPa = 20.9 ksf.		

Table C10.4.6.5-1—Summary of Elastic Moduli for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_R (ksi × 10^3)			Standard Deviation (ksi × 10^3)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Poisson's ratio for rock should be determined from tests on intact rock core.

Where tests on rock core are not practical, Poisson's ratio may be estimated from Table C10.4.6.5-2.

Table C10.4.6.5-2—Summary of Poisson's Ratio for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

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10.4.6.6—Erodibility of Rock

Consideration should be given to the physical characteristics of the rock and the condition of the rock mass when determining a rock's susceptibility to erosion in the vicinity of bridge foundations. Physical characteristics that should be considered in the assessment of erodibility include cementing agents, mineralogy, joint spacing, and weathering.

C10.4.6.6

There is no consensus on how to determine erodibility of rock masses near bridge foundations. Refer to Richardson and Davis (2001) "Evaluating Scour at Bridges—Fourth Edition," Mayne et al. (2001), Appendix M for guidance on two proposed methods. The first method was proposed in an FHWA memorandum of July 1991 and consists of evaluating various rock index properties. The second method is documented in Smith (1994) "Preliminary Procedure to Evaluate Scour in Bedrock" which uses the erodibility index proposed by G. W. Annandale. The Engineer should consider the appropriateness of these two methods when determining the potential for a rock mass to scour.

10.5—LIMIT STATES AND RESISTANCE FACTORS

10.5.1—General

The limit states shall be as specified in Article 1.3.2; foundation-specific provisions are contained in this Section.

Foundations shall be proportioned so that the factored resistance is not less than the effects of the factored loads specified in Section 3.

10.5.2—Service Limit States

10.5.2.1—General

Foundation design at the service limit state shall include:

- vertical movements (settlements),
- horizontal movements, and
- scour at the design flood.

Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation.

Bearing resistance estimated using the presumptive allowable bearing pressure for spread footings, if used, shall be applied only to address the service limit state.

The foundation movements shall be translated to the deck elevation to evaluate the effect of such movements on the superstructure.

C10.5.2.1

In bridges where the superstructure and substructure are not integrated, settlement corrections can be made by jacking and shimming bearings. Article 2.5.2.3 requires jacking provisions for these bridges.

The cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. The Owner may establish more stringent criteria.

The design flood for scour is defined in Article 2.6.4.4.2, and is specified in Article 3.7.5 as applicable at the service limit state.

Presumptive bearing pressures were developed for use with working stress design. These values may be used for preliminary sizing of foundations, but should generally not be used for final design. If used for final design, presumptive values are only applicable at service limit states.

Movements of the substructure, i.e., elements between foundation and superstructure, should be added to foundation movements as appropriate.

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10.5.2.2—Tolerable Movements and Movement Criteria

10.5.2.2.1—General

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Foundation movement shall include vertical, horizontal, and rotational movements. The tolerable movement criteria shall be established by either empirical procedures or structural analyses, or by consideration of both.

Foundation settlement shall be investigated using all applicable loads in the Service I Load Combination specified in Table 3.4.1-1. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependant consolidation settlements.

All applicable service limit state load combinations in Table 3.4.1-1 shall be used for evaluating horizontal movement and rotation of foundations.

C10.5.2.2.1

Experience has shown that bridges can and often do accommodate more movement and/or rotation than traditionally allowed or anticipated in design. Creep, relaxation, redistribution of force effects, as well as the conservatism inherent in the estimation of force effects by simplified methods accommodate these movements. Rotation movements should be evaluated at the top of the substructure unit in plan location and at the deck elevation.

Previous studies recommended differential settlement limits based on a factor applied to span length (Moulton et al., 1985) (Dimillio, 1982) (Barker et al., 1991). Recent work by Romano et al., 2017 developed separate differential settlement limits based on the differences in force effects computed from the live load distribution factor method (Article 4.6.2.2) and refined analysis. These limits apply to simple span and continuous multi-girder bridges constructed of steel and prestressed concrete that (a) meet the criteria for straight girders given in Article 4.6.1.2.4b, (b) have skew angles less than 45 degrees, and (c) were designed using the live load distribution factor method (Article 4.6.2.2).

To address bridge rideability, the angular break caused by differential settlement between either adjacent spans, or a span and the approach slab, should be limited to 0.004 radians. This criteria will always govern simply supported bridges and may govern continuous bridges.

For continuous steel girder bridges, the differential settlement limit was governed by the Strength I limit state and was found to be influenced by the girder spacing as well as the span length. The following expression may be used to estimate the tolerable differential settlement for continuous steel multi-girder bridges.

$$\Delta = 0.55\frac{L}{S} - 2.6 \qquad \text{(C10.5.2.2.1-1)}$$

Where, S is the girder spacing in ft, L is the minimum of adjacent span lengths, in ft, and Δ is the tolerable differential settlement, in inches.

For prestressed concrete girders rendered continuous for live load, the tension stress check of the Service III limit state controlled the level of tolerable differential settlement and resulted in relatively small tolerance. The following expression may be used to estimate the tolerable differential settlement for prestressed concrete multi-girder bridges rendered continuous for live load.

$$\Delta = 0.006L + 0.17 \qquad \text{(C10.5.2.2.1-2)}$$

If Service III limits are ignored, the controlling case for prestressed concrete multi-girder bridge rendered continuous for live load becomes Strength I, and the

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tolerable differential settlement may be estimated by the following expression.

$$\Delta = 0.13\frac{L}{S} - 0.17 \quad \text{(C10.5.2.2.1-3)}$$

As indicated in Table 3.4.1-1, the effects of settlement may need to be included in the evaluation of several different Load Combinations, including Strength and Service limit states. The estimation of differential settlement demands used in these evaluations should be computed using the Service I Load Combination.

For preliminary design, the previously developed limits of angular distortion between adjacent foundations of 0.004 rad. in continuous spans may be used for steel bridges.

Other angular distortion limits may be appropriate after consideration of:

- cost of mitigation through larger foundations, realignment or surcharge,
- rideability,
- vertical clearance,
- tolerable limits of deformation of other structures associated with a bridge, e.g., approach slabs, wing walls, pavement structures, drainage grades, utilities on the bridge, etc.
- roadway drainage,
- aesthetics, and
- safety.

Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement, with consideration of the column length and stiffness.

Tolerance of the superstructure to lateral movement will depend on bridge seat or joint widths, bearing type(s), structure type, and load distribution effects.

10.5.2.2.2—Assessment of Differential Movement and Its Effects

Determination of the relevant total foundation movement should include consideration of how and when movement occurs during the structure construction process and the uncertainty of the movement prediction. Movement that occurs after placement of the foundation and substructure elements, assessed using the construction-point approach, shall be considered the minimum relevant total movement when assessing the additional force effects applied to the superstructure due to differential movement between piers, foundation elements, or nonuniform movement across a foundation element.

C10.5.2.2.2

Differential movements and associated angular distortions are most accurately estimated using the construction-point concept. Details of how to conduct the construction-point approach is provided in Samtani et al. (2010). The load factor γ_{SE} is applied directly to the force effect resulting from the differential movement.

If the variability in the subsurface conditions between foundation elements is high, the S_f-0 approach as described in Samtani et al. (2010) and Samtani and Kulicki (2018) should be considered to determine the maximum differential movement between adjacent foundation elements.

10.5.2.3—Abutment Transitions

Vertical and horizontal movements caused by embankment loads behind bridge abutments shall be investigated.

C10.5.2.3

Settlement of foundation soils induced by embankment loads can result in excessive movements of substructure elements. Both short and long term settlement potential should be considered.

Settlement of improperly placed or compacted backfill behind abutments can cause poor rideability and a possibly dangerous bump at the end of the bridge. Guidance for proper detailing and material requirements for abutment backfill is provided in Samtani and Nowatzki (2006).

Lateral earth pressure behind and/or lateral squeeze below abutments can also contribute to lateral movement of abutments and should be investigated, if applicable.

10.5.3—Strength Limit States

10.5.3.1—General

Design of foundations at strength limit states shall include consideration of the nominal geotechnical and structural resistances of the foundation elements. Design at strength limit states shall not consider the deformations required to mobilize the nominal resistance, unless a definition of failure based on deformation is specified.

The design of all foundations at the strength limit state shall consider:

- structural resistance,
- loss of lateral and vertical support due to scour at the design flood event, and
- overall stability of the slope, if the foundation element is located on a slope, as specified in Article 3.4.1.

C10.5.3.1

For the purpose of design at strength limit states, the nominal resistance is considered synonymous with the ultimate capacity of an element as previously defined under allowable stress design, e.g., AASHTO (2002).

For design of foundations such as piles or drilled shafts that may be based directly on static load tests, or correlation to static load tests, the definition of failure may include a deflection-limited criteria.

Structural resistance includes checks for axial, lateral and flexural resistance.

The design event for scour is defined in Section 2 and is specified in Article 3.7.5 as applicable at the strength limit state.

10.5.3.2—Spread Footings

The design of spread footings at the strength limit state shall also consider:

- nominal bearing resistance,
- overturning or excessive loss of contact,
- sliding at the base of footing, and
- constructability.

C10.5.3.2

The designer should consider whether special construction methods are required to bear a spread footing at the design depth. Consideration should be given to the potential need for shoring, cofferdams, seals, and/or dewatering. Basal stability of excavations should be evaluated, particularly if dewatering or cofferdams are required.

Effort should be made to identify the presence of expansive/collapsible soils in the vicinity of the footing. If present, the structural design of the footing should be modified to accommodate the potential impact to the performance of the structure, or the expansive/collapsible soils should be removed or otherwise remediated. Special conditions such as the presence of karstic formations or mines should also be evaluated, if present.

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10.5.3.3—Driven Piles

The design of pile foundations at the strength limit state shall also consider:

- axial compression resistance for single piles,
- pile group compression resistance,
- uplift resistance for single piles,
- uplift resistance for pile groups,
- pile punching failure into a weaker stratum below the bearing stratum,
- single pile and pile group lateral resistance, and
- constructability, including pile drivability.

C10.5.3.3

The commentary in Article C10.5.3.2 is applicable if a pile cap is needed.

For pile foundations, as part of the evaluation for the strength limit states identified herein, the effects of downdrag, soil setup or relaxation, and buoyancy due to groundwater should be evaluated.

10.5.3.4—Drilled Shafts

The design of drilled shaft foundations at the strength limit state shall also consider:

- axial compression resistance for single drilled shafts,
- shaft group compression resistance,
- uplift resistance for single shafts,
- uplift resistance for shaft groups,
- single shaft and shaft group lateral resistance,
- shaft punching failure into a weaker stratum below the bearing stratum, and
- constructability, including method(s) of shaft construction.

C10.5.3.4

See commentary in Articles C10.5.3.2 and C10.5.3.3.

The design of drilled shafts for each of these limit states should include the effects of the method of construction, including construction sequencing, whether the shaft will be excavated in the dry or if wet methods must be used, as well as the need for temporary or permanent casing to control caving ground conditions. The design assumptions regarding construction methods must carry through to the contract documents to provide assurance that the geotechnical and structural resistance used for design will be provided by the constructed product.

10.5.3.5—Micropiles

The design of micropile foundations at the strength limit state shall also consider:

- axial compression resistance for single micropile,
- micropile group compression resistance,
- uplift resistance for single micropile,
- uplift resistance for micropile groups,
- micropile group punching failure into a weaker stratum below the bearing stratum, and single micropile punching failure where tip resistance is considered,
- single micropile and micropile group lateral resistance, and
- constructibility, including method(s) of micropile construction.

C10.5.3.5

The commentary in Article C10.5.3.2 is applicable if a pile cap is needed.

The design of micropiles for each of these limit states should include the effects of the method of construction for the micropile type to be constructed. The design assumptions regarding construction methods must carry through to the contract documents to provide assurance that the geotechnical and structural resistance used for design will be provided by the constructed product.

10.5.4—Extreme Events Limit States

10.5.4.1—Extreme Events Design

Foundations shall be designed for extreme events as applicable.

C10.5.4.1

Extreme events include the check flood for scour, vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included. Appendix A10 gives additional guidance regarding seismic analysis and design.

10.5.4.2—Liquefaction Design Requirements

A liquefaction assessment shall be conducted for Seismic Zones 3 and 4 if both of the following conditions are present:

- *Ground Water Level*—The groundwater level anticipated at the site is within 50.0 ft of the existing ground surface or the final ground surface, whichever is lower.
- *Soil Characteristics*—Low plasticity silts and sands within the upper 75.0 ft are characterized by one of the following conditions: (1) the corrected standard penetration test (*SPT*) blow count, $(N_1)_{60}$, is less than or equal to 25 blows/ft in sand and nonplastic silt layers, (2) the corrected cone penetration test (*CPT*) tip resistance, q_{ciN}, is less than or equal to 150 in sand, and nonplastic silt layers, (3) the normalized shear wave velocity, V_{s1}, is less than 660 fps, or (4) a geologic unit is present at the site that has been observed to liquefy in past earthquakes.

Where loose to very loose saturated sands are within the subsurface soil profile such that liquefaction of these soils could impact the stability of the structure, the potential for liquefaction in Seismic Zone 2 should also be considered.

For sites that require an assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction:

- loss in strength in the liquefied layer or layers,
- liquefaction-induced ground settlement, and
- flow failures, lateral spreading, and slope instability.

For sites where liquefaction occurs around bridge foundations, bridges should be analyzed and designed in two configurations as follows:

- *Nonliquefied Configuration*—The structure should be analyzed and designed, assuming no liquefaction occurs, using the ground response spectrum appropriate for the site soil conditions in a nonliquefied state.
- *Liquefied Configuration*—The structure as designed in nonliquefied configuration above should be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified *P-y* curves, modulus of subgrade reaction, or *t-z* curves). The design spectrum should be the same as that used in the nonliquefied configuration.

With the Owner's approval, or as required by the Owner, a site-specific response spectrum that accounts for the modifications in spectral content from the

C10.5.4.2

All of the following general conditions are necessary for liquefaction to occur:

- A sustained ground acceleration that is large enough and acting over a long enough period of time to develop excess pore-water pressure, thereby reducing effective stress and soil strength.
- Predominantly cohesionless soil that has the right gradation and composition. Liquefaction has occurred in soils ranging from low plasticity silts to gravels. Clean or silty sands and nonplastic silts are most susceptible to liquefaction.
- The state of the soil is characterized by a density that is low enough for the soil to exhibit contractive behavior when sheared undrained under the initial effective overburden stress.
- The presence of groundwater, resulting in a saturated or nearly saturated soil.

Methods used to assess the potential for liquefaction range from empirically-based design methods to complex numerical, effective stress methods that can model the time-dependent generation of pore-water pressure and its effect on soil strength and deformation. Furthermore, dynamic performance soil tests such as cyclic simple shear or cyclic triaxial tests can be used to assess liquefaction susceptibility and behavior to be used as input for liquefaction analysis and design.

The most common method of assessing liquefaction involves the use of empirical methods (e.g., Youd et al., 2001). These methods provide an estimate of liquefaction potential based on *SPT* blowcounts, *CPT* cone tip resistance, or shear wave velocity. This type of analysis should be conducted as a baseline evaluation, even when more rigorous methods are used.

Youd et al. (2001) summarizes the consensus of the profession up to year 2000 regarding the use of the simplified methods. Since the publication of this consensus paper, various other modifications to the consensus approach have been introduced, including those by Cetin et al. (2004), Moss et al. (2006), and Boulanger and Idriss (2006). These more recent methods account for additions to the database on liquefaction, as well as refinements in the interpretation of case history data. The newer methods potentially offer improved estimates of liquefaction potential and can be considered for use.

The simplified empirical methods are suited for use to a maximum depth of approximately 75.0 ft. This depth limit relates to the database upon which the original empirical method was developed. Most of the database was from observations of liquefaction at depths less than 50.0 to 60.0 ft. Extrapolation of the simplified method beyond 75.0 ft is therefore of uncertain validity. This limitation should not be interpreted as meaning liquefaction does not occur beyond 75.0 ft. Rather,

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liquefying soil may be developed. Unless approved otherwise by the Owner, the reduced response spectrum resulting from the site-specific analyses shall not be less than two-thirds of the spectrum developed at the ground surface using the general procedure described in Article 3.10.4.1 modified by the site factors in Article 3.10.3.2.

The Designer should provide explicit detailing of plastic hinging zones for both cases mentioned above since it is likely that locations of plastic hinges for the liquefied configuration are different than locations of plastic hinges for the nonliquefied configuration. Design requirements including shear reinforcement should be met for the liquefied and nonliquefied configuration. Where liquefaction is identified, plastic hinging in the foundation may be permitted with the Owner's approval.

For those sites where liquefaction-related permanent lateral ground displacements (e.g., flow, lateral spreading, or slope instability) are determined to occur, the effects of lateral displacements on the bridge and retaining structures should be evaluated. These effects can include increased lateral pressure on bridge foundations and retaining walls.

The effects of liquefaction-related, permanent lateral ground displacements on bridge and retaining wall performance should be considered separate from the inertial evaluation of the bridge structures. However, if large magnitude earthquakes dominate the seismic hazards, the bridge response evaluation should consider the potential simultaneous occurrence of:

- inertial response of the bridge, and loss in ground response from liquefaction around the bridge foundations, and
- predicted amounts of permanent lateral displacement of the soil.

If inelastic deformations are expected in the foundation due to liquefaction-induced effects, a quantitative assessment of such effects should be considered. Such assessment may follow the approach outlined for SDC D in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

different methods should be used for greater depths, including the use of site-specific ground motion response modeling in combination with liquefaction testing in the laboratory.

The magnitude for the design earthquake must be determined when conducting liquefaction assessments using the simplified empirical procedures. The earthquake magnitude used to assess liquefaction can be determined from earthquake deaggregation data for the site, available through the USGS national seismic hazard website http://earthquake.usgs.gov/hazards/hazmaps/ based on the 975-year return period (i.e., five percent in 50 years within the USGS website). If a single or a few larger magnitude earthquakes dominate the deaggregation, the magnitude of the single dominant earthquake or the mean of the few dominant earthquakes in the deaggregation should be used.

Liquefaction is generally limited to granular soils, such as sands and non-plastic silts. Loose gravels also can liquefy if drainage is prevented such as might occur if a layer of clay or frozen soil is located over the gravel. Methods for eliminating sites based on soil type have been developed, as discussed by Youd et al., (2001), Bray and Sancio (2006), and Boulanger and Idriss (2006). These methods can be used to screen the potential for liquefaction in certain soil types. In the past soil screening with regard to silts was done using the Chinese criteria (Kramer, 1996). Recent studies (Bray and Sancio, 2006; Boulanger and Idriss, 2006) indicate that the Chinese criteria are unconservative, and therefore their use should be discontinued.

Two criteria for assessing liquefaction susceptibility of soils have been recently proposed as replacements to the Chinese criteria:

> Boulanger and Idriss (2006) recommend considering a soil to have clay-like behavior (i.e., not susceptible to liquefaction) if the plasticity index (PI) ≥ 7.
> Bray and Sancio (2006) suggest that a soil with a $PI < 12$ and a ratio of water content to liquid limit (wc/LL) > 0.85 will be susceptible to liquefaction.

There is no current consensus on the preferred of the two criteria, and, therefore, either method may be used, unless the Owner has a specific preference.

To determine the location of soils that are adequately saturated for liquefaction to occur, the seasonally averaged groundwater elevation should be used. Groundwater fluctuations caused by tidal action or seasonal variations will cause the soil to be saturated only during a limited period of time, significantly reducing the risk that liquefaction could occur within the zone of fluctuation.

Liquefaction evaluation is required only for sites meeting requirements for Seismic Zones 3 and 4, provided that the soil is saturated and of a type that is susceptible to liquefaction. For loose to very loose sand sites (e.g., $(N_1)_{60} < 10$ bpf or $q_{c1N} < 75$), a potential exists for liquefaction in

Seismic Zone 2, if the acceleration coefficient, A_S, is 0.15 or higher. The potential for and consequences of liquefaction for these sites will depend on the dominant magnitude for the seismic hazard. As the magnitude decreases, the liquefaction resistance of the soil increases due to the limited number of earthquake loading cycles. Generally, if the magnitude is 6 or less, even in these very loose soils, either the potential for liquefaction is very low or the extent of liquefaction is very limited. Nevertheless, a liquefaction assessment should be made if loose to very loose sands are present to a sufficient extent to impact bridge stability and A_S is greater than or equal to 0.15. These loose-to-very-loose sands are likely to be present in hydraulically placed fills and alluvial or estuarine deposits near rivers and waterfronts.

During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement include:

- *Slope Failure, Flow Failure, or Lateral Spreading*—The strength loss associated with pore-water pressure build-up can lead to slope instability. Generally, if the factor of safety against liquefaction is less than approximately 1.2 to 1.3, a potential for pore-water pressure build-up will occur, and the effects of this build-up should be assessed. If the soil liquefies, the stability is determined by the residual strength of the soil. The residual strength of liquefied soils can be determined using empirical methods developed by Seed and Harder (1990), Olson and Stark (2002), and others. Loss of lateral resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure.
- *Reduced Foundation Bearing Resistance*—Liquefied strength is often a fraction of nonliquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason, spread footing foundations are not recommended where liquefiable soils occur unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction.
- *Reduced Soil Stiffness and Loss of Lateral Support for Deep Foundations*—This loss in strength can change the lateral response characteristics of piles and shafts under lateral load.
- *Vertical Ground Settlement as Excess Pore-water Pressures Induced by Liquefaction Dissipate, Resulting in Downdrag Loads on Deep Foundations*—If liquefaction-induced downdrag loads can occur, the downdrag loads should be assessed as specified in Article 3.11.8.

Most liquefaction-related damage to bridges during past earthquakes has been the result of lateral movement of the soil, causing severe column distortion and potential structure collapse. Therefore, a thorough analysis of the

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effects of lateral soil movement due to liquefaction on the structure is necessary. If there is potential for significant soil movement, the structure design should meet the requirements of Seismic Zone 4.

The effects of liquefaction will depend in large part on the amount of soil that liquefies and the location of the liquefied soil with respect to the foundation. On sloping ground, lateral flow, spreading, and slope instability can occur on relatively thin layers of liquefiable soils, whereas the effects of thin liquefied layer on the lateral response of piles or shafts (without lateral ground movement) may be negligible. Likewise, a thin liquefied layer at the ground surface results in essentially no downdrag loads, whereas the same liquefied layer deeper in the soil profile could result in large downdrag loads. Given these potential variations, site investigation plays a fundamental part of the liquefaction assessment. Article 10.4 identifies requirements for site investigations.

When assessing the effects of liquefaction on bridge response, the recommendations herein require that structure be designed for two cases, one in which the full seismic acceleration is applied to the structure assuming the soil does not liquefy, and one in which the full seismic acceleration is applied to the structure assuming the soil does liquefy but the spectrum is unchanged by liquefaction. This approach should produce conservative results for bridges with periods less than 1 sec. However, Youd and Carter (2005) suggest that at periods greater than 1 second, it is possible for liquefaction to result in higher spectral accelerations than occur for equivalent nonliquefied cases, all other conditions being equal. For Site Class C or D and bridges with periods greater than 1 sec., the Designer may consider using a response spectrum constructed using Site Class E for the liquefied condition. Alternately, site-specific ground motion response evaluations may be used to evaluate this potential.

There is currently no consensus on how to address this issue of timing of seismic acceleration and the development of full liquefaction and its combined impact on the structure without resorting to more rigorous analyses, such as by using nonlinear, effective stress methods. In general, the larger the earthquake magnitude (e.g., $M > 8$), the longer the period of time over which strong shaking acts, and the more likely the strong shaking and liquefaction effects will be acting concurrently. The smaller the earthquake magnitude, the more likely that these two effects will not be concurrent, in which case the peak inertial response of the bridge may occur before much, if any, reduction in soil support from liquefaction occurs.

Site-specific dynamic ground motion response analyses offers one method of evaluating the effects of pore-water pressure increases and timing on the development of the response spectrum. These analyses can be conducted using a nonlinear, effective stress method that accounts for the build-up in pore-water pressure and stiffness degradation in liquefiable layers.

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Use of this approach requires considerable skill in terms of selecting model parameters, particularly the pore pressure model. The complexity of this approach is such that Owner's approval is mandatory, and it is highly advisable that an independent peer review panel with expertise in nonlinear, effective stress modeling be used to review the methods and the resulting spectrum.

The limit of two-thirds for reduction of the liquefied response spectrum below the nonliquefied spectrum is meant to apply to any ordinate of the response spectrum. Generally, liquefied conditions may produce significant reductions in the shorter period range, but the reductions will be smaller or could be increased over nonliquefied conditions in the longer period range over about 1–2 sec. The developer of the site response analysis should capture accurate estimates of response for all periods that could be of importance in both nonliquefied and liquefied conditions. This consideration is particularly important if the conventional spectral shapes of Article 3.10.4.1 are being used.

The timing of liquefaction relative to the development of strong shaking also can be an important consideration for sites where lateral ground movement occurs. Both the development of liquefaction and the ground movement are dependent on the size and magnitude of the earthquake, but they do not necessarily occur at the same time. This issue is especially important when determining how to combine the inertial response of the structure and the response to lateral movement of the soil against the foundations and other substructure elements due to lateral spreading, slope instability, and flow failure. Current practice is to consider these two mechanisms to be independent, and therefore, the analyses are decoupled; i.e., the analysis is first performed to evaluate inertial effects during liquefaction following the same guidance as for level-ground sites, and then the foundation is evaluated for the moving ground, but without the inertial effects of the bridge superimposed. For critical bridges or in areas where very large-magnitude earthquakes could occur, detailed studies addressing the two mechanisms acting concurrently may be warranted. This timing issue also affects liquefaction-induced downdrag, in that settlement and downdrag generally does not occur until the pore pressures induced by ground shaking begin to dissipate after shaking ceases.

For assessment of existing structures, the Designer should consider using Seismic Zone 4 regardless of the magnitude of A_s, even when significant lateral soil movement is not expected, if the structure is particularly weak with regard to its ability to resist the forces and displacements that could be caused by liquefaction. Examples of weaknesses that could exacerbate the impact of liquefaction to the structure include presence of shallow foundations, deep foundations tipped in liquefiable soil, very limited bridge support lengths that have little tolerance of lateral movement of the substructure, deterioration of superstructure or substructure components due to advanced age of the

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structure or severe environmental conditions, and the absence of substructure redundancy.

The intent of these Specifications is to limit inelastic deformations under seismic loading to above-ground locations that can be inspected. However, if liquefaction occurs, it may be difficult or impossible to restrict inelastic action solely to above-ground locations without site improvement. If inelastic deformations are expected in the foundation, then the Owner may consider installation of devices that permit post-earthquake assessment; for example, installation of inclinometer tubes in drilled shafts permits limited evaluation of the deformations of the foundation, which would otherwise be impossible to inspect at any significant depth. Permitting inelastic behavior below the ground implies that the shaft or piles will be damaged, possibly along with other parts of the bridge, and may need to be replaced.

Design options range from (a) an acceptance of the movements with significant damage to the piles and columns if the movements are large (possibly requiring demolition but still preserving the no-collapse philosophy) to (b) designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective. However, tolerable structural movements should be evaluated quantitatively.

Quantitative assessments of liquefaction-induced deformations on foundations may be accomplished using the nonlinear static “push over” methodology. However, such analysis is complicated by the need to model nonlinear *P-y* behavior of the liquefied soil along with the nonlinear behavior of the structure. Analyses where the liquefied soil is represented by appropriate residual resistance (*P-y* curves or modulus of subgrade reaction values) will generally provide conservative results for the actual inelastic behavior of the foundation structural elements. The approach for such analyses should be developed on a case-by-case basis due to the varied conditions found in liquefiable sites. Careful coordination between the geotechnical and structural engineers is essential to estimating the expected response and to evaluating whether the structure can tolerate the response. Often mitigation strategies may be required to reduce structural movements.

Mitigation of the effects of liquefaction-induced settlement or lateral soil movement may include ground stabilization to either prevent liquefaction or add strength to keep soil deformation from occurring, foundation or superstructure modifications to resist the forces and accommodate the deformations that may occur, or both.

It is often cost prohibitive to design the bridge foundation system to resist the loads imposed by liquefaction-induced lateral loads, especially if the depth of liquefaction extends more than about 20.0 ft below the ground surface and if a nonliquefied crust is part of the failure surface. Ground improvement to mitigate the liquefaction hazard is the likely alternative if it is not practical to design the foundation system to accommodate the lateral loads.

The primary ground improvement techniques to mitigate liquefaction fall into five general categories, namely removal and replacement, densification, reinforcement, altering the soil composition, and enhanced drainage. Any one or a combination of methods can be used. However, drainage improvement is not currently considered adequately reliable to prevent liquefaction-induced, excess pore-water pressure build-up due to (1) the time required for excess pore-water pressures to dissipate through the drainage paths, and (2) the potential for drainage materials to become clogged during installation and in service. In addition, with drainage enhancements some settlement is still likely. Therefore, drainage enhancements should not be used as a means to fully mitigate liquefaction. For further discussion of ground improvement methods, see FHWA-SA-98-086, *Ground Improvement Technical Summaries* (Elias et al., 2000); FHWA-SA-95-037; Geotechnical Engineering Circular No. 1, *Dynamic Compaction* (Lukas, 1995); and FHWA/RD-83/O2C, *Design and Construction of Stone Columns* (Barkdale and Bachus, 1983).

The use of large diameter shafts in lieu of the conventional pile cap foundation type may be considered in order to achieve the lateral strength and stiffness required to sustain the column demand while minimizing the foundation exposed surface area normal to the lateral flow direction.

10.5.5—Resistance Factors

10.5.5.1—Service Limit States

Resistance factors for the service limit states shall be taken as 1.0.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after scour due to the design flood.

10.5.5.2—Strength Limit States

10.5.5.2.1—General

Resistance factors for different types of foundation systems at the strength limit state shall be taken as specified in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5, unless regionally specific values or substantial successful experience is available to justify higher values.

For overall stability, resistance factors shall be as specified in Article 11.6.3.7. Overall stability of foundations shall be investigated using Strength I Load Combination and the provisions of Article 3.4.1.

C10.5.5.2.1

Regionally specific values should be determined based on substantial statistical data combined with calibration or substantial successful experience to justify higher values. Smaller resistance factors should be used if site or material variability is anticipated to be unusually high or if design assumptions are required that increase design uncertainty that have not been mitigated through conservative selection of design parameters.

Certain resistance factors in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5 are presented as a function of soil type, e.g., sand or clay. Naturally occurring soils do not fall neatly into these two classifications. In general, the terms “sand” and “cohesionless soil” may be connoted to mean drained conditions during loading, while “clay” or “cohesive soil” imply undrained conditions. For other or intermediate soil classifications, such as silts or gravels, the designer should choose, depending on the load case under consideration, whether the resistance provided by the soil will be a drained or undrained strength, and select the method of computing resistance and associated resistance factor accordingly.

In general, resistance factors for bridge and other structure design have been derived to achieve a reliability index, β, of 3.5, an approximate probability of failure, P_f, of 1 in 5,000. However, past geotechnical design practice has resulted in an effective reliability index, β, of 3.0, or an approximate probability of a failure of 1 in 1,000, for foundations in general, and for highly redundant systems, such as pile groups, an approximate reliability index, β, of 2.3, an approximate probability of failure of 1 in 100 (Zhang et al., 2001)(Paikowsky et al., 2004) (Allen, 2005). If the resistance factors provided in this Article are adjusted to account for regional practices using statistical data and calibration, they should be developed using the β values provided above, with consideration given to the redundancy in the foundation system.

For bearing resistance, lateral resistance, and uplift calculations, the focus of the calculation is on the individual foundation element, e.g., a single pile or drilled shaft. Since these foundation elements are usually part of a foundation unit that contains multiple elements, failure of one of these foundation elements usually does not cause the entire foundation unit to reach failure, i.e., due to load sharing and overall redundancy. Therefore, the reliability of the foundation unit is usually more, and in many cases considerably more, than the reliability of the individual foundation element. Hence, a lower reliability can be successfully used for redundant foundations than is typically the case for the superstructure.

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

10.5.5.2.2—Spread Footings

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site-specific considerations in Article 10.5.5.2.

C10.5.5.2.2

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

The resistance factor for sliding of cast-in-place concrete on sand is slightly lower than the other sliding resistance factors based on reliability theory analysis (Barker et al., 1991). The higher interface friction coefficient used for sliding of cast-in-place concrete on sand relative to that used for precast concrete on sand causes the cast-in-place concrete sliding analysis to be less conservative, resulting in the need for the lower resistance factor. A more detailed explanation of the development of the resistance factors provided in Table 10.5.5.2.2-1 is provided in Allen (2005).

The resistance factors for plate load tests and passive resistance were based on engineering judgment and past ASD practice.

10.5.5.2.3—Driven Piles

Resistance factors shall be selected from Table 10.5.5.2.3-1 based on the method used for determining the driving criterion necessary to achieve the required nominal pile bearing resistance.

Regarding load tests, and dynamic tests with signal matching, the number of tests to be conducted to justify the design resistance factors selected should be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A

C10.5.5.2.3

Where nominal pile bearing resistance is determined by static load test, dynamic testing, wave equation, or dynamic formulas, the uncertainty in the nominal resistance is strictly due to the reliability of the resistance determination method used in the field during pile installation.

In most cases, the nominal bearing resistance of each production pile is field-verified based on compliance with a driving criterion developed using a dynamic method

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site shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification, i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

(see Articles 10.7.3.8.2, 10.7.3.8.3, 10.7.3.8.4, or 10.7.3.8.5). The actual penetration depth where the pile is stopped using the driving criterion (e.g., a blow count measured during pile driving) will likely not be the same as the estimated depth from the static analysis. Hence, the reliability of the nominal pile bearing resistance is dependent on the reliability of the method used to verify the nominal resistance during pile installation (see Allen, 2005, for additional discussion on this issue). Therefore, the resistance factor for the field verification method should be used to determine the number of piles of a given nominal resistance needed to resist the factored loads in the strength limit state.

If the resistance factors provided in Table 10.5.5.2.3-1 are to be applied to small pile groups, the resistance factor values in the table should be reduced by 20 percent to reflect the reduced ability for overstressing of an individual foundation element to be carried by adjacent foundation elements. The minimum size of a pile group necessary to provide significant opportunity for load sharing ranges from 2 or 3 (Isenhower and Long, 1997) to 5 (Paikowsky et al., 2004).

The ability to share load between structural elements should an overstress occur is addressed in Article 1.3.4 through the use of η_R. The values for η_R provided in that Article have been developed in general for the superstructure, and no specific guidance on the application of η_R to foundations is provided. The η_R factor values recommended in Article 1.3.4 are not adequate to address this ability to shed load to other foundation elements when some of the foundation elements become overstressed, based on the results provided by Paikowsky et al. (2004) and others (see also Allen, 2005). Therefore, the resistance factors specified in Table 10.5.5.2.3-1 should be reduced based on the guidance provided in this Article to account for the lack of load sharing opportunities due to the small pile group size.

Dynamic methods may underpredict the nominal axial resistance of piles driven in soft silts or clays where a large amount of setup is anticipated and it is not feasible to perform static load or dynamic tests over a sufficient length of time to assess soil setup.

See Allen (2005) for an explanation on the development of the resistance factors for pile foundation design.

For all axial resistance calculation methods, the resistance factors were, in general, developed from load test results obtained on piles with diameters of 24.0 in. or less. Very little data were available for larger diameter piles. Therefore, these resistance factors should be used with caution for design of significantly larger diameter piles. In general, experience has shown that the static analysis methods identified in Table 10.5.5.2.3-1 tend to significantly overestimate the available nominal resistance for larger diameter piles. A static or dynamic load test should be considered if piles larger than 24.0 in. in diameter are anticipated.

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Note that a site as defined herein may be only a portion of the area in which the structure (or structures) is located. For sites where conditions are highly variable, a site could even be limited to a single pier.

Where driving criteria are established based on a static load test, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program.

One of the following alternative approaches may be used to address site variability when extrapolating pile load test results, and the application of driving criteria from those load test results, to piles not load tested:

1. Divide up the site into zones where subsurface conditions are relatively uniform using engineering judgment, conducting one static pile load test in each zone, and dynamic testing with signal matching on a minimum of two percent of the production piles, but no less than two production piles. A resistance factor of 0.80 is recommended if this approach is used. If production pile dynamic testing is not conducted, then a resistance factor of 0.75 should be used.
2. Characterize the site variability and select resistance factors using the approach described by Paikowsky et al. (2004).

The dynamic testing with signal matching should be evenly distributed within a pier and across the entire structure. However, within a particular footing, an increase in safety is realized where the most heavily loaded piles are tested.

The resistance factors in Table 10.5.5.2.3-1 for the case where dynamic testing is conducted without static load testing were developed using reliability theory for beginning of redrive (BOR) conditions. These resistance factors may be used for end of driving (EOD) conditions, but it should be recognized that dynamic testing with signal matching at EOD will likely produce conservative results because soil set up, which causes nominal pile bearing resistance to increase, is not taken into account. If, instead, relaxation is anticipated to occur, these resistance factors for dynamic testing should only be used at BOR.

The 0.50 resistance factor in Table 10.5.5.2.3-1 for use of the wave equation without dynamic measurements to estimate nominal pile bearing resistance is based on calibration by fitting to past allowable stress design practice. Using default wave equation hammer and soil input values, reliability theory calibrations performed by Paikowsky et al. (2004) suggest that a resistance factor of 0.40 should be used if the wave equation is used to estimate nominal pile bearing resistance. Their recommendation is more conservative than the resistance factor implied by past allowable stress design practice. Their recommendation should be considered representative of the reliability of the wave equation to estimate nominal pile bearing resistance by designers who lack experience with the wave equation and its application to local or regional subsurface conditions. Application of default wave equation input parameters without consideration to local site conditions and observed hammer performance in combination with this lower resistance factor is not recommended.

Local experience or site-specific test results should be used to refine the wave equation soil input values, or to at least use the input values selected with greater confidence, and field verification of the hammer performance should be conducted to justify the use of the resistance factor of 0.50 provided in Table 10.5.5.2.3-1. Field verification of hammer performance is considered to be a direct measurement of either stroke or kinetic energy.

See Articles 10.7.3.8.2, 10.7.3.8.3, 10.7.3.8.4, and 10.7.3.8.5 for additional guidance regarding static pile load testing, dynamic testing and signal matching, wave equation analysis, and dynamic formulas, respectively, as they apply to the resistance factors provided in Table 10.5.5.2.3-1.

The dynamic pile formulas, i.e., FHWA-modified Gates and Engineering News, identified in Table 10.5.5.2.3-1, require the pile hammer energy as an input parameter. The developed hammer energy should be used for this purpose, defined as the product of actual stroke developed during the driving of the pile (or equivalent stroke as determined from the bounce chamber pressure for double acting hammers) and the hammer ram weight.

The resistance factors provided in Table 10.5.5.2.3-1 are specifically applicable to the dynamic pile formula as provided in Article 10.7.3.8.5. Note that for the Engineering News (EN) formula, the built-in safety factor of 6 has been removed so that it predicts nominal resistance. Therefore, the resistance factor shown in Table 10.5.5.2.3-1 for EN formula should not be applied to the traditional "allowable stress" form of the equation.

The resistance factors for the dynamic pile formulas, i.e., FHWA-modified Gates and EN, in Table 10.5.5.2.3-1 have been specifically developed for EOD conditions. Since static pile load test data, which include the effects of soil setup or relaxation (for the database used, primarily soil setup), were used to develop the resistance factors for these formulas, the resistance factors reflect soil setup occurring after the pile installation. At BOR, the blow count obtained already includes the soil setup. Therefore, a lower resistance factor for the driving formulas should be used for BOR conditions than the ones shown in Table 10.5.5.2.3-1 for EOD conditions. In general, dynamic testing should be conducted to verify nominal pile resistance at BOR in lieu of the use of driving formulas.

Paikowsky et al. (2004) indicate that the resistance factors for static pile resistance analysis methods can vary significantly for different pile types. The resistance factors presented are average values for the method. See Paikowsky et al. (2004) and Allen (2005) for additional information regarding this issue.

The resistance factor for the Nordlund/Thurman method was derived primarily using the Peck et al. (1974) correlation between $SPT\ N1_{60}$ and the soil friction angle, using a maximum design soil friction angle of 36 degrees, assuming the contributing zone for the bearing resistance

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is from the tip to two pile diameters below the tip. These assumptions should be considered when using the resistance factor specified in Table 10.5.5.2.3-1 for this static analysis method.

For the clay static pile analysis methods, if the soil cohesion was not measured in the laboratory, the correlation between *SPT N* and S_u by Hara et al. (1974) was used for the calibration. Use of other methods to estimate S_u may require the development of resistance factors based on those methods.

The resistance factors provided for uplift of single piles are generally less than the resistance factors for axial side resistance under compressive loading. This is consistent with past practice that recognizes the side resistance in uplift is generally less than the side resistance under compressive loading, and is also consistent with the statistical calibrations performed in Paikowsky et al. (2004). Since the reduction in uplift resistance that occurs in tension relative to the side resistance in compression is taken into account through the resistance factor, the calculation of side resistance using a static pile resistance analysis method should not be reduced from what is calculated from the methods provided in Article 10.7.3.8.6.

For uplift, the number of pile load tests required to justify a specific resistance factor are the same as that required for determining compression resistance. Extrapolating the pile load test results to other untested piles as specified in Article 10.7.3.10 does create some uncertainty, since there is not a way to directly verify that the desired uplift resistance has been obtained for each production pile. This uncertainty has not been quantified. Therefore, it is recommended that a resistance factor of not greater than 0.60 be used if an uplift load test is conducted.

Regarding pile drivability analysis, the only source of load is from the pile driving hammer. Therefore, the load factors provided in Section 3 do not apply. In past practice, e.g., AASHTO (2002), no load factors were applied to the stresses imparted to the pile top by the pile hammer. Therefore, a load factor of 1.0 should be used for this type of analysis. Generally, either a wave equation analysis or dynamic testing, or both, are used to determine the stresses in the pile resulting from hammer impact forces. See Article 10.7.8 for the specific calculation of the pile structural resistance available for analysis of pile drivability. The structural resistance available during driving determined as specified in Article 10.7.8 considers the ability of the pile to handle the transient stresses resulting from hammer impact, considering variations in the materials, pile/hammer misalignment, and variations in the pile straightness and uniformity of the pile head impact surface.

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Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile—Dynamic Analysis and Static Load Test Methods, φ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing* conducted on 100% of production piles	0.75
	Driving criteria established by dynamic testing*, quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
	Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)	0.10
Nominal Bearing Resistance of Single Pile—Static Analysis Methods, φ_{stat}	Side Resistance and End Bearing: Clay and Mixed Soils	
	α-method (Tomlinson, 1987; Skempton, 1951)	0.35
	β-method (Esrig & Kirby, 1979; Skempton, 1951)	0.25
	λ-method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40
	Side Resistance and End Bearing: Sand	
	Nordlund/Thurman Method (Hannigan et al., 2005)	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (Canadian Geotech. Society, 1985)	0.45
Block Failure, φ_{b1}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Static load test	0.60
	Dynamic test with signal matching	0.50
Group Uplift Resistance, φ_{ug}	All soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles: See the provisions of Article 6.5.4.2	
	Concrete piles: See the provisions of Article 5.5.4.2	
	Timber piles: See the provisions of Articles 8.5.2.2 and 8.5.2.3	
Pile Drivability Analysis, φ_{da}	Steel piles: See the provisions of Article 6.5.4.2	
	Concrete piles: See the provisions of Article 5.5.4.2	
	Timber piles: See the provisions of Article 8.5.2.2	
	In all three Articles identified above, use φ identified as "resistance during pile driving"	

*Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to the static load test, when available.

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10.5.5.2.4—Drilled Shafts

Resistance factors shall be selected based on the method used for determining the nominal shaft resistance. When selecting a resistance factor for shafts in clays or other easily disturbed formations, local experience with the geologic formations and with typical shaft construction practices shall be considered.

Where the resistance factors provided in Table 10.5.5.2.4-1 are to be applied to a single shaft supporting a bridge pier, the resistance factor values in the Table should be reduced by 20 percent. Where the resistance factor is decreased in this manner, the η_R factor provided in Article 1.3.4 shall not be increased to address the lack of foundation redundancy.

The number of static load tests to be conducted to justify the resistance factors provided in Table 10.5.5.2.4-1 shall be based on the variability in the properties and geologic stratification of the site to which the test results are to be applied. A site, for the purpose of assessing variability, shall be defined as a project site, or a portion of it, where the subsurface conditions can be characterized as geologically similar in terms of subsurface stratification; i.e., sequence, thickness, and geologic history of strata, the engineering properties of the strata, and groundwater conditions.

C10.5.5.2.4

The resistance factors in Table 10.5.5.2.4-1 were developed using either statistical analysis of shaft load tests combined with reliability theory (Paikowsky et al., 2004), fitting to allowable stress design (ASD), or both. Where the two approaches resulted in a significantly different resistance factor, engineering judgment was used to establish the final resistance factor, considering the quality and quantity of the available data used in the calibration. The available reliability theory calibrations were conducted for the Reese and O'Neill (1988) method, with the exception of shafts in intermediate geo-materials (IGMs), in which case the O'Neill and Reese (1999) method was used. In Article 10.8, the O'Neill and Reese (1999) method is recommended. See Allen (2005) for a more detailed explanation on the development of the resistance factors for shaft foundation design, and the implications of the differences in these two shaft design methods on the selection of resistance factors.

For single shafts, lower resistance factors are specified to address the lack of redundancy. See Article C10.5.5.2.3 regarding the use of η_R.

Where installation criteria are established based on one or more static load tests, the potential for site variability should be considered. The number of load tests required should be established based on the characterization of site subsurface conditions by the field and laboratory exploration and testing program. One or more static load tests should be performed per site to justify the resistance factor selection.

Site variability is the most important consideration in evaluating the limits of a site for design purposes. Defining the limits of a site therefore requires sufficient knowledge of the subsurface conditions in terms of general geology, stratigraphy, index and engineering properties of soil and rock, and groundwater conditions. This implies that the extent of the exploration program is sufficient to define the subsurface conditions and their variation across the site.

A designer may choose to design drilled shaft foundations for strength limit states based on a calculated nominal resistance, with the expectation that load testing results will verify that value. The question arises whether to use the resistance factor associated with the design equation or the higher value allowed for load testing. This choice should be based on engineering judgment. The potential risk is that axial resistance measured by load testing may be lower than the nominal resistance used for design, which could require increased shaft dimensions that may be problematic, depending on the capability of the drilled shaft equipment mobilized for the project and other project-specific factors.

For the specific case of shafts in clay, the resistance factor recommended by Paikowsky et al. (2004) is much lower than the recommendation from Barker et al. (1991). Since the shaft design method for clay is nearly the same for both the 1988 and 1999 methods, a resistance factor that represents the average of the two resistance factor

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recommendations is provided in Table 10.5.5.2.4-1. This difference may point to the differences in local geologic formations and local construction practices, pointing to the importance of taking such issues into consideration when selecting resistance factors, especially for shafts in clay.

Cohesive IGMs are materials that are transitional between soil and rock in terms of their strength and compressibility, such as clay shales or mudstones with undrained shear strength between 5 and 50 ksf.

Since the mobilization of shaft base resistance is less certain than side resistance due to the greater deformation required to mobilize the base resistance, a lower resistance factor relative to the side resistance is provided for the base resistance in Table 10.5.5.2.4-1. O'Neill and Reese (1999) make further comment that the recommended resistance factor for tip resistance in sand is applicable for conditions of high quality control on the properties of drilling slurries and base cleanout procedures. If high quality control procedures are not used, the resistance factor for the O'Neill and Reese (1999) method for tip resistance in sand should be also be reduced. The amount of reduction should be based on engineering judgment.

Shaft compression load test data should be extrapolated to production shafts that are not load tested as specified in Article 10.8.3.5.6. There is no way to verify shaft resistance for the untested production shafts, other than through good construction inspection and visual observation of the soil or rock encountered in each shaft. Because of this, extrapolation of the shaft load test results to the untested production shafts may introduce some uncertainty. Statistical data are not available to quantify this at this time. Historically, resistance factors higher than 0.70, or their equivalent safety factor in previous practice, have not been used for shaft foundations. If the recommendations in Paikowsky et al. (2004) are used to establish a resistance factor when shaft static load tests are conducted, in consideration of site variability, the resistance factors recommended by Paikowsky et al. for this case should be reduced by 0.05, and should be less than or equal to 0.70 as specified in Table 10.5.5.2.4-1.

This issue of uncertainty in how the load test is applied to shafts not load tested is even more acute for shafts subjected to uplift load tests, as failure in uplift can be more abrupt than failure in compression. Hence, a resistance factor of 0.60 for the use of uplift load test results is recommended.

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Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
Block Failure, φ_{b1}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

10.5.5.2.5—Micropiles

Resistance factors shall be selected from Table 10.5.5.2.5-1 based on the method used for determining the nominal axial pile resistance. If the resistance factors provided in Table 10.5.5.2.5-1 are to be applied to piles in potentially creeping soils, highly plastic soils, weak rock, or other marginal ground type, the resistance factor values in the Table should be reduced by 20 percent to reflect greater design uncertainty.

C10.5.5.2.5

The resistance factors in Table 10.5.5.2.5-1 were calibrated by fitting to ASD procedures tempered with engineering judgment. The resistance factors in Table 10.5.5.2.5-2 for structural resistance were calibrated by fitting to ASD procedures and are equal to or slightly more conservative than corresponding resistance factors from Section 5 of the *AASHTO LRFD Bridge Design Specifications* for reinforced concrete column design.

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Table 10.5.5.2.5-1—Resistance Factors for Geotechnical Resistance of Axially Loaded Micropiles

Limit State	Method/ Ground Condition	Resistance Factor
Compression Resistance of Single Micropile, ϕ_{stat}	Side Resistance (Bond Resistance): Presumptive Values	0.55[1]
	Tip Resistance on Rock O'Neill and Reese (1999)	0.50
	Side Resistance and Tip Resistance Load Test	Values in Table 10.5.5.2.3-1, but no greater than 0.70
Block Failure, ϕ_{bl}	Clay	0.60
Uplift Resistance of Single Micropile, ϕ_{up}	Presumptive Values	0.55[1]
	Tension Load Test	Values in Table 10.5.5.2.3-1, but no greater than 0.70
Group Uplift Resistance, ϕ_{ug}	Sand & Clay	0.50

[1] Apply to presumptive grout-to-ground bond values for preliminary design only in Article C10.9.3.5.2.

Table 10.5.5.2.5-2—Resistance Factors for Structural Resistance of Axially Loaded Micropiles

Section/Loading Condition		Resistance Factor
Pile Cased Length	Tension, φ_{TC}	0.80
	Compression, φ_{CC}	0.75
Pile Uncased Length	Tension, φ_{TU}	0.80
	Compression, φ_{CU}	0.75

10.5.5.3—Extreme Limit States

10.5.5.3.1—General

Design of foundations at extreme limit states shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

10.5.5.3.2—Scour

The provisions of Articles 2.6.4.4.2 and 3.7.5 shall apply to the changed foundation conditions resulting from scour. Resistance factors at the strength limit state shall be taken as specified herein. Resistance factors at the extreme event shall be taken as 1.0 except that for uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

C10.5.5.3.2

The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (2004) regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (2005).

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10.5.5.3.3—Other Extreme Limit States

Resistance factors for extreme limit state, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

C10.5.5.3.3

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see Article 10.5.5.2.3).

10.6—SPREAD FOOTINGS

10.6.1—General Considerations

10.6.1.1—General

Provisions of this Article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and other substructure and superstructure elements. Special attention shall be given to footings on fill, to make sure that the quality of the fill placed below the footing is well controlled and of adequate quality in terms of shear strength and compressibility to support the footing loads.

Spread footings shall be proportioned and designed such that the supporting soil or rock provides adequate nominal resistance, considering both the potential for adequate bearing strength and the potential for settlement, under all applicable limit states in accordance with the provisions of this Section.

Spread footings shall be proportioned and located to maintain stability under all applicable limit states, considering the potential for, but not necessarily limited to, overturning (eccentricity), sliding, uplift, overall stability and loss of lateral support.

C10.6.1.1

Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor, e.g., soft, wet, frozen, or nondurable, material is used, or if the material is not properly compacted.

Spread footings should not be used on soil or rock conditions that are determined to be too soft or weak to support the design loads without excessive movement or loss of stability. Alternatively, the unsuitable material can be removed and replaced with suitable and properly compacted engineered fill material, or improved in place, at reasonable cost as compared to other foundation support alternatives.

Footings should be proportioned so that the stress under the footing is as nearly uniform as practicable at the service limit state. The distribution of soil stress should be consistent with properties of the soil or rock and the structure and with established principles of soil and rock mechanics.

10.6.1.2—Bearing Depth

Where the potential for scour, erosion or undermining exists, spread footings shall be located to bear below the maximum anticipated depth of scour, erosion, or undermining as specified in Article 2.6.4.4.

C10.6.1.2

Consideration should be given to the use of either a geotextile or graded granular filter material to reduce the susceptibility of fine grained material piping into rip rap or open-graded granular foundation material.

For spread footings founded on excavated or blasted rock, attention should be paid to the effect of excavation and/or blasting. Blasting of highly resistant competent rock formations may result in overbreak and fracturing of the rock to some depth below the bearing elevation. Blasting may reduce the resistance to scour within the zone of overbreak or fracturing.

Evaluation of seepage forces and hydraulic gradients should be performed as part of the design of foundations that will extend below the groundwater table. Upward seepage forces in the bottom of excavations can result in piping loss of soil and/or heaving and loss of stability in the base of foundation excavations. Dewatering with wells or wellpoints can control these problems. Dewatering can result in settlement of adjacent ground or structures. If adjacent structures may be damaged by settlement induced by dewatering, seepage cut-off methods such as sheet piling or slurry walls may be necessary.

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Spread footings shall be located below the depth of frost potential. Depth of frost potential shall be determined on the basis of local or regional frost penetration data.

Consideration may be given to over-excavation of frost susceptible material to below the frost depth and replacement with material that is not frost susceptible.

10.6.1.3—Effective Footing Dimensions

For eccentrically loaded footings, a reduced effective area, $B' \times L'$, within the confines of the physical footing shall be used in geotechnical design for settlement or bearing resistance. The point of load application shall be at the centroid of the reduced effective area.

The reduced dimensions for an eccentrically loaded rectangular footing shall be taken as:

$$B' = B - 2e_B \qquad (10.6.1.3\text{-}1)$$

$$L' = L - 2e_L$$

where:

e_B = eccentricity parallel to dimension B (ft)
e_L = eccentricity parallel to dimension L (ft)

Footings under eccentric loads shall be designed to ensure that the factored bearing resistance is not less than the effects of factored loads at all applicable limit states.

C10.6.1.3

The reduced dimensions for a rectangular footing are shown in Figure C10.6.1.3-1.

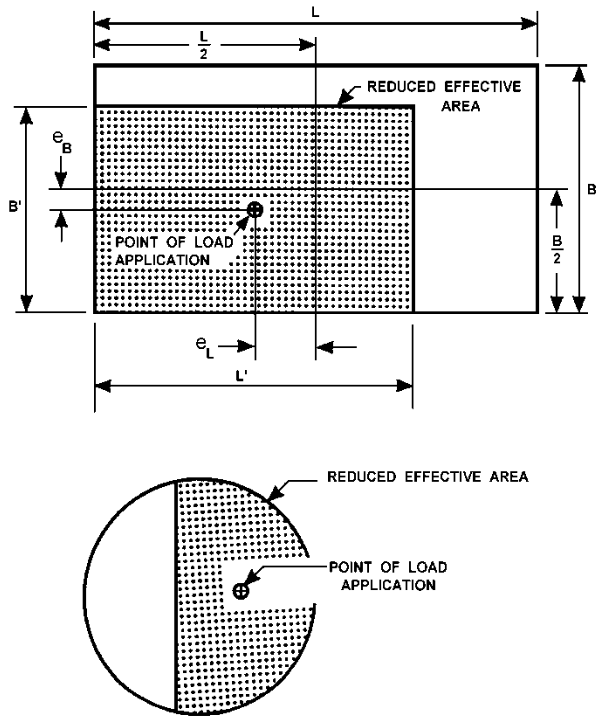

Figure C10.6.1.3-1—Reduced Footing Dimensions

For footings that are not rectangular, similar procedures should be used based upon the principles specified above.

For footings that are not rectangular, such as the circular footing shown in Figure C10.6.1.3-1, the reduced effective area is always concentrically loaded and can be estimated by approximation and judgment. Such an approximation could be made, assuming a reduced rectangular footing size having the same area and centroid as the shaded area of the circular footing shown in Figure C10.6.1.3-1.

10.6.1.4—Bearing Stress Distributions

When proportioning footing dimensions to meet settlement and bearing resistance requirements at all applicable limit states, the distribution of bearing stress on the effective area shall be assumed to be:

- uniform for footings on soils, or
- linearly varying, i.e., triangular or trapezoidal as applicable, for footings on rock.

The distribution of bearing stress shall be determined as specified in Article 11.6.3.2.

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Bearing stress distributions for structural design of the footing shall be as specified in Article 10.6.5.

10.6.1.5—Anchorage of Inclined Footings

Footings that are founded on inclined smooth solid rock surfaces and that are not restrained by an overburden of resistant material shall be effectively anchored by means of rock anchors, rock bolts, dowels, keys or other suitable means. Shallow keying of large footings shall be avoided where blasting is required for rock removal.

C10.6.1.5

Design of anchorages should include consideration of corrosion potential and protection.

10.6.1.6—Groundwater

Spread footings shall be designed in consideration of the highest anticipated groundwater table.

The influences of groundwater table on the bearing resistance of soils or rock and on the settlement of the structure shall be considered. In cases where seepage forces are present, they should also be included in the analyses.

10.6.1.7—Uplift

Where spread footings are subjected to uplift forces, they shall be investigated both for resistance to uplift and for structural strength.

10.6.1.8—Nearby Structures

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

10.6.2—Service Limit State Design

10.6.2.1—General

Service limit state design of spread footings shall include evaluation of total and differential settlement.

C10.6.2.1

The design of spread footings is frequently controlled by movement at the service limit state. It is therefore usually advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

10.6.2.2—Tolerable Movements

The requirements of Article 10.5.2.1 shall apply.

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10.6.2.3—Loads

Immediate settlement shall be determined using load combination Service I, as specified in Table 3.4.1-1. Time-dependent settlements in cohesive soils should be determined using only the permanent loads, i.e., transient loads should not be considered.

C10.6.2.3

The type of load or the load characteristics may have a significant effect on spread footing deformation. The following factors should be considered in the estimation of footing deformation:

- The ratio of sustained load to total load;
- The duration of sustained loads; and
- The time interval over which settlement or lateral displacement occurs.

The consolidation settlements in cohesive soils are time-dependent; consequently, transient loads have negligible effect. However, in cohesionless soils where the permeability is sufficiently high, elastic deformation of the supporting soil due to transient load can take place. Because deformation in cohesionless soils often takes place during construction while the loads are being applied, it can be accommodated by the structure to an extent, depending on the type of structure and construction method.

Deformation in cohesionless, or granular, soils often occurs as soon as loads are applied. As a consequence, settlements due to transient loads may be significant in cohesionless soils, and they should be included in settlement analyses.

10.6.2.4—Settlement Analyses

10.6.2.4.1—General

Foundation settlements should be estimated using computational methods based on the results of laboratory or insitu testing, or both. The soil parameters used in the computations should be chosen to reflect the loading history of the ground, the construction sequence, and the effects of soil layering.

Both total and differential settlements, including time dependent effects, shall be considered.

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$$S_t = S_e + S_c + S_s \qquad (10.6.2.4.1\text{-}1)$$

where:

S_e = elastic settlement (ft)
S_c = primary consolidation settlement (ft)
S_s = secondary settlement (ft)

C10.6.2.4.1

Elastic, or immediate, settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. The magnitude of elastic settlement is estimated as a function of the applied stress beneath a footing or embankment. Elastic settlement is usually small and neglected in design, but where settlement is critical, it is the most important deformation consideration in cohesionless soil deposits and for footings bearing on rock. For footings located on over-consolidated clays, the magnitude of elastic settlement is not necessarily small and should be checked.

In a nearly saturated or saturated cohesive soil, the pore water pressure initially carries the applied stress. As pore water is forced from the voids in the soil by the applied load, the load is transferred to the soil skeleton. Consolidation settlement is the gradual compression of the soil skeleton as the pore water is forced from the voids in the soil. Consolidation settlement is the most important deformation consideration in cohesive soil deposits that possess sufficient strength to safely support a spread footing. While consolidation settlement can occur in saturated cohesionless soils, the consolidation occurs quickly and is normally not distinguishable from the elastic settlement.

Secondary settlement, or creep, occurs as a result of the plastic deformation of the soil skeleton under a constant effective stress. Secondary settlement is of

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principal concern in highly plastic or organic soil deposits. Such deposits are normally so obviously weak and soft as to preclude consideration of bearing a spread footing on such materials.

The principal deformation component for footings on rock is elastic settlement, unless the rock or included discontinuities exhibit noticeable time-dependent behavior.

To avoid overestimation, relevant settlements should be evaluated using the construction-point concept described in Samtani et al. (2010).

The effects of the zone of stress influence, or vertical stress distribution, beneath a footing shall be considered in estimating the settlement of the footing.

Spread footings bearing on a layered profile consisting of a combination of cohesive soil, cohesionless soil and/or rock shall be evaluated using an appropriate settlement estimation procedure for each layer within the zone of influence of induced stress beneath the footing.

Unless otherwise specified for a specific settlement estimation method, the distribution of vertical stress increase below circular or square and long rectangular footings, i.e., where $L > 5B$, may be estimated using Figure 10.6.2.4.1-1.

For guidance on vertical stress distribution for complex footing geometries, see Poulos and Davis (1974) or Lambe and Whitman (1969).

Some methods used for estimating settlement of footings on sand include an integral method to account for the effects of vertical stress increase variations. For guidance regarding application of these procedures, see Gifford et al. (1987) and Samtani and Nowatzki (2006).

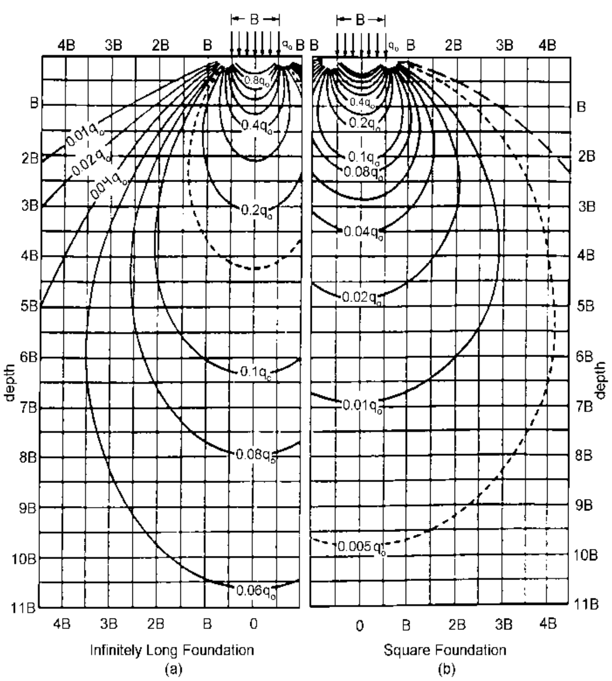

Figure 10.6.2.4.1-1—Boussinesq Vertical Stress Contours for Continuous and Square Footings Modified after Sowers (1979)

10.6.2.4.2—Settlement of Footings on Cohesionless Soils

10.6.2.4.2a—General

The settlement of spread footings bearing on cohesionless soil deposits shall be estimated as a function

C10.6.2.4.2a

For general guidance regarding the estimation of elastic settlement of footings on cohesionless soils, see

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of effective footing width and shall consider the effects of footing geometry and soil and rock layering with depth.

Gifford et al. (1987), and Kimmerling (2002), and Samtani and Nowatzki (2006).

Although methods are recommended for the determination of settlement of cohesionless soils, experience has indicated that settlements can vary considerably in a construction site, and this variation may not be predicted by conventional calculations.

Settlements of cohesionless soils occur rapidly, essentially as soon as the foundation is loaded. Therefore, the total settlement under the service loads may not be as important as the incremental settlement between intermediate load stages. For example, the total and differential settlement- due to loads applied by columns and cross beams is generally less important than the total and differential settlements due to girder placement and casting of continuous concrete decks.

The stress distributions used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth. The settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footings, respectively. The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

Spread footings of the dimensions normally used for bridges are generally assumed to be rigid, although the actual performance will be somewhere between perfectly rigid and perfectly flexible, even for relatively thick concrete footings, due to stress redistribution and concrete creep.

Settlements of footings on cohesionless soils shall be estimated using elastic theory or calibrated empirical procedures.

Generally conservative settlement estimates may be obtained using the empirical methods by Hough and Schmertmann. Additional information regarding the accuracy of the methods described herein is provided in Gifford et al. (1987) and Kimmerling (2002), Samtani and Nowatzki (2006), and Samtani and Allen (2018). This information, in combination with local experience and engineering judgment, should be used when determining the estimated settlement for a structure foundation, as there may be cases, such as attempting to build a structure grade high to account for the estimated settlement, when overestimating the settlement magnitude could be problematic. Samtani and Allen (2018) found that soil settlement estimation using these methods does not provide meaningful results for estimated settlements of less than 0.5 in. Therefore, assessment of the angular distortions and associated force effects such as shear and moment in a structure resulting from differential settlement within or between foundation elements should not depend on the ability to predict a foundation settlement of less than 0.5 in. Hence, in cases where the estimated total settlement of footings on soil is less than 0.5 in., a minimum differential settlement of 0.5 in. should be assumed for calculating the angular distortions and associated force effects for structural analysis.

If the structure foundation is located in or adjacent to a bridge approach fill or reinforced soil structure,

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settlement due the fill or reinforced soil structure should be included in the footing settlement estimate, unless the approach fill or reinforced soil structure settlement is allowed to occur before the bridge foundation is constructed. Both the Hough and Schmertmann methods have been successfully used to estimate fill/reinforced soil structure settlement when cohesionless soils are present (Samtani and Allen, 2018).

Details of other settlement estimation procedures can be found in textbooks and engineering manuals, including:

- Terzaghi and Peck (1967)
- Sowers (1979)
- U.S. Department of the Navy (1982)
- D'Appolonia (Gifford et al., 1987)—This method includes consideration for over-consolidated sands.
- Tomlinson (1986)
- Gifford et al. (1987)
- Elastic Half Space Method (Munkfakh et al., 2001)

These methods, however, have not been calibrated.

Use of methods based on local geologic conditions and calibration require approval from the Owner.

Calibration of local methods should be based on processes as described in Samtani and Kulicki (2018) and Samtani and Allen (2018).

10.6.2.4.2b—Hough Method

Estimation of spread footing settlement on cohesionless soils by the empirical Hough method shall be determined using Eqs. 10.6.2.4.2b-1 and 10.6.2.4.2b-2. *SPT* blow counts shall be corrected as specified in Article 10.4.6.2.4 for depth, i.e. overburden stress, and *SPT* hammer efficiency, before correlating the SPT blow counts to the bearing capacity index, C'.

$$S_e = \sum_{i=1}^{n} \Delta H_i \tag{10.6.2.4.2b-1}$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right) \tag{10.6.2.4.2b-2}$$

where:

n	=	number of soil layers within zone of stress influence of the footing
ΔH_i	=	elastic settlement of layer i (ft)
H_c	=	initial height of layer i (ft)
C'	=	bearing capacity index from Figure 10.6.2.4.2b-1 (dim)
σ'_o	=	initial vertical effective stress at the midpoint of layer i (ksf)
$\Delta\sigma_v$	=	increase in vertical stress at the midpoint of layer i (ksf)

C10.6.2.4.2b

The Hough method was developed for normally consolidated cohesionless soils.

The Hough method has several advantages over other methods used to estimate settlement in cohesionless soil deposits, including express consideration of soil layering and the zone of stress influence beneath a footing of finite size.

The subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about three times the footing width. The maximum layer thickness should be about 10.0 ft.

While Hough (1959) did not specifically state that the *SPT N* values should be corrected for hammer energy in addition to overburden pressure, due to the vintage of the original work, hammers that typically have an efficiency of approximately 60 percent were in general used to develop the empirical correlations contained in the method. If using *SPT* hammers with efficiencies that differ significantly from this 60 percent value, the N values should also be corrected for hammer energy, in effect requiring that $N1_{60}$ be used (Samtani and Nowatzki, 2006).

Studies conducted by Gifford et al. (1987) and Samtani and Nowatzki (2006) indicate that Hough's procedure may be more conservative, but with less prediction variability, than the Schmertmann Method. However, this difference is mostly taken into account through the load factor, γ_{SE}, since it has been calibrated using reliability theory (Kulicki et al. 2015) (Samtani and Kulicki, 2018) (Samtani and Allen (2018).

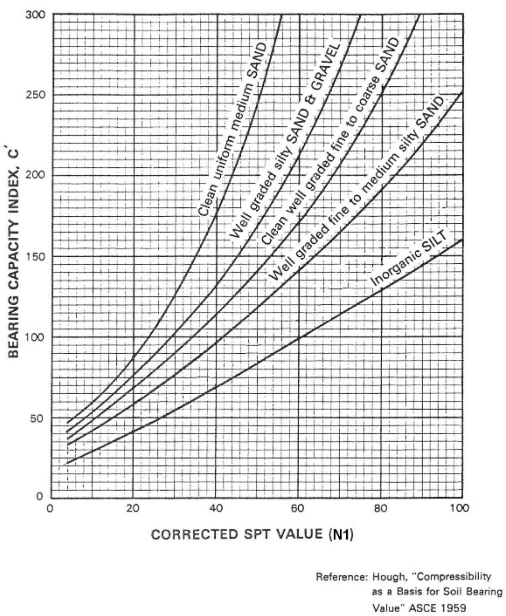

Figure 10.6.2.4.2b-1—Bearing Capacity Index versus Corrected *SPT* (Hough, 1959, as modified in Samtani and Nowatzki, 2006)

The Hough method is applicable to cohesionless soil deposits. The "Inorganic Silt" curve should generally not be applied to soils that exhibit plasticity because N-values in such soils are unreliable (Samtani and Nowatzki, 2006). The settlement characteristics of cohesive soils that exhibit plasticity should be investigated using undisturbed samples and laboratory consolidation tests as prescribed in Article 10.6.2.4.3.

10.6.2.4.2c—Schmertmann Method

Estimation of spread footing immediate, or elastic, settlement, S_i, on cohesionless soils by the empirical Schmertmann, method shall be calculated using Eq. 10.6.2.4.2c-1.

$$S_i = C_1 C_2 \Delta p \sum_{i=1}^{n} \Delta J_i \tag{10.6.2.4.2c-1}$$

in which:

$$\Delta J_i = H_c \left(\frac{I_z}{144 XE} \right) \tag{10.6.2.4.2c-2}$$

$$C_1 = 1 - 0.5 \left(\frac{p_o}{\Delta p} \right) \quad \geq 0.5 \tag{10.6.2.4.2c-3}$$

$$C_2 = 1 + 0.2 \log_{10} \left(\frac{t}{0.1} \right) \tag{10.6.2.4.2c-4}$$

where:

ΔJ_i = elastic spring stiffness of layer, *i* (ft/ksf)
H_c = height of compressible soil layer, *i* (ft)
I_z = strain influence factor from Figure 10.6.2.4.2c-1a. The dimensions L_f and B_f represent the least lateral dimension of the footing after correction for eccentricities, i.e., use effective footing dimension. The strain influence factor is a function of depth and is obtained from the strain influence diagram. The

C10.6.2.4.2c

Background information for this method, originally published in Schmertmann (1970) and Schmertmann et al. (1978), in the format as presented here can be found in Samtani and Nowatzki (2006). This method was originally developed for use with the static cone bearing resistance q_c, in which q_c was correlated to the soil modulus, *E*, and *E* is used directly in this method. The original formulation for this correlation by Schmertmann (1970) assumed *E* was in units of tsf (i.e., *E* (in tsf) = $2q_c$ (in tsf or kg/cm^2). The correlation in Table 10.6.2.4.2c-1 predicts *E* in ksi. Correlations between *E* and the *SPT N* values are also available and provided in Table 10.6.2.4.2c-1.

The variables in the equation for ΔJ_i (Eq. 10.6.2.4.2c-2) require specific units for H_c (ft) and *E* (provided in Table 10.6.2.4.2c-1) is in ksi. This results in the units for ΔJ_i being ft/ksf. Furthermore, in Eqs. 10.6.2.4.2c-1 and 10.6.2.4.2c-3, units of p_o and Δp must be ksf.

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strain influence diagram is constructed for the axisymmetric case (L_f/B_f = 1) and the plane strain case ($L_f/B_f \geq 10$) as shown in Figure 10.6.2.4.2c-1a. The strain influence diagram for intermediate conditions should be determined by simple linear interpolation.

n = number of soil layers within the zone of strain influence (strain influence diagram).

Δp = net uniform applied stress (load intensity) at the foundation depth as shown in Figure 10.6.2.4.2c-1b in which p is equal to the uniform applied footing stress, σ_v, as specified in Article 11.6.3.2 (see Figure 10.6.2.4.2c-1b) (ksf).

E = elastic modulus of layer i, estimated using Table 10.6.2.4.2c-1 (ksi).

X = a factor used to determine the value of elastic modulus. The value of elastic modulus is based on correlations with $N1_{60}$-values or q_c from Table 10.6.2.4.2c-1, then values of X shall be taken as follows:

X = 1.25 for axisymmetric case (L_f/B_f = 1)
X = 1.75 for plane strain case ($L_f/B_f \geq 10$)

Use interpolation for footings with values of L_f/B_f between 1 and 10. If the value of elastic modulus is based on in-situ testing (e.g., pressuremeter), use X = 1.0.

C_1 = correction factor to incorporate the effect of strain relief due to embedment

p_o = effective in-situ overburden stress at the foundation depth as shown in Figure 10.6.2.4.2c-1b (ksf)

p_{op} = effective in-situ overburden stress at the depth to peak strain influence factor, I_{zp}, as shown in Figure 10.6.2.4.2c-1b (ksf)

Δp = net uniform applied stress (load intensity) at the foundation depth as shown in Figure 10.6.2.4.2c-1b in which p is equal to the uniform applied footing stress, σ_v, as specified in Article 11.6.3.2 (ksf).

C_2 = correction factor to incorporate time-dependent (creep) increase in settlement for time, t, after construction

t = time from completion of construction to date under consideration for evaluation of C_2 (yrs)

The C_2 parameter shall not be used to estimate time-dependent consolidation settlements. Where consolidation settlement can occur within the depth of the strain distribution diagram, the magnitude of the consolidation settlement shall be estimated as per Article 10.6.2.4.3 and added to the immediate settlement of other layers within the strain distribution diagram where consolidation settlement may not occur.

For C_2, correction factor, the time duration, t, in Eq. 10.6.2.4.2c-4 is set to 0.1 years to evaluate the settlement immediately after construction, i.e., C_2 = 1. If long-term creep movement of the soil is suspected then an appropriate time duration, t, should be used in the computation of C_2. Creep movement is not the same as consolidation settlement. This factor can have an important influence on the reported settlement since it is included in Eq. 10.6.2.4.2c-1 as a multiplier. For example, the C_2 factor for time durations of 0.1 yrs, 1 yr, 10 yrs, and 50 yrs are 1.0, 1.2, 1.4, and 1.54, respectively. In cohesionless soils and unsaturated fine-grained cohesive soils with low plasticity, time durations of 0.1 yr and 1 yr, respectively, are generally appropriate and sufficient for cases of static loads.

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Table 10.6.2.4.2c-1—Correlations between Elastic Soil Modulus and *SPT* $N1_{60}$ or static Cone q_c values for the Schmertmann Method (modified after Schmertmann 1970, and Samtani and Nowatzki 2006)

Correlation between *E* and *SPT* $N1_{60}$ Value	
Soil Type	***E* (ksi)**
Silts, sandy silts, slightly cohesive mixtures	$0.056\ N1_{60}$
Clean fine to medium sands and slightly silty sands	$0.097\ N1_{60}$
Coarse sands and sands with little gravel	$0.139\ N1_{60}$
Sandy gravel and gravels	$0.167\ N1_{60}$
Correlation between *E* and q_c (static cone resistance, in ksi)	
Soil Type	***E* (ksi)**
Sandy soils	$0.028q_c$

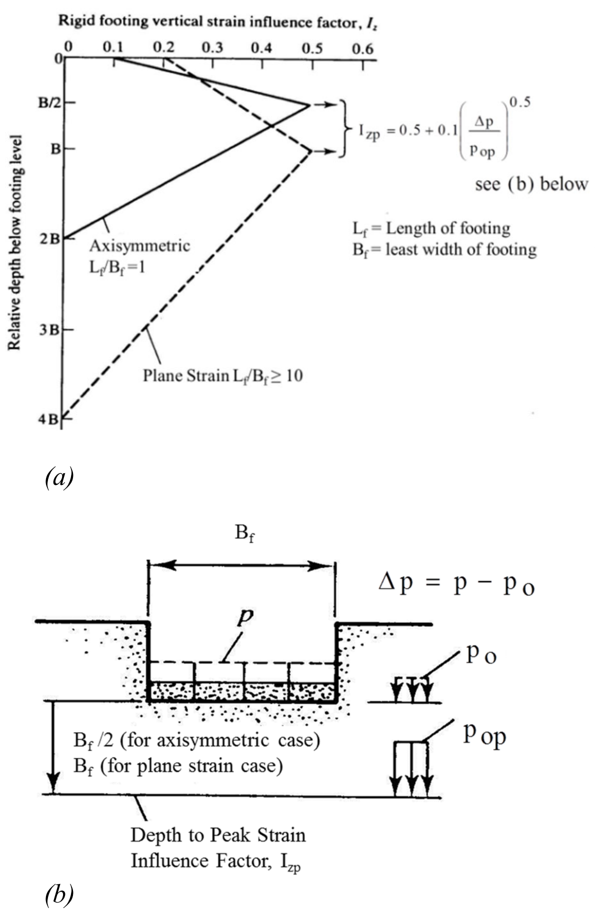

Figure 10.6.2.4.2c-1—(a) Simplified vertical strain influence factor distributions, (b) Explanation of pressure terms in equation for I_{zp} (after Schmertmann et al., 1978, as reported in Samtani and Nowatzki, 2006).

10.6.2.4.3—Settlement of Footings on Cohesive Soils

Spread footings in which cohesive soils are located within the zone of stress influence shall be investigated for consolidation settlement. Elastic and secondary settlement shall also be investigated in consideration of the timing and sequence of construction loading and the tolerance of the structure to total and differential movements.

Where laboratory test results are expressed in terms of void ratio, e, the consolidation settlement of footings shall be taken as:

- For overconsolidated soils where $\sigma'_p > \sigma'_o$, as illustrated in Figure 10.6.2.4.3-2:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[C_r \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_c \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right] \quad (10.6.2.4.3\text{-}1)$$

- For normally consolidated soils where $\sigma'_p = \sigma'_o$:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[\left(C_c \log\frac{\sigma'_f}{\sigma'_p}\right)\right] \quad (10.6.2.4.3\text{-}2)$$

- For underconsolidated soils where $\sigma'_p < \sigma'_o$:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[C_c \log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right)\right] \quad (10.6.2.4.3\text{-}3)$$

Where laboratory test results are expressed in terms of vertical strain, ε_v, the consolidation settlement of footings shall be taken as:

- For overconsolidated soils where $\sigma'_p > \sigma'_o$, as illustrated in Figure 10.6.2.4.3-2:

$$S_c = H_c\left[C_{r\varepsilon} \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right] \quad (10.6.2.4.3\text{-}4)$$

- For normally consolidated soils where $\sigma'_p = \sigma'_o$:

$$S_c = H_c C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_p}\right) \quad (10.6.2.4.3\text{-}5)$$

- For underconsolidated soils where $\sigma'_p<\sigma'_o$:

$$S_c = H_c C_{c\varepsilon} \log\left(\frac{\sigma'_f}{\sigma'_{pc}}\right) \quad (10.6.2.4.3\text{-}6)$$

C10.6.2.4.3

In practice, footings on cohesive soils are most likely founded on overconsolidated clays, and settlements can be estimated using elastic theory (Baguelin et al., 1978), or the tangent modulus method (Janbu, 1963, 1967). Settlements of footings on overconsolidated clay usually occur at approximately one order of magnitude faster than soils without preconsolidation, and it is reasonable to assume that they take place as rapidly as the loads are applied. Infrequently, a layer of cohesive soil may exhibit a preconsolidation stress less than the calculated existing overburden stress. The soil is then said to be underconsolidated because a state of equilibrium has not yet been reached under the applied overburden stress. Such a condition may have been caused by a recent lowering of the groundwater table. In this case, consolidation settlement will occur due to the additional load of the structure and the settlement that is occurring to reach a state of equilibrium. The total consolidation settlement due to these two components can be estimated by Eq. 10.6.2.4.3-3 or Eq. 10.6.2.4.3-6.

Normally consolidated and underconsolidated soils should be considered unsuitable for direct support of spread footings due to the magnitude of potential settlement, the time required for settlement, for low shear strength concerns, or any combination of these design considerations. Preloading or vertical drains may be considered to mitigate these concerns.

To account for the decreasing stress with increased depth below a footing and variations in soil compressibility with depth, the compressible layer should be divided into vertical increments, i.e., typically 5.0 to 10.0 ft for most normal width footings for highway applications, and the consolidation settlement of each increment analyzed separately. The total value of S_c is the summation of S_c for each increment.

The magnitude of consolidation settlement depends on the consolidation properties of the soil. These properties include the compression and recompression constants, C_c and C_r, or $C_{c\varepsilon}$, and $C_{r\varepsilon}$; the preconsolidation stress, σ'_p; the current, initial vertical effective stress, σ'_o; and the final vertical effective stress after application of additional loading, σ'_f. An overconsolidated soil has been subjected to larger stresses in the past than at present. This could be a result of preloading by previously overlying strata, desiccation, groundwater lowering, glacial overriding or an engineered preload. If $\sigma'_o = \sigma'_p$, the soil is normally consolidated. Because the recompression constant is typically about an order of magnitude smaller than the compression constant, an accurate determination of the preconsolidation stress, σ'_p, is needed to make reliable estimates of consolidation settlement.

The reliability of consolidation settlement estimates is also affected by the quality of the consolidation test sample and by the accuracy with which changes in σ'_p with depth are known or estimated. As shown in Figure C10.6.2.4.3-1, the slope of the e or ε_v versus log

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where:

H_c = initial height of compressible soil layer (ft)
e_o = void ratio at initial vertical effective stress (dim)
C_r = recompression index (dim)
C_c = compression index (dim)
$C_{r\varepsilon}$ = recompression ratio (dim)
$C_{c\varepsilon}$ = compression ratio (dim)
σ'_p = maximum past vertical effective stress in soil at midpoint of soil layer under consideration (ksf)
σ'_o = initial vertical effective stress in soil at midpoint of soil layer under consideration (ksf)

σ'_f = final vertical effective stress in soil at midpoint of soil layer under consideration (ksf)
σ'_{pc} = current vertical effective stress in soil, not including the additional stress due to the footing loads, at midpoint of soil layer under consideration (ksf)

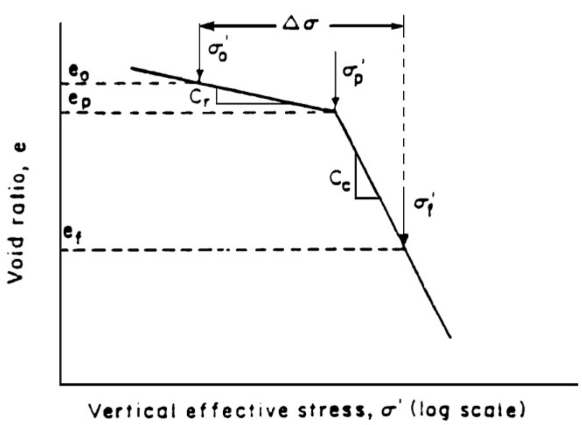

Figure 10.6.2.4.3-1—Typical Consolidation Compression Curve for Overconsolidated Soil: Void Ratio versus Vertical Effective Stress, EPRI (1983)

σ'_v curve and the location of σ'_p can be strongly affected by the quality of samples used for the laboratory consolidation tests. In general, the use of poor quality samples will result in an overestimate of consolidation settlement. Typically, the value of σ'_p will vary with depth as shown in Figure C10.6.2.4.3-2. If the variation of σ'_p with depth is unknown, e.g., only one consolidation test was conducted in the soil profile, actual settlements could be higher or lower than the computed value based on a single value of σ'_p.

The cone penetrometer test may be used to improve understanding of both soil layering and variation of σ'_p with depth by correlation to laboratory tests from discrete locations.

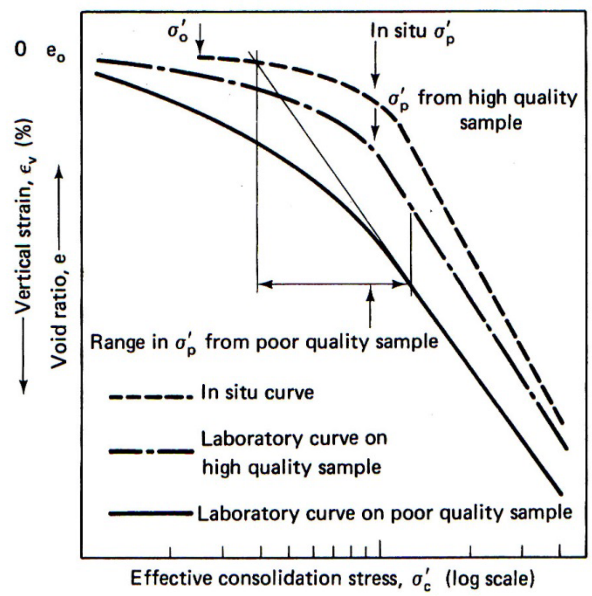

Figure C10.6.2.4.3-1—Effects of Sample Quality on Consolidation Test Results, Holtz and Kovacs (1981)

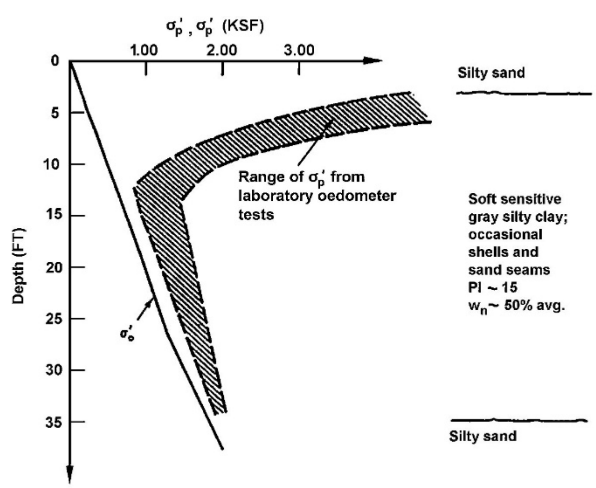

Figure C10.6.2.4.3-2—Typical Variation of Preconsolidation Stress with Depth, Holtz and Kovacs (1981)

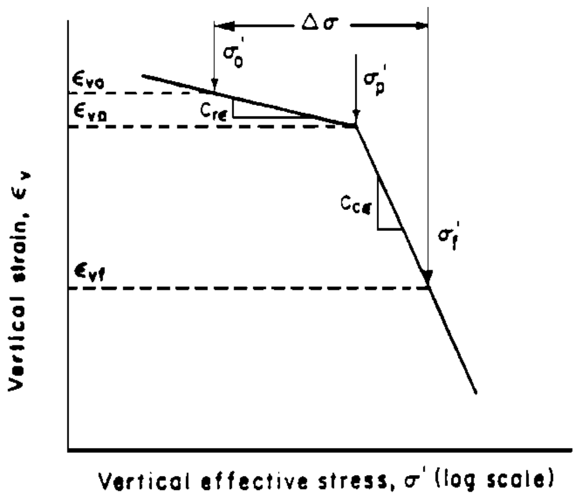

Figure 10.6.2.4.3-2—Typical Consolidation Compression Curve for Overconsolidated Soil: Vertical Strain versus Vertical Effective Stress, EPRI (1983)

If the footing width, B, is small relative to the thickness of the compressible soil, H_c, the effect of three-dimensional loading shall be considered and shall be taken as:

$$S_{c(3-D)} = \mu_c S_{c(1-D)} \quad (10.6.2.4.3\text{-}7)$$

where:

μ_c = reduction factor taken as specified in Figure 10.6.2.4.3-3 (dim)

$S_{c(1\text{-}D)}$ = single dimensional consolidation settlement (ft)

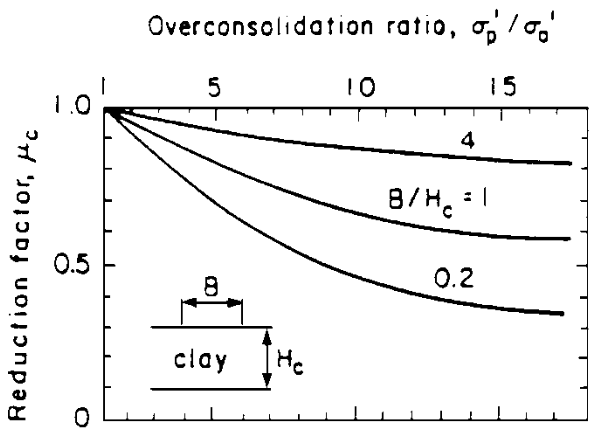

Figure 10.6.2.4.3-3—Reduction Factor to Account for Effects of Three-dimensional Consolidation Settlement (EPRI, 1983)

The time, t, to achieve a given percentage of the total estimated one-dimensional consolidation settlement shall be taken as:

Consolidation occurs when a saturated compressible layer of soil is loaded and water is squeezed out of the layer. The time required for the (primary) consolidation process to end will depend on the permeability of the soil. Because the time factor, T, is defined as logarithmic, the consolidation process theoretically never ends. The

$$t = \frac{TH_d^{\;2}}{c_v} \qquad (10.6.2.4.3\text{-}8)$$

where:

T = time factor taken as specified in Figure 10.6.2.4.3-4 for the excess pore pressure distributions shown in the Figure (dim)
H_d = length of longest drainage path in compressible layer under consideration (ft)
c_v = coefficient of consolidation (ft^2/yr)

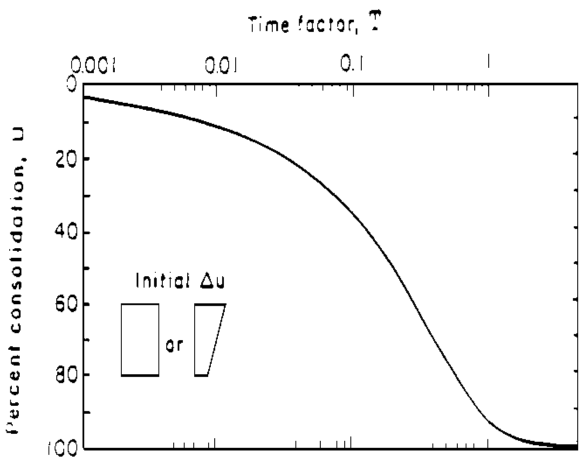

Figure 10.6.2.4.3-4 Percentage of Consolidation as a Function of Time Factor, *T* (EPRI, 1983)

Where laboratory test results are expressed in terms of void ratio, e, the secondary settlement of footings on cohesive soil shall be taken as:

$$S_s = \frac{C_\alpha}{1+e_o} H_c \log\left(\frac{t_2}{t_1}\right) \qquad (10.6.2.4.3\text{-}9)$$

Where laboratory test results are expressed in terms of vertical strain, ε_V, the secondary settlement of footings on cohesive soils shall be taken as:

$$S_s = C_{\alpha\varepsilon} H_c \log\left(\frac{t_2}{t_1}\right) \qquad (10.6.2.4.3\text{-}10)$$

practical assumption is usually made that the additional consolidation past 90 percent or 95 percent consolidation is negligible, or is taken into consideration as part of the total long-term settlement. Refer to Winterkorn and Fang (1975) for values of T for excess pore pressure distributions other than indicated in Figure 10.6.2.4.3-4.

The length of the drainage path is the longest distance from any point in a compressible layer to a drainage boundary at the top or bottom of the compressible soil unit. Where a compressible layer is located between two drainage boundaries, H_d equals one-half the actual height of the layer. Where a compressible layer is adjacent to an impermeable boundary (usually below), H_d equals the full height of the layer.

Computations to predict the time rate of consolidation based on the result of laboratory tests generally tend to over-estimate the actual time required for consolidation in the field. This over-estimation is principally due to:

- the presence of thin drainage layers within the compressible layer that are not observed from the subsurface exploration nor considered in the settlement computations,
- the effects of three-dimensional dissipation of pore water pressures in the field, rather than the one-dimensional dissipation that is imposed by laboratory odometer tests and assumed in the computations, and
- the effects of sample disturbance, which tend to reduce the permeability of the laboratory tested samples.

If the total consolidation settlement is within the serviceability limits for the structure, the time rate of consolidation is usually of lesser concern for spread footings. If the total consolidation settlement exceeds the serviceability limitations, superstructure damage will occur unless provisions are made for timing of closure pours as a function of settlement, simple support of spans and/or periodic jacking of bearing supports.

Secondary compression component if settlement results from compression of bonds between individual clay particles and domains, as well as other effects on the microscale that are not yet clearly understood (Holtz and Kovacs, 1981). Secondary settlement is most important for highly plastic clays and organic and micaceous soils. Accordingly, secondary settlement predictions should be considered as approximate estimates only.

If secondary compression is estimated to exceed serviceability limitations, either deep foundations or ground improvement should be considered to mitigate the effects of secondary compression. Experience indicates preloading and surcharging may not be effective in eliminating secondary compression.

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where:

H_c = initial height of compressible soil layer (ft)
e_o = void ratio at initial vertical effective stress (dim)
t_1 = time when secondary settlement begins, i.e., typically at a time equivalent to 90 percent average degree of primary consolidation (yr)
t_2 = arbitrary time that could represent the service life of the structure (yr)
C_α = secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (dim)
$C_{\alpha\varepsilon}$ = modified secondary compression index estimated from the results of laboratory consolidation testing of undisturbed soil samples (dim)

10.6.2.4.4—Settlement of Footings on Rock

For footings bearing on fair to very good rock, according to the Geomechanics Classification system and designed in accordance with the provisions of this Section, elastic settlements may generally be assumed to be less than 0.5 in. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics shall be made.

Where rock is broken or jointed (relative rating of ten or less for *RQD* and joint spacing), the rock joint condition is poor (relative rating of ten or less) or the criteria for fair to very good rock are not met, a settlement analysis should be conducted, and the influence of rock type, condition of discontinuities, and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock, in feet, should be taken as:

- For circular (or square) footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{rI_p}{144\,E_m} \tag{10.6.2.4.4-1}$$

in which:

$$I_p = \frac{\left(\sqrt{\pi}\right)}{\beta_z} \tag{10.6.2.4.4-2}$$

- For rectangular footings:

$$\rho = q_o\left(1-\nu^2\right)\frac{BI_p}{144\,E_m} \tag{10.6.2.4.4-3}$$

in which:

$$I_p = \frac{\left(L/B\right)^{1/2}}{\beta_z} \tag{10.6.2.4.4-4}$$

C10.6.2.4.4

In most cases, it is sufficient to determine settlement using the average bearing stress under the footing.

Where the foundations are subjected to a very large load or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus, E_m. In some cases, the value of E_m can be estimated through empirical correlation with the value of the modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in-situ tests, such as plate loading and pressuremeter tests.

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where:

q_o = applied vertical stress at base of loaded area (ksf)
ν = Poisson's Ratio (dim)
r = radius of circular footing or $B/2$ for square footing (ft)
I_p = influence coefficient to account for rigidity and dimensions of footing (dim)
E_m = rock mass modulus (ksi)
β_z = factor to account for footing shape and rigidity (dim)

Values of I_p should be computed using the β_z values presented in Table 10.6.2.4.4-1 for rigid footings. Where the results of laboratory testing are not available, values of Poisson's ratio, ν, for typical rock types may be taken as specified in Table C10.4.6.5-2. Determination of the rock mass modulus, E_m, should be based on the methods described in Sabatini (2002).

The magnitude of consolidation and secondary settlements in rock masses containing soft seams or other material with time-dependent settlement characteristics should be estimated by applying procedures specified in Article 10.6.2.4.3.

Table 10.6.2.4.4-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

10.6.2.5—Bearing Resistance at the Service Limit State

10.6.2.5.1—Presumptive Values for Bearing Resistance

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

C10.6.2.5.1

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.5.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

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Table C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.5.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3 f'_c$.

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10.6.3—Strength Limit State Design

10.6.3.1—Bearing Resistance of Soil

10.6.3.1.1—General

Bearing resistance of spread footings shall be determined based on the highest anticipated position of groundwater level at the footing location.

The factored bearing resistance, q_R, at the strength limit state shall be taken as:

$$q_R = \varphi_b \, q_n \quad (10.6.3.1.1\text{-}1)$$

where:

φ_b = resistance factor specified in Article 10.5.5.2.2
q_n = nominal bearing resistance (ksf)

Where loads are eccentric, the effective footing dimensions, L' and B', as specified in Article 10.6.1.3, shall be used instead of the overall dimensions L and B in all equations, tables, and figures pertaining to bearing resistance.

C10.6.3.1.1

The bearing resistance of footings on soil should be evaluated using soil shear strength parameters that are representative of the soil shear strength under the loading conditions being analyzed. The bearing resistance of footings supported on granular soils should be evaluated for both permanent dead loading conditions and short-duration live loading conditions using effective stress methods of analysis and drained soil shear strength parameters. The bearing resistance of footings supported on cohesive soils should be evaluated for short-duration live loading conditions using total stress methods of analysis and undrained soil shear strength parameters. In addition, the bearing resistance of footings supported on cohesive soils, which could soften and lose strength with time, should be evaluated for permanent dead loading conditions using effective stress methods of analysis and drained soil shear strength parameters.

The position of the groundwater table can significantly influence the bearing resistance of soils through its effect on shear strength and unit weight of the foundation soils. In general, the submergence of soils will reduce the effective shear strength of cohesionless (or granular) materials, as well as the long-term (or drained) shear strength of cohesive (clayey) soils. Moreover, the effective unit weights of submerged soils are about half of those for the same soils under dry conditions. Thus, submergence may lead to a significant reduction in the bearing resistance provided by the foundation soils, and it is essential that the bearing resistance analyses be carried out under the assumption of the highest groundwater table expected within the service life of the structure.

Footings with inclined bases should be avoided wherever possible. Where use of an inclined footing base cannot be avoided, the nominal bearing resistance determined in accordance with the provisions herein should be further reduced using accepted corrections for inclined footing bases in Munfakh et al. (2001).

Because the effective dimensions will vary slightly for each limit state under consideration, strict adherence to this provision will require re-computation of the nominal bearing resistance at each limit state.

Further, some of the equations for the bearing resistance modification factors based on L and B were not necessarily or specifically developed with the intention that effective dimensions be used. The designer should ensure that appropriate values of L and B are used, and that effective footing dimensions L' and B' are used appropriately.

Consideration should be given to the relative change in the computed nominal resistance based on effective versus gross footing dimensions for the size of footings typically used for bridges. Judgment should be used in deciding whether the use of gross footing dimensions for computing nominal bearing resistance at the strength limit state would result in a conservative design.

10.6.3.1.2—Theoretical Estimation

10.6.3.1.2a—Basic Formulation

The nominal bearing resistance shall be estimated using accepted soil mechanics theories and should be based on measured soil parameters. The soil parameters used in the analyses shall be representative of the soil shear strength under the considered loading and subsurface conditions.

The nominal bearing resistance of spread footings on cohesionless soils shall be evaluated using effective stress analyses and drained soil strength parameters.

The nominal bearing resistance of spread footings on cohesive soils shall be evaluated for total stress analyses and undrained soil strength parameters. In cases where the cohesive soils may soften and lose strength with time, the bearing resistance of these soils shall also be evaluated for permanent loading conditions using effective stress analyses and drained soil strength parameters.

For spread footings bearing on compacted soils, the nominal bearing resistance shall be evaluated using the more critical of either total or effective stress analyses.

Except as noted below, the nominal bearing resistance of a soil layer, in ksf, should be taken as:

$$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5\gamma_f B N_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

c = cohesion, taken as undrained shear strength (ksf)

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

C10.6.3.1.2a

The bearing resistance formulation provided in Eqs. 10.6.3.1.2a-1 though 10.6.3.1.2a-4 is the complete formulation as described in the Munfakh et al. (2001). However, in practice, not all of the factors included in these equations have been routinely used.

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N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
γ_q = total (moist) unit weight of soil above the bearing depth of the footing (kcf)
γ_f = total (moist) unit weight of soil below the bearing depth of the footing (kcf)
D_f = footing embedment depth (ft)
B = footing width (ft)
$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
s_c, s_γ, s_q = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)
d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation determined from Eq. 10.6.3.1.2a-10 (dim)
i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

For ϕ_f = 0:

$$i_c = 1 - (nH/cBLN_c) \tag{10.6.3.1.2a-5}$$

For ϕ_f > 0:

$$i_c = i_q - [(1 - i_q)/(N_q - 1)] \tag{10.6.3.1.2a-6}$$

in which:

$$i_q = \left[1 - \frac{H}{(V + cBL\cot\phi_f)}\right]^n \tag{10.6.3.1.2a-7}$$

$$i_\gamma = \left[1 - \frac{H}{V + cBL\cot\phi_f)}\right]^{(n+1)} \tag{10.6.3.1.2a-8}$$

$$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta \tag{10.6.3.1.2a-9}$$

where:

B = footing width (ft)
L = footing length (ft)
H = unfactored horizontal load (kips)
V = unfactored vertical load (kips)
θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

Most geotechnical engineers nationwide have not used the load inclination factors. This is due, in part, to the lack of knowledge of the vertical and horizontal loads at the time of geotechnical explorations and preparation of bearing resistance recommendations.

Furthermore, the basis of the load inclination factors computed by Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8 is a combination of bearing resistance theory and small scale load tests on 1.0 in. wide plates on London Clay and Ham River Sand (Meyerhof, 1953). Therefore, the factors do not take into consideration the effects of depth of embedment. Meyerhof further showed that for footings with a depth of embedment ratio of $D_f/B = 1$, the effects of load inclination on bearing resistance are relatively small. The theoretical formulation of load inclination factors were further examined by Brinch-Hansen (*1970*), with additional modification by Vesic (*1973*) into the form provided in Eqs. 10.6.3.1.2a-5 to 10.6.3.1.2a-8.

It should further be noted that the resistance factors provided in Article 10.5.5.2.2 were derived for vertical loads. The applicability of these resistance factors to design of footings resisting inclined load combinations is not currently known. The combination of the resistance factors and the load inclination factors may be overly conservative for footings with an embedment of approximately $D_f/B = 1$ or deeper because the load inclination factors were derived for footings without embedment.

In practice, therefore, for footings with modest embedment, consideration may be given to omission of the load inclination factors.

Figure C10.6.3.1.2a-1 shows the convention for determining the θ angle in Eq. 10.6.3.1.2a-9.

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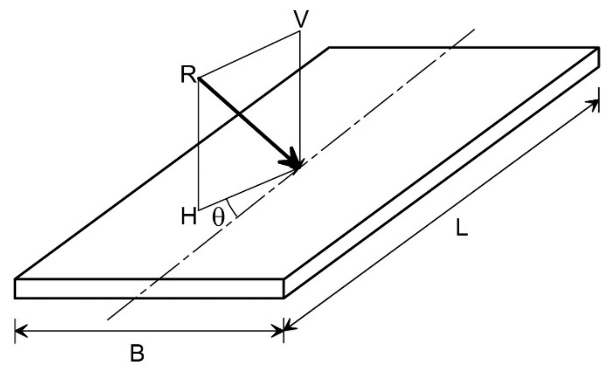

Figure C10.6.3.1.2a-1—Inclined Loading Conventions

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

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Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

$$d_q = 1 + 2\tan\phi_f(1-\sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right) \quad (10.6.3.1.2a\text{-}10)$$

where:

d_q = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation(dim)
ϕ_f = angle of internal friction of soil (degrees)
D_f = footing embedment depth (ft)
B = footing width (ft)

Arctan (D_f/B) is in radians.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0. The depth correction factor, d_q, shall not exceed 1.4.

Eq. 10.6.3.1.2a-10 has been verified to cover a range of friction angle, ϕ_f, of 32 degrees to 42 degrees, and a range of D_f/B of 1 to 8. Depth correction factor values beyond this range have not been verified at this time.

10.6.3.1.2b—Considerations for Punching Shear

If local or punching shear failure is possible, the nominal bearing resistance shall be estimated using reduced shear strength parameters c^* and ϕ^* in Eqs. 10.6.3.1.2b-1 and 10.6.3.1.2b-2. The reduced shear parameters may be taken as:

$$c^* = 0.67c \quad (10.6.3.1.2b\text{-}1)$$

$$\phi^* = \tan^{-1}(0.67\tan\phi_f) \quad (10.6.3.1.2b\text{-}2)$$

where:

c^* = reduced effective stress soil cohesion for punching shear (ksf)
ϕ^* = reduced effective stress soil friction angle for punching shear (degrees)

C10.6.3.1.2b

Local shear failure is characterized by a failure surface that is similar to that of a general shear failure but that does not extend to the ground surface, ending somewhere in the soil below the footing. Local shear failure is accompanied by vertical compression of soil below the footing and visible bulging of soil adjacent to the footing but not by sudden rotation or tilting of the footing. Local shear failure is a transitional condition between general and punching shear failure. Punching shear failure is characterized by vertical shear around the perimeter of the footing and is accompanied by a vertical movement of the footing and compression of the soil immediately below the footing but does not affect the soil outside the loaded area. Punching shear failure occurs in loose or compressible soils, in weak soils under slow (drained) loading, and in dense sands for deep footings subjected to high loads.

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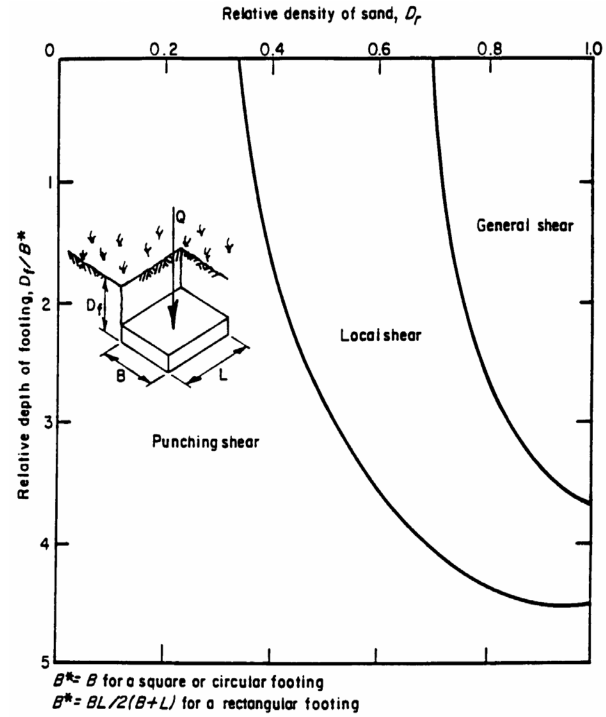

Figure C10.6.3.1.2b-1—Modes of Bearing Capacity Failure for Footings in Sand

10.6.3.1.2c—Considerations for Footings on Slopes

For footings constructed on or adjacent to slopes, the nominal bearing resistance shall be determined using a reduction coefficient (RC_{BC}) as presented in Tables 10.6.3.1.2c-1 and 10.6.3.1.2c-2. The reduction coefficient should be applied directly to the nominal bearing resistance calculated from Eq.10.6.3.1.2a-1 for footings on level ground and supported on the same foundation soil conditions.

The nominal resistance of footings on or adjacent to slopes shall be taken as:

$$q_{n-sloping\ ground} = RC_{BC} q_n = RC_{BC}\left(cN_c + 0.5\gamma BN_\gamma\right) \tag{10.6.3.1.2c-1}$$

where:

$q_{n\text{-}sloping\ ground}$ = the nominal footing bearing resistance considering the effect of sloping ground (ksf)

RC_{BC} = reduction coefficient for bearing resistance due to slope effects (dim)

and other variables are as defined in Article 10.6.3.1.2a and Figure 10.6.3.1.2c-1. The bearing capacity factors N_c and N_γ are obtained in accordance with Article 10.6.3.1.2a.

Reduction coefficients (RC_{BC}) should be determined using the definitions illustrated in Figure 10.6.3.1.2c-1

C10.6.3.1.2c

A rational approach for determining a modified bearing resistance for footings on or adjacent to a slope is presented in Leshchinsky (2015) and Leshchinsky and Xie (2016). These methods are considered valid and applicable to structure foundations in addition to the MSE retaining wall example presented in the reference papers. The reduction coefficients provided in Tables 10.6.3.1.2c-1 and 10.6.3.1.2c-2 are modified and reconfigured by the author of the cited papers to allow for more convenient use in practice. See the original papers for the complete tabulation of reduction coefficient values.

The reduction coefficients are applicable to purely cohesive, purely cohesionless and c-ϕ soils. The RC_{BC} factors are based on no footing embedment for footings either on or adjacent to slopes and may be conservative for deep footing embedment depths.

Limit analysis, or limit equilibrium analysis, should be considered to estimate the nominal bearing resistance of footings on or adjacent to slopes composed of soils and/or site conditions that are not consistent with the parameters and conditions described in the reference documents (i.e. embedment >0, layered soils, steeper slopes).

The schematic shown in Figure 10.6.3.1.2c-1 is provided only for illustrating and defining the terms used in the design equations and tables. This figure should not be used as the basis for locating footings on slopes regarding embedment depth and setback.

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for footings on or adjacent to slopes. Use linear interpolation to obtain reduction coefficients for values not provided. The slope stability factor, N_s, in Tables 10.6.3.1.2c-1 and 10.6.3.1.2c-2 shall be taken as:

$$N_s = \frac{\gamma H_s}{c} \qquad (10.6.3.1.2c\text{-}2)$$

where:

N_s = slope stability factor (dim)
H_s = height of sloping ground surface below bottom of footing (ft)

and other variables are as defined in Article 10.6.3.1.2a.

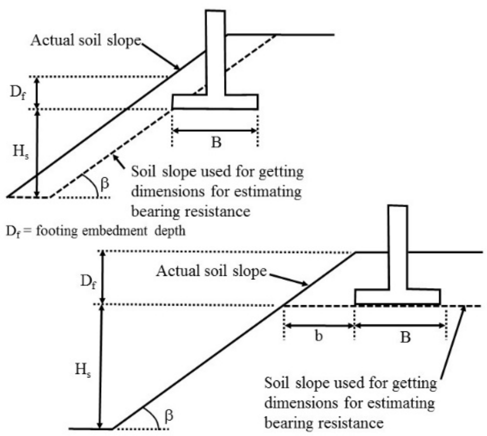

Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC_{BC}

Table 10.6.3.1.2c-1—Reduction Coefficients (RC_{BC}) for Footings Placed on Slopes Composed of either Purely Cohesive Soils, (ϕ = 0); Purely Cohesionless Soils (c'=0); or Soils with both Cohesive and Cohesionless Strength Components

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
φ (°)	***B/H***	***b/B***	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**
	0.1		0.89	0.89	0.88	0.00	0.89	0.88	0.87	0.00	0.85	0.84	0.83	0.00	0.77	0.76	0.74	0.00
	0.2		0.89	0.88	0.88	0.00	0.89	0.87	0.86	0.00	0.82	0.81	0.78	0.00	0.76	0.73	0.69	0.00
	0.4		0.88	0.87	0.86	0.00	0.89	0.86	0.82	0.00	0.81	0.77	0.66	0.00	0.74	0.68	0.53	0.00
0	**0.6**	**0 (On Slope)**	0.89	0.87	0.84	0.00	0.88	0.84	0.71	0.00	0.81	0.74	0.53	0.00	0.74	0.64	0.41	0.00
	1		0.87	0.84	0.75	0.00	0.87	0.79	0.56	0.00	0.80	0.66	0.42	0.00	0.73	0.56	0.33	0.00
	1.5		0.87	0.82	0.62	0.00	0.87	0.72	0.47	0.00	0.80	0.61	0.37	0.00	0.73	0.54	0.30	0.00
	3		0.87	0.73	0.47	0.00	0.87	0.67	0.37	0.00	0.83	0.62	0.31	0.00	0.80	0.59	0.28	0.00
	0.1		0.91	0.91	0.91	0.69	0.80	0.79	0.79	0.22	0.64	0.63	0.61	0.00	0.53	0.52	0.50	0.00
	0.2		0.90	0.89	0.90	0.68	0.75	0.73	0.72	0.21	0.62	0.59	0.56	0.00	0.52	0.49	0.45	0.00
	0.4		0.86	0.86	0.84	0.63	0.73	0.70	0.67	0.22	0.62	0.56	0.51	0.00	0.52	0.45	0.39	0.00
20	**0.6**	**0 (On Slope)**	0.85	0.84	0.82	0.58	0.73	0.68	0.63	0.22	0.61	0.54	0.47	0.00	0.51	0.41	0.33	0.00
	1		0.85	0.82	0.78	0.58	0.72	0.64	0.58	0.26	0.61	0.50	0.42	0.00	0.52	0.39	0.30	0.00
	1.5		0.86	0.80	0.75	0.58	0.73	0.62	0.54	0.31	0.65	0.50	0.42	0.00	0.60	0.44	0.34	0.00
	3		0.90	0.77	0.72	0.58	0.88	0.66	0.56	0.35	0.86	0.61	0.51	0.00	0.85	0.57	0.46	0.00
	0.1		0.93	0.92	0.91	0.77	0.65	0.64	0.63	0.40	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
	0.2		0.81	0.82	0.84	0.76	0.64	0.61	0.59	0.39	0.50	0.47	0.44	0.11	0.39	0.35	0.32	0.00
	0.4		0.79	0.79	0.78	0.72	0.63	0.59	0.55	0.37	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
30	**0.6**	**0 (On Slope)**	0.78	0.77	0.75	0.68	0.62	0.56	0.52	0.36	0.49	0.41	0.36	0.14	0.39	0.30	0.24	0.00
	1		0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.48	0.33	0.26	0.00
	1.5		0.79	0.73	0.69	0.66	0.72	0.56	0.50	0.46	0.68	0.47	0.39	0.33	0.64	0.41	0.33	0.00
	3		0.95	0.74	0.70	0.65	0.92	0.66	0.60	0.51	0.90	0.62	0.57	0.43	0.88	0.59	0.51	0.00
	0.1		0.74	0.77	0.79	0.80	0.52	0.51	0.50	0.38	0.37	0.36	0.34	0.17	0.28	0.26	0.24	0.05
	0.2		0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
	0.4		0.67	0.69	0.67	0.72	0.50	0.45	0.43	0.36	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.06
40	**0.6**	**0 (On Slope)**	0.67	0.67	0.64	0.66	0.50	0.43	0.43	0.34	0.40	0.34	0.26	0.17	0.32	0.22	0.18	0.08
	1		0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.54	0.33	0.27	0.24
	1.5		0.76	0.65	0.61	0.74	0.74	0.53	0.48	0.56	0.71	0.47	0.40	0.47	0.68	0.43	0.36	0.41
	3		0.95	0.74	0.71	0.77	0.94	0.68	0.65	0.66	0.91	0.67	0.62	0.62	0.92	0.67	0.59	0.57

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Table 10.6.3.1.2c-2—Reduction Coefficients (RC_{BC}) for Footings Placed Adjacent to Slopes Composed of either Purely Cohesive Soils, (ϕ = 0); Purely Cohesionless Soils (c'=0); or Soils with both Cohesive and Cohesionless Strength Components

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
φ (°)	***B/H***	***b/B***	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**
0	**0.2**	0	0.89	0.88	0.88	0.00	0.89	0.87	0.86	0.00	0.82	0.81	0.78	0.00	0.76	0.73	0.69	0.00
		0.5	0.97	0.96	0.96	0.00	0.95	0.93	0.91	0.00	0.92	0.89	0.87	0.00	0.86	0.83	0.76	0.00
		1.25	1.00	0.99	0.98	0.00	1.00	0.98	0.96	0.00	1.00	0.97	0.95	0.00	0.95	0.91	0.81	0.00
		2.5	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	0.97	0.84	0.00
		5	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	0.89	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	0.5	0	0.92	0.91	0.88	0.00	0.85	0.82	0.76	0.00	0.77	0.73	0.63	0.00	0.71	0.65	0.52	0.00
		0.5	0.96	0.95	0.89	0.00	0.92	0.89	0.78	0.00	0.87	0.84	0.68	0.00	0.83	0.76	0.56	0.00
		1.25	0.98	0.97	0.90	0.00	0.96	0.94	0.80	0.00	0.94	0.92	0.71	0.00	0.90	0.83	0.58	0.00
		2.5	1.00	1.00	1.00	0.00	1.00	1.00	0.86	0.00	1.00	1.00	0.79	0.00	1.00	0.93	0.68	0.00
		5	1.00	1.00	1.00	0.00	1.00	1.00	0.95	0.00	1.00	1.00	0.93	0.00	1.00	1.00	0.88	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	1	0	0.87	0.84	0.75	0.00	0.87	0.79	0.56	0.00	0.80	0.66	0.42	0.00	0.73	0.56	0.33	0.00
		0.5	0.95	0.91	0.82	0.00	0.92	0.83	0.65	0.00	0.86	0.73	0.46	0.00	0.81	0.67	0.40	0.00
		1.25	0.97	0.94	0.83	0.00	0.95	0.87	0.67	0.00	0.92	0.81	0.50	0.00	0.89	0.76	0.46	0.00
		2.5	1.00	0.98	0.88	0.00	1.00	0.97	0.77	0.00	1.00	1.00	0.84	0.00	0.99	0.92	0.63	0.00
		5	1.00	1.00	0.95	0.00	1.00	1.00	0.90	0.00	1.00	1.00	0.84	0.00	1.00	1.00	0.83	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	2	0	0.87	0.79	0.57	0.00	0.87	0.71	0.44	0.00	0.81	0.62	0.35	0.00	0.75	0.56	0.29	0.00
		0.5	0.97	0.93	0.65	0.00	0.94	0.79	0.49	0.00	0.89	0.72	0.42	0.00	0.85	0.69	0.37	0.00
		1.25	0.99	0.98	0.73	0.00	0.99	0.91	0.57	0.00	0.98	0.86	0.51	0.00	0.96	0.83	0.47	0.00
		2.5	1.00	0.99	0.82	0.00	1.00	0.96	0.69	0.00	1.00	0.95	0.64	0.00	1.00	0.95	0.61	0.00
		5	1.00	1.00	0.96	0.00	1.00	1.00	0.87	0.00	1.00	1.00	0.84	0.00	1.00	1.00	0.81	0.00
		10	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
20	**0.2**	0	0.90	0.89	0.90	0.68	0.75	0.73	0.72	0.21	0.62	0.59	0.56	0.00	0.52	0.49	0.45	0.00
		0.5	0.78	0.87	0.86	0.70	0.74	0.76	0.74	0.40	0.63	0.65	0.63	0.00	0.52	0.56	0.52	0.00
		1.25	0.86	0.92	0.92	0.82	0.83	0.84	0.83	0.70	0.74	0.75	0.74	0.00	0.63	0.66	0.63	0.00
		2.5	0.96	0.98	0.99	0.83	0.95	0.94	0.95	0.84	0.90	0.89	0.90	0.00	0.78	0.81	0.78	0.00
		5	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	0.96	0.98	0.96	0.00
		10	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	0.99	0.99	1.00	0.00
	0.5	0	0.86	0.86	0.84	0.60	0.73	0.70	0.67	0.22	0.62	0.56	0.51	0.00	0.52	0.45	0.39	0.00
		0.5	0.84	0.91	0.92	0.71	0.80	0.80	0.79	0.40	0.70	0.68	0.67	0.00	0.62	0.59	0.56	0.00
		1.25	0.88	1.00	0.97	0.82	0.85	0.88	0.86	0.70	0.76	0.75	0.75	0.00	0.68	0.66	0.64	0.00
		2.5	0.97	1.00	1.00	0.81	0.95	0.97	0.98	0.84	0.90	0.94	0.96	0.00	0.84	0.86	0.87	0.00
		5	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	1	0	0.85	0.82	0.78	0.58	0.72	0.64	0.58	0.26	0.61	0.50	0.42	0.00	0.52	0.39	0.30	0.00
		0.5	0.84	0.91	0.91	0.71	0.81	0.80	0.79	0.46	0.70	0.69	0.67	0.00	0.64	0.62	0.60	0.00
		1.25	0.87	0.95	0.96	0.82	0.85	0.85	0.85	0.73	0.76	0.76	0.75	0.00	0.71	0.70	0.69	0.00
		2.5	0.97	1.00	1.00	0.82	0.95	0.97	0.98	0.83	0.90	0.94	0.97	0.00	0.86	0.89	0.91	0.00
		5	1.00	1.00	1.00	0.83	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	0.83	1.00	1.00	1.00	0.81	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
	2	0	0.90	0.90	0.90	0.58	0.87	0.86	0.84	0.33	0.84	0.81	0.78	0.00	0.81	0.77	0.74	0.00
		0.5	0.90	0.93	0.93	0.70	0.88	0.88	0.87	0.54	0.84	0.83	0.81	0.00	0.84	0.82	0.81	0.00
		1.25	0.92	0.97	0.99	0.81	0.90	0.92	0.92	0.77	0.86	0.86	0.86	0.00	0.85	0.85	0.84	0.00
		2.5	0.98	1.00	1.00	0.81	0.97	0.98	1.00	0.81	0.93	0.97	1.00	0.00	0.92	0.96	0.99	0.00
		5	1.00	1.00	1.00	0.82	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	0.82	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.00	1.00	1.00	1.00	0.00

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Table 10.6.3.1.2c-2 (cont.)— Reduction Coefficients (RC_{BC}) for Footings Placed Adjacent to Slopes Composed of either Purely Cohesive Soils, (ϕ = 0); Purely Cohesionless Soils (c'=0); or Soils with both Cohesive and Cohesionless Strength Components

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
φ (°)	***B/H***	***b/B***	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**
30	**0.2**	0	0.93	0.92	0.91	0.76	0.65	0.64	0.63	0.39	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
		0.5	0.74	0.81	0.80	0.75	0.70	0.66	0.65	0.50	0.57	0.52	0.49	0.21	0.47	0.42	0.39	0.00
		1.25	0.78	0.85	0.86	0.86	0.74	0.73	0.72	0.72	0.63	0.60	0.59	0.38	0.54	0.50	0.47	0.00
		2.5	0.84	0.92	0.93	0.99	0.81	0.82	0.83	0.94	0.72	0.73	0.74	0.74	0.64	0.62	0.61	0.00
		5	0.95	1.00	1.00	1.00	0.93	0.98	1.00	1.00	0.88	0.95	1.00	0.97	0.80	0.85	0.87	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	0.5	0	0.79	0.79	0.78	0.70	0.63	0.59	0.55	0.36	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
		0.5	0.76	0.87	0.87	0.74	0.72	0.71	0.70	0.51	0.58	0.56	0.54	0.24	0.49	0.46	0.43	0.00
		1.25	0.79	0.85	0.92	0.87	0.75	0.73	0.76	0.72	0.63	0.62	0.61	0.45	0.54	0.52	0.50	0.00
		2.5	0.87	0.91	1.00	0.99	0.84	0.85	0.90	0.98	0.74	0.78	0.80	0.80	0.67	0.70	0.71	0.00
		5	0.97	1.00	1.00	1.00	0.95	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.85	0.94	0.98	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	1	0	0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.48	0.33	0.26	0.00
		0.5	0.78	0.87	0.89	0.74	0.75	0.74	0.74	0.51	0.64	0.62	0.60	0.35	0.59	0.56	0.54	0.00
		1.25	0.81	0.90	0.91	0.88	0.78	0.78	0.78	0.72	0.68	0.67	0.66	0.58	0.64	0.62	0.61	0.00
		2.5	0.88	0.99	1.00	0.96	0.85	0.90	0.92	0.95	0.78	0.81	0.84	0.88	0.75	0.78	0.80	0.00
		5	0.97	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.89	0.98	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	2	0	0.88	0.88	0.87	0.65	0.87	0.85	0.83	0.48	0.85	0.82	0.80	0.38	0.83	0.80	0.76	0.00
		0.5	0.89	0.91	0.91	0.75	0.89	0.89	0.87	0.58	0.88	0.86	0.84	0.51	0.87	0.85	0.82	0.00
		1.25	0.90	0.92	0.93	0.88	0.90	0.90	0.90	0.75	0.89	0.87	0.87	0.70	0.89	0.87	0.86	0.00
		2.5	0.97	1.00	1.00	1.00	0.96	0.97	0.98	0.98	0.92	0.94	0.96	0.95	0.91	0.92	0.94	0.00
		5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
40	**0.2**	0	0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
		0.5	0.65	0.73	0.71	0.74	0.60	0.55	0.53	0.38	0.64	0.38	0.35	0.25	0.34	0.29	0.25	0.13
		1.25	0.68	0.77	0.75	0.86	0.63	0.60	0.58	0.55	0.74	0.44	0.42	0.39	0.39	0.34	0.31	0.25
		2.5	0.72	0.83	0.84	1.00	0.68	0.68	0.68	0.76	0.87	0.53	0.53	0.62	0.45	0.43	0.41	0.48
		5	0.80	0.93	0.95	1.00	0.76	0.82	0.85	1.00	1.00	0.72	0.76	1.00	0.57	0.61	0.63	0.94
		10	0.94	1.00	1.00	1.00	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.76	0.93	1.00	1.00
	0.5	0	0.67	0.69	0.67	0.69	0.50	0.45	0.43	0.35	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.07
		0.5	0.68	0.81	0.81	0.73	0.63	0.62	0.61	0.46	0.47	0.44	0.41	0.25	0.39	0.35	0.32	0.09
		1.25	0.70	0.82	0.84	0.85	0.65	0.65	0.66	0.60	0.51	0.49	0.47	0.40	0.43	0.41	0.39	0.18
		2.5	0.76	0.92	0.96	1.00	0.72	0.77	0.80	0.81	0.59	0.62	0.63	0.60	0.54	0.56	0.56	0.37
		5	0.84	1.00	1.00	1.00	0.81	0.91	0.94	1.00	0.71	0.82	0.88	1.00	0.67	0.77	0.83	0.84
		10	0.96	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.89	1.00	1.00	1.00	0.86	1.00	1.00	1.00
	1	0	0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.54	0.33	0.27	0.24
		0.5	0.77	0.81	0.82	0.74	0.75	0.73	0.72	0.49	0.71	0.66	0.62	0.38	0.68	0.62	0.57	0.30
		1.25	0.78	0.84	0.85	0.84	0.77	0.76	0.75	0.64	0.73	0.69	0.66	0.55	0.71	0.66	0.63	0.48
		2.5	0.83	0.92	0.95	1.00	0.81	0.85	0.87	0.85	0.76	0.78	0.79	0.76	0.75	0.76	0.77	0.72
		5	0.89	1.00	1.00	1.00	0.87	0.95	0.98	1.00	0.80	0.90	0.95	1.00	0.80	0.89	0.94	1.00
		10	0.98	1.00	1.00	1.00	0.97	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.93	1.00	1.00	1.00
	2	0	0.93	0.92	0.89	0.45	0.92	0.90	0.87	0.60	0.91	0.88	0.84	0.53	0.89	0.85	0.81	0.47
		0.5	0.93	0.95	0.93	0.76	0.93	0.92	0.90	0.65	0.92	0.89	0.87	0.64	0.92	0.89	0.86	0.60
		1.25	0.93	0.95	0.94	0.86	0.93	0.93	0.92	0.78	0.93	0.91	0.89	0.74	0.93	0.90	0.88	0.74
		2.5	0.94	0.99	1.00	1.00	0.94	0.98	0.98	0.92	0.94	0.97	0.97	0.87	0.94	0.96	0.96	0.88
		5	0.95	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.98	1.00	1.00	1.00	0.96	1.00	1.00	1.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00

10.6.3.1.2d—Considerations for Two-Layer Soil Systems—Critical Depth

Where the soil profile contains a second layer of soil with different properties affecting shear strength within a distance below the footing less than H_{crit}, the bearing resistance of the layered soil profile shall be determined using the provisions for two-layered soil systems herein. The distance H_{crit}, in feet, may be taken as:

$$H_{crit} = \frac{(3B)\ \ln\left(\frac{q_1}{q_2}\right)}{2\left(1+\frac{B}{L}\right)} \quad (10.6.3.1.2d\text{-}1)$$

where:

q_1 = nominal bearing resistance of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (ksf)
q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (ksf)
B = footing width (ft)
L = footing length (ft)

10.6.3.1.2e—Two-Layered Soil System in Undrained Loading

Where a footing is supported on a two-layered soil system subjected to undrained loading, the nominal bearing resistance may be determined using Eq. 10.6.3.1.2a-1 with the following modifications:

c_1 = undrained shear strength of the top layer of soil as depicted in Figure 10.6.3.1.2e-1 (ksf)
N_{cm} = N_m, a bearing capacity factor as specified below (dim)
N_{qm} = 1.0 (dim)

Where the bearing stratum overlies a stiffer cohesive soil, N_m, may be taken as specified in Figure 10.6.3.1.2e-2.

Where the bearing stratum overlies a softer cohesive soil, N_m may be taken as:

$$N_m = \left(\frac{1}{\beta_m} + \kappa s_c N_c\right) \le s_c N_c \quad (10.6.3.1.2e\text{-}1)$$

in which:

$$\beta_m = \frac{BL}{2(B+L)H_{s2}} \quad (10.6.3.1.2e\text{-}2)$$

C10.6.3.1.2e

Vesic' (1970) developed a rigorous solution for the modified bearing capacity factor, N_m, for the weak undrained layer over strong undrained layer situation. This solution is given by the following equation:

$$N_m = \frac{\kappa N_c^*(N_c^* + \beta_m - 1)A}{B\ \ C - (\kappa N_c^* + \beta_m - 1))(N_c^* + 1)} \quad (C10.6.3.1.2e\text{-}1)$$

in which:

$$A = \left[(\kappa+1)N_c^{*2} + (1+\kappa\beta_m)N_c^* + \beta_m - 1\right] \quad (C10.6.3.1.2e\text{-}2)$$

$$B = \left[\kappa(\kappa+1)N_c^* + \kappa + \beta_m - 1\right] \quad (C10.6.3.1.2e\text{-}3)$$

$$C = \left[(N_c^* + \beta_m)N_c^* + \beta_m - 1\right] \quad (C10.6.3.1.2e\text{-}4)$$

- For circular or square footings:

$$\beta_m = \frac{B}{4H_{s2}} \quad (C10.6.3.1.2e\text{-}5)$$

$$N_c^* = 6.17$$

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$$\kappa = \frac{c_2}{c_1} \tag{10.6.3.1.2e-3}$$

where:

β_m = punching index (dim)
c_1 = undrained shear strength of upper soil layer (ksf)
c_2 = undrained shear strength of lower soil layer (ksf)
H_{s2} = distance from bottom of footing to top of the second soil layer (ft)
s_c = shape correction factor determined from Table 10.6.3.1.2a-3
N_c = bearing capacity factor determined herein (dim)
N_{qm} = bearing capacity factor determined herein (dim)

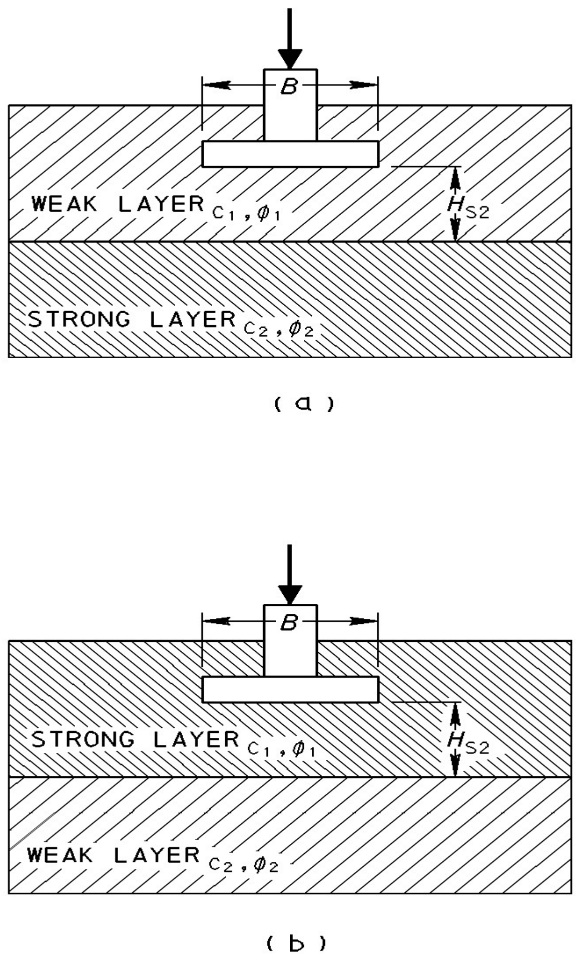

Figure 10.6.3.1.2e-1—Two-Layer Soil Profiles

- For strip footings:

$$\beta_m = \frac{B}{2H_{s2}} \tag{C10.6.3.1.2e-6}$$

$$N_c^* = 5.14$$

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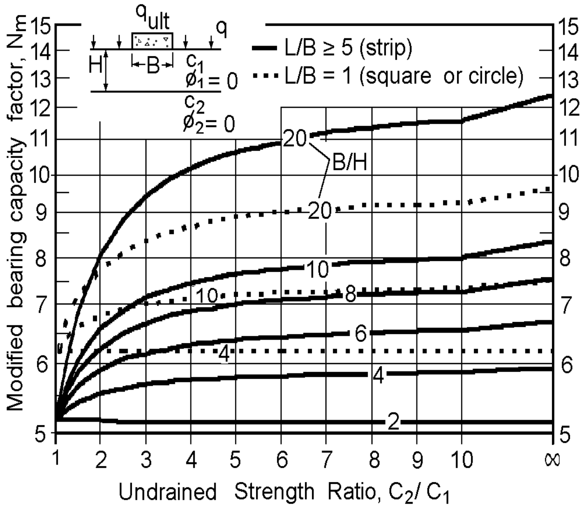

Figure 10.6.3.1.2e-2—Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (EPRI, 1983)

10.6.3.1.2f—Two-Layered Soil System in Drained Loading

Where a footing supported on a two-layered soil system is subjected to a drained loading, the nominal bearing resistance, in ksf, may be taken as:

$$q_n = \left[q_2 + \left(\frac{1}{K}\right)c_1' \cot\phi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right]K \tan\phi_1'\left(\frac{H_{s2}}{B}\right)} - \left(\frac{1}{K}\right)c_1' \cot\phi_1' \quad (10.6.3.1.2f\text{-}1)$$

in which:

$$K = \frac{1-\sin^2\phi_1'}{1+\sin^2\phi_1'} \quad (10.6.3.1.2f\text{-}2)$$

where:

c'_1 = drained shear strength of the top layer of soil in a two-layer system, such as depicted in Figure 10.6.3.1.2e-1 (ksf)

q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (ksf)

ϕ'_1 = effective stress angle of internal friction of the top layer of soil (degrees)

C10.6.3.1.2f

If the upper layer is a cohesionless soil and ϕ' equals 25–50 degrees, Eq. 10.6.3.1.2f-1 reduces to:

$$q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H_{s2}}{B}} \quad (C10.6.3.1.2f\text{-}1)$$

10.6.3.1.3—Semiempirical Procedures

The nominal bearing resistance of foundation soils may be estimated from the results of in-situ tests or by observed resistance of similar soils. The use of a particular in-situ test and the interpretation of test results should take local experience into consideration. The following in-situ tests may be used:

C10.6.3.1.3

In application of these empirical methods, the use of average *SPT* blow counts and *CPT* tip resistances is specified. The resistance factors recommended for bearing resistance included in Table 10.5.5.2.2-1 assume the use of average values for these parameters. The use of lower bound values may result in an overly conservative design. However, depending on the availability of soil

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- Standard Penetration Test (*SPT*)
- Cone Penetration Test (*CPT*)

The nominal bearing resistance in sand, in ksf, based on *SPT* results may be taken as:

$$q_n = \frac{\bar{N}1_{60}B}{5}\left(C_{wq}\frac{D_f}{B} + C_{w\gamma}\right) \quad (10.6.3.1.3\text{-}1)$$

where:

$\bar{N}1_{60}$ = average *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/ft) as specified in Article 10.4.6.2.4. Average the blow count over a depth range from the bottom of the footing to 1.5*B* below the bottom of the footing.
B = footing width (ft)
C_{wq}, $C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
D_f = footing embedment depth taken to the bottom of the footing (ft)

The nominal bearing resistance, in ksf, for footings on cohesionless soils based on *CPT* results may be taken as:

$$q_n = \frac{\overline{q_c}B}{40}\left(C_{wq}\frac{D_f}{B} + C_{w\gamma}\right) \quad (10.6.3.1.3\text{-}2)$$

where:

$\overline{q_c}$ = average cone tip resistance within a depth range, *B*, below the bottom of the footing (ksf)
B = footing width (ft)
C_{wq}, $C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
D_f = footing embedment depth taken to the bottom of the footing (ft)

property data and the variability of the geologic strata under consideration, it may not be possible to reliably estimate the average value of the properties needed for design. In such cases, the Engineer may have no choice but to use a more conservative selection of design input parameters to mitigate the additional risks created by potential variability or the paucity of relevant data.

The original derivation of Eqs. 10.6.3.1.3-1 and 10.6.3.1.3-2 did not include inclination factors (Meyerhof, 1956).

10.6.3.1.4—Plate Load Tests

The nominal bearing resistance may be determined by plate load tests, provided that adequate subsurface explorations have been made to determine the soil profile below the foundation. Where plate load tests are conducted, they should be conducted in accordance with ASTM D1194.

The nominal bearing resistance determined from a plate load test may be extrapolated to adjacent footings where the subsurface profile is confirmed by subsurface exploration to be similar.

C10.6.3.1.4

Plate load tests have a limited depth of influence and furthermore may not disclose the potential for long-term consolidation of foundation soils.

Scale effects should be addressed when extrapolating the results to performance of full scale footings. Extrapolation of the plate load test data to a full scale footing should be based on the design procedures provided herein for settlement (service limit state) and bearing resistance (strength and extreme event limit state), with consideration to the effect of the stratification, e.g., layer thicknesses, depths, and properties. Plate load test results should be applied only within a sub-area of the

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project site for which the subsurface conditions, e.g., stratification, geologic history, and properties, are relatively uniform.

10.6.3.2—Bearing Resistance of Rock

10.6.3.2.1—General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and *RQD* may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock and the orientation and condition of discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

C10.6.3.2.1

The design of spread footings bearing on rock is frequently controlled by either overall stability, e.g., the orientation and conditions of discontinuities, or load eccentricity considerations. The designer should verify adequate overall stability and size the footing based on eccentricity requirements at the strength limit state before checking nominal bearing resistance at both the service and strength limit states.

The design procedures for foundations in rock have been developed using the RMR, rock mass rating system. Classification of the rock mass should be according to the RMR system. For additional information on the RMR system, see Sabatini et al. (2002).

10.6.3.2.2—Semiempirical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geomechanics RMR system. Local experience shall be considered in the use of these semi-empirical procedures.

The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

C10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on rock mass quality measured in terms of RMR system, the nominal bearing resistance of a rock mass varies from a small fraction to six times the unconfined compressive strength of intact rock core samples.

10.6.3.2.3—Analytic Method

The nominal bearing resistance of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be considered.

C10.6.3.2.3

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing resistance for each of the failure modes can be found in Kulhawy and Goodman (1987), Goodman (1989), and Sowers (1979).

10.6.3.2.4—Load Test

Where appropriate, load tests may be performed to determine the nominal bearing resistance of foundations on rock.

10.6.3.3—Eccentric Load Limitations

The eccentricity of loading at the strength limit state, evaluated based on factored loads shall not exceed:

C10.6.3.3

A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil

- One-third of the corresponding footing dimension, B or L, for footings on soils, or 0.45 of the corresponding footing dimensions B or L, for footings on rock.

conditions. The base widths obtained using the LRFD load factors and eccentricity of $B/3$ were comparable to those of ASD with an eccentricity of $B/6$. For foundations on rock, to obtain equivalence with ASD specifications, a maximum eccentricity of $B/2$ would be needed for LRFD. However, a slightly smaller maximum eccentricity has been specified to account for the potential unknown future loading that could push the resultant outside the footing dimensions.

10.6.3.4—Failure by Sliding

Failure by sliding shall be investigated for footings that support horizontal or inclined load and/or are founded on slopes.

For foundations on clay soils, the possible presence of a shrinkage gap between the soil and the foundation shall be considered. If passive resistance is included as part of the shear resistance required for resisting sliding, consideration shall also be given to possible future removal of the soil in front of the foundation.

The factored resistance against failure by sliding, in kips, shall be taken as:

$$R_R = \varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \quad (10.6.3.4\text{-}1)$$

where:

φ = resistance factor (dim)
R_n = nominal sliding resistance against failure by sliding (kips)
φ_τ = resistance factor for shear resistance between soil and foundation specified in Table 10.5.5.2.2-1
R_τ = nominal sliding resistance between soil and foundation (kips)
φ_{ep} = resistance factor for passive resistance specified in Table 10.5.5.2.2-1
R_{ep} = nominal passive resistance of soil available throughout the design life of the structure (kips)

C10.6.3.4

Sliding failure occurs if the force effects due to the horizontal component of loads exceed the more critical of either the factored shear resistance of the soils or the factored shear resistance at the interface between the soil and the foundation.

For footings on cohesionless soils, sliding resistance depends on the roughness of the interface between the foundation and the soil.

The magnitudes of active earth load and passive resistance depend on the type of backfill material, the wall movement, and the compactive effort. Their magnitude can be estimated using procedures described in Sections 3 and 11.

In most cases, the movement of the structure and its foundation will be small. Consequently, if passive resistance is included in the resistance, its magnitude is commonly taken as 50 percent of the maximum passive resistance. This is the basis for the resistance factor, φ_{ep}, in Table 10.5.5.2.2-1.

The units for R_R, R_n, and R_{ep} are shown in kips. For elements designed on a unit length basis, these quantities will have the units of kips per unit length.

If the soil beneath the footing is cohesionless, the nominal sliding resistance between soil and foundation shall be taken as:

$$R_\tau = CV \tan \phi_f \quad (10.6.3.4\text{-}2)$$

for which:

C = 1.0 for concrete cast against soil
= 0.8 for precast concrete footing

where:

ϕ_f = internal friction angle of drained soil (degrees)
V = total vertical force (kips)

Sliding resistance between the base of a shallow foundation and a granular soil is governed by the coefficient of friction ($\tan \delta$) between the foundation soil and the footing. The value of the coefficient of friction is a function of the soil type and the roughness of the footing. For concrete cast against cohesionless or granular material, the footing base is rough, and the coefficient of friction, $\tan \delta$, will be equal to the tangent of the friction angle ($\tan \phi_f$) for the soil supporting the footing. If the bottom of the footing consists of something other than concrete cast against the ground, then the coefficient of friction ($\tan \delta$) is not solely a property of the soil but is also a function of the footing material type and the roughness of the footing material. For the case of a precast concrete footing, assuming the concrete is smooth, the coefficient of friction should be set equal to 80 percent of $\tan \phi_f$. See Article C3.11.5.3 for additional guidance and

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For footings that rest on clay, where footings are supported on at least 6.0 in. of compacted granular material,the sliding resistance may be taken as the lesser of:

- the cohesion of the clay, or
- one-half the normal stress on the interface between the footing and soil, as shown in Figure 10.6.3.4-1 for retaining walls.

The following notation shall be taken to apply to Figure 10.6.3.4-1:

q_s = unit shear resistance, equal to S_u or 0.5 σ'_v, whichever is less (ksf)
R_τ = nominal sliding resistance between soil and foundation (kips) expressed as the shaded area under the q_s diagram
S_u = undrained shear strength (ksf)
σ'_v = vertical effective stress (ksf)

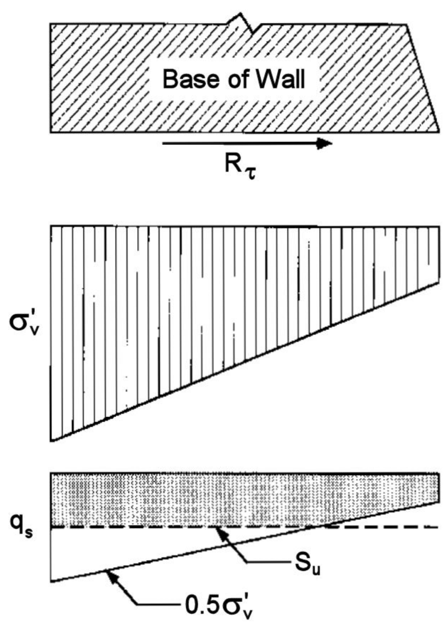

Figure 10.6.3.4-1—Procedure for Estimating Nominal Sliding Resistance for Walls on Clay

10.6.3.5—Overall Stability

Overall stability of a footing shall be evaluated where one or more of the following conditions exist:

background on the determination of interface friction values.

In the absence of specific data (e.g., measured ϕ_f from laboratory testing, ϕ_f determined through correlation to in-situ measured *SPT* or cone resistance values, or measured tan δ from laboratory testing), experience has shown that Table C3.11.5.3-1 may be used to estimate the coefficient of friction tan δ between soil and various footing material types in Eq. 10.6.3.4-2.

- Horizontal or inclined loads are present,
- The foundation is placed on embankment,
- The footing is located on, near or within a slope,
- The possibility of loss of foundation support through erosion or scour exists, or
- Bearing strata are significantly inclined.

10.6.4—Extreme Event Limit State Design

10.6.4.1—General

Extreme limit state design checks for spread footings shall include, but not necessarily be limited to:

- bearing resistance,
- eccentric load limitations (overturning),
- sliding, and
- overall stability.

Resistance factors shall be as specified in Article 10.5.5.3.

10.6.4.2—Eccentric Load Limitations

For footings, whether on soil or on rock, the eccentricity of loading for extreme limit states shall not exceed the limits provided in Article 11.6.5.

If live loads act to reduce the eccentricity for the Extreme I limit state, γ_{EQ} shall be taken as 0.0.

10.6.5—Structural Design

The structural design of footings shall comply with the requirements given in Section 5.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on factored loads shall be used for footings bearing on all soil and rock conditions.

C10.6.5

For purposes of structural design, it is usually assumed that the bearing stress varies linearly across the bottom of the footing. This assumption results in the slightly conservative triangular or trapezoidal contact stress distribution.

10.7—DRIVEN PILES

10.7.1—General

10.7.1.1—Application

Driven piling should be considered in the following situations:

- When spread footings cannot be founded on rock, or on competent soils at a reasonable cost;
- At locations where soil conditions would normally permit the use of spread footings but the potential exists for scour, liquefaction or lateral spreading, in which case driven piles bearing on suitable materials below susceptible soils should be considered for use as a protection against these problems;

- Where right-of-way or other space limitations would not allow the use spread footings;
- Where existing soil, contaminated by hazardous materials, must be removed for the construction of spread footings; or
- Where an unacceptable amount of settlement of spread footings may occur.

10.7.1.2—Minimum Pile Spacing, Clearance, and Embedment into Cap

Center-to-center pile spacing should not be less than 30.0 in. or 2.5 pile diameters. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 in.

The tops of piles shall project at least 12.0 in. into the pile cap after all damaged material has been removed. If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than 6.0 in. into the cap.

Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piles, the concrete cover on the sides of the piles shall not be less than 6.0 in., plus an allowance for permissible pile misalignment. Where pile reinforcement is anchored in the cap satisfying the requirements of Article 5.12.9.1, the projection may be less than 6.0 in.

10.7.1.3—Piles through Embankment Fill

Piles to be driven through embankments should penetrate a minimum of 10.0 ft through original ground unless refusal on bedrock or competent bearing strata occurs at a lesser penetration.

Fill used for embankment construction should be a select material, which does not obstruct pile penetration to the required depth.

C10.7.1.3

If refusal occurs at a depth of less than 10 ft, other foundation types, e.g., footings or shafts, may be more effective.

To minimize the potential for obstruction of the piles, the maximum size of any rock particles in the fill should not exceed 6.0 in. Pre-drilling or spudding pile locations should be considered in situations where obstructions in the embankment fill cannot be avoided, particularly for displacement piles. Note that predrilling or spudding may reduce the pile side resistance and lateral resistance, depending on how the predrilling or spudding is conducted. The diameter of the predrilled or spudded hole, and the potential for caving of the hole before the pile is installed will need to be considered to assess the effect this will have on side and lateral resistance.

If compressible soils are located beneath the embankment, piles should be driven after embankment settlement is complete, if possible, to minimize or eliminate downdrag forces.

10.7.1.4—Batter Piles

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles should be considered for use. Where negative side resistance (downdrag) loads are expected, batter piles should be avoided. If batter piles are used in areas of significant

C10.7.1.4

In some cases, it may be desirable to use batter piles. From a general viewpoint, batter piles provide a much stiffer resistance to lateral loads than would be possible with vertical piles. They can be very effective in resisting static lateral loads.

Due to increased foundation stiffness, batter piles may not be desirable in resisting lateral dynamic loads if

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seismic loading, the design of the pile foundation shall recognize the increased foundation stiffness that results.

10.7.1.5—Pile Design Requirements

Pile design shall address the following issues as appropriate:

- Nominal bearing resistance to be specified in the contract, type of pile, and size of pile group required to provide adequate support, with consideration of how nominal bearing pile resistance will be determined in the field.
- Group interaction.
- Pile quantity estimation and estimated pile penetration required to meet nominal axial resistance and other design requirements.
- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions.
- Foundation deflection to meet the established movement and associated structure performance criteria.
- Pile foundation nominal structural resistance.
- Pile drivability to confirm that acceptable driving stresses and blow counts can be achieved at the nominal bearing resistance, and at the estimated resistance to reach the minimum tip elevation, if a minimum tip elevation is required, with an available driving system.
- Long-term durability of the pile in service, i.e., corrosion and deterioration.

the structure is located in an area where seismic loads are potentially high.

C10.7.1.5

The driven pile design process is discussed in detail in Hannigan et al. (2006).

10.7.1.6—Determination of Pile Loads

10.7.1.6.1—General

The loads and load factors to be used in pile foundation design shall be as specified in Section 3. Computational assumptions that shall be used in determining individual pile loads are described in Section 4.

C10.7.1.6.1

The specification and determination of top of cap loads is discussed in Section 3. The Engineer should select different levels of analysis, detail and accuracy as appropriate for the structure under consideration. Details are discussed in Section 4.

10.7.1.6.2—Downdrag

The provisions of Article 3.11.8 shall apply for determination of load due to negative side resistance.

Where piles are driven to end bearing on a dense stratum or rock and the design of the pile is structurally controlled, downdrag shall be considered at the strength and extreme limit states.

C10.7.1.6.2

Downdrag occurs when settlement of soils along the side of the piles results in downward movement of the soil relative to the pile. See Article C3.11.8.

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For friction piles that can experience settlement at the pile tip, downdrag shall be considered at the service, strength and extreme limit states. Estimate pile and pile group settlement according to Article 10.7.2.

In the case of friction piles with limited tip resistance, the downdrag load can exceed the geotechnical resistance of the pile, causing the pile to move downward enough to allow service limit state criteria for the structure to be exceeded. Where pile settlement is not limited by nominal bearing resistance below the downdrag zone, service limit state tolerances will govern the geotechnical design.

This design situation is not desirable and the preferred practice is to mitigate the downdrag induced foundation settlement through a properly designed surcharge and/or preloading program, or by extending the piles deeper for higher resistance.

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to downdrag computed as specified in Article 3.11.8.

Instrumented static load tests, dynamic tests with signal matching, or static analysis procedures in Article 10.7.3.8.6 may be used to estimate the available nominal resistance to withstand the downdrag plus structure loads.

10.7.1.6.3—Uplift Due to Expansive Soils

Piles penetrating expansive soil shall extend to a depth into moisture-stable soils sufficient to provide adequate anchorage to resist uplift. Sufficient clearance should be provided between the ground surface and underside of caps or beams connecting piles to preclude the application of uplift loads at the pile/cap connection due to swelling ground conditions.

C10.7.1.6.3

Evaluation of potential uplift loads on piles extending through expansive soils requires evaluation of the swell potential of the soil and the extent of the soil strata that may affect the pile. One reasonably reliable method for identifying swell potential is presented in Table 10.4.6.3-1. Alternatively, ASTM D4829 may be used to evaluate swell potential. The thickness of the potentially expansive stratum must be identified by:

- examination of soil samples from borings for the presence of jointing, slickensiding, or a blocky structure and for changes in color, and
- laboratory testing for determination of soil moisture content profiles.

10.7.1.6.4—Nearby Structures

Where pile foundations are placed adjacent to existing structures, the influence of the existing structure on the behavior of the foundation, and the effect of the new foundation on the existing structures, including vibration effects due to pile installation, shall be investigated.

C10.7.1.6.4

Vibration due to pile driving can cause settlement of existing foundations as well as structural damage to the adjacent facility, especially in loose cohesionless soils. The combination of taking measures to mitigate the vibration levels through use of nondisplacement piles, predrilling, proper hammer choice, etc., and a good vibration monitoring program should be considered.

10.7.2—Service Limit State Design

10.7.2.1—General

Service limit state design of driven pile foundations includes the evaluation of settlement due to static loads, and downdrag loads if present, lateral squeeze, and lateral deformation.

The deformation caused by unbalanced lateral forces such as due to overall stability or lateral squeeze should be evaluated and addressed in the design of the pile foundation.

C10.7.2.1

Lateral analysis of pile foundations is conducted to establish the load distribution between the superstructure and foundations for all limit states, and to estimate the deformation in the foundation that will occur due to those loads. This Article only addresses the evaluation of the lateral deformation of the foundation resulting from the distributed loads.

10.7.2.2—Tolerable Movements

The provisions of Article 10.5.2.1 shall apply.

C10.7.2.2

See Article C10.5.2.1.

10.7.2.3—Settlement

10.7.2.3.1—Equivalent Footing Analogy

For purposes of calculating the settlements of pile groups, loads should be assumed to act on an equivalent footing based on the depth of embedment of the piles into the layer that provides support as shown in Figures 10.7.2.3.1-1 and 10.7.2.3.1-2.

Pile group settlement shall be evaluated for pile foundations in cohesive soils, soils that include cohesive layers, and piles in loose granular soils. The load used in calculating settlement shall be the permanently applied load on the foundation.

In applying the equivalent footing analogy for pile foundation, the reduction to equivalent dimensions B' and L' as used for spread footing design does not apply.

C10.7.2.3.1

Pile design should ensure that strength limit state considerations are satisfied before checking service limit state considerations.

For piles embedded adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived.

Methods for calculating settlement are discussed in Hannigan et al. (2006).

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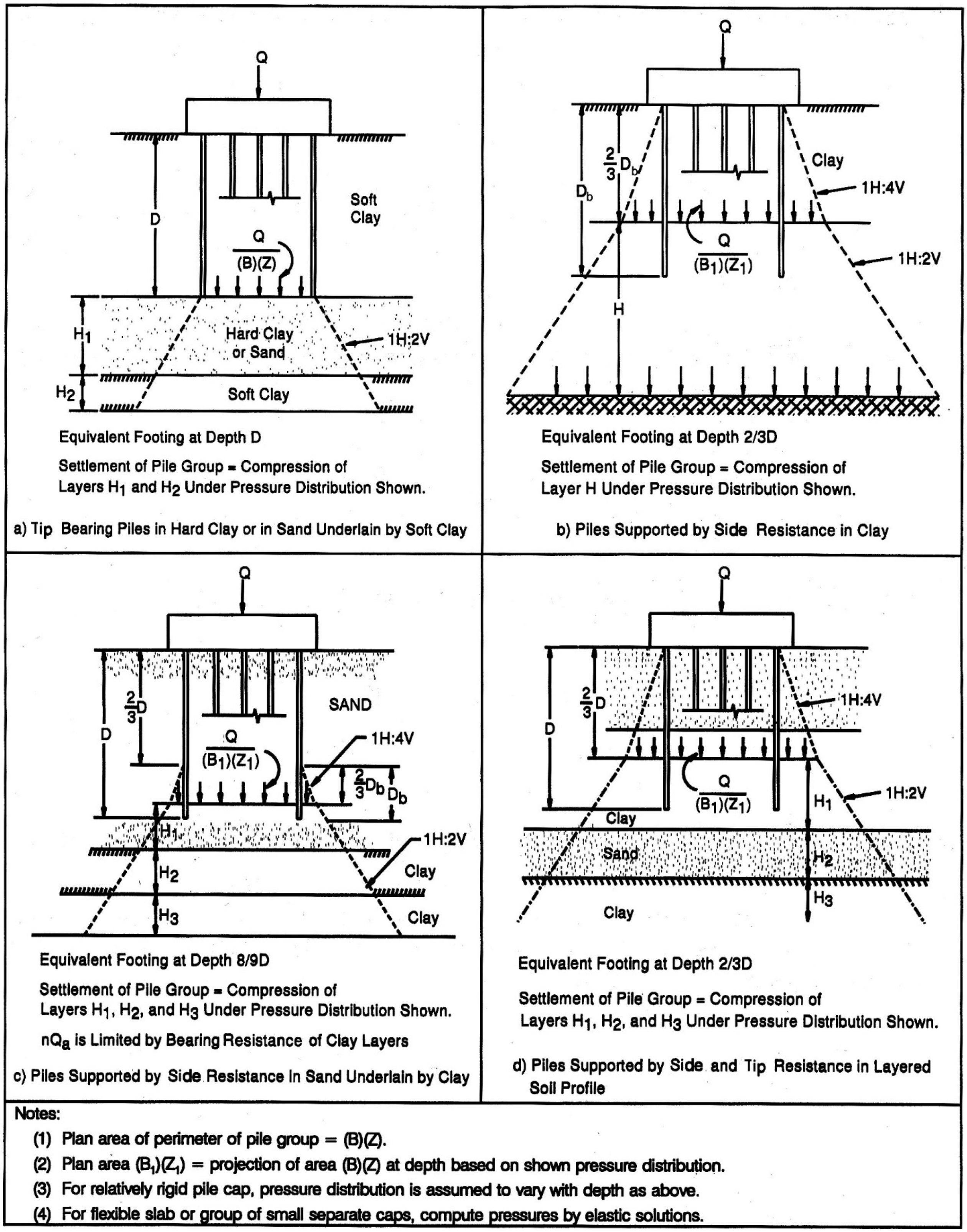

Figure 10.7.2.3.1-1—Stress Distribution below Equivalent Footing for Pile Group after Hannigan et al. (2006)

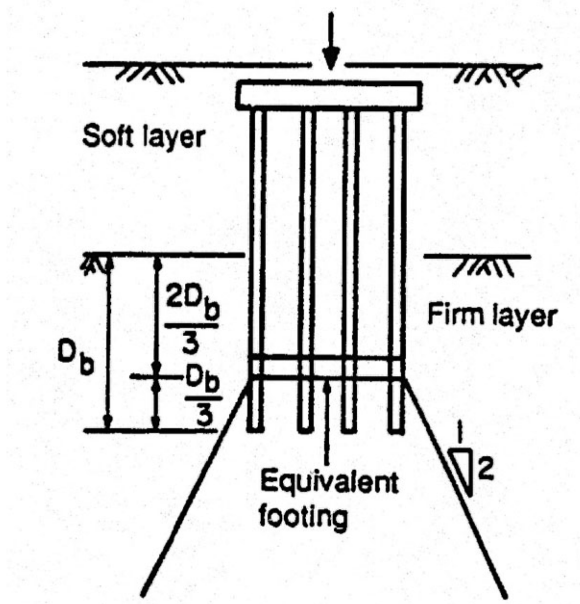

Figure 10.7.2.3.1-2—Location of Equivalent Footing (after Duncan and Buchignani, 1976)

10.7.2.3.2—Pile Groups in Cohesive Soil

Shallow foundation settlement estimation procedures shall be used to estimate the settlement of a pile group, using the equivalent footing location specified in Figure 10.7.2.3.1-1 or Figure 10.7.2.3.1-2.

The settlement of pile groups in cohesionless soils may be taken as:

Using *SPT*:
$$\rho = \frac{qI\sqrt{B}}{N1_{60}} \tag{10.7.2.3.2-1}$$

Using *CPT*:
$$\rho = \frac{qBI}{2q_c} \tag{10.7.2.3.2-2}$$

in which:

$$I = 1 - 0.125\frac{D'}{B} \geq 0.5 \tag{10.7.2.3.2-3}$$

where:

ρ = settlement of pile group (in.)

q = net foundation pressure applied at $2D_b/3$, as shown in Figure 10.7.2.3.1-1; this pressure is equal to the applied load at the top of the group divided by the area of the equivalent footing and does not include the weight of the piles or the soil between the piles (ksf)

B = width or smallest dimension of pile group (ft)

C10.7.2.3.2

The provisions are based upon the use of empirical correlations proposed by Meyerhof (1976). These are empirical correlations and the units of measure must match those specified for correct computations. This method may tend to over-predict settlements.

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I = influence factor of the effective group embedment (dim)
D' = effective depth taken as $2D_b/3$ (ft)
D_b = depth of penetration into bearing strata (ft)
$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/ft) as specified in Article 10.4.6.2.4.
q_c = static cone tip resistance (ksf)

Alternatively, other methods for computing settlement in cohesionless soil, such as the Hough method as specified in Article 10.6.2.4.2 may also be used in connection with the equivalent footing approach.

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width B below the equivalent footing. The *SPT* and *CPT* methods (Eqs. 10.7.2.3.2-1 and 10.7.2.3.2-2) shall only be considered applicable to the distributions shown in Figure 10.7.2.3.1-1b and Figure 10.7.2.3.1-2.

10.7.2.4—Horizontal Pile Foundation Movement

Horizontal movement induced by lateral loads shall be evaluated. The provisions of Article 10.5.2.1 shall apply regarding horizontal movement criteria.

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction. Tolerable horizontal movements of piles shall be established on the basis of confirming compatible movements of structural components, e.g., pile to column connections, for the loading condition under consideration.

The effects of the lateral resistance provided by an embedded cap may be considered in the evaluation of horizontal movement.

The orientation of nonsymmetrical pile cross-sections shall be considered when computing the pile lateral stiffness.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D3966.

The effects of group interaction shall be taken into account when evaluating pile group horizontal movement. When the *P-y* method of analysis is used, the values of P shall be multiplied by P-multiplier values, P_m, to account for group effects. The values of P_m provided in Table 10.7.2.4-1 should be used.

C10.7.2.4

Pile foundations are subjected to lateral loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles, and vertical piles are more effective against dynamic loads.

Methods of analysis that use manual computation were developed by Broms (1964a and 1964b). They are discussed in detail by Hannigan et al. (2006). Reese developed analysis methods that model the horizontal soil resistance using *P-y* curves. This analysis has been well developed and software is available for analyzing single piles and pile groups (Reese, 1986; Williams et al., 2003; and Hannigan et al., 2006).

Deep foundation horizontal movement at the foundation design stage may be analyzed using computer applications that consider soil-structure interaction. Application formulations are available that consider the total structure including pile cap, pier and superstructure (Williams et al., 2003).

If a lateral static load test is used to assess the site specific lateral resistance of a pile, information on the methods of analysis and interpretation of lateral load tests presented in the *Handbook on Design of Piles and Drilled Shafts Under Lateral Load*, Reese (1984) and *Static Testing of Deep Foundations*, Kyfor et al. (1992) should be used.

Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3B	0.8	0.4	0.3
5B	1.0	0.85	0.7

Loading direction and spacing shall be taken as defined in Figure 10.7.2.4-1. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the Figure), a group reduction factor of less than 1.0 should only be used if the pile spacing is 5*B* or less, i.e., a P_m of 0.8 for a spacing of 3*B*, as shown in Figure 10.7.2.4-1.

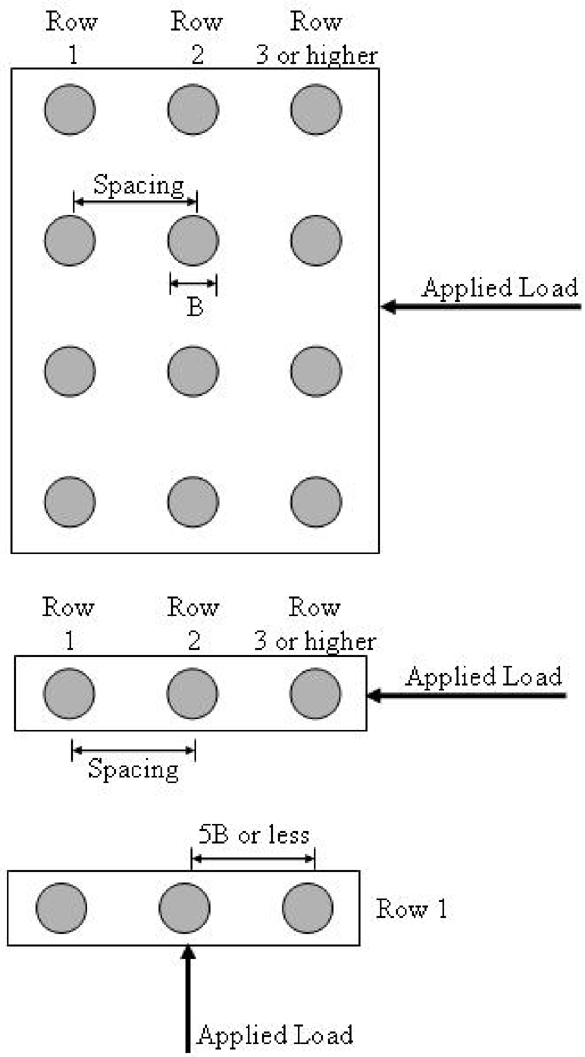

Figure 10.7.2.4-1—Definition of Loading Direction and Spacing for Group Effects

Since many piles are installed in groups, the horizontal resistance of the group has been studied and it has been found that multiple rows of piles will have less resistance than the sum of the single pile resistance. The front piles "shade" rows that are further back.

The P-multipliers, P_m, in Table 10.7.2.4-1 are a function of the center-to-center (*CTC*) spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, *B*. The values of P_m in Table 10.7.2.4-1 were developed for vertical piles only.

Lateral load tests have been performed on pile groups, and multipliers have been determined that can be used in the analysis for the various rows. Those multipliers have been found to depend on the pile spacing and the row number in the direction of loading. To establish values of P_m for other pile spacing values, interpolation between values should be conducted.

The multipliers are a topic of current research and may change in the future. Values from recent research have been tabulated by Hannigan et al. (2006).

Note that these *P-y* methods generally apply to foundation elements that have some ability to bend and deflect. For large diameter, relatively short foundation elements, e.g., drilled shafts or relatively short stiff piles, the foundation element rotates rather than bends, in which case strain wedge theory (Norris, 1986; Ashour et al., 1998) may be more applicable. When strain wedge theory is used to assess the lateral load response of groups of short, large diameter piles or shaft groups, group effects should be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each shaft in the group as lateral deflection increases. Note that P_m in Table 10.7.2.4-1 is not applicable if strain wedge theory is used.

Batter piles provide a much stiffer lateral response than vertical piles when loaded in the direction of the batter.

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10.7.2.5—Settlement Due to Downdrag

The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to the downdrag. In general, the available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the service limit state. In the case where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag, e.g., piles supported by side resistance, to fully resist the downdrag, the structure should be designed to tolerate the full amount of settlement resulting from the downdrag and the other applied loads.

If adequate geotechnical resistance is available to resist the downdrag plus structure loads in the service limit state, the amount of deformation needed to fully mobilize the geotechnical resistance should be estimated, and the structure designed to tolerate the anticipated movement.

C10.7.2.5

The static analysis procedures in Article 10.7.3.8.6 may be used to estimate the available pile nominal resistance to withstand the downdrag plus structure loads.

Nominal resistance may also be estimated using a dynamic method, e.g., dynamic measurements with signal matching analysis, wave equation, pile driving formula, etc., per Article 10.7.3.8, provided the side resistance within the zone contributing to downdrag is subtracted from the nominal bearing resistance determined from the dynamic method during pile installation. The side resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in Article 10.7.3.8.6, from signal matching analysis, or from instrumented pile load test results. Note that the static analysis methods may have bias that, on average, over or under predicts the side resistance. The bias of the method selected to estimate the side resistance within the downdrag zone should be taken into account as described in Article 10.7.3.3.

For the establishment of settlement tolerance limits, see Article 10.5.2.1.

10.7.2.6—Lateral Squeeze

Bridge abutments supported on pile foundations driven through soft soils that are subject to unbalanced embankment fill loading shall be evaluated for lateral squeeze.

C10.7.2.6

Guidance on evaluating the potential for lateral squeeze and potential mitigation methods are included in Hannigan et al., (2006).

10.7.3—Strength Limit State Design

10.7.3.1—General

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal bearing resistance;
- Size and configuration of the pile group to provide adequate foundation support;
- Estimated pile length to be used in the construction contract documents to provide a basis for bidding;
- A minimum pile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) depth needed to meet all of the applicable requirements identified in Article 10.7.6;
- The maximum driving resistance expected in order to reach the minimum pile penetration required, if applicable, including any soil/pile side resistance that will not contribute to the long-term nominal bearing resistance of the pile, e.g., soil contributing to downdrag, or soil that will be removed by scour;

C10.7.3.1

A minimum pile penetration should only be specified if needed to ensure that uplift, lateral stability, depth to resist downdrag, depth to satisfy scour concerns, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements for the service and extreme event limit states. See Article 10.7.6 for additional details. Assuming static load tests, dynamic methods, e.g., dynamic test with signal matching, wave equation, pile formulae, etc., are used during pile installation to establish when the nominal bearing resistance has been met, a minimum pile penetration should not be used to ensure that the required nominal pile bearing, i.e., compression, resistance is obtained.

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- The drivability of the selected pile to achieve the required nominal axial resistance or minimum penetration with acceptable driving stresses at a satisfactory blow count per unit length of penetration; and
- The nominal structural resistance of the pile and/or pile group.

Overall stability of a pile supported foundation shall be evaluated where:

- the foundation is placed through an embankment,
- the pile foundation is located on, near or within a slope,
- the possibility of loss of foundation support through erosion or scour exists, or
- bearing strata are significantly inclined.

Unbalanced lateral forces caused by lack of overall stability or lateral squeeze should be mitigated through stabilization measures to prevent failure of the foundation, structure, and what the foundation supports.

A nominal resistance measured during driving exceeding the compressive nominal resistance required by the contract may be needed in order to reach a minimum pile penetration specified in the contract.

The drivability analysis is performed to establish whether a hammer and driving system will likely install the pile in a satisfactory manner.

10.7.3.2—Point Bearing Piles on Rock

10.7.3.2.1—General

As applied to pile compressive resistance, this Article shall be considered applicable to soft rock, hard rock, and very strong soils such as very dense glacial tills that will provide high nominal bearing resistance in compression with little penetration.

C10.7.3.2.1

If pile penetration into rock is expected to be minimal, the prediction of the required pile length will usually be based on the depth to rock.

A definition of hard rock that relates to measurable rock characteristics has not been widely accepted. Local or regional experience with driving piles to rock provides the most reliable definition.

In general, it is not practical to drive piles into rock to obtain significant uplift or lateral resistance. The ability to obtain sufficient uplift resistance will depend on the softness of the rock formation. Local experience should also be considered. If significant lateral or uplift foundation resistance is required, drilled shaft foundations should be considered. If it is still desired to use piles, a pile drivability study should be performed to verify the feasibility of obtaining the desired penetration into rock.

10.7.3.2.2—Piles Driven to Soft Rock

Soft rock that can be penetrated by pile driving shall be treated in the same manner as soil for the purpose of design for bearing resistance, in accordance with Article 10.7.3.8.

C10.7.3.2.2

Steel piles driven into soft rock may not require tip protection.

10.7.3.2.3—Piles Driven to Hard Rock

The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the

C10.7.3.2.3

Care should be exercised in driving piles to hard rock to avoid tip damage. The tips of steel piles driven to hard rock should be protected by high strength, cast steel tip protection.

values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions. A pile-driving acceptance criteria shall be developed that will prevent pile damage. Dynamic pile measurements should be used to monitor for pile damage.

If the rock surface is reasonably flat, installation with pile tip protection should be considered. In the case of sloping rock, or when battered piles are driven to rock, greater difficulty can arise and the use of tip protection with teeth should be considered. The designer should perform a wave equation analysis to check anticipated stresses, and also consider the following to minimize the risk of pile damage during installation:

- Use a relatively small hammer. If a hammer with adjustable stroke or energy setting is used, it should be operated with a small stroke to seat the pile. The nominal axial resistance can then be proven with a few larger hammer blows.
- A large hammer should not be used if it cannot be adjusted to a low stroke. It may be impossible to detect possible toe damage if a large hammer with large stroke is used.
- For any hammer size, specify a limited number of hammer blows after the pile tip reaches the rock, and stop immediately. An example of a limiting criteria is five blows per one half inch.
- Extensive dynamic testing can be used to verify bearing resistance on a large percentage of the piles. This approach could be used to justify larger design nominal resistances.

If such measures are taken, and successful local experience is available, it may be acceptable to not conduct the dynamic pile measurements.

10.7.3.3—Pile Length Estimates for Contract Documents

Subsurface geotechnical information combined with static analysis methods (see Article 10.7.3.8.6), preconstruction probe pile programs (see Article 10.7.9), and/or pile load tests (see Article 10.7.3.8.2) shall be used to estimate the depth of penetration required to achieve the desired nominal bearing resistance to establish contract pile quantities. If static analysis methods are used, potential bias in the method selected should be considered when estimating the penetration depth required to achieve the desired nominal bearing resistance. Local pile driving experience shall also be considered when making pile quantity estimates. If the depth of penetration required to obtain the desired nominal bearing, i.e., compressive, resistance is less than the depth required to meet the provisions of Article 10.7.6, the minimum penetration required per Article 10.7.6 should be used as the basis for estimating contract pile quantities.

C10.7.3.3

The estimated pile length necessary to provide the required nominal resistance is determined using a static analysis, local pile driving experience, knowledge of the site subsurface conditions, and/or results from a static pile load test program. The required pile length is often defined by the presence of an obvious bearing layer. Local pile driving experience with such a bearing layer should be strongly considered when developing pile quantity estimates.

In variable soils, a program of probe piles across the site is often used to determine variable pile order lengths. Probe piles are particularly useful when driving concrete piles. The pile penetration depth (i.e., length) used to estimate quantities for the contract should also consider requirements to satisfy other design considerations, including service and extreme event limit states, as well as minimum pile penetration requirements for lateral stability, uplift, downdrag, scour, group settlement, etc.

One solution to the problem of predicting pile length is the use of a preliminary test program at the site. Such a program can range from a very simple operation of driving a few piles to evaluate drivability, to an extensive program where different pile types are driven and static load and dynamic testing is performed. For large projects, such test programs may be very cost effective.

In lieu of local pile driving experience, if a static analysis method is used to estimate the pile length required to achieve the desired nominal resistance for establishment of contract pile quantities, to theoretically account for method bias, the factored resistance used to determine the number of piles required in the pile group may be conservativley equated to the factored resistance estimated using the static analysis method as follows:

$$\varphi_{dyn} \times R_n = \varphi_{stat} \text{ x } R_{nstat} \quad \text{(C10.7.3.3-1)}$$

where:

φ_{dyn} = the resistance factor for the dynamic method used to verify pile bearing resistance during driving specified in Table 10.5.5.2.3-1
R_n = the nominal pile bearing resistance (kips)
φ_{stat} = the resistance factor for the static analysis method used to estimate the pile penetration depth required to achieve the desired bearing resistance specified in Table 10.5.5.2.3-1
R_{nstat} = the predicted nominal resistance from the static analysis method used to estimate the penetration depth required (kips)

Using Eq. C10.7.3.3-1 and solving for R_{nstat}, use the static analysis method to determine the penetration depth required to obtain R_{nstat}.

The resistance factor for the static analysis method inherently accounts for the bias and uncertainty in the static analysis method. However, local experience may dictate that the penetration depth estimated using this approach be adjusted to reflect that experience. Where piles are driven to a well defined firm bearing stratum, the location of the top of bearing stratum will dictate the pile length needed, and Eq. C10.7.3.3-1 is likely not applicable.

Note that R_n is considered to be nominal bearing resistance of the pile needed to resist the applied loads, and is used as the basis for determining the resistance to be achieved during pile driving, R_{ndr} (see Articles 10.7.6 and 10.7.7). R_{nstat} is only used in the static analysis method to estimate the pile penetration depth required.

Note that while there is a theoretical basis to this suggested approach, it can produce apparently erroneous results if attempting to use extremes in static analysis and dynamic methods, e.g., using static load test results and then using the Engineering News formula to control pile driving, or using a very inaccurate static analysis method in combination with dynamic testing and signal matching. Part of the problem is that the available resistance factors have been established in consideration of the risk and consequences of pile foundation failure rather than the risk and consequences of underrunning or overrunning pile quantities. Therefore, the approach provided in Eq. C10.7.3.3-1 should be used cautiously, especially when the difference between the resistance factors for method used to estimate pile penetration depth versus the one

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used for obtaining the required nominal axial resistance is large.

10.7.3.4—Nominal Axial Resistance Change after Pile Driving

10.7.3.4.1—General

Consideration should be given to the potential for change in the nominal axial pile resistance after the end of pile driving. The effect of soil relaxation or setup should be considered in the determination of nominal axial pile resistance for soils that are likely to be subject to these phenomena.

C10.7.3.4.1

Soil relaxation is not a common phenomenon but more serious than setup since it represents a reduction in the reliability of the foundation.

Soil setup is a common phenomenon that can provide the opportunity for using larger nominal resistances at no increase in cost. However, it is necessary that the resistance gain be adequately proven. This is usually accomplished by restrike testing with dynamic measurements (Komurka et. al, 2003).

10.7.3.4.2—Relaxation

If relaxation is possible in the soils at the site the pile shall be tested in restrike after a sufficient time has elapsed for relaxation to develop.

C10.7.3.4.2

Relaxation is a reduction in axial pile resistance. While relaxation typically occurs at the pile tip, it can also occur along the sides of the pile (Morgano and White, 2004). It can occur in dense sands or sandy silts and in some shales. Relaxation in the sands and silts will usually develop fairly quickly after the end of driving (perhaps in only a few minutes or hours) as a result of the return of the reduced pore pressure induced by dilation of the dense sands during driving. In some shales, relaxation occurs during the driving of adjacent piles and that will be immediate. There are other shales where the pile penetrates the shale and relaxation requires perhaps as much as two weeks to develop. In some cases, the amount of relaxation can be large.

10.7.3.4.3—Setup

Setup in the nominal axial resistance may be used to support the applied load. Where increase in resistance due to setup is utilized, the existence of setup shall be verified after a specified length of time by re-striking the pile.

C10.7.3.4.3

Setup is an increase in the nominal axial resistance that develops over time predominantly along the pile shaft. Pore pressures increase during pile driving due to a reduction of the soil volume, reducing the effective stress and the shear strength. Setup may occur rapidly in cohesionless soils and more slowly in finer grained soils as excess pore water pressures dissipate. In some clays, setup may continue to develop over a period of weeks and even months, and in large pile groups it can develop even more slowly.

Setup, sometimes called "pile freeze," can be used to carry applied load, providing the opportunity for using larger pile nominal axial resistances, if it can be proven. Signal matching analysis of dynamic pile measurements made at the end of driving and later in restrike can be an effective tool in evaluating and quantifying setup. (Komurka et al., 2003) (Bogard and Matlock, 1990).

If a wave equation or dynamic formula is used to determine the nominal pile bearing resistance on restrike, care should be used as these approaches require accurate blow count measurement which is inherently difficult at the beginning of redrive (BOR). Furthermore, the resistance factors provided in Table 10.5.5.2.3-1 for

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driving formulas were developed for end of driving conditions and empirically have been developed based on the assumption that soil setup will occur. See Article C10.5.5.2.3 for additional discussion on this issue.

Higher degrees of confidence for the assessment of setup effects are provided by dynamic measurements of pile driving with signal matching analyses or static load tests after a sufficient wait time following pile installation.

The restrike time and frequency should be based on the time dependent strength change characteristics of the soil. The following restrike durations are recommended:

Soil Type	Time Delay until Restrike
Clean Sands	1 day
Silty Sands	2 days
Sandy Silts	3–5 days
Silts and Clays	7–14 days*
Shales	7 days

* Longer times are sometimes required.

Specifying a restrike time for friction piles in fine grained soils which is too short may result in pile length overruns.

10.7.3.5—Groundwater Effects and Buoyancy

Nominal axial resistance shall be determined using the groundwater level consistent with that used to calculate the effective stress along the pile sides and tip. The effect of hydrostatic pressure shall be considered in the design.

C10.7.3.5

Unless the pile is bearing on rock, the bearing resistance is primarily dependent on the effective surcharge that is directly influenced by the groundwater level. For drained loading conditions, the vertical effective stress is related to the groundwater level and thus it affects pile axial resistance. Lateral resistance may also be affected.

Buoyant forces may also act on a hollow pile or unfilled casing if it is sealed so that water does not enter the pile. During pile installation, this may affect the driving resistance (blow count) observed, especially in very soft soils.

For design purposes, anticipated changes in the groundwater level during construction and over the life of the structure should be considered with regard to its effect on pile resistance and constructability.

10.7.3.6—Scour

The effect of scour shall be considered in determining the minimum pile embedment and the required nominal driving resistance, R_{ndr}. The pile foundation shall be designed so that the pile penetration after the design scour event satisfies the required nominal axial and lateral resistance.

The resistance factors shall be those used in the design without scour. The side resistance of the material lost due to scour should be determined using a static analysis and it should not be factored, but consideration should be given to the bias of the static analysis method used to predict resistance. Method bias is discussed in Article 10.7.3.3.

C10.7.3.6

The piles will need to be driven to the required nominal bearing resistance plus the side resistance that will be lost due to scour. The nominal resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal bearing resistance plus the magnitude of the side resistance lost as a result of scour, considering the prediction method bias.

Another approach that may be used takes advantage of dynamic measurements. In this case, the static analysis method is used to determine an estimated length. During the driving of test piles, the side resistance component of the bearing resistance of pile in the scourable material may be determined by a signal matching analysis of the restrike dynamic measurements obtained when the pile tip is below

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the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

The pile foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

In some cases, the flooding stream will carry debris that will induce horizontal loads on the piles.

Additional information regarding pile design for scour is provided in Hannigan et al. (2006).

10.7.3.7—Downdrag

The foundation should be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including the downdrag, at the strength limit state. The nominal pile resistance available to support structure loads plus downdrag shall be estimated by considering only the positive side and tip resistance below the lowest layer contributing to the downdrag. The pile foundation shall be designed to structurally resist the downdrag plus structure loads.

In the instance where it is not possible to obtain adequate geotechnical resistance below the lowest layer contributing to downdrag, e.g., piles supported by side resistance, to fully resist the downdrag, or if it is anticipated that significant deformation will be required to mobilize the geotechnical resistance needed to resist the factored loads including the downdrag load, the structure should be designed to tolerate the settlement resulting from the downdrag and the other applied loads as specified in Article 10.7.2.5.

C10.7.3.7

The static analysis procedures in Article 10.7.3.8.6 may be used to estimate the available pile nominal resistance to withstand the downdrag plus structure loads.

Nominal resistance may also be estimated using an instrumented static load test or dynamic testing during restrike with signal matching, provided the side resistance within the zone contributing to downdrag is subtracted from the resistance determined from the static load or dynamic test. The side resistance within the zone contributing to downdrag may be estimated using the static analysis methods specified in Article 10.7.3.8.6, from restrike signal matching analysis, or from instrumented static pile load test results. Note that the static analysis method may have a bias, on average over or under predicting the side resistance. The bias of the method selected to estimate the skin friction should be taken into account as described in Article C10.7.3.3.

Pile design for downdrag is illustrated in Figure C10.7.3.7-1.

where:

R_{Sdd} = side resistance which must be overcome during driving through downdrag zone (kips)
$Q_p = \Sigma\gamma_i Q_i$ = factored load per pile, excluding downdrag load (kips)
DD = downdrag load per pile (kips)
$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile (ft)
φ_{dyn} = resistance factor, assuming that a dynamic method is used to estimate nominal pile resistance during installation of the pile (if a static analysis method is used instead, use φ_{stat})
γ_p = load factor for downdrag

The summation of the factored loads ($\Sigma\gamma_i Q_i$) should be less than or equal to the factored resistance ($\varphi_{dyn} R_n$). Therefore, the nominal resistance R_n should be greater than or equal to the sum of the factored loads divided by the resistance factor φ_{dyn}. The nominal bearing resistance (kips) of the pile needed to resist the factored loads, including downdrag, is therefore taken as:

$$R_n = \frac{(\Sigma\gamma_i Q_i)}{\varphi_{dyn}} + \frac{\gamma_p DD}{\varphi_{dyn}} \qquad \text{(C10.7.3.7-1)}$$

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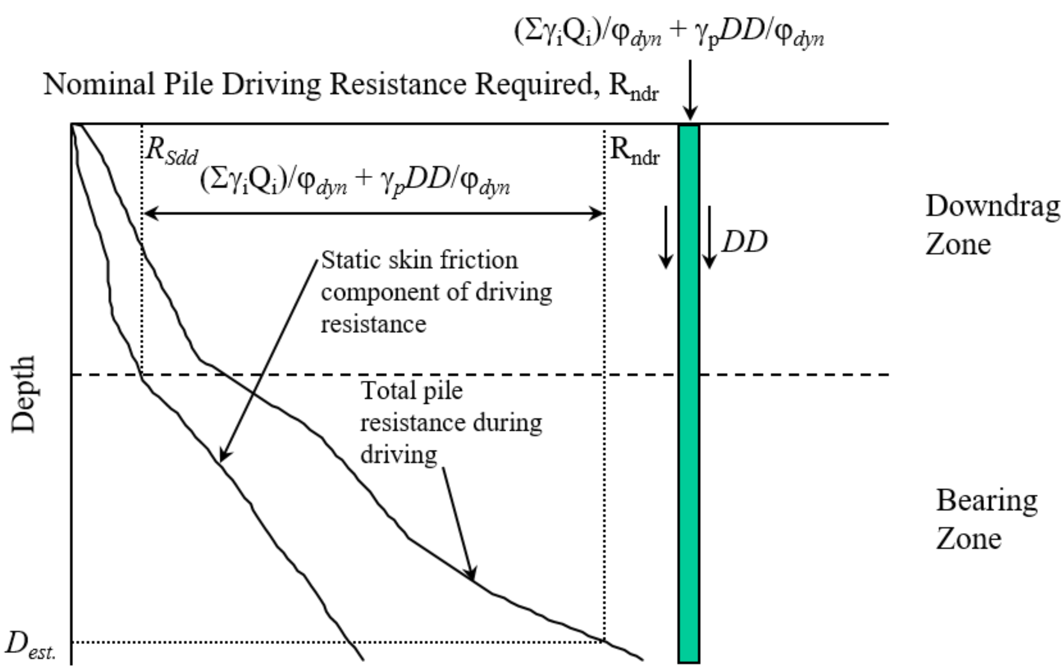
The total nominal driving resistance, R_{ndr} (kips), needed to obtain R_n, accounting for the side resistance that must be overcome during pile driving that does not contribute to the nominal resistance of the pile, is taken as:

$$R_{ndr} = R_{Sdd} + R_n \qquad \text{(C10.7.3.7-2)}$$

where:

R_{ndr} = nominal pile driving resistance required (kips)

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.



Q_{Sdd} = skin friction which must be overcome during driving through downdrag zone

(SgiQi)/φ_{dyn} + $\gamma_p DD/\varphi_{dyn}$ = nominal pile resistance needed to resist all applied loads per pile, including downdrag

Q_p = (SgiQi) = factored load per pile, excluding downdrag load

Q_{DD} = downdrag load per pile

$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile

φ_{dyn} = resistance factor, assuming that a dynamic method is used to estimate pile resistance

γ_p = load factor for downdrag

Figure C10.7.3.7-1—Design of Pile Foundations for Downdrag

10.7.3.8—Determination of Nominal Bearing Resistance for Piles

10.7.3.8.1—General

Nominal pile bearing resistance should be field verified during pile installation using static load tests, dynamic tests, wave equation analysis, or dynamic formula. The resistance factor selected for design shall be based on the method used to verify pile bearing resistance as specified in Article 10.5.5.2.3. The production piles

C10.7.3.8.1

This Article addresses the determination of the nominal bearing (compression) resistance needed to meet strength limit state requirements, using factored loads and factored resistance values. From this design step, the number of piles and pile nominal resistance needed to resist the factored loads applied to the foundation are shall be driven to the minimum blow count determined from the static load test, dynamic test, wave equation, or dynamic formula and, if required, to a minimum penetration needed for uplift, scour, lateral resistance, or other requirements as specified in Article 10.7.6. If it is determined that static load testing is not feasible and dynamic methods are unsuitable for field verification of nominal bearing resistance, the piles shall be driven to the tip elevation determined from the static analysis, and to meet other limit states as required in Article 10.7.6.

determined. Both the loads and resistance values are factored as specified in Articles 3.4.1 and 10.5.5.2.3, respectively, for this determination.

In most cases, the nominal resistance of production piles should be controlled by driving to a required blow count. In a few cases, usually piles driven into cohesive soils with little or no toe resistance and very long wait times to achieve the full pile resistance increase due to soil setup, piles may be driven to depth. However, even in those cases, a pile may be selected for testing after a sufficient waiting period, using either a static load test or a dynamic test.

In cases where the project is small and the time to achieve soil setup is large compared with the production time to install all of the piles, no field testing for the verification of nominal resistance may be acceptable.

10.7.3.8.2—Static Load Test

If a static pile load test is used to determine the pile nominal axial resistance, the test shall not be performed less than five days after the test pile was driven unless approved by the Engineer. The load test shall follow the procedures specified in ASTM D1143, and the loading procedure should follow the Quick Load Test Procedure.

Unless specified otherwise by the Engineer, the nominal bearing resistance shall be determined from the test data as follows:

- for piles 24.0 in. or less in diameter (length of side for square piles), the davisson method;
- for piles larger than 36.0 in. in diameter (length of side for square piles), at a pile top movement, s_f(in.), as determined from Eq. 10.7.3.8.2-1; and
- for piles greater than 24.0 in. but less than 36.0 in. in diameter, criteria to determine the nominal bearing resistance that is linearly interpolated between the criteria determined at diameters of 24.0 and 36.0 in.

$$s_f = \frac{QL}{12\,AE} + \frac{B}{2.5} \qquad (10.7.3.8.2\text{-}1)$$

where:

Q = test load (kips)
L = pile length (ft)
A = pile cross-sectional area (ft^2)
E = pile modulus (ksi)
B = pile diameter (length of side for square piles) (ft)

C10.7.3.8.2

The Quick Load Test Procedure is preferred because it avoids problems that frequently arise when performing a static load test that cannot be completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Load Test has proven to be easily performed in the field and the results usually are satisfactory. Static load tests should be conducted to failure whenever possible and practical to extract the maximum information, particularly when correlating with dynamic tests or static analysis methods. However, if the formation in which the pile is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143 should be considered.

The Davisson Method to determine nominal bearing resistance evaluation is performed by constructing a line on the static load test curve that is parallel to the elastic compression line of the pile. The elastic compression line is calculated by assuming equal compressive forces are applied to the pile ends. The elastic compression line is offset by a specified amount of displacement. The Davisson Method is illustrated in Figure C10.7.3.8.2-1 and described in more detail in Hannigan et al. (2006).

Driving criteria should be established in consideration of the static load test results.

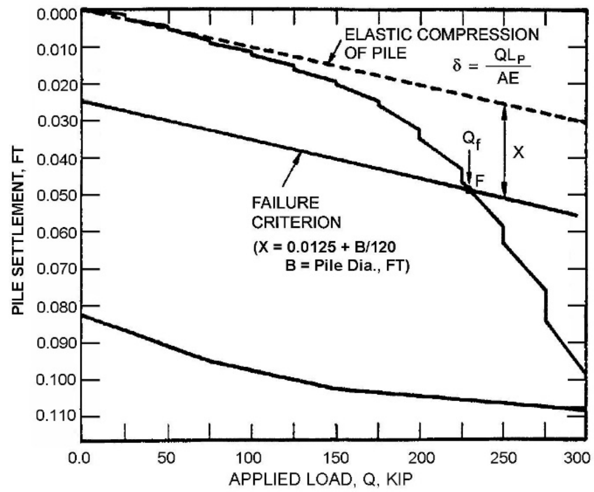


Figure C10.7.3.8.2-1—Alternate Method Load Test Interpretation (Cheney and Chassie, 2000, modified after Davisson, 1972) Note: The failure criterion, *X*, is in feet in this figure.

For piles with large cross-sections, i.e., diameters greater than 24.0 in., the Davisson Method will under predict the nominal pile bearing resistance.

Development of driving criteria in consideration of static load test results is described in Hannigan et al. (2006).

10.7.3.8.3—Dynamic Testing

Dynamic testing shall be performed according to the procedures given in ASTM D4945. If possible, the dynamic test should be performed as a restrike test if the Engineer anticipates significant time dependent strength change. The nominal pile bearing resistance shall be determined by a signal matching analysis of the dynamic pile test data if the dynamic test is used to establish the driving criteria.

C10.7.3.8.3

The dynamic test may be used to establish the driving criteria at the beginning of production driving. A signal matching analysis (Rausche et al., 1972) of the dynamic test data should always be used to determine bearing resistance if a static load test is not performed. See Hannigan et al. (2006) for a description of and procedures to conduct a signal matching analysis. Restrike testing should be performed if setup or relaxation is anticipated.

For example, note that it may not be possible to adjust the dynamic measurements with signal matching analysis to match the static load test results if the driving resistance at the time the dynamic measurement is taken is too large, i.e., the pile set per hammer blow is too small. In this case, adequate hammer energy is not reaching the pile tip to assess end bearing and produce an accurate match, though in such cases, the prediction will usually be very conservative. In general, a tip movement (pile set) of 0.10 to 0.15 in. is needed to provide an accurate signal matching analysis. See Hannigan et al. (2006) for additional guidance on this issue.

In cases where a significant amount of soil setup occurs and the set at the beginning of redrive (BOR) is less than 0.10 inch per blow, a more accurate nominal resistance may be obtained by combining the end bearing determined using the signal matching analysis obtained for the end of driving (EOD) with the signal matching analysis for the shaft resistance at the beginning of redrive.

Dynamic testing and interpretation of the test data should only be performed by certified, experienced testers.

10.7.3.8.4—Wave Equation Analysis

If a wave equation analysis is to be used to establish the driving criteria, it shall be performed based on the hammer and pile driving system to be used for pile installation.

If a wave equation analysis is used for the determination of the nominal bearing resistance, then the driving criterion (blow count) may be the value taken either at the end of driving (EOD) or at the beginning of redrive (BOR). The latter should be used where the soils exhibit significant strength changes (setup or relaxation) with time. When restrike (i.e., BOR) blow counts are taken, the hammer shall be warmed up prior to restrike testing and the blow count shall be taken as accurately as possible for the first inch of restrike.

If the wave equation is used to assess the potential for pile damage, driving stresses shall not exceed the values obtained in Article 10.7.8, using the resistance factors specified or referred to in Table 10.5.5.2.3-1. Furthermore, the blow count needed to obtain the maximum driving resistance anticipated shall be less than the maximum value established based on the provisions in Article 10.7.8.

C10.7.3.8.4

Note that without dynamic test results with signal matching analysis and/or pile load test data (see Articles 10.7.3.8.2 and 10.7.3.8.3), some judgment is required to use the wave equation to predict the pile bearing resistance. Unless experience in similar soils exists, the recommendations of the software provider should be used for dynamic resistance input. Key soil input values that affect the predicted nominal resistance include the soil damping and quake values, the skin friction distribution, e.g., such as could be obtained from a static pile bearing analysis, and the anticipated amount of soil setup or relaxation. The actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though field observations such as hammer stroke or measured ram velocity can and should be used to improve the accuracy of the wave equation prediction.

In general, improved prediction accuracy of nominal bearing resistance is obtained when targeting the driving criteria at BOR conditions, if soil setup or relaxation is anticipated. Using the wave equation to predict nominal bearing resistance from EOD blow counts requires that an accurate estimate of the time-dependent changes in bearing resistance due to soil setup or relaxation be made. This is generally difficult to do unless site-specific, longer-term measurements of bearing resistance from static load tests or dynamic measurements with signal matching are available. Hence, driving criteria based on BOR measurements are recommended when using the wave equation for driving criteria development.

A wave equation analysis should also be used to evaluate pile drivability during design.

10.7.3.8.5—Dynamic Formula

If a dynamic formula is used to establish the driving criterion, the FHWA Gates Formula (Eq. 10.7.3.8.5-1) should be used. The nominal pile resistance as measured during driving using this method shall be taken as:

$$R_{ndr} = 1.75\sqrt{E_d}\,\log_{10}(10N_b) - 100 \quad (10.7.3.8.5\text{-}1)$$

where:

R_{ndr} = nominal pile driving resistance measured during pile driving (kips)

E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the actual stroke (ft-lb)

N_b = Number of hammer blows for 1.0 in. of pile permanent set (blows/in.)

C10.7.3.8.5

It is preferred to use more accurate methods such as wave equation or dynamic testing with signal matching to establish driving criteria (i.e., blow count). However, driving formulas have been in use for many years. Therefore, driving formulas are provided as an option for the development of driving criteria.

Two dynamic formulas are provided here for the Engineer. If a dynamic formula is used for either determination of the nominal resistance or the driving criterion, the FHWA Modified Gates formula is preferred over the Engineering News formula. It is discussed further in the Design and Construction of Driven Pile Foundations (Hannigan et al., 2006). Note that the units in the FHWA Gates formula are not consistent. The specified units in Eq. 10.7.3.8.5-1 must be used.

The Engineering News formula in its traditional form contains a factor of safety of 6.0. For LRFD applications, to produce a nominal resistance, the factor of safety has been removed. As is true of the FHWA Gates formula, The Engineering News formula, modified to predict a nominal bearing resistance, may be used. The nominal pile resistance using this method shall be taken as:

$$R_{ndr} = \frac{12E_d}{(s+0.1)} \quad (10.7.3.8.5\text{-}2)$$

where:

R_{ndr} = nominal pile resistance measured during driving (kips)
E_d = developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the stroke, taken as the ram weight times the stroke (ft-kips)
s = pile permanent set (in.)

the units specified in Eq. 10.7.3.8.5-2 must be used for the Engineering News formula. See Allen (2005; 2007) for additional discussion on the development of the Engineering News formula and its modification to produce a nominal resistance.

Driving formula should only be used to determine end of driving blow count criteria. These driving formula are empirically based on pile load test results, and therefore inherently include some degree of soil setup or relaxation (see Allen, 2007).

If a dynamic formula other than those provided herein is used, it shall be calibrated based on measured load test results to obtain an appropriate resistance factor, consistent with Article C10.5.5.2.

If a drivability analysis is not conducted, for steel piles, design stresses shall be limited as specified in Article 6.15.2.

Dynamic formulas should not be used when the required nominal resistance exceeds 600 kips.

As the required nominal bearing resistance increases, the reliability of dynamic formulas tends to decrease. The FHWA Gates formula tends to underpredict pile nominal resistance at higher resistances. The Engineering News formula tends to become unconservative as the nominal pile resistance increases. If other driving formulas are used, the limitation on the maximum driving resistance to be used should be based upon the limits for which the data is considered reliable, and any tendency of the formula to over or under predict pile nominal resistance.

10.7.3.8.6—Static Analysis

10.7.3.8.6a—General

Where a static analysis prediction method is used to determine pile installation criteria, i.e., for bearing resistance, the nominal pile resistance shall be factored at the strength limit state using the resistance factors in Table 10.5.5.2.3-1 associated with the method used to compute the nominal bearing resistance of the pile. The factored nominal bearing resistance of piles, R_R, may be taken as:

$$R_R = \varphi R_n \quad (10.7.3.8.6a\text{-}1)$$

or:

$$R_R = \varphi R_n = \varphi_{stat} R_p + \varphi_{stat} R_s \quad (10.7.3.8.6a\text{-}2)$$

in which:

C10.7.3.8.6a

While the most common use of static analysis methods is solely for estimating pile quantities, a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal bearing resistance. This is applicable on projects where pile quantities are relatively small, pile loads are relatively low, and/or where the setup time is long so that restrike testing would require an impractical wait-period by the Contractor on the site, e.g., soft silts or clays where a large amount of setup is anticipated.

For use of static analysis methods for contract pile quantity estimation, see Article 10.7.3.3.

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$$R_p = q_p A_p \quad (10.7.3.8.6a\text{-}3)$$

$$R_s = q_s A_s \quad (10.7.3.8.6a\text{-}4)$$

where:

φ_{stat} = resistance factor for the bearing resistance of a single pile specified in Article 10.5.5.2.3
R_p = pile tip resistance (kips)
R_s = pile side resistance (kips)
q_p = unit tip resistance of pile (ksf)
q_s = unit side resistance of pile (ksf)
A_s = surface area of pile side (ft^2)
A_p = area of pile tip (ft^2)

Both total stress and effective stress methods may be used, provided the appropriate soil strength parameters are available. The resistance factors for the side resistance and tip resistance, estimated using these methods, shall be as specified in Table 10.5.5.2.3-1. The limitations of each method as described in Article C10.5.5.2.3 should be applied in the use of these static analysis methods.

10.7.3.8.6b—α-Method

The α-method, based on total stress, may be used to relate the adhesion between the pile and clay to the undrained strength of the clay. For this method, the nominal unit side resistance, in ksf, shall be taken as:

$$q_s = \alpha S_u \quad (10.7.3.8.6b\text{-}1)$$

where:

S_u = undrained shear strength (ksf)
α = adhesion factor applied to S_u (dim)

The adhesion factor for this method, α, shall be assumed to vary with the value of the undrained strength, S_u, as shown in Figure 10.7.3.8.6b-1.

C10.7.3.8.6b

The α-method has been used for many years and gives reasonable results for both displacement and nondisplacement piles in clay.

In general, this method assumes that a mean value of S_u will be used. It may not always be possible to establish a mean value, as in many cases, data are too limited to reliably establish the mean value. The Engineer should apply engineering judgment and local experience as needed to establish an appropriate value for design (see Article C10.4.6).

For H-piles, the perimeter or "box" area should generally be used to compute the surface area of the pile side.

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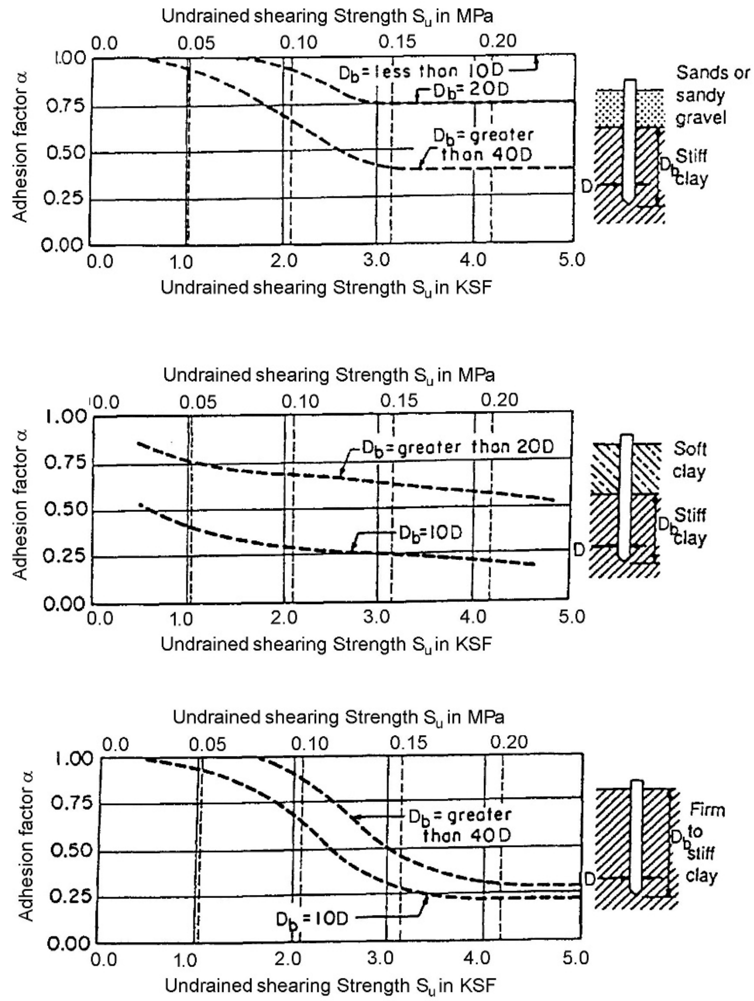

Figure 10.7.3.8.6b-1—Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (1980)

10.7.3.8.6c—β-Method

The β-method, based on effective stress, may be used for predicting side resistance of prismatic piles. The nominal unit skin friction for this method, in ksf, shall be related to the effective stresses in the ground as:

$$q_s = \beta \sigma'_v \tag{10.7.3.8.6c-1}$$

where:

σ'_v = vertical effective stress (ksf)
β = a factor taken from Figure 10.7.3.8.6c-1

C10.7.3.8.6c

The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overestimate side resistance of piles in heavily overconsolidated soils. Esrig and Kirby (1979) suggested that for heavily overconsolidated clays, the value of β should not exceed two.

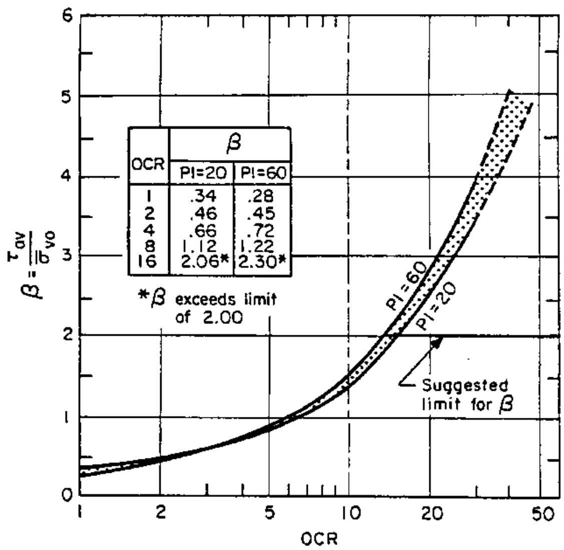

Figure 10.7.3.8.6c-1—β Versus OCR for Displacement Piles after Esrig and Kirby (1979)

10.7.3.8.6d—λ-Method

The λ-method, based on effective stress (though it does contain a total stress parameter), may be used to relate the unit side resistance, in ksf, to passive earth pressure. For this method, the unit skin friction shall be taken as:

$$q_s = \lambda(\sigma'_v + 2S_u) \tag{10.7.3.8.6d-1}$$

where:

$\sigma'_v + 2S_u$ = passive lateral earth pressure (ksf)

σ'_v = the effective vertical stress at midpoint of soil layer under consideration (ksf)

λ = an empirical coefficient taken from Figure 10.7.3.8.6d-1 (dim)

C10.7.3.8.6d

The value of λ decreases with pile length and was found empirically by examining the results of load tests on steel pipe piles.

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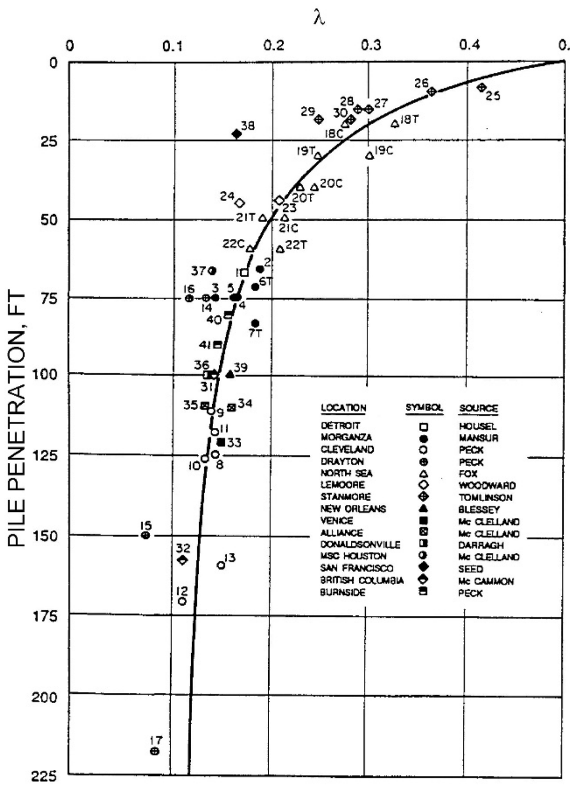

Figure 10.7.3.8.6d-1—λ Coefficient for Driven Pipe Piles after Vijayvergiya and Focht (1972)

10.7.3.8.6e—Tip Resistance in Cohesive Soils

The nominal unit tip resistance of piles in saturated clay, in ksf, shall be taken as:

$$q_p = 9S_u \qquad (10.7.3.8.6e\text{-}1)$$

where:

S_u = undrained shear strength of the clay near the pile tip (ksf)

10.7.3.8.6f—Nordlund/Thurman Method in Cohesionless Soils

This effective stress method should be applied only to sands and nonplastic silts. The nominal unit side resistance, q_s, for this method, in ksf, shall be taken as:

$$q_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos\ \omega} \qquad (10.7.3.8.6f\text{-}1)$$

where:

K_δ = coefficient of lateral earth pressure at mid-point of soil layer under consideration from Figures 10.7.3.8.6f-1 through 10.7.3.8.6f-4 (dim)

C10.7.3.8.6f

Detailed design procedures for the Nordlund/Thurman method are provided in Hannigan et al. (2006). This method was derived based on load test data for piles in sand. In practice, it has been used for gravelly soils as well.

The effective overburden stress is not limited in Eq. 10.7.3.8.6f-1.

For H-piles, the perimeter or “box” area should generally be used to compute the surface area of the pile side.

C_F = correction factor for K_δ when $\delta \neq \phi_f$, from Figure 10.7.3.8.6f-5

σ'_v = effective overburden stress at midpoint of soil layer under consideration (ksf)

δ = friction angle between pile and soil obtained from Figure 10.7.3.8.6f-6 (degrees)

ω = angle of pile taper from vertical (degrees)

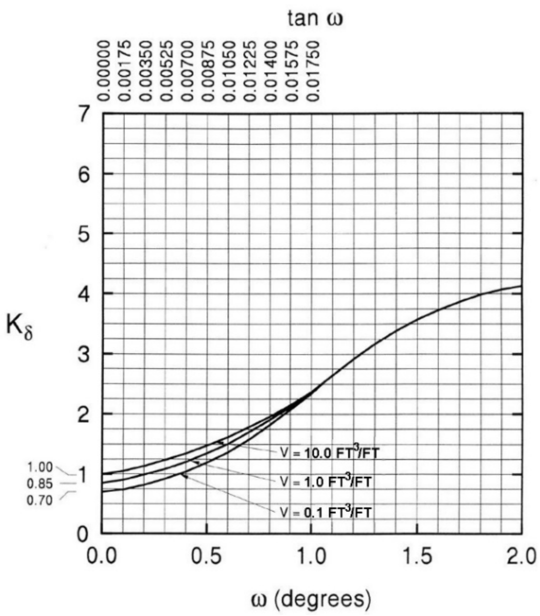

Figure 10.7.3.8.6f-1—Design Curve for Evaluating K_δ for Piles where ϕ_f = 25 degrees (Hannigan et al., 2006 after Nordlund, 1979)

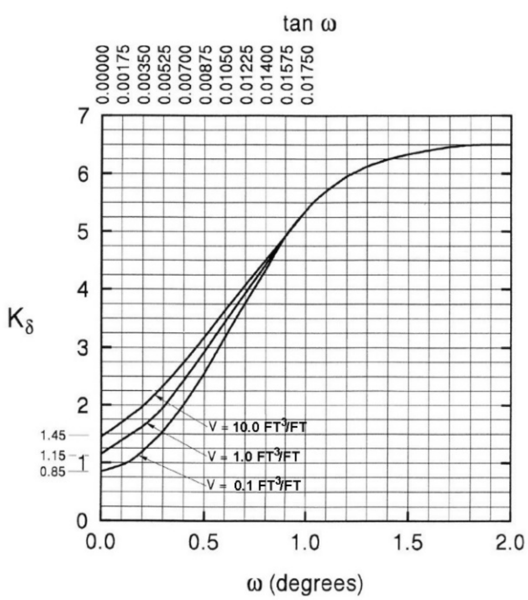

Figure 10.7.3.8.6f-2—Design Curve for Evaluating K_δ for Piles where ϕ_f = 30 degrees (Hannigan et al., 2006 after Nordlund, 1979)

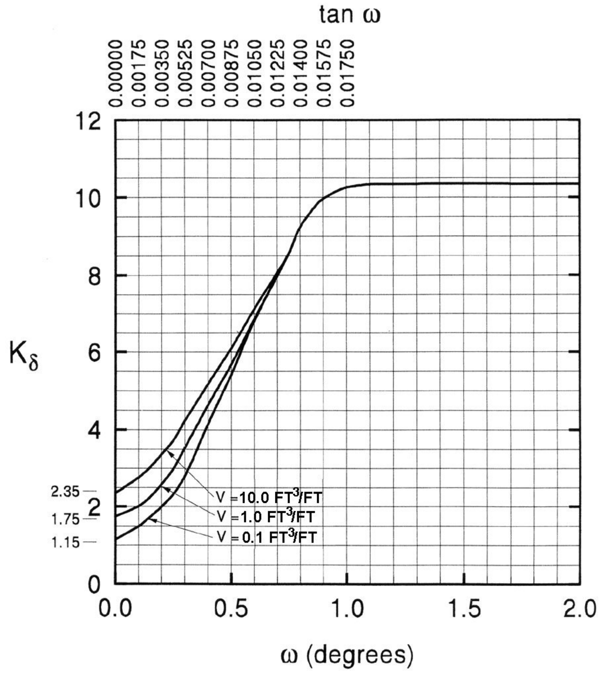

Figure 10.7.3.8.6f-3—Design Curve for Evaluating K_δ for Piles where ϕ_f = 35 degrees (Hannigan et al., 2006 after Nordlund, 1979)

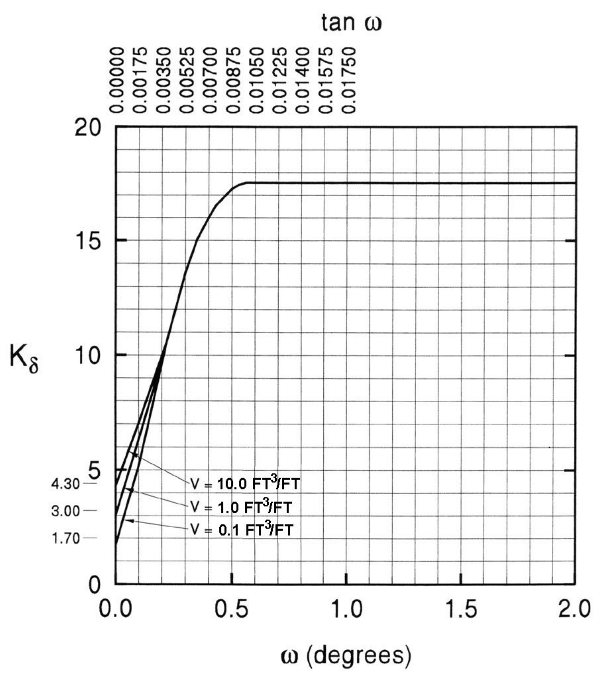

Figure 10.7.3.8.6f-4—Design Curve for Evaluating K_δ for Piles where ϕ_f = 40 degrees (Hannigan et al., 2006 after Nordlund, 1979)

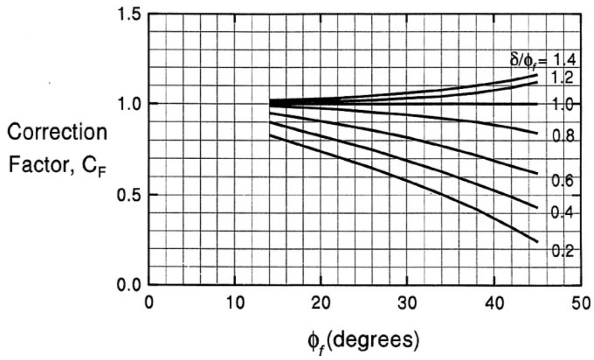

Figure 10.7.3.8.6f-5—Correction Factor for K_δ where $\delta \neq \phi_f$ (Hannigan et al., 2006 after Nordlund, 1979)

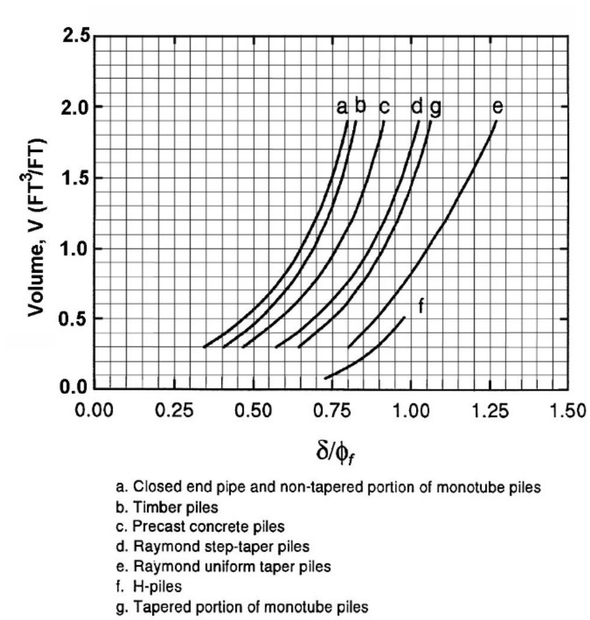

Figure 10.7.3.8.6f-6—Relation of δ/ϕ_f and Pile Displacement, *V*, for Various Types of Piles (Hannigan et al., 2006 after Nordlund, 1979)

The nominal unit tip resistance, q_p, in ksf by the Nordlund/Thurman method shall be taken as:

$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \qquad (10.7.3.8.6f\text{-}2)$$

where:

α_t = coefficient from Figure 10.7.3.8.6f-7 (dim)
N'_q = bearing capacity factor from Figure 10.7.3.8.6f-8
σ'_v = effective overburden stress at pile tip (ksf) ≤3.2 ksf
q_L = limiting unit tip resistance from Figure 10.7.3.8.6f-9

If the friction angle, ϕ_f, is estimated from average, corrected *SPT* blow counts, $N1_{60}$, the $N1_{60}$ values should be averaged over the zone from the pile tip to two diameters below the pile tip.

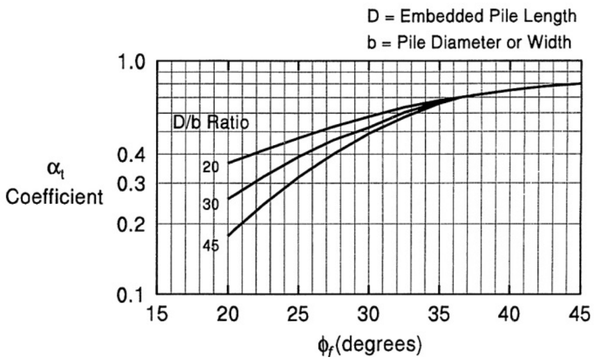

Figure 10.7.3.8.6f-7—α_t Coefficient (Hannigan et al., 2006 modified after Bowles, 1977)

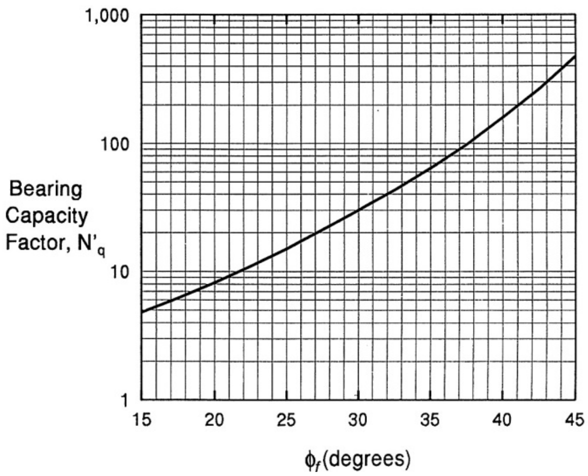

Figure 10.7.3.8.6f-8—Bearing Capacity Factor, N'_q (Hannigan et al., 2006 modified after Bowles, 1977)

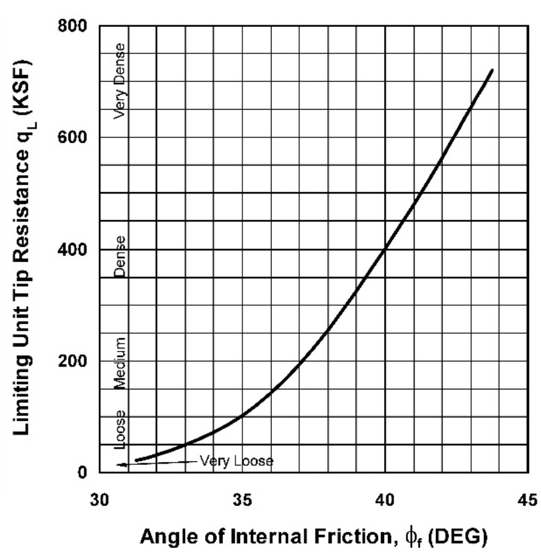

Figure 10.7.3.8.6f-9—Limiting Unit Pile Tip Resistance (Hannigan et al., 2006 after Meyerhof, 1976)

10.7.3.8.6g—Using SPT or CPT in Cohesionless Soils

These methods shall be applied only to sands and nonplastic silts.

The nominal unit tip resistance for the Meyerhof method, in ksf, for piles driven to a depth D_b into a cohesionless soil stratum shall be taken as:

$$q_p = \frac{0.8(N1_{60})D_b}{D} \leq q_\ell \quad (10.7.3.8.6g\text{-}1)$$

where:

$N1_{60}$ = representative *SPT* blow count near the pile tip corrected for overburden pressure as specified in Article 10.4.6.2.4 (blows/ft)
D = pile width or diameter (ft)
D_b = depth of penetration into bearing strata (ft)
q_ℓ = limiting pile tip resistance (ksf)

q_ℓ shall be taken as eight times the value of $N1_{60}$ for sands and six times the value of $N1_{60}$ for nonplastic silt (ksf).

The nominal side resistance of piles in cohesionless soils for the Meyerhof method, in ksf, shall be taken as:

- For driven displacement piles:

$$q_s = \frac{\bar{N}1_{60}}{25} \quad (10.7.3.8.6g\text{-}2)$$

- For nondisplacement piles, e.g., steel H-piles:

$$q_s = \frac{\bar{N}1_{60}}{50} \quad (10.7.3.8.6g\text{-}3)$$

where:

q_s = unit side resistance for driven piles (ksf)
$\bar{N}1_{60}$ = average corrected *SPT*-blow count along the pile side (blows/ft)

Tip resistance, q_p, for the Nottingham and Schmertmann method, in ksf, shall be determined as shown in Figure 10.7.3.8.6g-1.

In which:

$$q_p = \frac{q_{c1} + q_{c2}}{2} \quad (10.7.3.8.6g\text{-}4)$$

where:

q_{c1} = average q_c over a distance of yD below the pile tip (path a-b-c); sum q_c values in both the downward (path a-b) and upward (path b-c)

C10.7.3.8.6g

In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In-situ test parameters may be used to estimate the tip resistance and side resistance of piles.

Two frequently used in-situ test methods for predicting pile axial resistance are the standard penetration test (*SPT*) method (Meyerhof, 1976) and the cone penetration test (*CPT*) method (Nottingham and Schmertmann, 1975).

Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Nondisplacement piles usually have relatively small cross-sectional areas, e.g., steel H-piles and open-ended pipe piles that have not yet plugged. Plugging occurs when the soil between the flanges in a steel H-pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

CPT may be used to determine:

- The cone penetration resistance, q_c, which may be used to determine the tip resistance of piles, and
- Sleeve friction, f_s, which may be used to determine the side resistance.

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directions; use actual q_c values along path a-b and the minimum path rule along path b-c; compute q_{c1} for y-values from 0.7 to 4.0 and use the minimum q_{c1} value obtained (ksf)

q_{c2} = average q_c over a distance of $8D$ above the pile tip (path c-e); use the minimum path rule as for path b-c in the q_{c1}, computations; ignore any minor "x" peak depressions if in sand but include in minimum path if in clay (ksf)

The minimum average cone resistance between 0.7 and four pile diameters below the elevation of the pile tip shall be obtained by a trial and error process, with the use of the minimum-path rule. The minimum-path rule shall also be used to find the value of cone resistance for the soil for a distance of eight pile diameters above the tip. The two results shall be averaged to determine the pile tip resistance.

This process is described in Nottingham and Schmertmann (1975).

The nominal side resistance of piles for this method, in kips, shall be taken as:

$$R_s = K_{s,c}\left[\sum_{i=1}^{N_1}\left(\frac{L_i}{8D_i}\right) f_{si} a_{si} h_i + \sum_{i=1}^{N_2} f_{si} a_{si} h_i\right] \qquad (10.7.3.8.6g\text{-}5)$$

where:

- $K_{s,c}$ = correction factors: K_c for clays and K_s for sands from Figure 10.7.3.8.6g-2 (dim)
- L_i = depth to middle of length interval at the point considered (ft)
- D_i = pile width or diameter at the point considered (ft)
- f_{si} = unit local sleeve friction resistance from *CPT* at the point considered (ksf)
- a_{si} = pile perimeter at the point considered (ft)
- h_i = length interval at the point considered (ft)
- N_1 = number of intervals between the ground surface and a point $8D$ below the ground surface
- N_2 = number of intervals between $8D$ below the ground surface and the tip of the pile

For a pile of constant cross-section (nontapered), Eq. 10.7.3.8.6g-5 can be written as:

$$R_s = K_{s,c}\left[\frac{a_s}{8D}\sum_{i=1}^{N_1} L_i f_{si} h_i + a_s \sum_{i=1}^{N_2} f_{si} h_i\right] \qquad (C10.7.3.8.6g\text{-}1)$$

If, in addition to the pile being prismatic, f_s is approximately constant at depths below $8D$, Eq. C10.7.3.8.6g-1 can be simplified to:

$$R_s = K_{s,c}\left[a_s f_s (Z - 4D)\right] \qquad (C10.7.3.8.6g\text{-}2)$$

where:

- Z = total embedded pile length (ft)

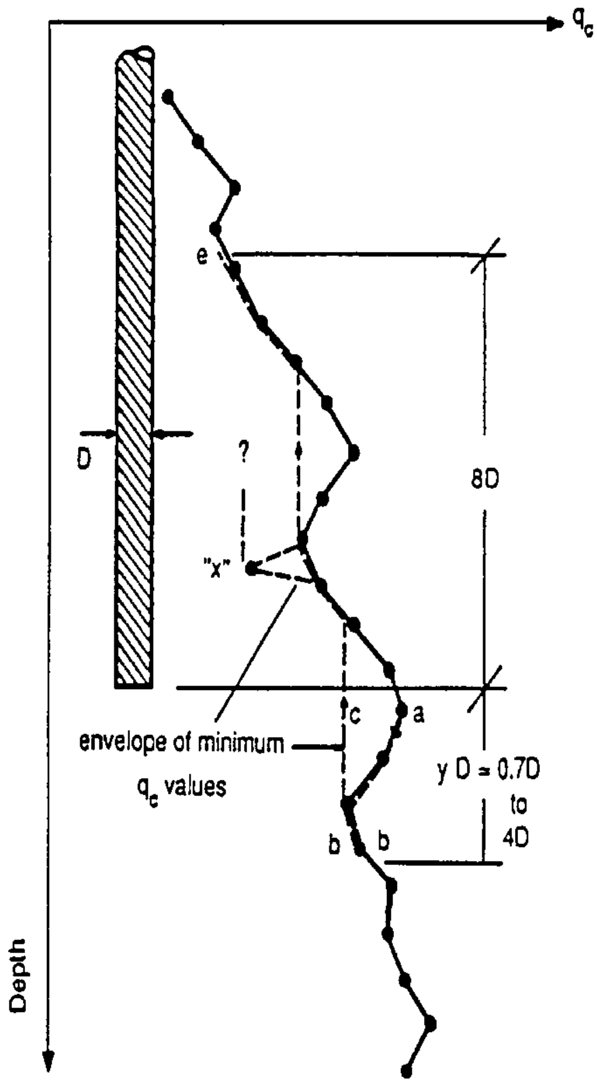

Figure 10.7.3.8.6g-1—Pile End-Bearing Computation Procedure after Nottingham and Schmertmann (1975)

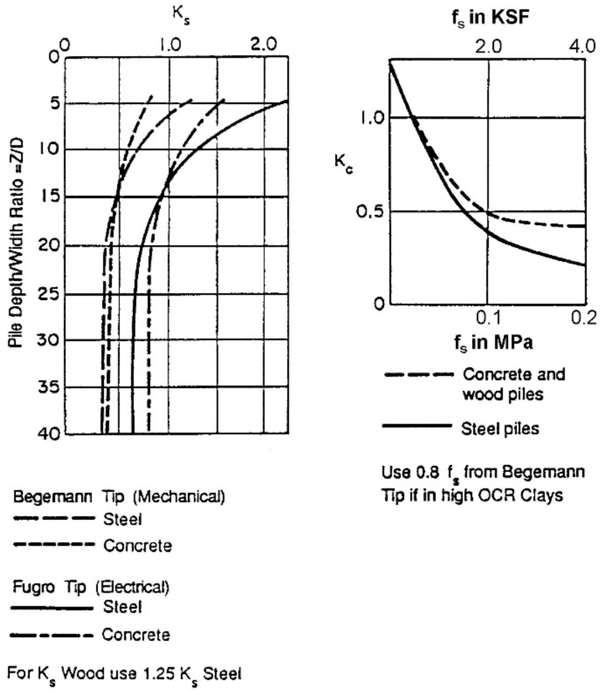

Figure 10.7.3.8.6g-2 —Side Resistance Correction Factors K_s and K_c after Nottingham and Schmertmann (1975)

10.7.3.9—Resistance of Pile Groups in Compression

For pile groups in clay, the nominal bearing resistance of the pile group shall be taken as the lesser of:

- The sum of the individual nominal resistances of each pile in the group, or
- The nominal resistance of an equivalent pier consisting of the piles and the block of soil within the area bounded by the piles.

If the cap is not in firm contact with the ground and if the soil at the surface is soft, the individual nominal resistance of each pile shall be multiplied by an efficiency factor η, taken as:

- η = 0.65 for a center-to-center spacing of 2.5 diameters,
- η = 1.0 for a center-to-center spacing of 6.0 diameters.

For intermediate spacings, the value of η should be determined by linear interpolation.

If the cap is in firm contact with the ground, no reduction in efficiency shall be required. If the cap is not in firm contact with the ground and if the soil is stiff, no reduction in efficiency shall be required.

The nominal bearing resistance of pile groups in cohesionless soil shall be the sum of the resistance of all the piles in the group. The efficiency factor, η, shall be 1.0 where the pile cap is or is not in contact with the ground for a center-to-center pile spacing of 2.5 diameters or

C10.7.3.9

The equivalent pier approach checks for block failure and is generally only applicable for pile groups within cohesive soils. For pile groups in cohesionless soils, the sum of the nominal resistances of the individual piles always controls the group resistance.

When analyzing the equivalent pier, the full shear strength of the soil should be used to determine the friction resistance. The total base area of the equivalent pier should be used to determine the end bearing resistance.

In cohesive soils, the nominal resistance of a pile group depends on whether the cap is in firm contact with the ground beneath. If the cap is in firm contact, the soil between the pile and the pile group behave as a unit.

At small pile spacings, a block type failure mechanism may prevail, whereas individual pile failure may occur at larger pile spacings. It is necessary to check for both failure mechanisms and design for the case that yields the minimum capacity.

For a pile group of width X, length Y, and depth Z, as shown in Figure C10.7.3.9-1, the bearing capacity for block failure, in kips, is given by:

$$Q_g = \left(2X + 2Y\right) Z\overline{S_u} + XYN_cS_u \qquad \text{(C10.7.3.9-1)}$$

in which:

for $\dfrac{Z}{X} \leq 2.5$:

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greater. The resistance factor is the same as that for single piles, as specified in Table 10.5.5.2.3-1.

For pile groups in clay or sand, if a pile group is tipped in a strong soil deposit overlying a weak deposit, the block bearing resistance shall be evaluated with consideration to pile group punching as a group into the underlying weaker layer. The methods in Article 10.6.3.1.2a of determining bearing resistance of a spread footing in a strong layer overlying a weaker layer shall apply, with the notional footing located as shown in Article 10.7.2.3.

$$N_c = 5\left(1 + \frac{0.2X}{Y}\right)\left(1 + \frac{0.2Z}{X}\right) \qquad \text{(C10.7.3.9-2)}$$

for $\dfrac{Z}{X} > 2.5$:

$$N_c = 7.5\left(1 + \frac{0.2X}{Y}\right) \qquad \text{(C10.7.3.9-3)}$$

where:

$\overline{S_u}$ = average undrained shear strength along the depth of penetration of the piles (ksf)

S_u = undrained shear strength at the base of the group (ksf)

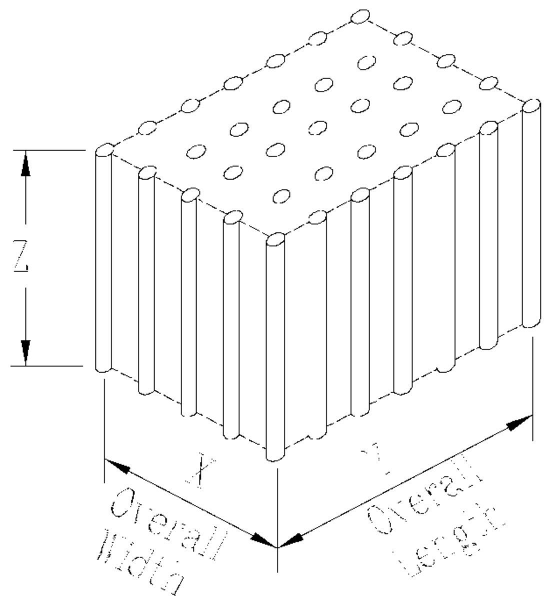

Figure C10.7.3.9-1—Pile Group Acting as a Block Foundation

10.7.3.10—Uplift Resistance of Single Piles

Uplift on single piles shall be evaluated when tensile forces are present. The factored nominal tensile resistance of the pile due to soil failure shall be greater than the factored pile loads.

The nominal uplift resistance of a single pile should be estimated in a manner similar to that for estimating the side resistance of piles in compression specified in Article 10.7.3.8.6.

Factored uplift resistance in kips shall be taken as:

$$R_R = \varphi R_n = \varphi_{up} R_s \qquad \text{(10.7.3.10-1)}$$

C10.7.3.10

The factored load effect acting on any pile in a group may be estimated using the traditional elastic strength of materials procedure for a cross-section under thrust and moment. The cross-sectional properties should be based on the pile as a unit area.

Note that the resistance factor for uplift already is reduced to 80 percent of the resistance factor for static side resistance. Therefore, the side resistance estimated based on Article 10.7.3.8.6 does not need to be reduced to account for uplift effects on side resistance.

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where:

R_s = nominal uplift resistance due to side resistance (kips)

φ_{up} = resistance factor for uplift resistance specified in Table 10.5.5.2.3-1

Nominal uplift resistance of single piles may be determined by static load test or by dynamic test with signal matching. If a static uplift test is to be performed, it shall follow the procedures specified in ASTM D3689. Dynamic tests with signal matching, if conducted, shall be performed as specified in Article 10.7.3.8.3. If dynamic tests with signal matching are used to determine uplift, a maximum of 80 percent of the uplift determined from the dynamic test should be used.

The static pile uplift load test(s) should be used to calibrate the static analysis method, i.e., back calculate soil properties, to adjust the calculated uplift resistance for variations in the stratigraphy. The minimum penetration criterion to obtain the desired uplift resistance should be based on the calculated uplift resistance using the static pile uplift load test results.

Static uplift tests should be evaluated using a modified Davisson Method as described in Hannigan et al. (2006).

If using dynamic tests with signal matching to determine uplift resistance, it may be difficult to separate the measured end bearing resistance from the side resistance acting on the bottom section of the pile, especially if the soil stiffness at the pile tip is not significantly different from the soil stiffness acting on the sides of the pile near the pile tip. If it is not clear what is end bearing and what is side friction near the pile tip, the side resistance acting on the bottom pile element should be ignored when estimating uplift resistance using this method. If the pile length is shorter than 30 ft. in length, caution should be exercised when using dynamic tests with signal matching to estimate uplift.

10.7.3.11—Uplift Resistance of Pile Groups

The nominal uplift resistance of pile groups shall be evaluated when the foundation is subjected to uplift loads.

Pile group factored uplift resistance, in kips, shall be taken as:

$$R_R = \varphi R_n = \varphi_{ug} R_{ug} \qquad (10.7.3.11\text{-}1)$$

where:

φ_{ug} = resistance factor specified in Table 10.5.5.2.3-1

R_{ug} = nominal uplift resistance of the pile group (kips)

The nominal uplift resistance, R_{ug}, of a pile group shall be taken as the lesser of:

- the sum of the individual pile uplift resistance, or
- the uplift resistance of the pile group considered as a block.

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be determined using a spread of load of 1*H* in 4*V* from the base of the pile group taken from Figure 10.7.3.11-1. Buoyant unit weights shall be used for soil below the groundwater level.

In cohesive soils, the block used to resist uplift in undrained shear shall be taken from Figure 10.7.3.11-2.

C10.7.3.11

A net uplift force can act on the foundation. An example of such a load is the construction load induced during the erection of concrete segmental girder bridges.

The nominal group uplift resistance may be taken as:

$$R_n = R_{ug} = (2XZ + 2YZ)\bar{S}_u + W_g \qquad (10.7.3.11\text{-}2)$$

where:

X = width of the group, as shown in Figure 10.7.3.11-2 (ft)
Y = length of the group, as shown in Figure 10.7.3.11-2 (ft)
Z = depth of the block of soil below pile cap taken from Figure 10.7.3.11-2 (ft)
$\overline{S}_u$ = average undrained shear strength along the sides of the pile group (ksf)
W_g = weight of the block of soil, piles, and pile cap (kips)

The resistance factor for the nominal group uplift resistance, R_{ug}, determined as the sum of the individual pile resistances, shall be taken as the same as that for the uplift resistance of single piles as specified in Table 10.5.5.2.3-1.

The resistance factor for the uplift resistance of the pile group considered as a block shall be taken as specified in Table 10.5.5.2.3-1 for pile groups in all soils.

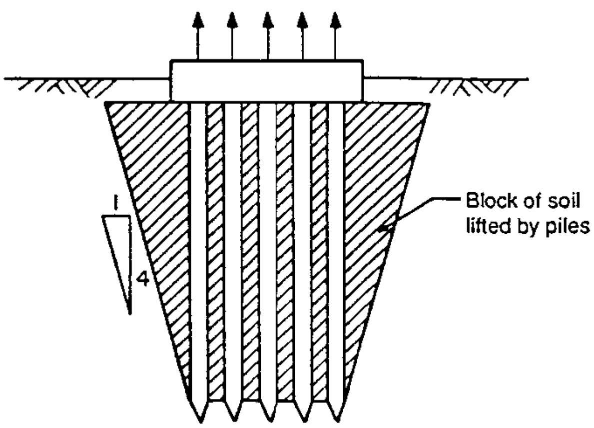

Figure 10.7.3.11-1—Uplift of Group of Closely Spaced Piles in Cohesionless Soils after Tomlinson (1987)

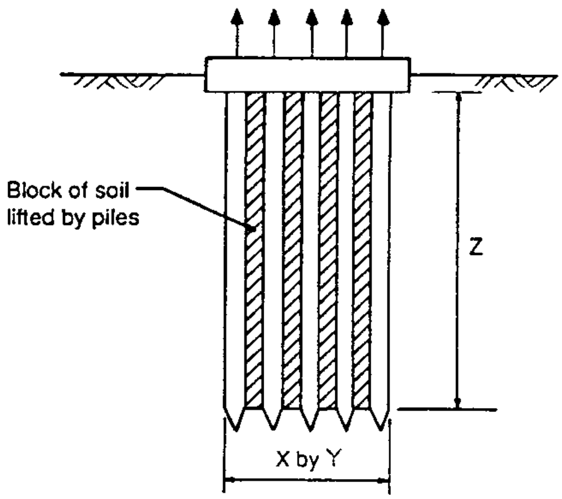

Figure 10.7.3.11-2—Uplift of Group of Piles in Cohesive Soils after Tomlinson (1987)

10.7.3.12—Nominal Lateral Resistance of Pile Foundations

The nominal resistance of pile foundations to lateral loads shall be evaluated based on both geomaterial and structural properties. The lateral soil resistance along the piles should be modeled using *P-y* curves developed for the soils at the site.

The applied loads shall be factored loads and they must include both lateral and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. The *P-y* curves shall be modified for group effects. The *P*-multipliers in Table 10.7.2.4-1 should be used to modify the curves. If the pile cap will always be embedded, the *P-y* lateral resistance of the soil on the cap face may be included in the nominal lateral resistance.

The minimum penetration of the piles below ground (see Article 10.7.6) required in the contract should be established such that fixity is obtained. For this determination, the loads applied to the pile are factored as specified in Section 3, and a soil resistance factor of 1.0 shall be used as specified in Table 10.5.5.2.3-1.

If fixity cannot be obtained, additional piles should be added, larger diameter piles used if feasible to drive them to the required depth, or a wider spacing of piles in the group should be considered to provide the necessary lateral resistance. Batter piles may be added to provide the lateral resistance needed, unless downdrag is anticipated. If downdrag is anticipated, batter piles should not be used. The design procedure, if fixity cannot be obtained, should take into consideration the lack of fixity of the pile.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D3966.

C10.7.3.12

Pile foundations are subjected to lateral loads due to wind, traffic loads, bridge curvature, stream flow, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles and vertical piles are more effective against dynamic loads.

Additional details regarding methods of analysis using *P-y* curves, both for single piles and pile groups, are provided in Article 10.7.2.4. As an alternative to *P-y* analysis, strain wedge theory may be used (see Article 10.7.2.4).

When this analysis is performed, the loads are factored since the strength limit state is under consideration, but the resistances as represented by the *P-y* curves are not factored since they already represent the ultimate condition.

The strength limit state for lateral resistance is only structural (see Sections 5 and 6 for structural limit state design requirements), though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state, and this limit state is reached, in the general case, when the nominal combined bending and axial resistance is reached.

If the lateral resistance of the soil in front of the pile cap is included in the lateral resistance of the foundation, the effect of soil disturbance resulting from construction of the pile cap should be considered. In such cases, the passive resistance may need to be reduced to account for the effects of disturbance.

For information on analysis and interpretation of load tests, see Article 10.7.2.4.

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10.7.3.13—Pile Structural Resistance

10.7.3.13.1—Steel Piles

The nominal axial compression resistance in the structural limit state for piles loaded in compression shall be as specified in Article 6.9.4.1 for noncomposite piles and Article 6.9.5.1 for composite piles. If the pile is fully embedded, λ in Eq. 6.9.5.1-1 shall be taken as zero.

The nominal axial resistance of horizontally unsupported noncomposite piles that extend above the ground surface in air or water shall be determined from Eqs. 6.9.4.1.1-1 or 6.9.4.1.1-2. The nominal axial resistance of horizontally unsupported composite piles that extend above the ground surface in air or water shall be determined from Eqs. 6.9.5.1-1 or 6.9.5.1-2.

The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

The resistance factors for the compression limit state are specified in Article 6.5.4.2.

C10.7.3.13.1

Composite members refer to steel pipe piles that are filled with concrete.

The effective length given in Article C10.7.3.13.4 is an empirical approach to determining effective length. Computer methods are now available that can determine the axial resistance of a laterally unsupported compression member using a P-Δ analysis that includes a numerical representation of the lateral soil resistance (Williams et al., 2003). These methods are preferred over the empirical approach in Article C10.7.3.13.4.

10.7.3.13.2—Concrete Piles

The nominal axial compression resistance for concrete piles and prestressed concrete piles shall be as specified in Article 5.6.4.4.

The nominal axial compression resistance for concrete piles that are laterally unsupported in air or water shall be determined using the procedures given in Articles 5.6.4.3 and 4.5.3.2. The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

The resistance factor for the compression limit state for concrete piles shall be that given in Article 5.5.4.2 for concrete loaded in axial compression.

C10.7.3.13.2

Article 5.6.4 includes specified limits on longitudinal reinforcement, spirals and ties. Methods are given for determining nominal axial compression resistance but they do not include the nominal axial compression resistance of prestressed members. Compression members are usually prestressed only where they are subjected to high levels of flexure. Therefore, a method of determining nominal axial compression resistance is not given.

Article 5.6.4.5 specifically permits an analysis based on equilibrium and strain compatibility. Methods are also available for performing a stability analysis (Williams et al., 2003).

10.7.3.13.3—Timber Piles

The nominal axial compression resistance for timber piles shall be as specified in Article 8.8.2. The methods presented there include both laterally supported and laterally unsupported members.

The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

C10.7.3.13.3

Article 8.5.2.3 requires that a reduction factor for long term loads of 0.75 be multiplied times the resistance factor for Strength Load Combination IV.

10.7.3.13.4—Buckling and Lateral Stability

In evaluating stability, the effective length of the pile shall be equal to the laterally unsupported length, plus an embedded depth to fixity.

The potential for buckling of unsupported pile lengths and the determination of stability under lateral loading should be evaluated by methods that consider soil-structure interaction as specified in Article 10.7.3.12.

C10.7.3.13.4

For preliminary design, the depth to fixity below the ground, in ft, may be taken as:

- For clays:

$$1.4\,[E_p\,I_w / E_s\,]^{0.25} \qquad \text{(C10.7.3.13.4-1)}$$

- For sands:

$$1.8\,[E_p\,I_w / n_h\,]^{0.2} \qquad \text{(C10.7.3.13.4-2)}$$

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where:

E_p = modulus of elasticity of pile (ksi)
I_w = weak axis moment of inertia for pile (ft^4)
E_s = soil modulus for clays = 0.465 S_u (ksi)
S_u = undrained shear strength of clays (ksf)
n_h = rate of increase of soil modulus with depth for sands as specified in Table C10.4.6.3-2 (ksi/ft)

This procedure is taken from Davisson and Robinson (1965).

In Eqs. C10.7.3.13.4-1 and C10.7.3.13.4-2, the loading condition has been assumed to be axial load only, and the piles are assumed to be fixed at their ends. Because the equations give depth to fixity from the ground line, the Engineer must determine the boundary conditions at the top of the pile to determine the total unbraced length of the pile. If other loading or pile tip conditions exist, see Davisson and Robinson (1965).

The effect of pile spacing on the soil modulus has been studied by Prakash and Sharma (1990), who found that, at pile spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling resistance. However, at a pile spacing of three times the pile width, the effective soil modulus is reduced to 25 percent of the value applicable to a single pile. For intermediate spacings, modulus values may be estimated by interpolation.

10.7.4—Extreme Event Limit State

The provisions of Article 10.5.5.3 shall apply.

For the applicable factored loads, including those specified in Article 10.7.1.6, for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance. For seismic design, all soil within and above the liquefiable zone, if the soil is liquefiable, shall not be considered to contribute bearing resistance. Downdrag resulting from liquefaction induced settlement shall be determined as specified in Article 3.11.8 and included in the loads applied to the foundation. Static downdrag loads should not be combined with seismic downdrag loads due to liquefaction.

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the *P-y* curve soil parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

When designing for scour, the pile foundation design shall be conducted as described in Article 10.7.3.6, except that the check flood and resistance factors consistent with Article 10.5.5.3.2 shall be used.

C10.7.4

See Article C10.5.5.3.3.

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10.7.5—Corrosion and Deterioration

The effects of corrosion and deterioration from environmental conditions shall be considered in the selection of the pile type and in the determination of the required pile cross-section.

As a minimum, the following types of deterioration shall be considered:

- corrosion of steel pile foundations, particularly in fill soils, low ph soils, and marine environments;
- sulfate, chloride, and acid attack of concrete pile foundations; and
- decay of timber piles from wetting and drying cycles or from insects or marine borers.

The following soil or site conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- resistivity less than 2,000 ohm-cm,
- pH less than 5.5,
- pH between 5.5 and 8.5 in soils with high organic content,
- sulfate concentrations greater than 1,000 ppm,
- landfills and cinder fills,
- soils subject to mine or industrial drainage,
- areas with a mixture of high resistivity soils and low resistivity high alkaline soils, and
- insects (wood piles).

The following water conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- chloride content greater than 500 ppm,
- sulfate concentration greater than 500 ppm,
- mine or industrial runoff,
- high organic content,
- pH less than 5.5,
- marine borers, and
- piles exposed to wet/dry cycles.

When chemical wastes are suspected, a full chemical analysis of soil and groundwater samples shall be considered.

C10.7.5

Resistivity, pH, chloride content, and sulfate concentration values have been adapted from those in Fang (1991) and Tomlinson (1987).

Some states use a coal tar epoxy paint system as a protective coating with good results.

The criterion for determining the potential for deterioration varies widely. An alternative set of recommendations is given by Elias (1990).

A field electrical resistivity survey or resistivity testing and pH testing of soil and groundwater samples may be used to evaluate the corrosion potential.

The deterioration potential of steel piles may be reduced by several methods, including protective coatings, concrete encasement, cathodic protection, use of special steel alloys, or increased steel area. Protective coatings should be resistant to abrasion and have a proven service record in the corrosive environment identified. Protective coatings should extend into noncorrosive soils a few feet because the lower portion of the coating is more susceptible to abrasion loss during installation.

Concrete encasement through the corrosive zone may also be used. The concrete mix should be of low permeability and placed properly. Steel piles protected by concrete encasement should be coated with a dielectric coating near the base of the concrete jacket.

The use of special steel alloys of nickel, copper, and potassium may also be used for increased corrosion resistance in the atmosphere or splash zone of marine piling.

Sacrificial steel area may also be used for corrosion resistance. This technique over sizes the steel section so that the available section after corrosion meets structural requirements.

Deterioration of concrete piles can be reduced by design procedures. These include use of a dense impermeable concrete, sulfate resisting Portland cement, increased steel cover, air-entrainment, reduced chloride content in the concrete mix, cathodic protection, and epoxy-coated reinforcement. Piles that are continuously submerged are less subject to deterioration. ACI 318, Section 4.5.2, provides maximum water-cement ratio requirements for special exposure conditions. ACI 318, Section 4.5.3, lists the appropriate types of cement for various types of sulfate exposure.

For prestressed concrete, ACI 318 recommends a maximum water-soluble chloride ion of 0.06 percent by weight of cement.

Cathodic protection of reinforcing and prestressing steel may be used to protect concrete from corrosion effects. This process induces electric flow from the anode to the cathode of the pile and reduces corrosion. An external DC power source may be required to drive the current. However, cathodic protection requires electrical continuity between all steel and that necessitates bonding the steel for electric connection. This bonding is expensive and usually precludes the use of cathodic protection of concrete piles.

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Epoxy coating of pile reinforcement has been found in some cases to be useful in resisting corrosion. It is important to ensure that the coating is continuous and free of holidays.

More detail on design for corrosion or other forms of deterioration is contained in Hannigan et al. (2006).

10.7.6—Determination of Minimum Pile Penetration

The minimum pile penetration, if required for the particular site conditions and loading, shall be based on the maximum depth (i.e., tip elevation) needed to meet the following requirements as applicable:

- single and pile group settlement (service limit state)
- lateral deflection (service limit state)
- uplift (strength limit state)
- penetration into bearing soils needed to get below soil causing downdrag loads on the pile foundation resulting from static consolidation stresses on soft soil or downdrag loads due to liquefaction (strength and extreme event limit state, respectively)
- penetration into bearing soils needed to get below soil subject to scour
- penetration into bearing soils necessary to obtain fixity for resisting the applied lateral loads to the foundation (strength limit state)
- axial uplift, and nominal lateral resistance to resist extreme event limit state loads

The contract documents should indicate the minimum pile penetration, if applicable, as determined above only if one or more of the requirements listed above are applicable to the pile foundation. The contract documents should also include the required nominal axial compressive resistance, R_{ndr} as specified in Article 10.7.7 and the method by which this resistance will be verified, if applicable, such that the resistance factor(s) used for design are consistent with the construction field verification methods of nominal axial compressive pile resistance.

C10.7.6

A minimum pile penetration should only be specified if necessary to ensure that all of the applicable limit states are met. A minimum pile penetration should not be specified solely to meet axial compression resistance, i.e., bearing, unless field verification of the pile nominal bearing resistance is not performed as described in Article 10.7.3.8.

10.7.7—Determination of R_{ndr} Used to Establish Contract Driving Criteria for Nominal Bearing Resistance

The value of R_{ndr} used for the construction of the pile foundation to establish the driving criteria to obtain the nominal bearing resistance shall be the value that meets or exceeds the following limit states, as applicable:

- strength limit state nominal bearing resistance specified in Article 10.7.3.8
- strength limit state nominal bearing resistance, including downdrag specified in Article 10.7.3.7
- strength limit state nominal bearing resistance, accounting for scour specified in Article 10.7.3.6
- extreme event limit state nominal bearing resistance for seismic specified in Article 10.7.4

- extreme event limit state nominal bearing resistance for scour specified in Article 10.7.4

10.7.8—Drivability Analysis

The establishment of the installation criteria for driven piles should include a drivability analysis. Except as specified herein, the drivability analysis shall be performed by the Engineer using a wave equation analysis, and the driving stresses (σ_{dr}) anywhere in the pile determined from the analysis shall be less than the following limits:

Steel piles, compression and tension:

$$\sigma_{dr} = 0.9\varphi_{da} f_y \qquad (10.7.8\text{-}1)$$

where:

f_y = specified minimum yield strength of the steel (ksi)

φ_{da} = resistance factor as specified in Table 10.5.5.2.3-1

Concrete piles:

- In compression:

$$\sigma_{dr} = \varphi_{da} 0.85 f'_c \qquad (10.7.8\text{-}2)$$

- In tension, considering only the steel reinforcement:

$$\sigma_{dr} = 0.7\varphi_{da} f_y \qquad (10.7.8\text{-}3)$$

where:

f'_c = compressive strength of the concrete (ksi)

f_y = specified minimum yield strength of the steel reinforcement (ksi)

Prestressed concrete piles, normal environments:

- In compression:

$$\sigma_{dr} = \varphi_{da}\left(0.85 f'_c - f_{pe}\right) \qquad (10.7.8\text{-}4)$$

- In tension:

$$\sigma_{dr} = \varphi_{da}\left(0.095\sqrt{f'_c} + f_{pe}\right) \qquad (10.7.8\text{-}5)$$

where:

f_{pe} = effective prestressing stress in concrete (ksi)

Prestressed concrete piles, severe corrosive environments:

- In tension:

C10.7.8

Wave equation analyses should be conducted during design using a range of likely hammer/pile combinations, considering the soil and installation conditions at the foundation site. See Article 10.7.3.8.4 for additional considerations for conducting wave equation analyses. These analyses should be used to assess feasibility of the proposed foundation system and to establish installation criteria with regard to driving stresses to limit driving stresses to acceptable levels. For routine pile installation applications, e.g., smaller diameter, low nominal resistance piles, the development of installation criteria with regard to the limitation of driving stresses, e.g., minimum or maximum ram weight, hammer size, maximum acceptable driving resistance, etc., may be based on local experience, rather than conducting a detailed wave equation analysis that is project specific. Local experience could include previous drivability analysis results and actual pile driving experience that are applicable to the project specific situation at hand. Otherwise, a project specific drivability study should be conducted.

Drivability analyses may also be conducted as part of the project construction phase. When conducted during the construction phase, the drivability analysis shall be conducted using the contractor's proposed driving system. This information should be supplied by the contractor. This drivability analysis should be used to determine if the contractor's proposed driving system is capable of driving the pile to the maximum resistance anticipated without exceeding the factored structural resistance available, i.e., σ_{dr}.

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$$\sigma_{dr} = \varphi_{da} f_{pe} \quad (10.7.8\text{-}6)$$

Timber piles, in compression and tension:

$$\sigma_{dr} = \varphi_{da}(2.6F_{co}) \quad (10.7.8\text{-}7)$$

where:

F_{co} = base resistance of wood in compression parallel to the grain as specified in Article 8.4.1.3 (ksi)

This drivability analysis shall be based on the maximum driving resistance needed:

- to obtain minimum penetration requirements specified in Article 10.7.6,
- to overcome resistance of soil that cannot be counted upon to provide axial or lateral resistance throughout the design life of the structure, e.g., material subject to scour, or material subject to downdrag, and
- to obtain the required nominal bearing resistance.

In addition to this drivability analysis, the best approach to controlling driving stresses during pile installation is to conduct dynamic testing with signal matching to verify the accuracy of the wave equation analysis results. Note that if a drivability analysis is conducted using the wave equation for acceptance of the contractor's proposed driving system, but a different method is used to develop driving resistance, i.e., blow count, criterion to obtain the specified nominal pile resistance, e.g., a driving formula, the difference in the methods regarding the predicted driving resistance should be taken into account when evaluating the contractor's driving system. For example, the wave equation analysis could indicate that the contractor's hammer can achieve the desired bearing resistance, but the driving formula could indicate the driving resistance at the required nominal bearing is too high. Such differences should be considered when setting up the driving system acceptance requirements in the contract documents, though it is preferable to be consistent in the method used for acceptance of the contractor's driving system and the one used for developing driving criteria.

The selection of a blow count limit as a definition of refusal is difficult because it can depend on the site soil profile, the pile type, hammer performance, and possibly hammer manufacturer limitations to prevent hammer damage. In general, blow counts greater than 10–15 blows per inch should be used with care, particularly with concrete or timber piles. In cases where the driving is easy until near the end of driving, a higher blow count may sometimes be satisfactory, but if a high blow count is required over a large percentage of the depth, even ten blows per inch may be too large.

10.7.9—Probe Piles

Probe piles should be driven at several locations on the site to establish order length. If dynamic measurements are not taken, these probe piles should be driven after the driving criteria have been established.
If dynamic measurements during driving are taken, both order lengths and driving criteria should be established after the probe pile(s) are driven.

C10.7.9

Probe piles are sometimes known as test piles or indicator piles. It is common practice to drive probe piles at the beginning of the project (particularly with concrete piles) to establish pile order lengths and/or to evaluate site variability whether or not dynamic measurements are taken.

10.8—DRILLED SHAFTS

10.8.1—General

10.8.1.1—Scope

The provisions of this Section shall apply to the design of drilled shafts. Throughout these provisions, the use of the term "drilled shaft" shall be interpreted to mean a shaft constructed using either drilling (open hole or with drilling slurry) or casing plus excavation equipment and technology.

These provisions shall also apply to shafts that are constructed using casing advancers that twist or rotate

C10.8.1.1

Drilled shafts may be an economical alternative to spread footing or pile foundations, particularly when spread footings cannot be founded on suitable soil or rock strata within a reasonable depth or when driven piles are not viable. Drilled shafts may be an economical alternative to spread footings where scour depth is large. Drilled shafts may also be considered to resist high lateral or axial loads, or when deformation tolerances are small.

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casings into the ground concurrent with excavation rather than drilling.

The provisions of this Section shall not be taken as applicable to drilled piles, e.g., augercast piles, installed with continuous flight augers that are concreted as the auger is being extracted.

For example, a movable bridge is a bridge where it is desirable to keep deformations small.

Drilled shafts are classified according to their primary mechanism for deriving load resistance either as floating (friction) shafts, i.e., shafts transferring load primarily by side resistance, or end-bearing shafts, i.e., shafts transferring load primarily by tip resistance.

It is recommended that the shaft design be reviewed for constructability prior to advertising the project for bids.

10.8.1.2—Shaft Spacing, Clearance, and Embedment into Cap

If the center-to-center spacing of drilled shafts is less than 4.0 diameters, the interaction effects between adjacent shafts shall be evaluated. If the center-to-center spacing of drilled shafts is less than 6.0 diameters, the sequence of construction should be specified in the contract documents.

Shafts used in groups should be located such that the distance from the side of any shaft to the nearest edge of the cap is not less than 12.0 in. Shafts shall be embedded sufficiently into the cap to develop the required structural resistance.

C10.8.1.2

Larger spacing may be required to preserve shaft excavation stability or to prevent communication between shafts during excavation and concrete placement.

Shaft spacing may be decreased if casing construction methods are required to maintain excavation stability and to prevent interaction between adjacent shafts.

10.8.1.3—Shaft Diameter and Enlarged Bases

If the shaft is to be manually inspected, the shaft diameter should not be less than 30.0 in. The diameter of columns supported by shafts should be smaller than or equal to the diameter of the drilled shaft.

In stiff cohesive soils, an enlarged base (bell, or underream) may be used at the shaft tip to increase the tip bearing area to reduce the unit end bearing pressure or to provide additional resistance to uplift loads.

C10.8.1.3

Nominal shaft diameters used for both geotechnical and structural design of shafts should be selected based on available diameter sizes.

If the shaft and the column are the same diameter, it should be recognized that the placement tolerance of drilled shafts is such that it will likely affect the column location. The shaft and column diameter should be determined based on the shaft placement tolerance, column and shaft reinforcing clearances, and the constructability of placing the column reinforcing in the shaft. A horizontal construction joint in the shaft at the bottom of the column reinforcing will facilitate constructability. Making allowance for the tolerance where the column connects with the superstructure, which could affect column alignment, can also accommodate this shaft construction tolerance.

In drilling rock sockets, it is common to use casing through the soil zone to temporarily support the soil to prevent cave-in, allow inspection and to produce a seal along the soil-rock contact to minimize infiltration of groundwater into the socket. Depending on the method of excavation, the diameter of the rock socket may need to be sized at least 6.0 in. smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment.

Where practical, consideration should be given to extension of the shaft to a greater depth to avoid the difficulty and expense of excavation for enlarged bases.

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Where the bottom of the drilled hole is dry, cleaned and inspected prior to concrete placement, the entire base area may be taken as effective in transferring load.

10.8.1.4—Battered Shafts

Battered shafts should be avoided. Where increased lateral resistance is needed, consideration should be given to increasing the shaft diameter or increasing the number of shafts.

C10.8.1.4

Due to problems associated with hole stability during excavation, installation, and with removal of casing during installation of the rebar cage and concrete placement, construction of battered shafts is very difficult.

10.8.1.5—Drilled Shaft Resistance

Drilled shafts shall be designed to have adequate axial and structural resistances, tolerable settlements, and tolerable lateral displacements.

The axial resistance of drilled shafts shall be determined through a suitable combination of subsurface investigations, laboratory and/or in-situ tests, analytical methods, and load tests, with reference to the history of past performance. Consideration shall also be given to:

- the difference between the resistance of a single shaft and that of a group of shafts;
- the resistance of the underlying strata to support the load of the shaft group;
- the effects of constructing the shaft(s) on adjacent structures;
- the possibility of scour and its effect;
- the transmission of forces, such as downdrag forces, from consolidating soil;
- minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads and seismic conditions;
- satisfactory behavior under service loads;
- drilled shaft nominal structural resistance; and
- long-term durability of the shaft in service, i.e., corrosion and deterioration.

Resistance factors for shaft axial resistance for the strength limit state shall be as specified in Table 10.5.5.2.4-1.

The method of construction may affect the shaft axial and lateral resistance. The shaft design parameters shall take into account the likely construction methodologies used to install the shaft.

C10.8.1.5

The drilled shaft design process is discussed in detail in Drilled Shafts: Construction Procedures and Design Methods (Brown et al., 2010).

The performance of drilled shaft foundations can be greatly affected by the method of construction, particularly side resistance. The Designer should consider the effects of ground and groundwater conditions on shaft construction operations and delineate, where necessary, the general method of construction to be followed to ensure the expected performance. Because shafts derive their resistance from side and tip resistance, which is a function of the condition of the materials in direct contact with the shaft, it is important that the construction procedures be consistent with the material conditions assumed in the design. Softening, loosening, or other changes in soil and rock conditions caused by the construction method could result in a reduction in shaft resistance and an increase in shaft displacement. Therefore, evaluation of the effects of the shaft construction procedure on resistance should be considered an inherent aspect of the design. Use of slurries, varying shaft diameters, and post grouting can also affect shaft resistance.

Soil parameters should be varied systematically to model the range of anticipated conditions. Both vertical and lateral resistance should be evaluated in this manner.

Procedures that may affect axial or lateral shaft resistance include, but are not limited to, the following:

- Artificial socket roughening, if included in the design nominal axial resistance assumptions.
- Removal of temporary casing where the design is dependent on concrete-to-soil adhesion.
- The use of permanent casing.
- Use of tooling that produces a uniform cross-section where the design of the shaft to resist lateral loads cannot tolerate the change in stiffness if telescoped casing is used.

It should be recognized that the design procedures provided in these Specifications assume compliance to construction specifications that will produce a high-quality shaft. Performance criteria should be included in the construction specifications that require:

- shaft bottom cleanout criteria,
- appropriate means to prevent side wall movement or failure (caving) such as temporary casing, slurry, or a combination of the two,
- slurry maintenance requirements including minimum slurry head requirements, slurry testing requirements, and maximum time the shaft may be left open before concrete placement.

If for some reason one or more of these performance criteria are not met, the design should be reevaluated and the shaft repaired or replaced as necessary.

10.8.1.6—Determination of Shaft Loads

10.8.1.6.1—General

The factored loads to be used in shaft foundation design shall be as specified in Section 3. Computational assumptions that shall be used in determining individual shaft loads are also specified in Section 3.

C10.8.1.6.1

The specification and determination of top of cap loads is discussed extensively in Section 3. It should be noted that Article 3.6.2.1 states that dynamic load allowance need not be applied to foundation elements that are below the ground surface. Therefore, if shafts extend above the ground surface to act as columns the dynamic load allowance should be included in evaluating the structural resistance of that part of the shaft above the ground surface. The dynamic load allowance may be ignored in evaluating the geotechnical resistance.

10.8.1.6.2—Downdrag

The provisions of Articles 10.7.1.6.2 and 3.11.8 shall apply for determination of load due to downdrag.

For shafts with tip bearing in a dense stratum or rock where design of the shaft is structurally controlled, downdrag shall be considered at the strength and extreme event limit states.

For shafts with tip bearing in soil, downdrag shall not be considered at the strength and extreme limit states if settlement of the shaft is less than failure criterion.

C10.8.1.6.2

See Article C3.11.8.

Downdrag loads may be estimated using the α-method, as specified in Article 10.8.3.5.1b, to calculate negative shaft friction. As with positive shaft resistance, shaft length assumed to not contribute to nominal side resistance should also be assumed to not contribute to downdrag loads.

When using the α-method, an allowance should be made for a possible increase in the undrained shear strength as consolidation occurs. Downdrag loads may also come from cohesionless soils above settling cohesive soils. The downdrag caused by settling cohesionless soils may be estimated using the β method presented in Article 10.8.3.5.2.

Downdrag occurs in response to relative downward deformation of the surrounding soil to that of the shaft, and may not exist if downward movement of the drilled shaft in response to axial compression forces exceeds the vertical deformation of the soil. The response of a drilled shaft to downdrag in combination with the other forces acting at the head of the shaft therefore is complex and a realistic evaluation of actual limit states that may occur requires careful consideration of two issues: (1) drilled shaft load-settlement behavior and (2) the time period over which downdrag occurs relative to the time period over which nonpermanent components of load occur. When these factors are taken into account, it is appropriate to consider different downdrag forces for evaluation of geotechnical strength limit states than for

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structural strength limit states. These issues are addressed in Brown et al. (2010).

10.8.1.6.3—Uplift

The provisions in Article 10.7.1.6.3 shall apply.

C10.8.1.6.3

See Article C10.7.1.6.3.

10.8.2—Service Limit State Design

10.8.2.1—Tolerable Movements

The requirements of Article 10.5.2.1 shall apply.

C10.8.2.1

See Article C10.5.2.1.

10.8.2.2—Settlement

10.8.2.2.1—General

The settlement of a drilled shaft foundation involving either single-drilled shafts or groups of drilled shafts shall not exceed the movement criteria selected in accordance with Article 10.5.2.1.

10.8.2.2.2—Settlement of Single-Drilled Shaft

The settlement of single-drilled shafts shall be estimated as a sum of the following:

- short-term settlement resulting from load transfer,
- consolidation settlement if constructed where cohesive soil exhist beneath the shaft tip, and
- axial compression of the shaft.

The normalized load-settlement curves shown in Figures 10.8.2.2.2-1 through 10.8.2.2.2-4 should be used to limit the nominal shaft axial resistance computed as specified for the strength limit state in Article 10.8.3 for service limit state tolerable movements. Consistent values of normalized settlement shall be used for limiting the base and side resistance when using these Figures. Long-term settlement should be computed according to Article 10.7.2 using the equivalent footing method and added to the short-term settlements estimated using Figures 10.8.2.2.2-1 through 10.8.2.2.2-4.

C10.8.2.2.2

O'Neill and Reese (1999) have summarized load-settlement data for drilled shafts in dimensionless form, as shown in Figures 10.8.2.2.2-1 through 10.8.2.2.2-4. These curves do not include consideration of long-term consolidation settlement for shafts in cohesive soils. Figures 10.8.2.2.2-1 and 10.8.2.2.2-2 show the load-settlement curves in side resistance and in end bearing for shafts in cohesive soils. Figures 10.8.2.2.2-3 and 10.8.2.2.2-4 are similar curves for shafts in cohesionless soils. These curves should be used for estimating short-term settlements of drilled shafts.

The designer should exercise judgment relative to whether the trend line, one of the limits, or some relation in between should be used from Figures 10.8.2.2.2-1 through 10.8.2.2.2-4.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. Although elastic shortening may be small in relatively short shafts, it may be substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. O'Neill and Reese (1999) have described an approximate procedure for estimating the elastic shortening of long- drilled shafts.

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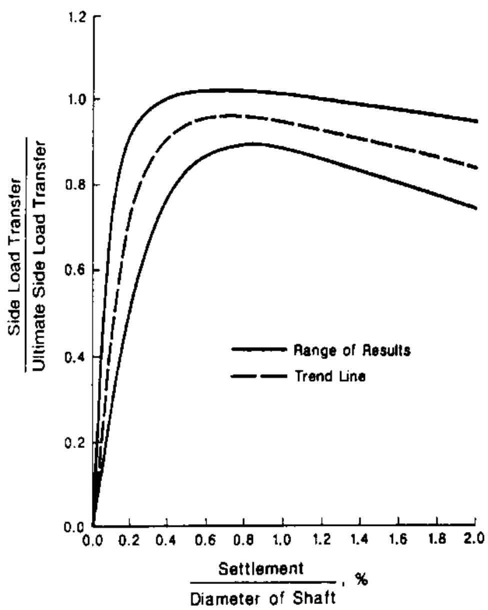

Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

Induced setttlements for isolated drilled shafts are different for elements in cohesive soils and in cohesionless soils. In cohesive soils, the failure threshold, or nominal axial resistance corresponds to mobilization of the full available side resistance, plus the full available base resistance. In cohesive soils, the failure threshold has been shown to occur at an average normalized deformation of four percent of the shaft diameter. In cohesionless soils, the failure threshold is the force corresponding to mobilization of the full side resistance plus the base resistance, corresponding to settlement at a defined failure criterion. This has been traditionally defined as the bearing pressure required to cause vertical deformation equal to five percent of the shaft diameter, even though this does not correspond to complete failure of the soil beneath the base of the shaft.

Note that nominal base resistance in cohesionless soils is calculated according to the empirical correlation given by Eq. 10.8.3.5.2c-1 in terms of *N*-value. That relationship was developed using a base resistance corresponding to five percent normalized displacement. If a normalized displacement other than five percent is used, the base resistance calculated by Eq. 10.8.3.5.2c-1 must be corrected.

The curves in Figures 10.8.2.2.2-1 and 10.8.2.2.2-3 also show the settlements at which the side resistance is mobilized. The shaft skin friction, R_s, is typically fully mobilized at displacements of 0.2 percent to 0.8 percent of the shaft diameter for shafts in cohesive soils. For shafts in cohesionless soils, this value is 0.1 percent to 1.0 percent.

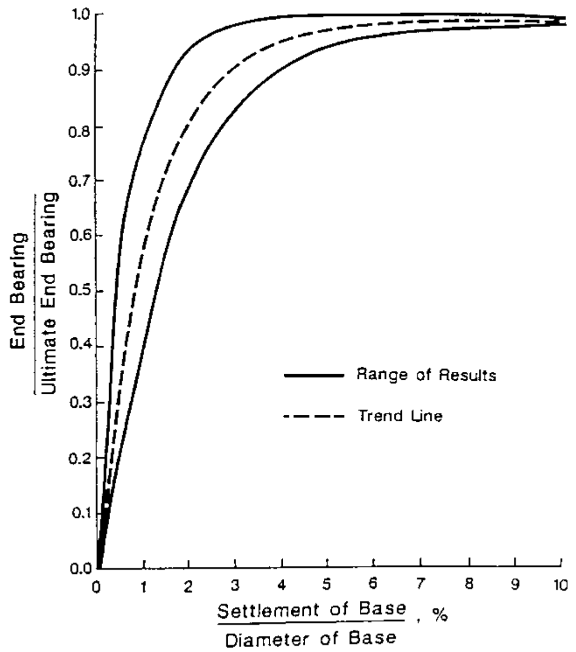

Figure 10.8.2.2.2-2—Normalized Load Transfer in End Bearing versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

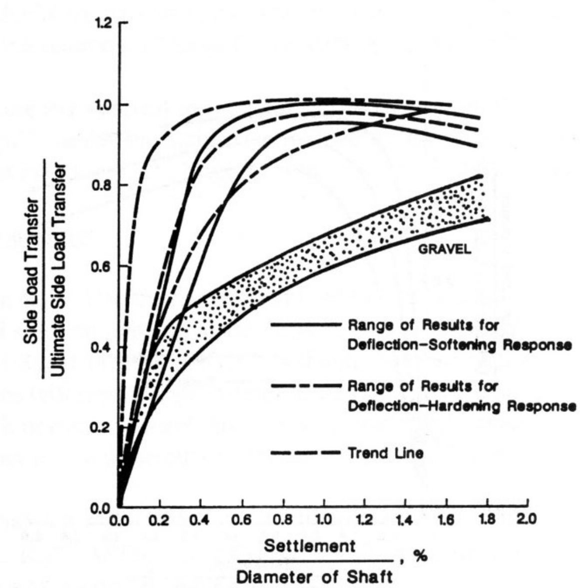

Figure 10.8.2.2.2-3—Normalized Load Transfer in Side Resistance versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

The deflection-softening response typically applies to cemented or partially-cemented soils, or other soils that exhibit brittle behavior, which have low residual shear strengths at larger deformations. Note that the trend line for sands is a reasonable approximation for either the deflection-softening or deflection-hardening response.

The normalized load-settlement curves require separate evaluation of an isolated drilled shaft for side and base resistance. Brown et al. (2010) provide alternate normalized load-settlement curves that may be used for estimation of settlement of a single drilled shaft considering combined side and base resistance. The method is based on modeling the average load deformation behavior observed from field load tests and incorporates the load test data used in development of the curves provided by O'Neill and Reese (1999). Additional methods that consider numerical simulations of axial load transfer and approximations based on elasto-plastic solutions are available in Brown et al. (2010).

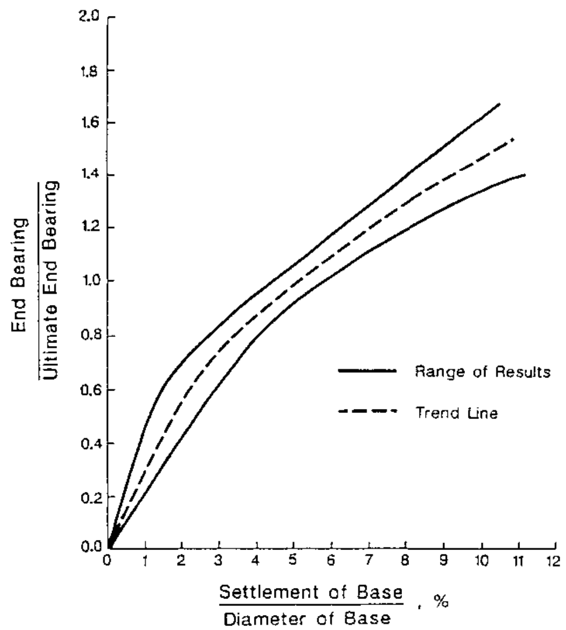

Figure 10.8.2.2.2-4—Normalized Load Transfer in End Bearing versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

10.8.2.2.3—Intermediate Geo Materials (IGMs)

For detailed settlement estimation of shafts in IGMs, the procedures described by Brown et al. (2010) should be used.

C10.8.2.2.3

IGMs are defined by Brown et al. (2010) as follows:

- *Cohesive IGM*—clay shales or mudstones with an S_u of 5 to 50 ksf

10.8.2.2.4—Group Settlement

The provisions of Article 10.7.2.3 shall apply. Shaft group effect shall be considered for groups of two shafts or more.

C10.8.2.2.4

See Article C10.7.2.3.

O'Neill and Reese (1999) summarize various studies on the effects of shaft group behavior. These studies were for groups that consisted of 1 × 2 to 3 × 3 shafts. These studies suggest that group effects are relatively unimportant for shaft center-to-center spacing of 5*D* or greater.

10.8.2.3—Horizontal Movement of Shafts and Shaft Groups

The provisions of Articles 10.5.2.1 and 10.7.2.4 shall apply.

For shafts socketed into rock, the input properties used to determine the response of the rock to lateral loading shall consider both the intact shear strength of the rock and the rock mass characteristics. The designer shall also consider the orientation and condition of discontinuities of the overall rock mass. Where specific adversely oriented discontinuities are not present but the rock mass is fractured such that its intact strength is considered compromised, the rock mass shear strength parameters should be assessed using the procedures for GSI rating in Article 10.4.6.4. For lateral deflection of the rock adjacent to the shaft greater than 0.0004*b*, where *b* is

C10.8.2.3

See Articles C10.5.2.1 and C10.7.2.4.

For shafts socketed into rock, approaches to developing *P-y* response of rock masses include both a weak rock response and a strong rock response. For the strong rock response, the potential for brittle fracture should be considered. If horizontal deflection of the rock mass is greater than 0.0004*b*, a lateral load test to evaluate the response of the rock to lateral loading should be considered. Brown et al. (2010) provides a summary of a methodology that may be used to estimate the lateral load response of shafts in rock. Additional background on lateral loading of shafts in rock is provided in Turner (2006).

the diameter of the rock socket, the potential for brittle fracture of the rock shall be considered.

These methods for estimating the response of shafts in rock subjected to lateral loading use the unconfined compressive strength of the intact rock as the main input property. While this property is meaningful for intact rock, and was the key parameter used to correlate to shaft lateral load response in rock, it is not meaningful for fractured rock masses. If the rock mass is fractured enough to justify characterizing the rock shear strength using the GSI, the rock mass should be characterized as a c-ϕ material, and confining stress (i.e., σ'_3) present within the rock mass should be considered when establishing a rock mass shear strength for lateral response of the shaft. If the P-y method of analysis is used to model horizontal resistance, user-specified P-y curves should be derived. A method for developing hyperbolic P-y curves isdescribed by Liang et al. (2009).

10.8.2.4—Settlement Due to Downdrag

The provisions of Article 10.7.2.5 shall apply.

C10.8.2.4

See Article C10.7.2.5.

10.8.2.5—Lateral Squeeze

The provisions of Article 10.7.2.6 shall apply.

C10.8.2.5

See Article C10.7.2.6.

10.8.3—Strength Limit State Design

10.8.3.1—General

The nominal shaft resistances that shall be considered at the strength limit state include:

- axial compression resistance,
- axial uplift resistance,
- punching of shafts through strong soil into a weaker layer,
- lateral geotechnical resistance of soil and rock stratum,
- resistance when scour occurs,
- axial resistance when downdrag occurs, and
- structural resistance of shafts.

10.8.3.2—Groundwater Table and Buoyancy

The provisions of Article 10.7.3.5 shall apply.

C10.8.3.2

See Article C10.7.3.5.

10.8.3.3—Scour

The provisions of Article 10.7.3.6 shall apply.

C10.8.3.3

See Article C10.7.3.6.

10.8.3.4—Downdrag

The foundation should be designed so that the available factored axial geotechnical resistance is greater than the factored loads applied to the shaft, including the downdrag, at the strength limit state. The nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the

C10.8.3.4

The static analysis procedures in Article 10.8.3.5 may be used to estimate the available drilled shaft nominal side and tip resistances to withstand the downdrag plus other axial force effects.

Nominal resistance may also be estimated using an instrumented static load test provided the side resistance positive skin and tip resistance below the lowest layer contributing to the downdrag. The drilled shaft shall be designed structurally to resist the downdrag plus structure loads.

within the zone contributing to downdrag is subtracted from the resistance determined from the load test.

As stated in Article C10.8.1.6.2, it is appropriate to apply different downdrag forces for evaluation of geotechnical strength limit states than for structural strength limit states. A drilled shaft with its tip bearing in stiff material, such as rock or hard soil, would be expected to limit settlement to very small values. In this case, the full downdrag force could occur in combination with the other axial force effects because downdrag will not be reduced if there is little or no downward movement of the shaft. Therefore, the factored force effects resulting from all load components, including full factored downdrag, should be used to check the structural strength limit state of the drilled shaft.

A rational approach to evaluating this strength limit state will incorporate the force effects occurring at this magnitude of downward displacement. This will include the factored axial force effects transmitted to the head of the shaft plus the downdrag loads occurring at a downward displacement defining the failure criterion. In many cases, this amount of downward displacement will reduce or eliminate downdrag. For soil layers that undergo settlement exceeding the failure criterion (for example, five percent of B for shafts bearing in sand), downdrag loads are likely to remain and should be included. This approach requires the designer to predict the magnitude of downdrag load occurring at a specified downward displacement. This can be accomplished using the hand calculation procedure described in Brown et al. (2010) or with commercially available software.

When downdrag loads are determined to exist at a downward displacement defining failure, evaluation of drilled shafts for the geotechnical strength limit state in compression should be conducted under a load combination that is limited to permanent loads only, including the calculated downdrag load at a settlement defining the failure criterion but excluding nonpermanent loads, such as live load, temperature changes, etc. See Brown et al. (2010) for further discussion.

When analysis of a shaft subjected to downdrag shows that the downdrag load would be eliminated in order to achieve a defined downward displacement, evaluation of geotechnical and structural strength limit states in compression should be conducted under the full-load combination corresponding to the relevant strength limit state, including the non-permanent components of load but not including downdrag.

10.8.3.5—Nominal Axial Compression Resistance of Single Drilled Shafts

The factored resistance of drilled shafts, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad (10.8.3.5\text{-}1)$$

C10.8.3.5

The nominal axial compression resistance of a shaft is derived from the tip resistance and/or shaft side resistance, i.e., skin friction. Both the tip and shaft resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement, as described in Article 10.8.2.2.2.

in which:

$$R_p = q_p A_p \tag{10.8.3.5-2}$$

$$R_S = q_S A_S \tag{10.8.3.5-3}$$

where:

R_p = nominal shaft tip resistance (kips)
R_s = nominal shaft side resistance (kips)
φ_{qp} = resistance factor for tip resistance specified in Table 10.5.5.2.4-1
φ_{qs} = resistance factor for shaft side resistance specified in Table 10.5.5.2.4-1
q_p = unit tip resistance (ksf)
q_s = unit side resistance (ksf)
A_p = area of shaft tip (ft^2)
A_s = area of shaft side surface (ft^2)

For consistency in the interpretation of both static load tests (see Article 10.8.3.5.6) and the normalized curves of Article 10.8.2.2.2, it is customary to establish the failure criterion at the strength limit state at a gross deflection equal to five percent of the base diameter for drilled shafts.

O'Neill and Reese (1999) identify several methods for estimating the resistance of drilled shafts in cohesive and granular soils, intermediate geomaterials, and rock. The most commonly used methods are provided in this Article. Methods other than the ones provided in detail in this Article may be used provided that adequate local or national experience with the specific method is available to have confidence that the method can be used successfully and that appropriate resistance factors can be determined. At present, it must be recognized that these resistance factors have been developed using a combination of calibration by fitting to previous allowable stress design (ASD) practice and reliability theory (see Allen, 2005 for additional details on the development of resistance factors for drilled shafts). Such methods may be used as an alternative to the specific methodology provided in this Article, provided that:

- the method selected consistently has been used with success on a regional or national basis.
- significant experience is available to demonstrate that success.
- as a minimum, calibration by fitting to allowable stress design is conducted to determine the appropriate resistance factor, if inadequate measured data are available to assess the alternative method using reliability theory. A similar approach as described by Allen (2005) should be used to select the resistance factor for the alternative method.

The methods for estimating drilled shaft resistance provided in this Article should be used. Shaft strength limit state resistance methods not specifically addressed in this Article for which adequate successful regional or national experience is available may be used, provided adequate information and experience is also available to develop appropriate resistance factors.

10.8.3.5.1—Estimation of Drilled Shaft Resistance in Cohesive Soils

10.8.3.5.1a—General

Drilled shafts in cohesive soils should be designed by total and effective stress methods for undrained and drained loading conditions, respectively.

10.8.3.5.1b—Side Resistance

The nominal unit side resistance, q_S, in ksf, for shafts in cohesive soil loaded under undrained loading conditions by the α-Method shall be taken as:

$$q_s = \alpha S_u \tag{10.8.3.5.1b-1}$$

in which:

$$\alpha = 0.55 \quad \text{for} \quad \frac{S_u}{p_a} \leq 1.5 \tag{10.8.3.5.1b-2}$$

C10.8.3.5.1b

The α-method is based on total stress. For effective stress methods for shafts in clay, see Brown et al. (2010).

The adhesion factor is an empirical factor used to correlate the results of full-scale load tests with the material property or characteristic of the cohesive soil. The adhesion factor is usually related to S_u and is derived from the results of full-scale pile and drilled shaft, load tests. Use of this approach presumes that the measured value of S_u is correct and that all shaft behavior resulting from construction and loading can be lumped into a single

$$\alpha = 0.55 - 0.1\left(S_u / p_a - 1.5\right)$$

for $1.5 \le S_u / p_a \le 2.5$ (10.8.3.5.1b-3)

where:

S_u = undrained shear strength (ksf)
α = adhesion factor (dim)
p_a = atmospheric pressure (= 2.12 ksf)

The following portions of a drilled shaft, illustrated in Figure 10.8.3.5.1b-1, should not be taken to contribute to the development of resistance through skin friction:

- at least the top 5.0 ft of any shaft; and
- periphery of belled ends, if used.

When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Values of α for contributing portions of shafts excavated dry in open or cased holes should be as specified in Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3.

parameter. Neither presumption is strictly correct, but the approach is used due to its simplicity.

Steel casing will generally reduce the side resistance of a shaft. No specific data for drilled shafts in clay are available regarding the reduction in skin friction resulting from the use of permanent casing relative to concrete placed directly against the soil when the casing is vibrated or rotated into the soil. Interface shear resistance for steel against clay can vary from 50 to 80 percent of the interface shear resistance for poured in place concrete against clay, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). A reduction factor applied to the calculated shaft side resistance of 0.6 to 0.75 is commonly used to account for the presence of permanent steel casing. Greater reduction in the side resistance may be necessary when installation methods result in reduced contact between the ground and the casing.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in Article 10.7.3.8.6.

The upper 5.0 ft of the shaft is ignored in estimating R_n, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

Bells or underreams constructed in stiff fissured clay often settle sufficiently to result in the formation of a gap above the bell that will eventually be filled by slumping soil. Slumping will tend to loosen the soil immediately above the bell and decrease the side resistance along the lower portion of the shaft.

The value of α is often considered to vary as a function of S_u. Values of α for drilled shafts are recommended as shown in Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3, based on the results of back-analyzed, full-scale load tests. The load tests were conducted in insensitive cohesive soils. Therefore, if shafts are constructed in sensitive clays, values of α may be different than those obtained from Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3. Other values of α may be used if based on the results of load tests.

The depth of 5.0 ft at the top of the shaft may need to be increased if the drilled shaft is installed in expansive clay, if scour deeper than 5.0 ft is anticipated, if there is substantial groundline deflection from lateral loading, or if there are other long-term loads or construction factors that could affect shaft resistance.

The values of α obtained from Eqs. 10.8.3.5.1b-2 and 10.8.3.5.1b-3 are considered applicable for both compression and uplift loading.

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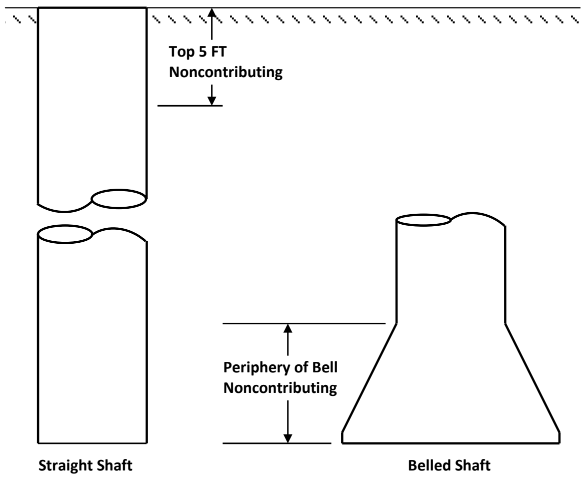

Figure 10.8.3.5.1b-1—Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (Brown et al., 2010)

10.8.3.5.1c—Tip Resistance

For axially loaded shafts in cohesive soil, the nominal unit tip resistance, q_p, by the total stress method as provided in Brown et al. (2010) shall be taken as:

$$q_p = N_c S_u \leq 80.0 \text{ ksf} \qquad (10.8.3.5.1c\text{-}1)$$

in which:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \qquad (10.8.3.5.1c\text{-}2)$$

where:

D = diameter of drilled shaft (ft)
Z = penetration of shaft (ft)
S_u = undrained shear strength (ksf)

The value of S_u should be determined from the results of in-situ and/or laboratory testing of undisturbed samples obtained within a depth of 2.0 diameters below the tip of the shaft. If the soil within 2.0 diameters of the tip has S_u <0.50 ksf, the value of N_c should be multiplied by 0.67.

C10.8.3.5.1c

These equations are for total stress analysis. For effective stress methods for shafts in clay, see Brown et al. (2010).

The limiting value of 80.0 ksf for q_p is not a theoretical limit but a limit based on the largest measured values. A higher limiting value may be used if based on the results of a load test, or previous successful experience in similar soils.

10.8.3.5.2—Estimation of Drilled Shaft Resistance in Cohesionless Soils

10.8.3.5.2a—General

Shafts in cohesionless soils should be designed by effective stress methods for drained loading conditions or by empirical methods based on in-situ test results.

C10.8.3.5.2a

The factored resistance should be determined in consideration of available experience with similar conditions.

The shear strength of cohesionless soils can be characterized by an angle of internal friction, ϕ_f, or empirically related to its *SPT* blow count, *N*. Methods of estimating shaft resistance and end bearing are presented below. Judgment and experience should always be considered.

10.8.3.5.2b—Side Resistance

The side resistance for shafts in cohesionless soils shall be determined using the β method, take as:

$$q_s = \beta\, \sigma'_v \qquad (10.8.3.5.2b\text{-}1)$$

in which:

$$\beta = \left(1 - \sin \varphi'_f\right)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin \varphi'_f} \tan \varphi'_f \qquad (10.8.3.5.2b\text{-}2)$$

where:

β = load transfer coefficient (dim)
σ'_f = friction angle of cohesionless soil layer (°)
σ'_p = effective vertical preconsolidation stress
σ'_v = vertical effective stress at soil layer mid-depth

The correlation for effective soil friction angle for use in the above equations shall be taken as:

$$\varphi'_f = 27.5 + 9.2 \log\left[\left(N_1\right)_{60}\right] \qquad (10.8.3.5.2b\text{-}3)$$

where:

$(N_1)_{60}$ = *SPT N*-value corrected for effective overburden stress

The preconsolidation stress in Eq. 10.8.3.5.2b-2 should be approximated through correlation to *SPT N*-values. For sands:

$$\frac{\sigma'_p}{p_a} = 0.47\left(N_{60}\right)^m \qquad (10.8.3.5.2b\text{-}4)$$

where:

m = 0.6 for clean quartzitic sands
m = 0.8 for silty sand to sandy silts
p_a = atmospheric pressure (same units as σ'_p : 2.12 ksf or 14.7 psi)

For gravelly soils:

$$\frac{\sigma'_p}{p_a} = 0.15\left(N_{60}\right) \qquad (10.8.3.5.2b\text{-}5)$$

C10.8.3.5.2b

The method described herein is based on axial load tests on drilled shafts as presented by Chen and Kulhawy (2002) and updated by Kulhawy and Chen (2007). This method provides a rational approach for relating unit side resistance to N-values and to the state of effective stress acting at the soil-shaft interface. This approach replaces the previously used depth-dependent β-method developed by O'Neill and Reese (1999), which does not account for variations in *N*-value or effective stress on the calculated value of β. Further discussion, including the detailed development of Eq. 10.8.3.5.2b-2, is provided in (Brown et al., 2010).

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When permanent casing is used, the side resistance shall be adjusted with consideration to the type and length of casing to be used, and how it is installed.

Steel casing will generally reduce the side resistance of a shaft. No specific data for drilled shafts in cohesionless soil are available regarding the reduction in skin friction resulting from the use of permanent casing relative to concrete placed directly against the soil. Interface shear resistance for steel against cohesionless soil can vary from 50 to 80 percent of the interface shear resistance for poured in place concrete against cohesionless soil, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961). Note that unit side resistance for poured in place concrete against cohesionless soil is nearly equal to the soil shear strength in most cases. A reduction factor applied to the calculated shaft side resistance of 0.6 to 0.75 is commonly used to account for the presence of permanent steel casing. Greater reduction in the side resistance may be necessary when installation methods result in reduced contact between the ground and the casing.

If open-ended pipe piles are driven full depth with an impact hammer before soil inside the pile is removed, and left as a permanent casing, driven pile static analysis methods may be used to estimate the side resistance as described in Article 10.7.3.8.6.

10.8.3.5.2c—Tip Resistance

The nominal tip resistance, q_p, in ksf, for drilled shafts in cohesionless soils by the method described in Brown et al. (2010) shall be taken as:

$$\text{If } N_{60} \leq 50, \text{ then } q_p = 1.2N_{60} \qquad (10.8.3.5.2c\text{-}1)$$

where:

N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

The value of q_p in Eq. 10.8.3.5.2c-1 should be limited to 60 ksf, unless greater values can be justified though load test data.

C10.8.3.5.2c

Brown et al. (2010) provide additional discussion regarding the computation of nominal tip resistance and on tip resistance in specific geologic environments.

10.8.3.5.3—Shafts in Strong Soil Overlying Weaker Compressible Soil

Where a shaft is tipped in a strong soil layer overlying a weaker layer, the base resistance shall be reduced if the shaft base is within a distance of 1.5B of the top of the weaker layer. A weighted average should be used that varies linearly from the full base resistance in the overlying strong layer at a distance of 1.5B above the top of the weaker layer to the base resistance of the weaker layer at the top of the weaker layer.

C10.8.3.5.3

The distance of 1.5B represents the zone of influence for general bearing capacity failure based on bearing capacity theory for deep foundations.

10.8.3.5.4—Estimation of Drilled Shaft Resistance in Rock

10.8.3.5.4a—General

Drilled shafts in rock subject to compressive loading shall be designed to support factored loads in:

- side-wall shear comprising skin friction on the wall of the rock socket; or
- end bearing on the material below the tip of the drilled shaft; or
- a combination of both.

The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in the design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value.

C10.8.3.5.4a

Methods presented in this Article to calculate drilled shaft axial resistance require an estimate of the uniaxial compressive strength of rock core. Unless the rock is massive, the strength of the rock mass is most frequently controlled by the discontinuities, including orientation, length, and roughness, and the behavior of the material that may be present within the discontinuity, e.g., gouge or infilling. The methods presented are semi-empirical and are based on load test data and site-specific correlations between measured resistance and rock core strength.

Design based on side-wall shear alone should be considered for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large movements of the shaft would be required to mobilize resistance in end bearing.

Design based on end-bearing alone should be considered where sound bedrock underlies low strength overburden materials, including highly weathered rock. In these cases, however, it may still be necessary to socket the shaft into rock to provide lateral stability.

Where the shaft is drilled some depth into sound rock, a combination of sidewall shear and end bearing can be assumed (Kulhawy and Goodman, 1980).

If the rock is degradable, use of special construction procedures, larger socket dimensions, or reduced socket resistance should be considered.

Factors that should be considered when making an engineering judgment to neglect any component of resistance (side or base) are discussed in Article 10.8.3.5.4d. In most cases, both side and base resistances should be included in limit state evaluation of rock-socketed shafts.

For drilled shafts installed in karstic formations, exploratory borings should be advanced at each drilled shaft location to identify potential cavities. Layers of compressible weak rock along the length of a rock socket and within approximately three socket diameters or more below the base of a drilled shaft may reduce the resistance of the shaft.

For rock that is stronger than concrete, the concrete shear strength will control the available side friction, and the strong rock will have a higher stiffness, allowing significant end bearing to be mobilized before the side wall shear strength reaches its peak value. Note that concrete typically reaches its peak shear strength at about 250 to 400 microstrain (for a 10-ft long rock socket, this is approximately 0.5 in. of deformation at the top of the rock socket). If strains or deformations greater than the value at the peak shear stress are anticipated to mobilize the desired end bearing in the rock, a residual value for the skin friction can still be used. Article 10.8.3.5.4d provides procedures for computing a residual value

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of the skin friction based on the properties of the rock and shaft.

10.8.3.5.4b—Side Resistance

For drilled shafts socketed into rock, unit side resistance, q_s in ksf, shall be taken as (Kulhawy et al., 2005):

$$\frac{q_S}{p_a} = C\sqrt{\frac{q_u}{p_a}} \qquad (10.8.3.5.4b\text{-}1)$$

where:

p_a = atmospheric pressure taken as 2.12 ksf
C = regression coefficient taken as 1.0 for normal conditions
q_u = uniaxial compressive strength of rock (ksf)

If the uniaxial compressive strength of rock forming the sidewall of the socket exceeds the drilled shaft concrete compressive strength, the value of concrete compressive strength (f'_c) shall be substituted for q_u in Eq. 10.8.3.5.4b-1.

For fractured rock that caves and cannot be drilled without some type of artificial support, the unit side resistance shall be taken as:

$$\frac{q_S}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}} \qquad (10.8.3.5.4b\text{-}2)$$

The joint modification factor, α_E is given in Table 10.8.3.5.4b-1 based on RQD and visual inspection of joint surfaces.

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

RDQ (%)	Joint Modification Factor, α_E	
	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

C10.8.3.5.4b

Eq. 10.8.3.5.4b-1 is based on regression analysis of load test data as reported by Kulhawy et al. (2005) and includes data from pervious studies by Horvath and Kenney (1979), Rowe and Armitage (1987), Kulhawy and Phoon (1993), and others. The recommended value of the regression coefficient C = 1.0 is applicable to normal rock sockets, defined as sockets constructed with conventional equipment and resulting in nominally clean sidewalls without resorting to special procedures or artificial roughening. Rock that is prone to smearing or rapid deterioration upon exposure to atmospheric conditions, water, or slurry are outside the normal range and may require additional measures to insure reliable side resistance. Rocks exhibiting this type of behavior include clay shales and other argillaceous rocks. Rock that cannot support construction of an unsupported socket without caving is also outside the normal and will likely exhibit lower side resistance than given by Eq. 10.8.3.5.4b-1 with C = 1.0. For additional guidance on assessing the magnitude of C, see Brown et al. (2010).

Shafts are sometimes constructed by supporting the hole with temporary casing or by grouting the rock ahead of the excavation. When using these construction methods, disturbance of the sidewall results in lower unit side resistances. Based on O'Neill and Reese (1999) and as discussed in Brown et al. (2010), the reduction in side resistance can be related empirically to the RQD and joint conditions.

10.8.3.5.4c—Tip Resistance

End-bearing for drilled shafts in rock may be taken as follows:

- If the rock below the base of the drilled shaft to a depth of 2.0B is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5B:

$$q_p = 2.5 q_u \tag{10.8.3.5.4c-1}$$

- If the rock below the base of the shaft to a depth of 2.0B is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

$$q_p = A + q_u \left[m_b \left(\frac{A}{q_u} \right) + s \right]^a \tag{10.8.3.5.4c-2}$$

In which:

$$A = \sigma'_{vb} + q_u \left[m_b \frac{\left(\sigma'_{v,b}\right)}{q_u} + s \right]^a \tag{10.8.3.5.4c-3}$$

where:

σ'_{vb} = vertical effective stress at the socket bearing elevation (tip elevation)

s, a, and m_b = Hoek–Brown strength parameters for the fractured rock mass determined from GSI (see Article 10.4.6.4)

q_u = uniaxial compressive strength of intact rock

Eq. 10.8.3.5.4c-1 should be used as an upper-bound limit to base resistance calculated by Eq. 10.8.2.5.4c-2, unless local experience or load tests can be used to validate higher values.

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of Eq. 10.8.3.5.4c-1 also requires that there are no solution cavities or voids below the base of the drilled shaft.

For further information, see Brown et al. (2010).

Bearing capacity theory provides a framework for evaluation of base resistance for cases where the bearing rock can be characterized by its GSI. Eq. 10.8.3.5.4c-2 (Turner and Ramey, 2010) is a lower bound solution for bearing resistance of a drilled shaft bearing on or socketed into a fractured rock mass. Fractured rock describes a rock mass intersected by multiple sets of intersecting joints such that the strength is controlled by the overall mass response and not by failure along pre-existing structural discontinuities. This generally applies to rock that can be characterized by the descriptive terms shown in Figure 10.4.6.4-1 (e.g., blocky, disintegrated, etc.).

10.8.3.5.4d—Combined Side and Tip Resistance

Design methods that consider the difference in shaft movement required to mobilize skin friction in rock versus what is required to mobilize end bearing shall be used to estimate axial compressive resistance of shafts embedded in rock.

C10.8.3.5.4d

A design decision to be addressed when using rock sockets is whether to neglect one or the other component of resistance (side or base). For example, design based on side resistance alone is sometimes assumed for cases in which the base of the drilled hole cannot be cleaned and inspected or where it is determined that large downward movement of the shaft would be required to mobilize tip resistance. However, before making a decision to omit tip resistance, careful consideration should be given to applying available methods of quality construction and inspection that can provide confidence in tip resistance. Quality construction practices can result in adequate clean-out at the base of rock sockets, including those constructed by wet methods. In many cases, the cost of quality control and assurance is offset by the economies

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achieved in socket design by including tip resistance. Load testing provides a means to verify tip resistance in rock.

Reasons cited for neglecting side resistance of rock sockets include:

(1) the possibility of strain-softening behavior of the sidewall interface,
(2) the possibility of degradation of material at the borehole wall in argillaceous rocks, and
(3) uncertainty regarding the roughness of the sidewall.

Brittle behavior along the sidewall, in which side resistance exhibits a significant decrease beyond its peak value, is not commonly observed in load tests on rock sockets. If there is reason to believe strain softening will occur, laboratory direct shear tests of the rock-concrete interface can be used to evaluate the load-deformation behavior and account for it in design. These cases would also be strong candidates for conducting field load tests. Investigating the sidewall shear behavior through laboratory or field testing is generally more cost-effective than neglecting side resistance in the design. Application of quality control and assurance through inspection is also necessary to confirm that sidewall conditions in production shafts are of the same quality as laboratory or field test conditions.

Materials that are prone to degradation at the exposed surface of the borehole and are prone to a “smooth” sidewall generally are argillaceous sedimentary rocks such as shale, claystone, and siltstone. Degradation occurs due to expansion, opening of cracks and fissures combined with groundwater seepage, and by exposure to air and/or water used for drilling. Hassan and O’Neill (1997) note that this behavior is most prevalent in cohesive IGMs and that in the most severe cases degradation results in a smear zone at the interface. Smearing may reduce load transfer significantly. As reported by Abu-Hejleh et al. (2003), both smearing and smooth sidewall conditions can be prevented in cohesive IGMs by using roughening tools during the final pass with the rock auger or by grooving tools. Careful inspection prior to concrete placement is required to confirm roughness of the sidewalls. Only when these measures cannot be confirmed would there be cause for neglecting side resistance in design.

Analytical tools for evaluating the load transfer behavior of rock socketed shafts are given in Turner (2006) and Brown et al. (2010).

10.8.3.5.5—Estimation of Drilled Shaft Resistance in Intermediate Geo Materials (IGMs)

For detailed base and side resistance estimation procedures for shafts in cohesive IGMs, the procedures provided by Brown et al. (2010) should be used.

C10.8.3.5.5

See Article 10.8.2.2.3 for a definition of an IGM.

10.8.3.5.6—Shaft Load Test

When used, load tests shall be conducted in representative soil conditions using shafts constructed in a manner and of dimensions and materials similar to those planned for the production shafts. The load test shall follow the procedures specified in ASTM D1143. The loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data is needed, in which case the standard loading procedure should be used.

The nominal resistance shall be determined according to the failure definition of either:

- "plunging" of the drilled shaft, or
- a gross settlement or uplift of five percent of the diameter of the shaft if plunging does not occur.

The resistance factors for axial compressive resistance or axial uplift resistance shall be taken as specified in Table 10.5.5.2.4-1.

Regarding the use of shaft load test data to determine shaft resistance, the load test results should be applied to production shafts that are not load tested by matching the static resistance prediction to the load test results. The calibrated static analysis method should then be applied to adjacent locations within the site to determine the shaft tip elevation required, in consideration of variations in the geologic stratigraphy and design properties at each production shaft location. The definition of a site and number of load tests required to account for site variability shall be as specified in Article 10.5.5.2.3.

C10.8.3.5.6

For a larger project where many shafts are to be used, it may be cost-effective to perform a full-scale load test on a drilled shaft during the design phase of a project to confirm response to load.

Load tests should be conducted following prescribed written procedures that have been developed from accepted standards and modified, as appropriate, for the conditions at the site. The Quick Test Procedure is desirable because it avoids problems that frequently arise when performing a static test that cannot be started and completed within an eight-hour period. Tests that extend over a longer period are difficult to perform due to the limited number of experienced personnel that are usually available. The Quick Test has proven to be easily performed in the field, and the results usually are satisfactory. However, if the formation in which the shaft is installed may be subject to significant creep settlement, alternative procedures provided in ASTM D1143 should be considered.

Load tests are conducted on full-scale drilled shaft foundations to provide data regarding nominal axial resistance, load-displacement response, and shaft performance under the design loads, and to permit assessment of the validity of the design assumptions for the soil conditions at the test shaft(s).

Tests can be conducted for compression, uplift, lateral loading, or for combinations of loading. Full-scale load tests in the field provide data that include the effects of soil, rock, and groundwater conditions at the site; the dimensions of the shaft; and the procedures used to construct the shaft.

The results of full-scale load tests can differ even for apparently similar ground conditions. Therefore, care should be exercised in generalizing and extrapolating the test results to other locations.

For large diameter shafts, where conventional reaction frames become unmanageably large, load testing using Osterberg load cells may be considered. Additional discussion regarding load tests is provided in O'Neill and Reese (1999). Alternatively, smaller diameter shafts may be load tested to represent the larger diameter shafts (but no less than one-half the full scale production shaft diameter), provided that appropriate measures are taken to account for potential scale effects when extrapolating the results to the full scale production shafts.

Plunging occurs when a steady increase in movement results from incrementally small increases in load, e.g., 2.0 kips.

10.8.3.6—Shaft Group Resistance

10.8.3.6.1—General

Reduction in resistance from group effects shall be evaluated.

C10.8.3.6.1

If casing is advanced in front of the excavation heading, this reduction need not be made.

10.8.3.6.2—Cohesive Soil

The provisions of Article 10.7.3.9 shall apply.

The resistance factor for the group resistance of an equivalent pier or block failure provided in Table 10.5.5.2.4-1 shall apply where the cap is, or is not, in contact with the ground.

The resistance factors for the group resistance calculated using the sum of the individual drilled shaft resistances are the same as those for the single-drilled shaft resistances.

C10.8.3.6.2

The efficiency of groups of drilled shafts in cohesive soil may be less than that of the individual shaft due to the overlapping zones of shear deformation in the soil surrounding the shafts.

10.8.3.6.3—Cohesionless Soil

The individual nominal resistance of each shaft in a group should be reduced by applying an adjustment factor η taken as shown in Table 10.8.3.6.3-1.

For intermediate spacings, the value of η may be determined by linear interpolation.

C10.8.3.6.3

The bearing resistance of drilled shaft groups in sand is less than the sum of the individual shafts due to overlap of shear zones in the soil between adjacent shafts and loosening of the soil during construction. The recommended reduction factors are based in part on theoretical considerations and on limited load test results. See O'Neill and Reese (1999) for additional details and a summary of group load test results. It should be noted that most of the available group load test results were obtained for sands above the water table and for relatively small groups, e.g., groups of 3–9 shafts. For larger shaft groups or for shaft groups of any size below the water table, more conservative values of η should be considered.

These reduction factors presume that good shaft installation practices are used to minimize or eliminate the relaxation of the soil between shafts and caving. If this cannot be adequately controlled due to difficult soil conditions or for other reasons, lower group reduction factors should be considered, or steps should be taken during and after shaft construction to restore the soil to its original condition.

Table 10.8.3.6.3-1—Group Reduction Factors for Bearing Resistance of Shafts in Sand

Shaft Group Configuration	Shaft Center-to-Center Spacing	Special Conditions	Reduction Factor for Group Effects, η
Single Row	2*D*		0.90
	3*D* or more		1.0
Multiple Row	2.5*D*		0.67
	3*D*		0.80
	4*D* or more		1.0
Single and Multiple Rows	2*D* or more	Shaft group cap in intimate contact with ground consisting of medium dense or denser soil, and no scour below the shaft cap is anticipated	1.0
Single and Multiple Rows	2*D* or more	Pressure grouting is used along the shaft sides to restore lateral stress losses caused by shaft installation, and the shaft tip is pressure grouted	1.0

10.8.3.6.4—Shaft Groups in Strong Soil Overlying Weak Soil

For shaft groups that are collectively tipped within a strong soil layer overlying a soft, cohesive layer, block bearing resistance shall be evaluated in accordance with Article 10.7.3.9.

10.8.3.7—Uplift Resistance

10.8.3.7.1—General

Uplift resistance shall be evaluated when upward loads act on the drilled shafts. Drilled shafts subjected to uplift forces shall be investigated for resistance to pullout, for their structural strength, and for the strength of their connection to supported components.

10.8.3.7.2—Uplift Resistance of Single Drilled Shaft

The uplift resistance of a single straight-sided drilled shaft should be estimated in a manner similar to that for determining side resistance for drilled shafts in compression, as specified in Article 10.8.3.5.

In determining the uplift resistance of a belled shaft, the side resistance above the bell should conservatively be neglected if the resistance of the bell is considered, and it can be assumed that the bell behaves as an anchor.

The factored nominal uplift resistance of a belled drilled shaft in a cohesive soil, R_R, in kips, should be determined as:

$$R_R = \varphi R_n = \varphi_{up} R_{sbell} \tag{10.8.3.7.2-1}$$

in which:

$$R_{s\ bell} = q_{s\ bell} A_u \tag{10.8.3.7.2-2}$$

where:

q_{sbell} = $N_u S_u$ (ksf)
A_u = $\pi(D_p^2 - D^2)/4$ (ft^2)
N_u = uplift adhesion factor (dim)
D_p = diameter of the bell (ft)
D_b = depth of embedment in the founding layer (ft)
D = shaft diameter (ft)
S_u = undrained shear strength averaged over a distance of 2.0 bell diameters ($2D_p$) above the base (ksf)
φ_{up} = resistance factor specified in Table 10.5.5.2.4-1

If the soil above the founding stratum is expansive, S_u should be averaged over the lesser of either $2.0D_p$ above the bottom of the base or over the depth of penetration of the drilled shaft in the founding stratum.

C10.8.3.7.2

The resistance factors for uplift are lower than those for axial compression. One reason for this is that drilled shafts in tension unload the soil, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft. Empirical justification for uplift resistance factors is provided in Article C10.5.5.2.3 and in Allen (2005).

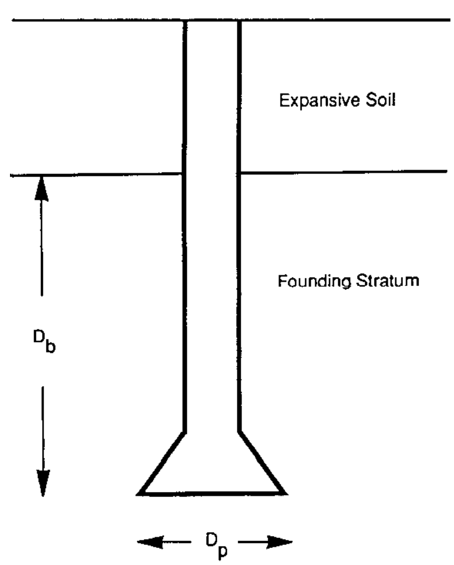

Figure C10.8.3.7.2-1—Uplift of a Belled Drilled Shaft

The value of N_u may be assumed to vary linearly from 0.0 at $D_b/D_p = 0.75$ to a value of 8.0 at $D_b/D_p = 2.5$, where D_b is the depth below the founding stratum. The top of the founding stratum should be taken at the base of zone of seasonal moisture change.

The assumed variation of N_u is based on Yazdanbod et al. (1987).

This method does not include the uplift resistance contribution due to soil suction and the weight of the shaft.

10.8.3.7.3—Group Uplift Resistance

The provisions of Article 10.7.3.11 shall apply.

10.8.3.7.4—Uplift Load Test

The provisions of Article 10.7.3.10 shall apply.

C10.8.3.7.4

See Article C10.7.3.10.

10.8.3.8—Nominal Horizontal Resistance of Shaft and Shaft Groups

The provisions of Article 10.7.3.12 apply.

The design of horizontally loaded drilled shafts shall account for the effects of interaction between the shaft and ground, including the number of shafts in the group.

For shafts used in groups, the drilled shaft head shall be fixed into the cap.

C10.8.3.8

See Article C10.7.3.12.

10.8.3.9—Shaft Structural Resistance

10.8.3.9.1—General

The structural design of drilled shafts shall be in accordance with the provisions of Section 5 for the design of reinforced concrete.

10.8.3.9.2—Buckling and Lateral Stability

The provisions of Article 10.7.3.13.4 shall apply.

C10.8.3.9.2

See Article C10.7.3.13.4.

10.8.3.9.3—Reinforcement

Where the potential for lateral loading is insignificant, drilled shafts may be reinforced for axial loads only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced concrete columns in accordance with Article 5.6.4. Reinforcing steel shall extend a minimum of 10.0 ft below the plane where the soil provides fixity.

Where the potential for lateral loading is significant, the unsupported portion of the shaft shall be designed in accordance with Article 5.11.

The minimum spacing between longitudinal bars, as well as between transverse bars or spirals, shall be sufficient to allow free passage of the concrete through the cage and into the annulus between the cage and the borehole wall.

C10.8.3.9.3

Shafts constructed using generally accepted procedures are not normally stressed to levels such that the allowable concrete stress is exceeded. Exceptions include:

- Shafts with sockets in hard rock,
- Shafts subjected to lateral loads,
- Shafts subjected to uplift loads from expansive soils or direct application of uplift loads, and
- Shafts with unreinforced bells.

Maintenance of the spacing of reinforcement and the maximum aggregate size requirements are important to ensure that the high-slump concrete mixes normally used for drilled shafts can flow readily between the steel bars during concrete placement. See Article 5.12.9.5.2 for specifications regarding the minimum clear spacing required between reinforcing cage bars.

A shaft can be considered laterally supported:

- below the zone of liquefaction or seismic loads,

- in rock, or
- 5.0 ft below the ground surface or the lowest anticipated scour elevation.

Laterally supported does not mean fixed. Fixity would occur somewhat below this location and depends on the stiffness of the supporting soil.

The out-to-out dimension of the assembled reinforcing cage should be sufficiently smaller than the diameter of the drilled hole to ensure free flow of concrete around the reinforcing as the concrete is placed. See Article 5.12.9.

The minimum requirements to consider the steel shell to be load carrying shall be as specified in Article 5.12.9.5.2.

See commentary to Article 10.7.5 regarding assessment of corrosivity. In addition, consideration should be given to the ability of the concrete and steel shell to bond together.

10.8.3.9.4—Transverse Reinforcement

Transverse reinforcement may be constructed as hoops of spiral steel.

Seismic provisions shall be in accordance with Article 5.11.

10.8.3.9.5—Concrete

The maximum aggregate size, slump, wet or dry placement, and necessary design strength should be considered when specifying shaft concrete. The concrete selected should be capable of being placed and adequately consolidated forthe anticipated construction condition, and shaft details should be specified. The maximum size aggregate shall meet the requirements of Article 10.8.3.9.3.

C10.8.3.9.5

When concrete is placed in shafts, vibration is often not possible except for the uppermost cross-section. Vibration should not be used for high slump concrete.

10.8.3.9.6—Reinforcement into Superstructure

Sufficient reinforcement shall be provided at the junction of the shaft with the shaft cap or column to make a suitable connection. The embedment of the reinforcement into the cap shall comply with the provision for cast-in-place piles in Section 5.

10.8.3.9.7—Enlarged Bases

Enlarged bases shall be designed to ensure that the plain concrete is not overstressed. The enlarged base shall slope at a side angle not greater than 30 degrees from the vertical and have a bottom diameter not greater than three times the diameter of the shaft. The thickness of the bottom edge of the enlarged base shall not be less than 6.0 in.

10.8.4—Extreme Event Limit State

The provisions of Article 10.5.5.3 and 10.7.4 shall apply.

C10.8.4

See Articles C10.5.5.3 and C10.7.4.

10.9—MICROPILES

10.9.1—General

The provisions of Article 10.7.1 shall apply, except as noted herein.

Micropiles shall be classified by type based on their method of installation as follows:

- Type A micropiles are constructed by placing a sand-cement mortar or neat cement grout in the pile under a gravity head only;
- Type B micropiles are constructed by injecting a neat cement grout under pressure (typically 6–21 ksf) into the drilled hole while the temporary drill casing or auger is withdrawn;
- Type C micropiles are grouted as for Type A, followed 15–25 min after primary grouting by injection of additional grout under pressure (typically greater than 21 ksf) via a preplaced sleeved grout pipe.
- Type D micropiles are grouted similar to Type C, but the primary grout is allowed to harden before injecting the secondary grout under pressure (typically 42–170 ksf) with a packer to achieve treatment of specific pile intervals or material horizons; or
- Type E micropiles are constructed by drilling with grout injection through a continuous-thread, hollow-core steel bar. The grout injection serves to flush cuttings, achieve grout penetration into the ground and stabilize the drill hole. Often the initial grout has a high water to cement ratio and is then replaced with a thicker structural grout near the completion of drilling.

Primary grout, where it provides direct load transfer along the micropile to the surrounding ground, shall be Portland cement-based grout injected into the micropile hole before or after reinforcement installation.

Post grouting, also known as regrouting or secondary grouting, shall be taken as the injection of additional Portland cement grout into the bond length of the micropile after set up of primary grout to enhance the grout–ground bond.

C10.9.1

Micropiles should be considered:

- Where footings cannot be founded on rock, stiff cohesive, or granular foundation material at a reasonable expense;
- At locations where soil conditions would normally permit the use of spread footings, but the potential for erosion exists;
- At locations where pile foundations must penetrate rock;
- At locations where difficult subsurface conditions (e.g., cobbles, boulders, debris fill, running sands) would hinder installation of driven piles or drilled shafts;
- At locations where difficult access or limited headroom preclude use of other deep foundation systems;
- At locations where foundations must bridge over or penetrate subsurface voids;
- Where vibration limits preclude conventional pile driving operations or access by drilled shaft rigs; or
- When underpinning or retrofitting existing foundations.

A typical detail for a composite reinforced micropile is illustrated in Figure C10.9.1-1.

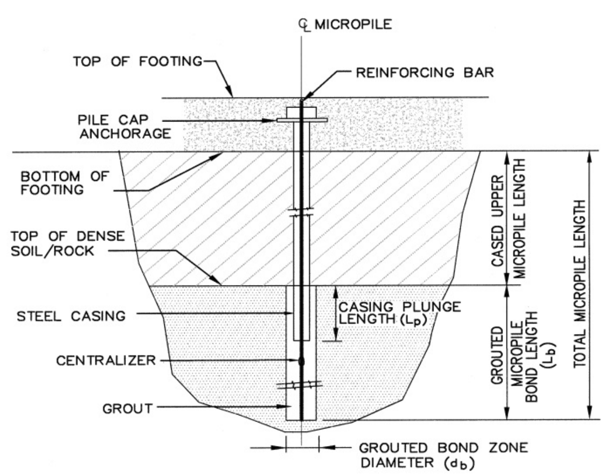

Figure C10.9.1-1—Typical Detail of Composite Reinforced Micropile (after Sabatini et al., 2005)

10.9.1.1—Scope

The provisions of this Article shall apply to the design of micropiles.

The provisions of this section shall not be taken as applicable to drilled piles, e.g., augercast piles, installed with continuous flight augers that are concreted as the auger is being extracted.

10.9.1.2—Minimum Micropile Spacing, Clearance, and Embedment into Cap

Center-to-center pile spacing should not be less than 30.0 in. or 3.0 pile diameters, whichever is greater. Otherwise, the provisions of Article 10.7.1.2 shall apply.

10.9.1.3—Micropiles through Embankment Fill

Micropiles extending through embankments shall penetrate a minimum of 10.0 ft into original ground, unless the required nominal axial and lateral resistance occurs at a lesser penetration below the embankment within bedrock or other suitable support materials.

C10.9.1.3

If compressible soils are located beneath the embankment, micropiles should be installed after embankment settlement is complete, if possible, to minimize or eliminate downdrag forces.

10.9.1.4—Battered Micropiles

The provisions of Article 10.7.1.4 shall apply.

C10.9.1.4

See Article C10.7.1.4.

10.9.1.5—Micropile Design Requirements

Micropile design shall address the following issues as appropriate:

- Nominal axial resistance to be specified in the contract and size of micropile group required to provide adequate support, with consideration of how nominal axial micropile resistance will be determined in the field;
- Group interaction;
- Pile quantity estimation from estimated pile penetration required to meet nominal axial resistance and other design requirements;
- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, liquefaction, lateral loads, and seismic conditions;
- Foundation deflection to meet the established movement and associated structure performance criteria;
- Pile foundation nominal structural resistance; and
- Long-term durability of the micropile in service, i.e., corrosion and deterioration.

C10.9.1.5

The micropile design process is discussed in detail in *Micropile Design and Construction* (Sabatini et al., 2005).

10.9.1.6—Determination of Micropile Loads

The provisions of Article 10.7.1.6 shall apply.

C10.9.1.6

See Article C10.7.1.6.

10.9.1.6.1—Downdrag

The provisions of Articles 10.7.1.6.2 and 3.11.8 shall apply.

C10.9.1.6.1

See Articles C10.7.1.6.2 and C3.11.8.

10.9.1.6.2—Uplift Due to Expansive Soils

The provisions in Article 10.7.1.6.3 shall apply.

C10.9.1.6.2

See Article C10.7.1.6.3.

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10.9.1.6.3—Nearby Structures

The provisions of Article 10.7.1.6.4 shall apply.

10.9.2—Service Limit State Design

10.9.2.1—General

The provisions of Article 10.7.2.1 shall apply.

C10.9.2.1

See Article C10.7.2.1.

10.9.2.2—Tolerable Movements

The provisions of Articles 10.5.2.1 and 10.5.2.2 shall apply.

C10.9.2.2

See Articles C10.5.2.1 and C10.5.2.2.

10.9.2.3—Settlement

The provisions of Article 10.7.2.3 shall apply.

C10.9.2.3

See Article C10.7.2.3.

Methods for calculating the settlement of micropiles are discussed in Sabatini et al. (2005).

10.9.2.3.1—Micropile Groups in Cohesive Soil

The provisions of Article 10.7.2.3.1 shall apply.

C10.9.2.3.1

See Article 10.7.2.3.1.

10.9.2.3.2—Micropile Groups in Cohesionless Soil

The provisions of Article 10.7.2.3.2 shall apply.

C10.9.2.3.2

See Article C10.7.2.3.2.

10.9.2.4—Horizontal Micropile Foundation Movement

The provisions of Articles 10.5.2.1 and 10.7.2.4 shall apply.

C10.9.2.4

See Articles C10.5.2.1 and C10.7.2.4.

10.9.2.5—Settlement Due to Downdrag

The provisions of Article 10.7.2.5 shall apply.

C10.9.2.5

See Article C10.7.2.5.

10.9.2.6—Lateral Squeeze

The provisions of Article 10.7.2.6 shall apply.

C10.9.2.6

See Article C10.7.2.6.

10.9.3—Strength Limit State Design

10.9.3.1—General

For strength limit state design, the following shall be determined:

- loads and performance requirements;
- micropile dimensions and nominal axial micropile resistance;
- size and configuration of the micropile group to provide adequate foundation support;
- estimated micropile length to be used in the construction contract documents to provide a basis for bidding;

C10.9.3.1

- a minimum micropile penetration, if required, for the particular site conditions and loading, determined based on the maximum (deepest) penetration needed to meet all of the applicable requirements identified in Article 10.7.6; and
- the nominal structural resistance of the micropile and/or micropile group.

A minimum micropile penetration should only be specified if needed to insure that uplift, lateral stability, depth to resist downdrag, depth to resist scour, and depth for structural lateral resistance are met for the strength limit state, in addition to similar requirements for the service and extreme event limit states. See Article C10.7.6 for additional details.

Punching of micropiles through strong soil into a weaker layer is not likely for micropiles designed for a resistance by bond transfer only.

10.9.3.2—Ground Water Table and Bouyancy

The provisions of Article 10.7.3.4 shall apply.

C10.9.3.2

See Article C10.7.3.4.

10.9.3.3—Scour

The provisions of Article 10.7.3.5 shall apply.

C10.9.3.3

See Article C10.7.3.5.

10.9.3.4—Downdrag

The provisions of Article 10.7.3.6 shall apply.

C10.9.3.4

See Article C10.7.3.6.

10.9.3.5—Nominal Axial Compression Resistance of a Single Micropile

10.9.3.5.1—General

Micropiles shall be designed to resist failure of the bonded length in soil and rock, or for micropiles bearing on rock, failure of the rock at the micropile tip.

The factored resistance of a micropile, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \tag{10.9.3.5.1-1}$$

in which:

$$R_p = q_p A_p \tag{10.9.3.5.1-2}$$

$$R_s = q_s A_s \tag{10.9.3.5.1-3}$$

where:

φ = resistance factor (dim)
R_n = nominal sliding resistance against failure by sliding (kips)
R_p = nominal tip resistance (kips)
R_s = nominal grout-to-ground bond resistance (kips)
φ_{qp} = resistance factor for tip resistance specified in Table 10.5.5.2.5-1
φ_{qs} = resistance factor for grout-to-ground bond resistance specified in Table 10.5.5.2.5-1
q_p = unit tip resistance (ksf)

C10.9.3.5.1

Micropiles are typically designed based on bond into soil and rock neglecting tip resistance due to their relatively small diameter and high grout-to-ground bond resistance. Tip resistance may be considered for micropiles bearing on hard rock although the axial capacity for this case is often controlled by the structural resistance of the micropile.

Both the tip and bond resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement. The bond resistance is typically fully mobilized at displacements of about 0.1 to 0.4 in. The tip capacity, however, is mobilized after the micropile settles about six percent of its diameter (Jeon and Kulhawy, 2001), and is generally neglected in the design of micropiles in soil.

The methods for estimating micropile axial resistance provided in this article should be used. Micropile strength limit state resistance methods not specifically addressed in this Article for which adequate successful regional or national experience is available may be used, provided adequate information and experience is also available to develop appropriate resistance factors.

q_s = unit grout-to-ground bond resistance (ksf)
A_p = area of micropile tip (ft^2)
A_s = area of grout-to-ground bond surface (ft^2)

For final design, micropile resistance shall be verified through the performance of micropile load tests as described in Article 10.9.3.5.4. The resistance factors for micropiles shall be taken as specified in Table 10.5.5.2.5-1.

10.9.3.5.2—Estimation of Grout-to-Ground Bond Resistance

The nominal grout-to-ground bond resistance over the bonded length of a micropile, R_s, in kips shall be taken as:

$$R_s = \pi d_b \alpha_b L_b \qquad (10.9.3.5.2\text{-}1)$$

where:

d_b = diameter of micropile drill hole through bonded length (ft)
α_b = nominal micropile grout-to-ground bond strength (ksf)
L_b = micropile bonded length (ft)

For final design, micropile capacity shall be verified through the performance of micropile load tests as described in Article 10.9.3.5.4.

C10.9.3.5.2

The value of nominal unit grout-to-ground bond strength, either estimated empirically or determined through load testing, is typically taken as the average value over the entire bond length.

Micropile grout-to-ground bond strength is influenced by soil and rock conditions, method of micropile drilling and installation, and grouting pressure. The final micropile geotechnical design should be performed by a specialty contractor qualified to perform micropile design and construction. As a guide, Table C10.9.3.5.2-1 may be used to estimate the nominal (ultimate) unit grout-to-ground bond strength for Types A, B, C, D, and E micropiles bonded into soil and/or rock for preliminary design.

For preliminary design, the grout-to-ground bond resistance of micropiles may be based on the results of micropile load tests; estimated based on a review of geologic and boring data, soil and rock samples, laboratory testing, and pervious experience; or estimated using published soil/rock-grout bond guidelines.

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Table C10.9.3.5.2-1—Summary of Typical α_b Values (Grout-to-Ground Bond) for Preliminary Micropile Design (modified after Sabatini et al., 2005)

Soil/Rock Description	Typical Range of Grout-to-Ground Bond Nominal Resistance for Micropile Types[1] (ksf)				
	Type A	Type B	Type C	Type D	Type E
Silt & Clay (some sand) (soft medium plastic)	0.7–1.4	0.7–2.0	0.7–2.5	0.7–3.0	0.7–2.0
Silt & Clay (some sand) (stiff, dense to very dense)	0.7–2.5	1.4–4.0	2.0–4.0	2.0–4.0	1.4–4.0
Sand (some silt) (fine, loose-medium dense)	1.4–3.0	1.4–4.0	2.0–4.0	2.0–5.0	1.4–5.0
Sand (some silt, gravel) (fine-coarse, medium-very dense)	2.0–4.5	2.5–7.5	3.0–7.5	3.0–8.0	2.5–7.5
Gravel (some sand) (medium-very dense)	2.0–5.5	2.5–7.5	3.0–7.5	3.0–8.0	2.5–7.5
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	2.0–4.0	2.0–6.5	2.5–6.5	2.5–7.0	2.0–6.5
Soft Shales (fresh-moderate fracturing, little to no weathering)	4.3–11.5	N/A	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	10.8–28.8	N/A	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	21.6–43.2	N/A	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	10.8–36.0	N/A	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	28.8–87.7	N/A	N/A	N/A	N/A

(1) Refer to Article 10.9.1 for description of micropile types.

10.9.3.5.3—Estimation of Micropile Tip Resistance in Rock

The methods used for design of micropiles bearing on rock shall consider the presence, orientation, and condition of discontinuities, weathering profiles, and other similar profiles as they apply at a particular site. The designer shall judge the competency of a rock mass in accordance with the provisions of Article 10.4.6.4.

For micropiles founded on competent rock, tip resistance may be estimated in accordance with the provisions of Article 10.8.3.5.4c.

C10.9.3.5.3

Micropiles are generally designed based on bond into rock rather than tip resistance. Tip resistance is generally considered only for micropiles bearing on competent rock.

For micropiles founded on competent rock, the rock is usually so sound that the structural capacity will govern the design.

Weak rock includes some shales and mudstones or poor-quality weathered rocks. The term “weak” has no generally accepted, quantitative definition; therefore, judgment and experience are required to make this determination.

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10.9.3.5.4—Micropile Load Test

The load test shall follow the procedures specified in ASTM D1143 for compression and ASTM D3689 for tension. The loading procedure should follow the Quick Load Test Method, unless detailed longer-term load-settlement data is needed, in which case the standard loading procedure should be used. Unless specified otherwise by the Engineer, the pile axial resistance shall be determined from the test data using the Davisson Method as presented in Article 10.7.3.8.2.

The number of load tests required to account for site variability shall be as specified in Article 10.5.5.2.3. The number of test micropiles required should be increased in nonuniform subsurface conditions.

In addition, proof tests loaded to the required factored load shall be performed on one pile per substructure unit or five percent of the piles, whichever is greater, unless specified otherwise by the Engineer.

The resistance factors for axial compressive resistance or axial uplift resistance shall be taken as specified in Table 10.5.5.2.5-1.

C10.9.3.5.4

See Article C10.8.3.5.6.

Load Tests on micropiles are performed to determine micropile installation characteristics, evaluate micropile capacity with depth, and establish micropile bond lengths.

During the performance of ASTM tests, the Contractor may perform several load cycles on the test micropile for diagnostic purposes.

Test micropiles may not be required where previous experience exists with the same micropile type and ultimate micropile capacity in similar subsurface conditions. However, load tests can differ even for apparently similar ground conditions. Therefore, care should be exercised in generalizing and extrapolating the test results to other locations.

Test micropiles are frequently planned for each substructure.

With approval of the Engineer, the number of load tests and proof tests can be reduced based on:

- Previous micropile load tests in similar ground using similar methods, or
- Site-specific tests showing much higher than required factored resistance or consistent proof test.

10.9.3.6—Resistance of Micropile Groups in Compression

Reduction in resistance from group effects shall be evaluated in accordance with the provisions of Article 10.7.3.9.

C10.9.3.6

See Article C10.7.3.9.

10.9.3.7—Nominal Uplift Resistance of a Single Micropile

Uplift resistance shall be evaluated when upward loads act on the micropiles. Micropiles subjected to uplift forces shall be investigated for resistance to pullout, for their structural strength, and for the strength of their connection to supported components.

C10.9.3.7

Resistance factors in Article 10.5.5.2.5 assume a tension load test is performed. In the event that tension load tests are not performed, the resistance factor for presumptive values may be used or the tension resistance estimated as 50 percent of the compression resistance.

10.9.3.8—Nominal Uplift Resistance of Micropile Groups

The provisions of Article 10.7.3.11 shall apply.

C10.9.3.8

Group uplift resistance in rock should consider the depth of soil overburden, rock discontinuity spacing and condition, and rock mass shear strength, as well as bond between micropiles and rock.

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10.9.3.9—Nominal Horizontal Resistance of Micropiles and Micropile Groups

The provisions of Article 10.7.3.12 apply.

The design of horizontally loaded micropiles shall account for the effects of interaction between the micropiles and ground, including the number and spacing of micropiles in the group.

For micropiles used in groups, the micropile head shall be embedded into the cap and the degree of fixity shall be considered in the design.

C10.9.3.9

See Article C10.7.3.12.

10.9.3.10—Structural Resistance

10.9.3.10.1—General

The structural design of micropiles shall be in accordance with the provisions of Section 5 for the design of reinforced concrete and Section 6 for the design of steel.

The cased and uncased length of each micropile shall be designed to resist the forces distributed to the micropile based on the micropile inclination and spacing.

The resistance factors for structural design shall be as specified in Table 10.5.5.2.5-2.

C10.9.3.10.1

Articles 5.8.2.4, 5.6.4, 5.6.6, 5.12.9, and 6.15 provide specific provisions applicable to design of concrete and steel micropiles.

The design of micropiles supporting axial compression load only requires an allowance for unintended eccentricity. This has been accounted for by use of the equations in Article 5.6.4.4 for reinforced concrete columns that already contain an eccentricity allowance.

10.9.3.10.2—Axial Compressive Resistance

The upper cased section of a micropile subjected to compression loading shall be designed structurally to support the full factored load on the micropile. The lower uncased section of a micropile subjected to compression loading shall be designed structurally to support the maximum full factored load on the micropile less the load transferred to the surrounding ground from the cased portion of the pile in the plunge length (if used), as described in Article 10.9.3.10.4.

For micropiles extending through a weak upper soil layer, extending above ground, subject to scour, extending through mines/caves, or extending through soil that may liquefy, the effect of any laterally unsupported length shall be considered in the determination of axial compression resistance.

The factored structural resistance of a micropile to axial compression loading, R_C, in kips may be taken as:

$$R_C = \varphi_C R_n \quad (10.9.3.10.2\text{-}1)$$

where:

φ_C = resistance factor specified in Table 10.5.5.2.5-2 for structural resistance of micropiles in axial compression

R_n = nominal axial compression resistance of micropile specified in Articles 10.9.3.10.2a and 10.9.3.10.2b (kips)

10.9.3.10.2a—Cased Length

The factored structural resistance of the upper cased length of a micropile having no unsupported length and loaded in compression, R_{CC}, in kips may be taken as:

$$R_{CC} = \varphi_{CC} R_n \quad (10.9.3.10.2a\text{-}1)$$

for which:

$$R_n = 0.85\left[0.85 f_c' A_g + f_y \left(A_b + A_c\right)\right] \quad (10.9.3.10.2a\text{-}2)$$

where:

φ_{CC} = resistance factor specified in Table 10.5.5.2.5-2 for structural resistance of the cased section of a micropile subjected to compression loading
f'_c = specified compressive strength of micropile grout at 28 days unless another age is specified (ksi)
A_g = cross-sectional area of grout within micropile (in.^{2})
f_y = specified minimum yield strength of reinforcement bar or steel casing, or stress in steel reinforcement bar or casing at a strain of 0.003, whichever is less (ksi)
A_b = cross-sectional area of steel reinforcing bar (in.^{2})
A_c = cross-sectional area of steel casing (in.^{2})

C10.9.3.10.2a

The design compressive stress in the steel is limited to the stress at which the strain equals 0.003 to maintain compatibility with the strain in the grout.

10.9.3.10.2b—Uncased Length

The factored structural resistance of the lower, uncased length of a micropile having no unsupported length and loaded in compression, R_{CU}, in kips may be taken as:

$$R_{CU} = \varphi_{CU} R_n \quad (10.9.3.10.2b\text{-}1)$$

for which:

$$R_n = 0.85\left[0.85 f_c' A_g + f_y A_b\right] \quad (10.9.3.10.2b\text{-}2)$$

where:

φ_{CU} = resistance factor specified in Table 10.5.5.2.5-2 for structural resistance of the uncased section of a micropile subjected to compression loading
f'_c = specified compressive strength of micropile grout at 28 days unless another age is specified (ksi)
A_g = cross-sectional area of grout within micropile (in.^{2})
f_y = specified minimum yield strength of reinforcement bar or stress in steel reinforcement bar at a strain of 0.003, whichever is less (ksi)
A_b = cross-sectional area of steel reinforcing bar (in.^{2})

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10.9.3.10.3—Axial Tension Resistance

The upper cased section of a micropile subjected to tension loading shall be designed structurally to support the full factored load on the micropile. The lower uncased section of a micropile subjected to tension loading shall be designed structurally to support the maximum full factored load on the micropile less the load transferred to the surrounding ground from the cased portion of the micropile in the plunge length, as described in Article 10.9.3.10.4.

The factored structural resistance of a micropile subjected to tension, R_T, may be taken as:

$$R_T = \varphi_T R_n \qquad (10.9.3.10.3\text{-}1)$$

where:

φ_T = resistance factor specified in Table 10.5.5.2.5-2 for structural resistance of a micropile subjected to tension loading (dim)

R_n = nominal axial tension resistance of micropile specified in Articles 10.9.3.10.3a and 10.9.3.10.3b

10.9.3.10.3a—Cased Length

The factored structural resistance of the upper cased length of a micropile subjected to tension loading, R_{TC}, in kips may be taken as:

$$R_{TC} = \varphi_{TC} R_n \qquad (10.9.3.10.3a\text{-}1)$$

for which:

$$R_n = f_y \left(A_b + A_{ct} \right) \qquad (10.9.3.10.3a\text{-}2)$$

where:

φ_{TC} = resistance factor specified in Table 10.5.5.2.5-2 for structural resistance of the cased section of a micropile subjected to tension loading (dim)

f_y = specified minimum yield strength of reinforcement bar or steel casing, whichever is less (ksi)

A_b = cross-sectional area of steel reinforcing bar (in.^{2})

A_{ct} = cross-sectional area of steel casing considering reduction for threads (in.^{2})

C10.9.3.10.3a

The design compressive stress in the steel is limited to the stress at which the strain equals 0.003 to maintain compatibility with the strain in the grout.

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10.9.3.10.3b—Uncased Length

The factored structural resistance of the lower uncased length of a micropile subjected to tension loading, R_{TU}, in kips may be taken as:

$$R_{TU} = \varphi_{TU} R_n \qquad (10.9.3.10.3b\text{-}1)$$

for which:

$$R_n = f_y A_b \qquad (10.9.3.10.3b\text{-}2)$$

where:

φ_{TU} = resistance factor specified in Table 10.5.5.2.5-2 for structural resistance of the uncased section of a micropile subjected to tension loading (dim)
f_y = specified minimum yield strength of reinforcement bar (kip)
A_b = cross-sectional area of steel reinforcing bar (in.^{2})

10.9.3.10.4—Plunge Length Transfer Load

The factored axial load transferred to the ground through the plunge length of the cased portion of a micropile, P_t, in kips, may be taken as:

$$P_t = \varphi\left[\pi d_b \alpha_b L_p\right] \qquad (10.9.3.10.4\text{-}1)$$

where:

φ = resistance factor specified in Table 10.5.5.2.5-1 for geotechnical bearing or uplift resistance, as appropriate, of a single micropile
d_b = diameter of micropile drill hole through bonded length (ft)
α_b = nominal micropile grout-to-ground bond strength (ksf)
L_p = micropile casing plunge length (ft)

If load transfer through the plunge length of the cased portion of a micropile is considered to reduce the load on the lower uncased portion of the micropile, the factored axial load on the uncased portion of the micropile in compression or tension, P_u, in kips, may be taken as:

$$P_u = Q - P_t \qquad (10.9.3.10.4\text{-}2)$$

where:

Q = $\sum \eta_i \gamma_i Q_i$
= total factored axial load on micropile (kips)
P_t = factored axial load transferred to the ground through the plunge length of the cased portion of a micropile from Eq. 10.9.3.10.4-1 (kips)

C10.9.3.10.4

An optional procedure for construction of a composite reinforced micropile includes insertion of the pile casing into the top of the grouted bond zone to effect a transition between the upper cased portion to the lower uncased portion of a micropile. The length of casing inserted into the bond zone by the plunge length is shown in Figure C10.9.1-1. As a result, a portion of the factored axial load on a micropile is transferred to the surrounding ground by the cased portion of the pile, reducing the load that must be supported by the weaker uncased portion of the pile. The reduction in load applied to the uncased length is termed the transfer load P_t.

10.9.3.10.5—Grout-to-Steel Bond

Casing-to-grout bond shall be checked and reinforcement bar development length shall be checked in accordance with the provisions of Section 5.

C10.9.3.10.5

Grout-to-steel bond does not typically govern micropile design, except for overlap of reinforcing bars into upper casing.

The bond between the cement grout and the reinforcing steel is the mechanism for transfer of the pile load from the reinforcing steel to the ground. Typical ultimate bond values range from 0.15 to 0.25 ksi for smooth bars and pipe, and 0.30 to 0.50 ksi for deformed bars (Armour et al., 2000). Refer to Section 5 for bar development requirements.

As is the case with any reinforcement, the surface condition will affect the attainable bond. A film of rust may be beneficial, but the presence of loose debris or lubricant or paint is not desirable. Normal methods for the handling and storage of reinforcing bars apply to micropile construction. For the permanent casing that is also used to drill the hole, cleaning of the casing surface can occur during drilling, particularly in granular soils.

10.9.3.10.6—Buckling and Lateral Stability

The provisions of Article 10.7.3.13.4 shall apply.

10.9.3.10.7—Reinforcement into Superstructure

Sufficient reinforcement shall be provided at the junction of the micropile with the micropile footing or column to make a suitable connection. The embedment of the reinforcement into the cap shall comply with the provision for cast-in-place piles in Section 5.

C10.9.3.10.7

Refer to Sabatini et al. (2005) for typical micropile to footing connection details.

10.9.4—Extreme Event Limit State

The provisions of Articles 10.5.5.3 and 10.7.4 shall apply.

C10.9.4

See Articles C10.5.5.3 and C10.7.4.

10.9.5—Corrosion and Deterioration

The provisions of Article 10.7.5 shall apply.

C10.9.5

Corrosion protection methods and design presented in Article C10.7.5 apply to micropiles as well. In addition, other micropile specific design options including plastic encapsulation of central reinforcing bars is provided in Sabatini (2005).

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APPENDIX A10—SEISMIC ANALYSIS AND DESIGN OF FOUNDATIONS

A10.1—INVESTIGATION

Slope instability, liquefaction, fill settlement, and increases in lateral earth pressure have often been major factors contributing to bridge damage in earthquakes. These earthquake hazards may be significant design factors for peak earthquake accelerations in excess of 0.1 *g* and should form part of a site-specific investigation if the site conditions and the associated acceleration levels and design concepts suggest that such hazards may be of importance.

A10.2—FOUNDATION DESIGN

The commonly-accepted practice for the seismic design of foundations is to utlize a pseudo-static approach, where earthquake-induced foundation loads are determined from the reaction forces and moments necessary for structural equilibrium. Although traditional bearing capacity design approaches are also applied, with appropriate capacity reduction factors if a margin of safety against "failure" is desired, a number of factors associated with the dynamic nature of earthquake loading should always be borne in mind.

Under cyclic loading at earthquake frequencies, the strength capable of being mobilized by many soils is greater than the static strength. For unsaturated cohesionless soils, the increase may be about ten percent, whereas for cohesive soils, a 50 percent increase could occur. However, for softer saturated clays and saturated sands, the potential for strength and stiffness degradation under repeated cycles of loading must also be recognized. For bridges classified as Zone 2, the use of static soil strengths for evaluating ultimate foundation capacity provides a small implicit measure of safety and, in most cases, strength and stiffness degradation under repeated loading will not be a problem because of the smaller magnitudes of seismic events. However, for bridges classified as Zones 3 and 4, some attention should be given to the potential for stiffness and strength degradation of site soils when evaluating ultimate foundation capacity for seismic design.

As earthquake loading is transient in nature, "failure" of soil for a short time during a cycle of loading may not be significant. Of perhaps greater concern is the magnitude of the cyclic foundation displacement or rotation associated with soil yield, as this could have a significant influence on structural displacements or bending moments and shear distributions in columns and other members.

As foundation compliance influences the distribution of forces or moments in a structure and affects computation of the natural period, equivalent stiffness factors for foundation systems are often required. In many cases, use is made of various analytical solutions that are available for footings or piles where it is assumed that soil behaves in an elastic medium. In using these formulae, it should be recognized that equivalent elastic moduli for soils are a function of strain amplitude, and for seismic loads modulus values could be significantly less than those appropriate for low levels of seismic loading. Variation of shear modulus with shearing strain amplitude in the case of sands is shown in Figure A10.2-1. Additional discussion of this topic can be found in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

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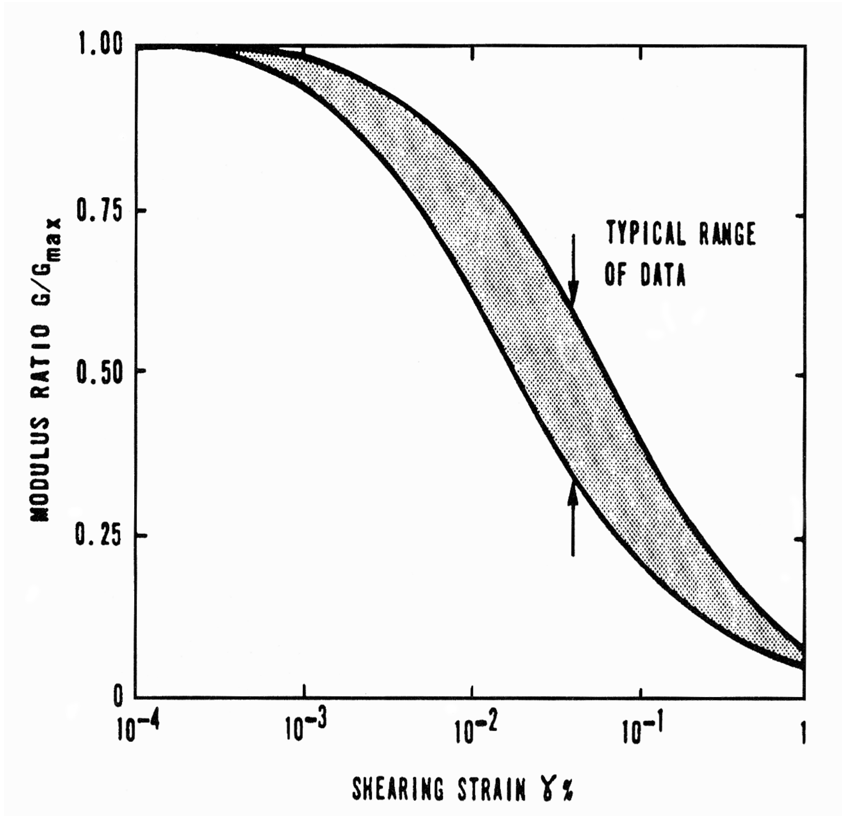

Figure A10.2-1—Variation of Shear Modulus with Shearing Strain for Sands

On the basis of field and experimental observations, it is becoming more widely recognized that transient foundation uplift or rocking during earthquake loading, resulting in separation of the foundation from the subsoil, is acceptable provided that appropriate design precautions are taken (Taylor and Williams, 1979). Experimental studies suggest that rotational yielding beneath rocking foundation can provide a useful form of energy dissipation. However, care must be taken to avoid significant induced vertical deformations accompanying possible soil yield during earthquake rocking as well as excessive pier movement. These could lead to design difficulties with relative displacements.

Lateral Loading of Piles—Most of the well-known solutions for computing the lateral stiffness of vertical piles are based on the assumption of elastic behavior and utilize equivalent cantilever beam concepts (Davisson and Gill, 1960), the beam on an elastic Inkler foundation method (Matlock and Reese, 1960), or elastic continuum solutions (Poulos, 1971). However, the use of methods incorporating nonlinear subgrade reaction behavior that allows for soil failure may be important for high lateral loading of piles in soft clay and sand. Such a procedure is encompassed in the American Petroleum Institute (API) recommendations for offshore platform design. The method utilizes nonlinear subgrade reaction or *P*-*y* curves for sands and clays that have been developed experimentally from field loading tests.

The general features of the API analysis in the case of sands are illustrated in Figure A10.2-2. Under large loads, a passive failure zone develops near the pile head. Test data indicate that the ultimate resistance, p_u, for lateral loading is reached for pile deflections, y_u, of about $3d/80$, where d is the pile diameter. Note that most of the lateral resistance is mobilized over a depth of about $5d$. The API method also recognizes degradation in lateral resistance with cylic loading, although in the case of saturated sands, the degradation postulated does not reflect pore water pressure increases. The degradation in lateral resistance due to earthquake-induced, free-field pore water pressure increases in saturated sands has been described by Finn and Martin (1979). A numerical method that allows the use of API *P*-*y* curves to compute pile stiffness characteristics forms the basis of the computer program BMCOL 76 described by Bogard and Matlock (1977).

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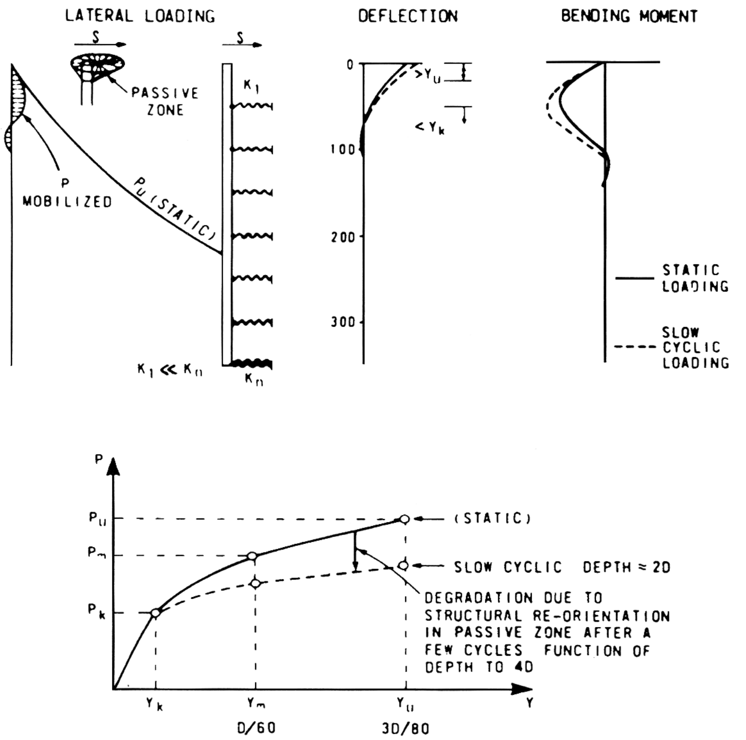

Figure A10.2-2—Lateral Loading of Piles in Sand Using API Criteria

The influence of group action on pile stiffness is a somewhat controversial subject. Solutions based on elastic theory can be misleading where yield near the pile head occurs. Experimental evidence tends to suggest that group action is not significant for pile spacings greater than 4*d* to 6*d*.

For batter pile systems, the computation of lateral pile stiffness is complicated by the stiffness of the piles in axial compression and tension. It is also important to recognize that bending deformations in batter pile groups may generate high reaction forces on the pile cap.

It should be noted that although battered piles are economically attractive for resisting horizontal loads, such piles are very rigid in the lateral direction if arranged so that only axial loads are induced. Hence, large relative lateral displacements of the more flexible surrounding soil may occur during the free-field earthquake response of the site (particularly if large changes in soil stiffness occur over the pile length), and these relative displacements may in turn induce high pile bending moments. For this reason, more flexible vertical pipe systems where lateral load is resisted by bending near the pile heads are recommended. However, such pile systems must be designed to be ductile because large lateral displacements may be necessary to resist the lateral load. A compromise design using battered piles spaced some distance apart may provide a system that has the benefits of limited flexibility and the economy of axial load resistance to lateral load.

Soil-Pile Interaction—The use of pile stiffness characteristics to determine earthquake-induced pile bending moments based on a pseudo-static approach assumes that moments are induced only by lateral loads arising from inertial effects on the bridge structure. However, it must be remembered that the inertial loads are generated by interaction of the free-field earthquake ground motion with the piles and that the free-field displacements themselves can influence bending moments. This is illustrated in an idealized manner in Figure A10.2-3. The free-field earthquake displacement time histories provide input into the lateral resistance interface elements, which in turn transfer motion to the pile. Near the pile heads, bending moments will be dominated by the lateral interaction loads generated by inertial effects on the bridge structure. At greater depth (e.g., greater than 10*d*), where soil stiffness progressively increases with respect to pile stiffness, the pile will be constrained to deform in a manner similar to that of the free field, and pile bending moments become a function of the curvatures induced by free-field displacements.

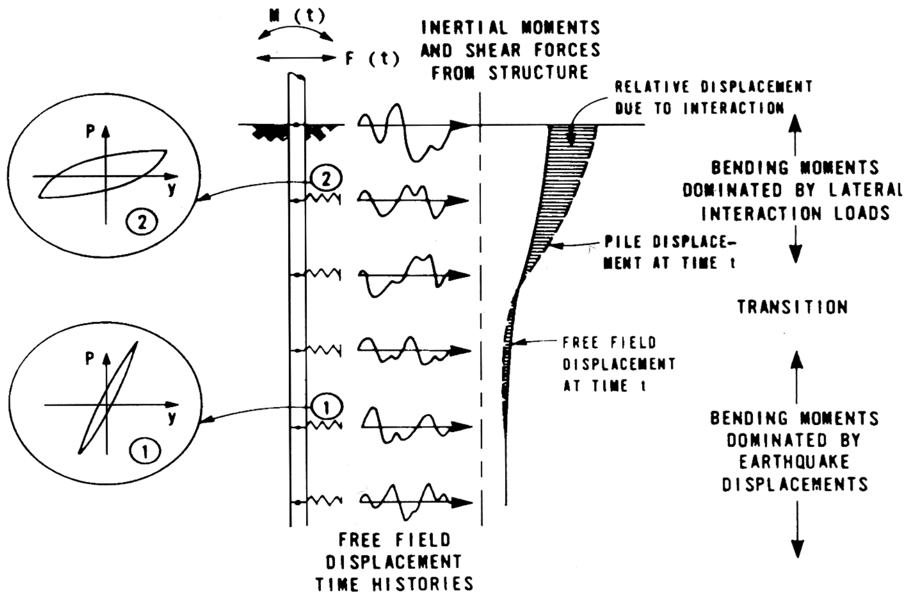

Figure A10.2-3—Mechanism of Soil-Pile Interaction during Seismic Loading

To illustrate the nature of free-field displacements, reference is made to Figure A10.2-4, which represents a 200-ft deep cohesionless soil profile subjected to the El Centro earthquake. The free-field response was determined using a nonlinear, one-dimensional response analysis. From the displacement profiles shown at specific times, curvatures can be computed and pile bending moments calculated if it is assumed that the pile is constrained to displace in phase with the free-field response.

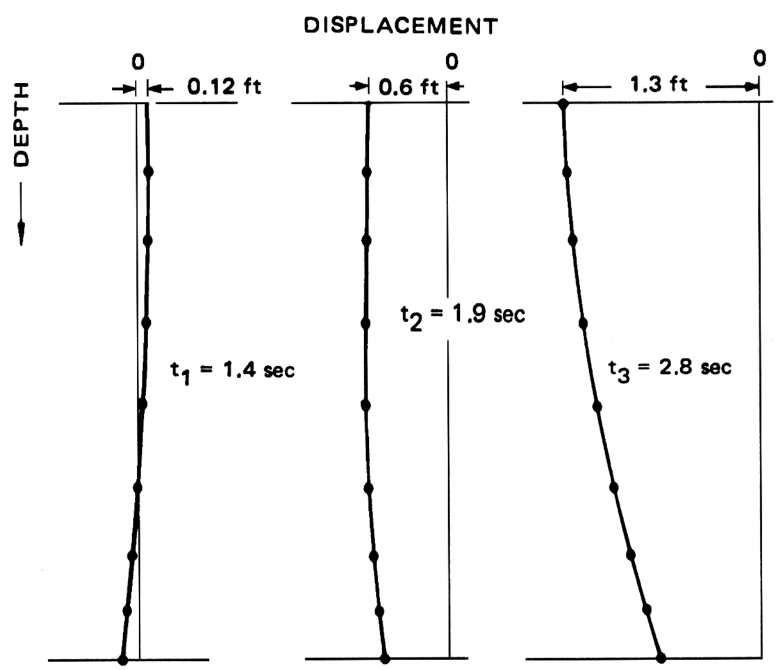

Figure A10.2-4—Typical Earthquake Displacement Profiles

Large curvatures could develop at interfaces between soft and rigid soils and, clearly, in such cases emphasis should be placed on using flexible ductile piles. Margason (1979) suggests that curvatures of up to 6×10^{-4} in.$^{-1}$ could be induced by strong earthquakes, but these should pose no problem to well-designed steel or prestressed concrete piles.

Studies incorporating the complete soil-pile structure interaction system, as presented in Figure A10.2-3, have been described by Penzien (1970) for a bridge piling system in deep soft clay. A similar but somewhat simpler soil-pile structure interaction system (SPASM) to that used by Penzien has been described by Matlock et al. (1978). The model used is, in effect, a dynamic version of the previously mentioned BMCOL program.

A10.3—SPECIAL PILE REQUIREMENTS

The uncertainties of ground and bridge response characteristics lead to the desirability of providing tolerant pile and foundation systems. Toughness under induced curvature and shears is required, and hence piles such as steel H-sections and concrete filled steel-cased piles are favored for highly seismic areas. Unreinforced concrete piles are brittle in nature, so nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie elements together and to assist in load transfer from the pile to the pile cap.

Experience has shown that reinforced concrete piles tend to hinge or shatter immediately below the pile cap. Hence, tie spacing is reduced in this area so that the concrete is better confined. Driven precast piles should be constructed with considerable spiral confining steel to ensure good shear strength and tolerance of yield curvatures should these be imparted by the soil or structural response. Clearly, it is desirable to ensure that piles do not fail below ground level and that flexural yielding in the columns is forced to occur above ground level. The additional pile design requirements imposed on piles for bridges classified as Zones 3 and 4, for which earthquake loading is more severe, reflect a design philosophy aimed at minimizing below-ground damage that is not easily inspected following a major earthquake.

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