10.2—DEFINITIONS

Replace the following definition:

Casing — Steel pipe introduced during the drilling process to temporarily or permanently stabilize the soil within the drill hole. Depending on the details of construction, this casing may be fully extracted during concrete placement of a Cast-In-Drilled Hole (CIDH) concrete pile, or after grouting of a micropile, or may remain partially or completely in place, e.g., permanent casing, as part of the final pile configuration.

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10.3—NOTATION

Replace the following notations:

φ_{qp} = resistance factor for tip resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5) (10.8.3.5.1)

φ_{qs} = resistance factor for side resistance (dim) (10.5.5.2.3) (10.5.5.2.4) (10.8.3.5)

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10.5.2.1—General

Replace the 1st paragraph with the following:

Foundation design at the service limit state shall include:

- settlements,
- horizontal movements,
- overall stability, and
- total scour at the design flood.

10.5.2.2—Tolerable Movements and Movement Criteria

Add two paragraphs after the 3rd paragraph:

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

Under Service-I load combination eccentricity shall be limited to *B*/6 and *B*/4 when spread footings are founded on soil and rock, respectively.

The permissible (allowable) horizontal load for piles/shafts at abutments shall be evaluated at 0.25 inch pile/shaft top horizontal movement. Horizontal load on the pile from Service-I load combination shall be less than the permissible horizontal load.

C10.5.2.2

Add two paragraphs to the end of the article as follows:

No rotation analysis is necessary when eccentricity under Service-I load combination is limited to *B*/6 and *B*/4 or less for spread footings founded on soil and rock, respectively. Otherwise, it is necessary to establish permissible foundation movement criteria and the corresponding permissible eccentricity limits. When necessary for bridge abutments, such analysis is performed only for eccentricity in the longitudinal direction of the bridge.

The horizontal component of a battered pile's axial load may be subtracted from the total lateral load to determine the applied horizontal or lateral loads on pile foundations.

10.5.3.1—General

Replace the 2nd paragraph with the following:

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

- structural resistance and
- loss of lateral and axial support due to scour at the design flood event.

C10.5.3.1

Replace the 4th paragraph with the following:

The design flood for scour is defined in Article 2.6 and is specified in Article 3.7.5 as applicable at the strength limit state.

10.5.4.1—Extreme Events Design

C10.5.4.1

Replace the commentary with the following:

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. *Seismic Design Criteria* (SDC) gives additional guidance regarding seismic analysis and design. Scour should be considered with extreme events as per Articles 3.4.1 and 3.7.5.

10.5.5.2.1—General

Replace the article with the following:

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.

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

C10.5.5.2.1

Replace the commentary with the following:

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. When a single pile or drilled shaft supports a bridge column, reduction of the resistance factors in Articles 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5 should be considered.

Certain resistance factors in Articles 10.5.5.2.2, 10.5.5.2.3 and 10.5.5.2.4 are presented as a function of soil type, e.g., cohesionless or cohesive. Many naturally occurring soils do not fall neatly into these two classifications. In general, the terms "cohesionless soil" or “sand” may be connoted to mean drained conditions during loading, while "cohesive soil" or “clay” implies undrained conditions in the short-term. For other or intermediate soil classifications, such as clayey sand or silts, the designer should choose, depending on the load case under consideration, whether the resistance provided by the soil in the short term will be a drained, undrained, or a combination of the two strengths 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).*

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 adequate quantity and/or quality of data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the Caltrans *Bridge Design Specifications* (2000), dated November 2003.

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 some 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).

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

Replace the 1st paragraph with the following:

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings.

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Replace Table 10.5.5.2.2-1 with the following:

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

Nominal Resistance	Resistance Determination Method/Condition		Resistance Factor
Bearing in Compression	φ_b	Theoretical method - (*Munfakh et al., 2001*), in cohesive soils	0.50
		Theoretical method - (*Munfakh et al., 2001*), in cohesionless soil, using *CPT*	0.50
		Theoretical method - (*Munfakh et al., 2001*), in cohesionless soil, 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 cohesionless soil	0.90
		Cast-in-place concrete on cohesionless soil	0.80
		Cast-in-place or pre-cast concrete on cohesive soil	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

10.5.5.2.3—Driven Piles

Replace the entire article with the following:

Resistance factors for driven piles shall be selected from Table 10.5.5.2.3-1.

C10.5.5.2.3

Replace commentary with the following:

The resistance factors in Table 10.5.5.2.3-1 are calibrated to past WSD and Load Factored Design (LFD) practice.

Replace Table 10.5.5.2.3-1 with the following:

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

Nominal Resistance	Resistance Determination Method/Conditions	Resistance Factor	
Axial Compression or Tension	All resistance determination methods, all soils and rock	φ_{stat}, φ_{dyn}, φ_{qp}, φ_{qs}, φ_{bl}, φ_{up}, φ_{ug}, φ_{load}	0.70
Lateral or Horizontal Resistance	All soils and rock		1.0
Pile Drivability Analysis	Steel Piles	φ_{da}	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
	In all three Articles identified above, use φ identified as "resistance during pile driving"		
Structural Limit States	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	

10.5.5.2.4—Drilled Shafts

Replace the entire article with the following:

Resistance factors for drilled shafts shall be selected from Table 10.5.5.2.4-1.

C10.5.5.2.4

Replace the entire commentary and with the following:

The resistance factors in Table 10.5.5.2.4-1 are calibrated to WSD and LFD practices.

The maximum value of the resistance factors in Table 10.5.5.2.4-1 are based on full-time inspection and field quality control during shaft construction. If a full time inspection and field quality control can not be assured, lower resistance factors should be used.

The mobilization of drilled shaft tip resistance is uncertain as it depends on many factors including soil types, groundwater conditions, drilling and hole support methods, the degree of quality control on the drilling slurry and the base cleanout, etc. Allowance of the full effectiveness of the tip resistance should be permitted only when cleaning of the bottom of the drilled shaft hole is specified and can be acceptably completed before concrete placement.

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Replace Table 10.5.5.2.4-1 with the following:

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

Nominal Resistance	Resistance Determination Method/Conditions	Resistance Factor	
Axial Compression and Tension or Uplift	All soils, rock and IGM All calculation methods	φ_{stat}, φ_{up}, φ_{bl}, φ_{ug}, φ_{load}, φ_{upload}, φ_{qs}	0.70
Axial Compression	All soils, rock and IGM All calculation methods	φ_{qp}	0.50
Lateral Geotechnical Resistance	All soils, rock and IGM All calculation methods		1.0

10.5.5.3.2—Scour

Delete the entire article.

C10.5.5.3.2

Replace the 1st paragraph with the following:

See Commentary to Article 3.4.1, Extreme Events, and Article 3.7.5.

Replace Article 10.5.5.3.3 title with the following:

10.5.5.3.3—Other Extreme Event Limit States

Replace the 1st paragraph with the following:

Resistance factors for extreme event limit states, including the design of foundations to resist earthquake, blast, ice, vehicle or vessel impact loads, shall be taken as 1.0.

C10.5.5.3.3

Delete the entire commentary

10.6.1.1—General

Replace the 1st paragraph with the following:

Provisions of this article shall apply to design of isolated, continuous strip and combined footings for use in support of columns, walls and others 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, swell or expansion potential and compressibility to support the footing loads.

C10.6.1.1

Replace the 2nd paragraph with the following:

Spread footing should not be used on soil or rock conditions that are determined to be expansive, collapsible, or too soft or weak to support the design loads, without excessive movements, or loss of stability.

10.6.1.3—Effective Footing Dimensions

Replace the 1st paragraph with the following:

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 and bearing resistance. The point of load application shall be at the centroid of the reduced effective area.

Replace the 2nd paragraph with the following:

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

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

$$L' = L - 2e_L$$

where,

e_B = M_L / V = Eccentricity parallel to dimension B (ft)

e_L = M_B / V = Eccentricity parallel to dimension L (ft)

M_B = Factored moment about the central axis along dimension B (kip-ft)

M_L = Factored moment about the central axial along dimension L (kip-ft)

V = Factored vertical load (kips)

C10.6.1.3

Add the following at the end of the commentary:

For additional guidance, see Munfakh (2001) and Article 10.6.3.2.

10.6.1.4—Bearing Stress Distribution

Replace 1st paragraph with the following:

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

- Uniform over the effective area for footing on soils, or
- Linearly varying, i.e., triangular or trapezoidal as applicable, for footings on rock

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10.6.1.6—Groundwater

Replace the last paragraph with the following:

The influences of groundwater on the bearing resistance of soil or rock, the expansion and collapse potential of soil or rock, and on the settlements of the structure should be considered. In cases where seepage forces are present, they should also be included in the analyses.

10.6.2.4.1—General

C10.6.2.4.1

Add the following after the last paragraph:

For eccentrically loaded footings on soils, replace L and B in these specifications with the effective dimensions L' and B', respectively.

10.6.2.4.2—Settlement of Footing on Cohesionless Soils

Replace the 3rd paragraph with the following:

The elastic half-space method assumes the footing is supported on a homogeneous soil of infinite depth. The elastic settlement of spread footings, in feet, by the elastic half-space method shall be estimated as:

$$S_e = \frac{[q_O(1-v^2)\sqrt{A'}]}{144\ E_S\ \beta_Z} \qquad (10.6.2.4.2\text{-}1)$$

where:

q_O = applied vertical stress (ksf)
A' = effective area of footing (ft^2)
E_s = Young's modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of in-situ or laboratory tests (ksi)
β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)
v = Poisson's Ratio, taken as specified in Article 10.4.6.3 if direct measurements of v are not available from the results of in-situ or laboratory tests (dim)

C10.6.2.4.2

Replace the 6th paragraph with the following:

In Table 10.6.2.4.2-1, the β_z values for the flexible foundations correspond to the average settlement. The elastic 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 footing, respectively. For low values of *L/B* ratio, the average settlement for flexible footing is about 85 percent of the maximum settlement near the center. The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

Replace the 8th paragraph with the following:

The accuracy of settlement estimates using elastic theory are strongly affected by the selection of soil modulus and the inherent assumptions of infinite elastic half space. Accurate estimates of soil moduli are difficult to obtain because the analyses are based on only single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent foundations.

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10.6.2.4.2—Settlement of Footing on Cohesionless Soils

Replace the last paragraph with the following:

In Figure 10.6.2.4.2-1, $N1$ shall be taken as $(N_1)_{60}$, Standard Penetration Resistance, N (blows/ft), corrected for hammer energy efficiency and overburden pressure as specified in Article 10.4.6.2.4.

10.6.2.4.3—Settlement of Footing on Cohesive Soils

Add the following after the 1st paragraph:

Immediate or elastic settlement of footings founded on cohesive soils can be estimated using Eq. 10.6.2.4.2-1 with the appropriate value of the soil modulus.

Add the following under Figure 10.6.2.4.3-3:

For eccentrically loaded footings, replace B/H_c with B'/H_c in Figure 10.6.2.4.3-3.

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Replace Article 10.6.2.6 title with the following:

10.6.2.6—Permissible Net Contact Stress

Replace the entire Articles 10.6.2.6.1 & 10.6.2.6.2 with the following:

The permissible net contact stress for spread footings shall be taken as the net footing bearing stress over the effective footing area due to Service-I Load Combination that results in an estimated foundation soil or rock settlement equal to the support-specific permissible settlement. Spread footings shall be located and sized such that the applied net bearing stress due to Service-I Load Combination does not exceed the support-specific permissible net contact stress.

C10.6.2.6

Replace C10.6.2.6.1 and Table C10.6.2.6.1-1 with the following:

The permissible settlement (total) for a bridge support is the maximum tolerable foundation settlement due to Service-I Load Combination in accordance with Article 10.5.2.2. The adequacy of a given footing size to satisfy the permissible settlement can be verified by performing a settlement analysis for the effective size and applied bearing stress, both based on the Service-I Load Combinations. However, in most cases, the design footing size can more conveniently be optimized by evaluating first the minimum effective size of a spread footing required to satisfy the permissible settlement criterion. This can be achieved by using support-specific permissible net contact pressures presented in a plot or table as a function of the effective footing width (B') for a range of L'/B' ratios.

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10.6.3.1.2a—Basic Formulation

C10.6.3.1.2a

Replace the 4th paragraph with the following:

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 modest embedment of 4 feet or deeper because the load inclination factors were derived for footings without embedment.

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Replace the 5th paragraph with the following:

In practice, therefore, footings that are normal to a column with modest embedment, should omit the use of the load inclination factors.

Add a new paragraph after the 5th paragraph:

Unusual column geometry or loading configurations may require consideration be given to evaluating load inclination factors. A column that is not aligned normal to the footing bearing surface would be one example where inclination factors would be given consideration. In cases where inclinations are to be evaluated, the simultaneous application of both shape and inclination factors will result in overly conservative design, therefore using the lower of the two factors is recommended (Munfakh et al., 2001).

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10.6.3.1.2c—Considerations for Footings on Slopes

Replace the entire article with the following:

For footings bearing on or near slopes:

$$N_q = 0.0 \qquad (10.6.3.1.2c\text{-}1)$$

In Equations 10.6.3.1.2a-2 and 10.6.3.1.2a-4, N_c and N_γ shall be replaced with N_{cq} and $N_{\gamma q}$, respectively, from Figures 10.6.3.1.2c-1 and 10.6.3.1.2c-2 for footings bearing on or near slopes. In Figure 10.6.3.1.2c-1, the slope stability factor, N_s, shall be taken as:

- For $B < H_s$:

$$N_s = 0 \qquad (10.6.3.1.2c\text{-}2)$$

- For $B \geq H_s$:

$$N_s = \frac{\gamma H_s}{c} \qquad (10.6.3.1.2c\text{-}3)$$

where:

B = footing width (ft)

H_s = height of sloping ground mass (ft)

C10.6.3.1.2c

Replace the 1st paragraph with the following:

A rational numerical approach for determining a modified bearing capacity factor, N_{cq}, for footings on or near a slope is given in Bowles (1988).

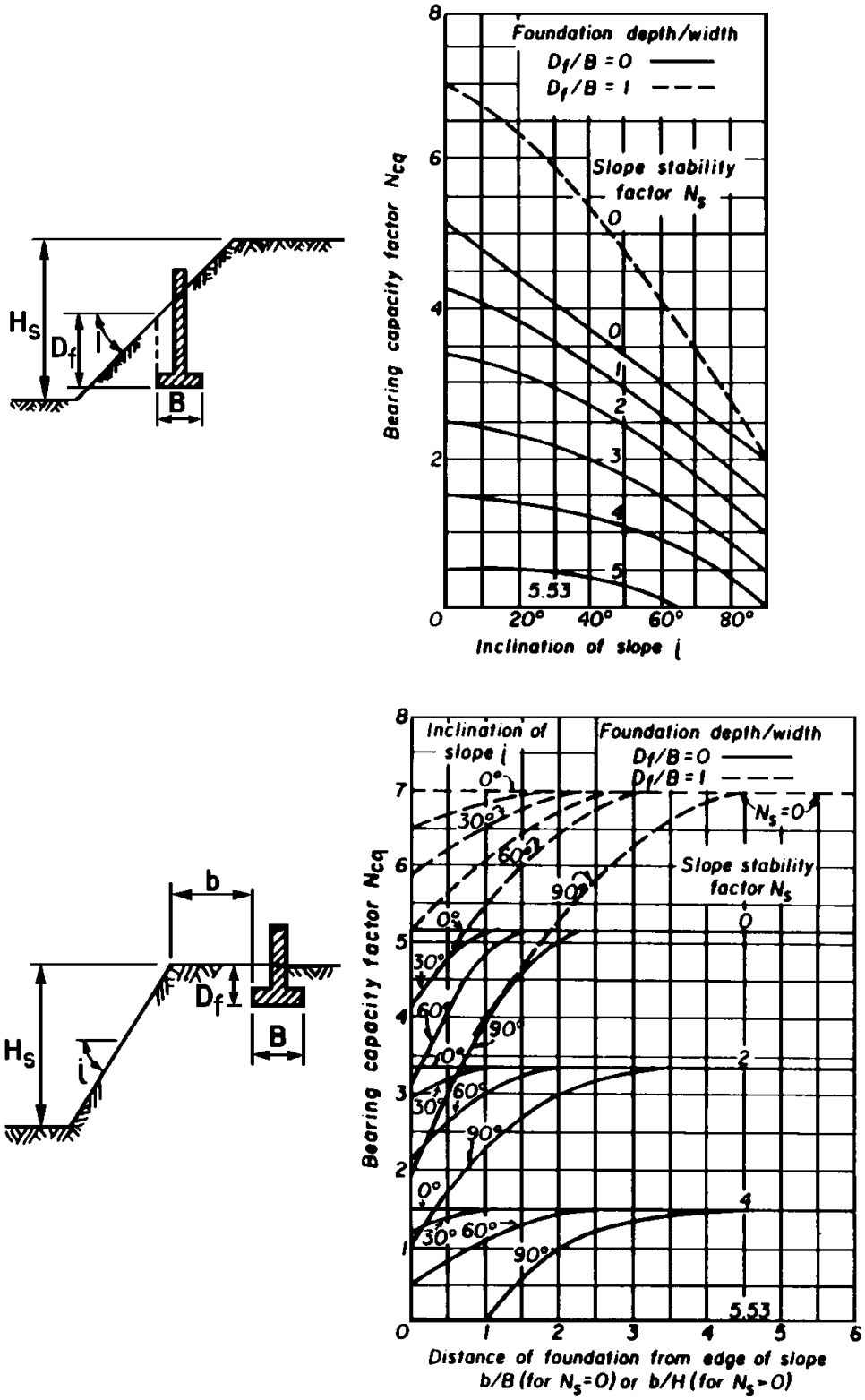
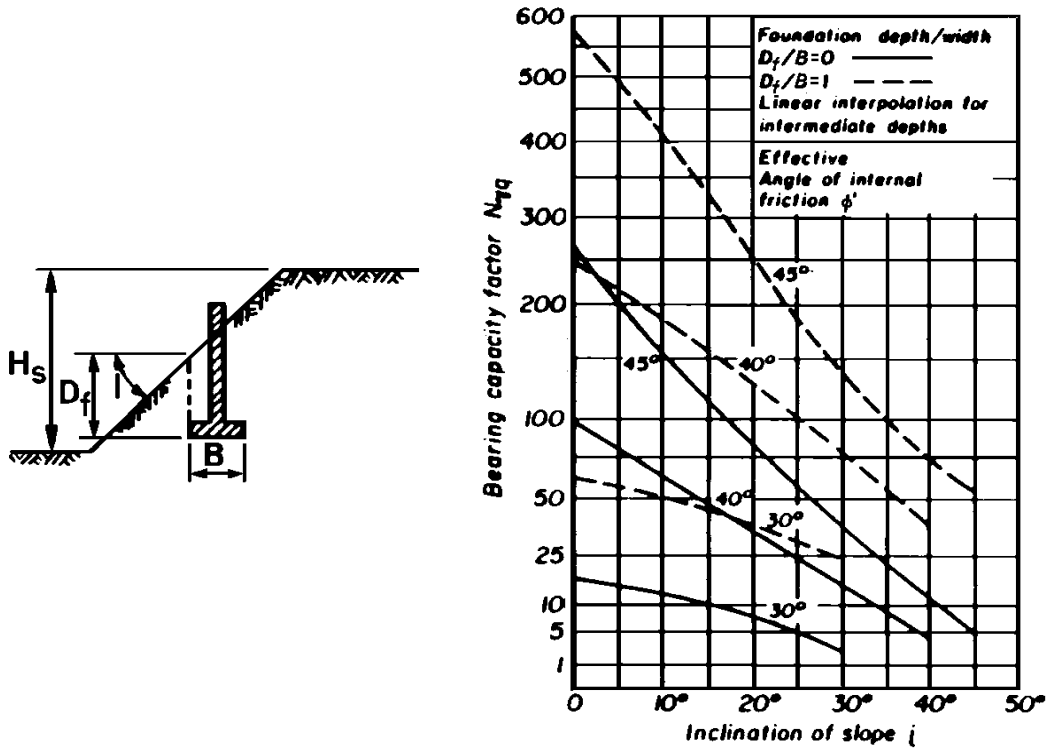


Figure 10.6.3.1.2c-1—Modified Bearing Capacity Factors for Footing in Cohesive Soils and on or adjacent to Sloping Ground after Meyerhof (1957)



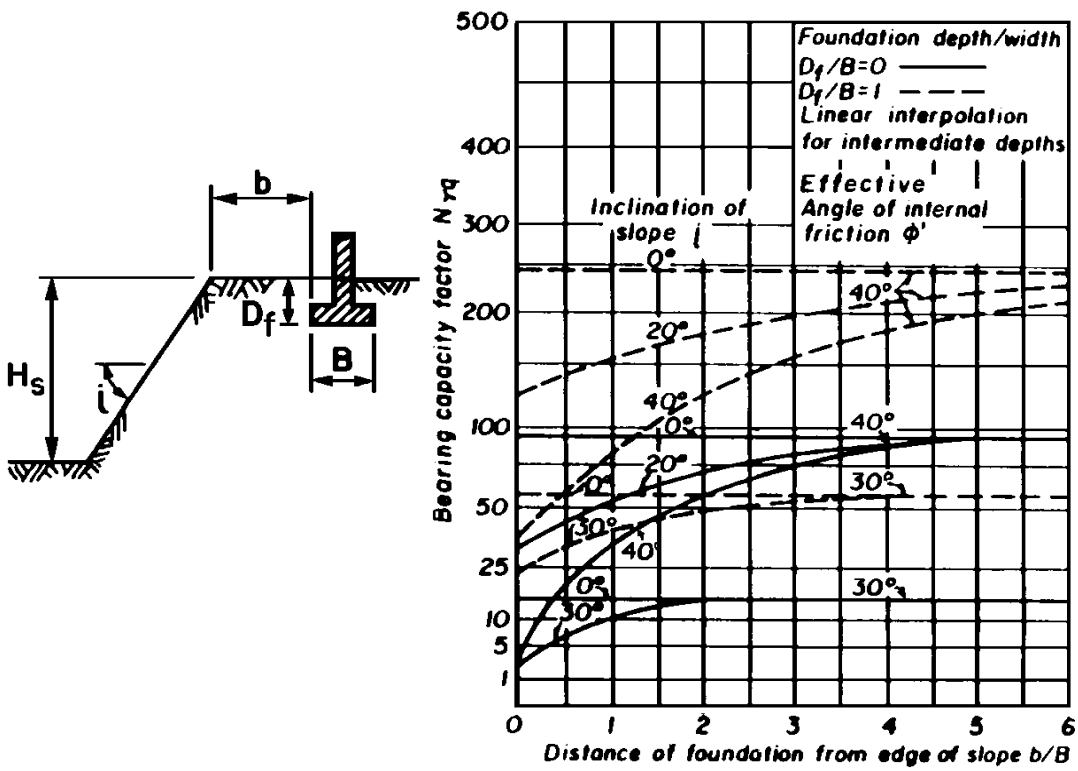


Figure 10.6.3.1.2c-2—Modified Bearing Capacity Factors for Footing in Cohesionless Soils and on or adjacent to Sloping Ground after Meyerhof (1957)

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10.6.3.1.2e—Two-Layered Soil Systems in Undrained Loading

C10.6.3.1.2e

Replace equations C10.6.3.1.2e-5 and C1.0.6.3.1.2e-6 with the following:

- For circular or square footings:

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

$$N^*_c = 6.17$$

- For strip footings:

$$\beta_m = \frac{B}{2\,H_{s2}} \qquad \text{(C10.6.3.1.2e-6)}$$

$$N^*_C = 5.14$$

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Replace H with H_{s2} in Figure 10.6.3.1.2e-2,

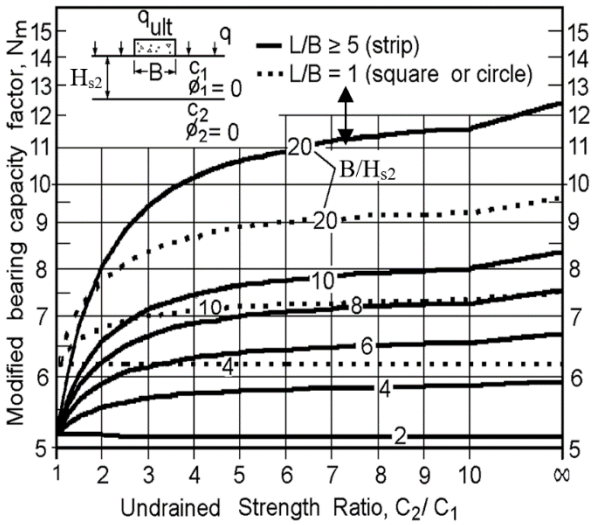

Figure 10.6.3.1.2e-2—Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (EPRI, 1983)

10.6.3.1.2f—Two –Layered Soil System in Drained Loading

Replace Eq. 10.6.3.1.2f-1 with the following:

$$q_n = \left[q_2 + \left(\frac{1}{K}\right) c'_1 \cot \varphi'_1\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\phi'_1 \left(\frac{H_{s2}}{B}\right)} - \left(\frac{1}{K}\right) c'_1 \cot \varphi'_1 \qquad (10.6.3.1.2f\text{-}1)$$

C10.6.3.1.2f

Replace Eq. C10.6.3.1.2f-1 with the following:

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

Replace the title for 10.6.3.1.3 with the following:

10.6.3.1.3—Semiempirical Procedures for Cohesionless Soils.

C10.6.3.1.3

Add the following to the end of the commentary:

It is recommended that the *SPT* based method not be used.

10.6.3.2.1—General

C10.6.3.2.1

Replace the 1st paragraph with the following:

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

Replace the article and title for *10.6.3.2.4* with the following:

10.6.3.2.4—Plate Load Test

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

10.6.3.3—Eccentric Load Limitations

Replace the article with the following:

The factored nominal bearing resistance of the effective footing area shall be equal to or greater than the factored bearing stress.

C10.6.3.3

Replace the commentary with the following:

Excessive differential contact stress due to eccentric loading can cause a footing to rotate excessively leading to failure. To prevent rotation, the footing must be sized to provide adequate factored bearing resistance under the vertical eccentric load that causes the highest equivalent uniform bearing stress. As any increase in eccentricity will reduce the effective area of the footing (on soil), or will increase the maximum bearing stress (on rock), bearing resistance check for all potential factored load combinations will ensure that eccentricity will not be excessive.

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

Replace the 1st paragraph with the following:

Center-to-center spacing shall not be less 36.0 in. or 2.0 pile diameters (whichever is greater). The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 in. or 0.5 pile diameters (whichever is greater). For abutments without a pile cap, the distance from the side of any pile to the nearest edge of the abutment shall not be less than 6.0 inches.

Replace the 2nd paragraph with the following:

If the pile is attached to the cap by embedded bars or strands, the pile shall extend no less than 3.0 in. into the cap for concrete piles and 5 in. into the cap for steel piles.

10.7.1.4—Battered Piles

Add the following at the end of the article:

Battered piles shall not be used at foundations of bents and piers in class S2 soil.

Battered piles may be considered for use at foundations of bents and piers in class S1 soil with approval from Owner.

10.7.1.5—Pile Design Requirements

Replace the article with the following:

Pile design shall address the following issues as appropriate:

- Pile cut off elevation, type of pile, and size and layout of pile group required to provide adequate support, with consideration of subsurface conditions, loading, constructability, and how nominal bearing pile resistance will be determined in the field.
- Group interaction.
- Pile quantity estimation and estimated pile penetration to meet nominal axial resistance and other design requirements.
- Uplift, lateral loads, scour, downdrag, liquefaction, lateral spreading, and other seismic conditions.
- Foundation deflection to meet the established movement and associated structure performance criteria.
- Minimum pile penetration necessary to satisfy the requirements caused by settlement, uplift and lateral loads.
- Pile foundation nominal structural resistance.
- Pile foundation buckling and lateral stability
- 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.

10.7.2.2—Tolerable Movements

Replace the article with the following:

The provisions of Article 10.5.2.2 shall apply

C10.7.2.2

Replace the commentary with the following:

See Article C10.5.2.2

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10.7.2.3—Settlement

C10.7.2.3

Add the following commentary:

Since most piles are placed as groups, estimation of settlement is more commonly performed for pile groups than a single pile. The equivalent footing or the equivalent pier methods may be used to estimate pile group settlement.

The short-term load-settlement relationship for a single pile can be estimated by using procedures provided by Poulos and Davis (1974), Randolph and Wroth, (1978) and empirical load-transfer relationship or skin friction t-z curves and base resistance q-z curves. Load transfer relationships presented in API (2003) and in Article 10.8.2.2.2 can be used. Long-term or consolidation settlement for a single pile may be estimated according to the equivalent footing or pier method.

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Replace the title for 10.7.2.3.2 with the following:

10.7.2.3.2—Pile Group Settlement

Replace the 1st paragraph with the following:

Shallow foundation settlement estimation procedures in Article 10.6.2.4 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.

Replace the 1st sentence of the 2nd paragraph with the following:

The settlement of pile groups in homogeneous cohesionless soil~~s~~ deposits not underlain by more compressible soil at deeper depth may be taken as:

Using SPT: $$\rho = \frac{qI\sqrt{B}}{N1_{60}} \quad (10.7.2.3.2\text{-}1)$$

Using CPT: $$\rho = \frac{qBI}{2q_c} \quad (10.7.2.3.2\text{-}2)$$

in which:

$$I = 1 - 0.125\frac{D'}{B} \geq 0.5 \quad (10.7.2.3.2\text{-}3)$$

where:

ρ = settlement of pile group (in.)

q = net foundation pressure; 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. For friction piles, this pressure is applied at two-thirds of the pile embedment depth, D_b, in the cohesionless bearing layer. For a group of end bearing piles, this pressure is applied at the elevation of the pile tip. (ksf)

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

I = influence factor of the effective group embedment (dim)

D' = effective depth taken as $2D_b/3$ (ft)

D_b = depth of embedment of piles in the cohesionless layer that provides support (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_e = static cone tip resistance (ksf)

Replace the 4th paragraph with the following:

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.

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10.7.2.4—Horizontal Pile Foundation Movement

Replace Table 10.7.2.4-1 with the following:

Table 10.7.2.4-1—Pile P-Multipliers, P_m for Multiple Row Shading

Pile *CTC* spacing (in the Direction of Loading)	P-Multipliers, P_m		
	Row 1	Row 2	Row 3
2.0*B*	0.60	0.35	0.25
3.0*B*	0.75	0.55	0.40
5.0*B*	1.0	0.85	0.70
7.0*B*	1.0	1.0	0.90

Replace the 7th paragraph with the following:

Loading direction and spacing shall be taken as defined in Figure 10.7.2.4-1. A P-multiplier of 1.0 shall be used for pile *CTC* spacing of 8*B* or greater. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the Figure), a P-multiplier of less than 1.0 shall only be used if the pile spacing is 4*B* or less. A P-multiplier of 0.80, 0.90 and 1.0 shall be used for pile spacing of 2.5*B*, 3*B* and 4*B*, respectively.

C10.7.2.4

Replace the 8th paragraph with the following:

The multipliers on the pile rows are a topic of current research and may change in the future. Values from recent research have been compiled in Reese and Van Impe (2000), Caltrans (2003), Hannigan et al. (2006), and Rollins et al (2006).

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10.7.2.5—Settlement Due to Downdrag

Replace the article with the following:

The effects of downdrag, if present, shall be considered when estimating pile settlement under service limit state.

C10.7.2.5

Replace the commentary with the following:

Guidance to estimate the pile settlement considering the effects of downdrag is provided in Meyerhof (1976), Briaud and Tucker (1997) and Hennigan et al (2005).

10.7.3.1—General

Replace the article with the following:

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;
- The specified pile tip elevation to be used in the construction contract document 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 specified tip elevation, including any soil/pile side resistance that will not contribute to the long-term nominal bearing resistance of the pile, e.g., surficial soft or loose soil layers, soil contributing to downdrag, or soil that will be removed by scour;
- The drivability of the selected pile to the specified tip elevation 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.

C10.7.3.1

Replace the 1st paragraph with the following:

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.

Replace the title of article 10.7.3.3 with the following:

10.7.3.3—Pile Length Estimates

Replace the article with the following:

Subsurface geotechnical information combined with static analysis methods (Article 10.7.3.8.6), preconstruction test pile programs (Article 10.7.9), and/or pile load tests (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 the specified tip elevation and estimating contract pile quantities.

C10.7.3.3

Replace the 1st and 2nd paragraphs with the following:

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 specified pile tip elevation 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 test piles across the site may be used to determine variable pile order lengths. Test piles are particularly useful when driving concrete piles. The specified pile tip elevation 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.

Delete the 4th paragraph.

Replace the 5th paragraph with the following:

Where piles are driven to a well defined firm bearing stratum, the location of the top of the bearing stratum will dictate the pile length needed.

Delete the 6th paragraph.

Delete the 7th paragraph.

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10.7.3.4.3—Setup

C10.7.3.4.3

Replace the 3rd paragraph with the following:

If a wave equation or dynamic formula is used to determine the nominal pile bearing resistance on re-strike, care should be used as these approaches require accurate blow count measurement which is inherently difficult at the beginning of redrive (BOR).

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10.7.3.6—Scour

C10.7.3.6

Replace the 1st paragraph with the following:

The piles will need to be driven to the specified tip elevation and 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.

Replace the 2nd paragraph with the following:

The magnitude of skin friction that will be lost due to scour may be estimated by static analysis. The static analysis used to determine the nominal axial resistance after the scour event must consider the reduction of the effective overburden stresses due to scour. 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 the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

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10.7.3.8.1—General

Replace the article with the following:

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 as specified in Article 10.5.5.2.3. The production piles shall be driven to the specified tip elevation and the minimum blow count determined from the static load test, dynamic test, wave equation, or dynamic formula.

C10.7.3.8.1

Replace the commentary with the following:

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. 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 the specified tip elevation and 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 maybe 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.

10.7.3.8.2—Static Load Test

Replace the 1st paragraph with the following:

If a static pile load test is used to determine the pile axial resistance, the test shall not be performed prior to completion of the pile set up period as determined by the Engineer. The load test shall follow the procedures specified in ASTM D 1143, and the loading procedure should follow the Quick Load Test Procedure.

C10.7.3.8.2

Replace Figure C10.7.3.8.2-1 with the following:

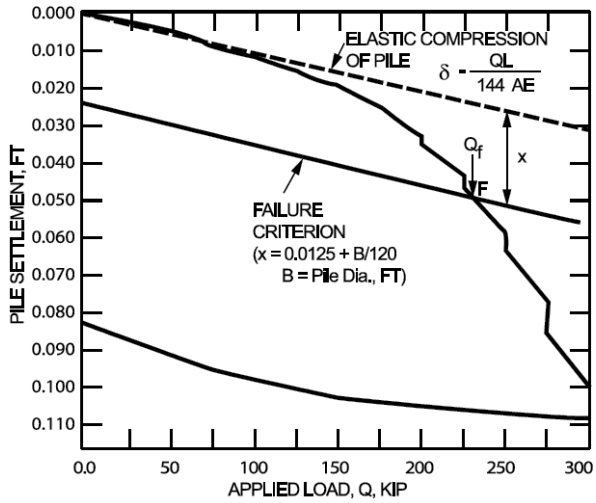

Figure C10.7.3.8.2-1—Davissons' Method for Load Test Interpretation (Cheney and Chassie, 2000, modified after Davisson, 1972).

10.7.3.8.3—Dynamic Testing

Replace the article with the following:

Dynamic testing shall be performed according to the procedures given in ASTM D 4945. If possible, the dynamic test should be performed as a re-strike test if the Engineer anticipates significant time dependent soil strength change. The pile nominal 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.

Dynamic testing shall be calibrated to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.3

Replace the 1st paragraph with the following:

The dynamic test may be used to establish the driving criteria at the beginning of production driving. When dynamic testing is performed on piles up to 36 inches in diameter, 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. Re-strike testing should be performed if setup or relaxation is anticipated. The minimum number of piles that should be tested are as specified by the Engineer.

10.7.3.8.4—Wave Equation Analysis

Add the following to the end of the article:

When the pile nominal resistance is greater than 600 kips or the pile diameter is greater than or equal to 18 inches, the wave equation analysis used for establishing the bearing acceptance criteria shall be based on dynamic test results with signal matching.

The wave equation shall be calibrated to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.

C10.7.3.8.4

Replace the 1st paragraph with the following:

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. The reliability of the predicted pile axial nominal resistance can be improved by selecting the key input parameters based on local experience.

10.7.3.8.5—Dynamic Formula

Replace the 1st paragraph with the following:

If a dynamic formula is used to establish the driving criterion, the following modified Gates Formula (Eq. 10.7.3.8.5-1) shall be used. The nominal pile resistance as measured during driving using this method shall be taken as:

$$R_{ndr} = \left[1.83\sqrt{E_r}\log_{10}(0.83N_b)\right]\text{-}124 \qquad (10.7.3.8.5\text{-}1)$$

where:

R_{ndr} = nominal pile resistance measured during pile driving (kips)

E_r = Manufacturer's rating for energy developed by the hammer at the observed field drop height (ft.-lbs.)

N_b = Number of hammer blows in the last foot, (maximum value to be used for N_b is 96) (blows/ft).

Delete the 2nd paragraph.

Delete the 3rd paragraph.

Replace the 5th paragraph with the following:

Dynamic formulas shall not be used when the required nominal resistance exceeds 600 kips or the pile diameter is greater than or equal to 18-inches.

C10.7.3.8.5

Delete the 2nd paragraph.

Delete the 3rd paragraph.

Replace the 5th paragraph with the following:

As the required nominal bearing resistance increases, the reliability of dynamic formulae tends to decrease. The modified Gates Formula tends to underpredict pile nominal resistance at higher resistances.

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10.7.3.8.6a—General

Add to the end of the article as follows:

The static analysis methods presented in this article shall be limited to driven piles 24 in. or less in diameter (length of side for square piles). For steel pipe and cast-in-steel shell (CISS) piles larger than 18 inches in diameter, the static analysis methods from the American Petroleum Institute (API, 2000) publication RP 2A shall be used.

C10.7.3.8.6a

Delete the entire commentary.

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10.7.3.10—Uplift Resistance of Single Piles

Replace the 1st paragraph with the following:

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 in uplift or tension.

Replace the 2nd paragraph with the following:

The uplift resistance of a single pile should be estimated in a manner similar to that for estimating the skin friction resistance of piles in compression specified in Article 10.7.3.8.6, and when appropriate, by considering reduction due to the effects of uplift.

C10.7.3.10

Add the following before the 1st paragraph as follows:

In general, piles may be considered to resist a transient, but not sustained, uplift load by skin friction.

Replace the 2nd paragraph with the following:

See Hannigan et al (2006) for guidance on the reduction of skin friction due to the effects of uplift.

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Replace the 5th paragraph of the article with the following:

The static pile uplift load test(s), when performed, 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, when available.

10.7.3.11—Uplift Resistance of Pile Groups

Replace the 4th paragraph with the following:

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. The nominal uplift resistance of the pile group when considered as a block shall be taken as equal to the weight of this soil block. Buoyant unit weights shall be used for soil below the groundwater level. In this case, the resistance factor φ_{ug} in Eq. 10.7.3.11-1 shall be taken as equal to 1.0.

Delete the 6th and 7th paragraphs.

C10.7.3.11

Add the following to the end of the commentary:

In cohesionless soils, the shear resistance around the perimeter of the soil block that will be uplifted is ignored. This results in a conservative estimate of the nominal uplift resistance of the block and justifies the use of a higher resistance factor of 1.0.

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10.7.3.13.1—Steel Piles

Add to the end of the article:

Shear rings are required in CISS piles and drilled shafts with permanent casing to ensure composite action.

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Replace the title of Article 10.7.5 with the following:

10.7.5—Protection Against Corrosion and Deterioration

Replace the 3rd paragraph with the following:

Soil, water, or site conditions that have a minimum resistivity equal to or less than 1100 ohm-cm shall be considered as indicators of potential pile corrosion or deterioration.

Delete the 4th paragraph.

Add the following after the 3rd paragraph:

A site is considered corrosive if one or more of the following soil, water, or site conditions exist:

- chloride concentration equal to or greater than 500ppm,
- sulfate concentration equal to or greater than 1500ppm,
- pH equal to or less than 5.5.

Steel piling may be used in corrosive soil and water environments provided that adequate corrosion mitigation measures are specified. When increased steel area is used for corrosion protection, the following corrosion rates shall be used to determine the corrosion allowance (sacrificial metal loss):

- 0.001 in. per year for soil embedment zone,
- 0.0015 in. per year for fill or disturbed natural soils,
- 0.002 in. per year for atmospheric zone (marine),
- 0.004 in. per year for immersed zone (marine),
- 0.006 in. per year for splash zone.

Designer must consider site specific corrosion rate for steel piling in scour zones.

The corrosion rates used to determine the corrosion allowance for steel piling shall be doubled for steel H-piling since there are two surfaces for the web and flanges that would be exposed to the corrosive environment.

C10.7.5

Replace the 9th paragraph with the following:

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.

Delete the 10th paragraph.

10.8.1.1—Scope

C10.8.1.1

Add the following after the 2nd paragraph:

When casing is used to stabilize the soil within the excavation for construction of a Cast-In-Drilled Hole (CIDH) concrete pile, the method of installation will influence how the pile is designed and the resulting side and tip resistance. Special consideration shall be given to cases where oscillator or rotator drill equipment is used to construct CIDH concrete piles. Steel pipe (sometimes referred to as "casing") advanced into the ground using oscillator or rotator drilling equipment in most cases should be considered equivalent to large drilling rod with drilling teeth at the tip of the steel "casing." The drill teeth typically extend out slightly beyond the diameter of the oscillator or rotator drill rod resulting in a drilled hole larger than the outside diameter of the drill rod that is not "tight" in the hole. When oscillators and rotators are used to excavate earth materials, cuttings are produced outside of the drill rod (aka "casing") that are not typically removed during the drilling process or during the drill rod removal process, which can result in cuttings be trapped between the sidewalls of the excavations and the concrete of the pile.

In situations, where relatively large side resistance is relied upon in the design of CIDH concrete pile (e.g., IGM or rock), the contract specifications need to provide adequate requirements to ensure that the side resistance in the rock is not significantly reduced due to the use of oscillator/rotator drilling equipment to construct the pile. Some examples would include: 1) prohibiting the use of the oscillator/rotator drill rod in the rock socket portion of a CIDH concrete pile, or 2) after reaching the pile tip elevation, pull the rotator/oscillator drill rod up to the top of the rock and remove cuttings from the sidewalls of the excavation by other means prior to constructing the pile.

Studies have shown cases where significant reduction in side resistance of a CIDH concrete pile socketed in rock when an oscillator/rotator drill rod was used to construct the pile. The significant reduction in side resistance was presumably due to cuttings trapped between the concrete and rock along the sidewall of the excavation due to the method of installation. For further discussion regarding this topic, refer to Section 6 of the *Drilled Shafts: Construction Procedures and LRFD Design Methods* (Brown et al 2010) or the article titled "Deep Foundation Challenges At The New Benicia-Martinez Bridge" by the American Society of Civil Engineers (2004).

There have been a number of situations on projects in California where large ground subsidences have developed at the ground surface that was supporting the oscillator/rotator drilling equipment as a result of the means and methods used by the drilling contractor. To help prevent these situations, it is recommended that construction specifications be included in the contract documents to address these issues and the pile placement plans contain specific measure to avoid these issues. These include but are not limited to: 1) maintaining an adequate positive fluid head in wet excavations, 2) using only the approved slurry in wet excavations, 3) maintaining a soil plug (i.e. 10 ft) at the tip of the oscillator/rotator drill rod during excavation of the pile, and 4) specifying that contractors provide access to the top of the oscillator/rotator drill rod (i.e boom lift), so that inspectors can inspect the fluid head and monitor the progress of the excavation.

10.8.1.2—Shaft Spacing, Clearance, and Embedment Into Cap

Replace the 1st paragraph with the following:

The center-to-center spacing of drilled shafts in a group shall be not less than 2.5 times the shaft diameter. If the center-to-center spacing of drilled shafts is less than 4.0 diameters, the sequence of construction shall be specified in the contract documents.

C10.8.1.2

Delete the commentary.

Add after the first paragraph of the article:

For abutments without a pile cap, the distance from the side of the shaft to the nearest edge of the abutment shall not be less than 6.0 inches.

Replace the article and title for Article 10.8.1.3 with the following:

10.8.1.3—Shaft Diameter, Concrete Cover, Rebar Spacing, 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 order to facilitate construction of the drilled shafts (CIDH Piles), the minimum concrete cover to reinforcement shall be as specified in Table 10.8.1.3-1. For shaft capacity calculations, only 3” of cover is assumed effective and shall be used in calculations.

Table 10.8.1.3-1—Minimum Concrete Cover for Drilled Shafts (CIDH Piles)

Diameter of the Drilled Shaft (CIDH Pile) “D”	Side Concrete Cover
16” and 24” Standard Plan Piles	Refer to the applicable Standard Plans
24” ≤ D ≤ 36”	3”
42” ≤ D ≤ 54”	4”
60” ≤ D < 96”	5”
96” and larger	6”

In order to improve concrete flow when constructing drilled shafts, a 5 in. x 5 in. clear window between the horizontal and vertical shaft reinforcing steel shall be maintained, except at the locations of the inspection pipes where the minimum clear spacing between the longitudinal reinforcing bars and the inspection pipes is 3.0 in.

The maximum center-to-center spacing of longitudinal bars in drilled shafts (CIDH Piles) is limited to 10 in. when the shaft diameter is less than 5 ft., and 12 in. for larger shafts, except at the locations of inspection pipes where 8.5 in of clear spacing between the main longitudinal bars is required.

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.

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.

C10.8.1.3

Replace the 3rd paragraph with the following:

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 8.0 in. smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment.

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10.8.2.2.2—Settlement of Single-Drilled Shaft

Add the following to the end of the article:

Superstructure tolerance to support movements shall be verified for the displacements assumed in the geotechnical design of the shaft at the strength limit states.

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10.8.3.5.1b—Side Resistance

Replace the 3rd paragraph with the following:

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. Method of installation of a steel casing will dictate what design method shall be used in determining side resistance for the drilled shaft and casing portion of the pile. If a corrugated metal pipe is placed in an oversized hole and the annular space is properly backfilled with concrete or grout (e.g., tremie methods in wet conditions), use equation 10.8.3.5.1b-1 for estimating side resistance without reduction factors as long as concrete or grout placed in the annular space can be verified in the field to the satisfaction of the geotechnical designer. Smooth-wall steel casings installed by vibratory methods, oscillatory methods, rotational methods or placed in an excavated oversized hole shall not use equation 10.8.3.5.1b-1.

C10.8.3.5.1b

Replace the 3rd paragraph with the following:

Steel casing will generally reduce the side resistance of a cast-in-drilled hole concrete pile also known as a drilled shaft. No specific data is available regarding the reduction in skin friction of a drilled shaft in cohesive soil resulting from the use of permanent casing relative to concrete placed directly against the soil when the casing is vibrated, oscillated or rotated into the soil. Interface shear resistance for steel against cohesive soil can vary from 50 to 75 percent of the interface shear resistance for poured in place concrete against cohesive soil, depending on whether the steel is clean or rusty, respectively (Potyondy, 1961).

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10.8.3.5.1c—Tip Resistance

Replace the 1st paragraph with the following:

For axially loaded shafts in cohesive soil, the net nominal unit tip resistance, q_p, in ksf, by the total stress method as provided in Brown et al (2010) shall be calculated as follows:

If $Z \geq 3D$,

$$q_p = N^*_c \, S_u \qquad (10.8.3.5.1c\text{-}1)$$

in which:

Table 10.8.3.5.1c-1—Bearing Capacity Factor N^*_c

Undrained shear strength, S_u (ksf)	N^*_c
0.5	6.5
1	8.0
2 - 5	9.0

Note: For S_u >5 to 50 ksf, use cohesive Intermediate Geomaterial procedures (Article 10.8.3.5.5).

If $Z \geq 3D$,

$$q_p = \left(\frac{2}{3}\right)\left[1+\left(\frac{1}{6}\right)\left(\frac{Z}{D}\right)\right] N^*_c \, S_u \qquad (10.8.3.5.1c\text{-}2)$$

where,

D = diameter of drilled shaft (ft)

Z = penetration length of drilled shaft in base cohesive layer (ft)

S_u = design undrained shear strength (ksf)

C10.8.3.5.1c

Delete the 2nd paragraph.

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10.8.3.5.2b—Side Resistance

Replace the article with the following:

The nominal axial resistance of drilled shafts in cohesionless soils by the β-method shall be taken as:

$$q_s = \beta\sigma'_v \leq 4.0 \quad \text{for} \quad 0.25 \leq \beta \leq 1.2 \qquad (10.8.3.5.2b\text{-}1)$$

in which, for sandy soils:

- $N_{60} \geq 15$:

$$\beta = 1.5 - 0.135\sqrt{z} \qquad (10.8.3.5.2b\text{-}2)$$

- $N_{60} < 15$:

$$\beta = \frac{N_{60}}{15}\left(1.5 - 0.135\sqrt{z}\right) \qquad (10.8.3.5.2b\text{-}3)$$

where:

σ'_v = vertical effective stress at soil layer mid-depth (ksf)
β = load transfer coefficient (dim)
z = depth below ground, at soil layer mid-depth (ft)
N_{60} = average *SPT* blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft)

Higher side resistance values may be used if verified by load tests.

For gravelly sands and gravels, Eq. 10.8.3.5.2b-4 should be used for computing β where $N_{60} \geq 15$. If $N_{60} < 15$, Eq. 10.8.3.5.2b-3 should be used.

$$B = 2.0 - 0.06(z)^{0.75} \qquad (10.8.3.5.2b\text{-}4)$$

When permanent casing is used, the method of installation of a steel casing will dictate what design method shall be used in determining side resistance for the cased portion of the drilled shaft. If a corrugated metal pipe is placed in an oversized hole and the annular space is properly backfilled with grout (e.g., tremie methods), use equation 10.8.3.5.2b-1 for estimating side resistance without reduction to the side resistance. Smooth-wall steel casings installed by vibratory methods, oscillatory methods, rotational methods or placed in an excavated oversized hole shall not use the equations in 10.8.3.5.2b.

C10.8.3.5.2b

Replace the 1st paragraph with the following:

O'Neill and Reese (1999) provide additional discussion of computation of shaft side resistance and recommend allowing β to increase to 1.8 in gravels and gravelly sands, however, they recommend limiting the unit side resistance to 4.0 ksf in all soils.

O'Neill and Reese (1999) proposed a method for uncemented soils that uses a different approach in that the shaft resistance is independent of the soil friction angle or the SPT blow count. According to their findings, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.

The detailed development of Eq. 10.8.3.5.2b-4 is provided in O'Neill and Reese (1999).

The design method by Chen and Kulhawy (2002) provides an alternate approach to calculating side resistance for drilled shafts. The design method was shown to be very sensitive to soil type and allowed for a reduction of 2/3 to the horizontal stress coefficient when construction quality was not properly controlled. For these reasons, the Chen and Kulhawy (2002) design method can be considered for use only if verified with load tests.

Replace the 2nd paragraph with the following:

Steel casing will generally reduce the side resistance of a cast-in-drilled-hole concrete pile also known as a drilled shaft. No specific data is available regarding the reduction in skin friction for drilled shafts in cohesionless soil resulting from the use of permanent casing relative to concrete placed directly against the soil when the casing is vibrated, oscillated or rotated into the soil. Interface shear resistance for steel against cohesionless soil can vary from 50 to 75 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.

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10.8.3.6.3—Cohesionless Soil

Replace Table 10.8.3.6.3-1 with the following:

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	2.5*D*		0.95
	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	2.5*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	2.5*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

10.8.3.7.2—Uplift Resistance of Single Drilled Shaft

Replace the 1st paragraph with the following:

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, and, when appropriate, by considering reduction due to effects of uplift.

C10.8.3.7.2

Replace the 1st paragraph with the following:

The side resistance for uplift is lower than that for axial compression. One reason for this is that drilled shafts in tension unload soils, thus reducing the overburden effective stress and hence the uplift side resistance of the drilled shaft.

10.8.3.9.3—Reinforcement

Replace 1st paragraph with the following:

Where the potential for lateral loading is insignificant, drilled shafts may be reinforced for axial load 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. For drilled shafts with a diameter larger than 24 inches, reinforcing steel shall extend 6 inches above the pile specified tip elevation. For *Standard Plan* CIDH piles, the cover to reinforcing steel shall be as shown on the plans.

10.8.3.9.4—Transverse Reinforcement

Add a new paragraph to the end of the article:

The design shear force demand in CIDH shafts and rock sockets need not be more than two and a half times the seismic overstrength shear force of the column: $V_u \leq 2.5V_o$

Add a new commentary:

C10.8.3.9.4

Caltrans policy imposes an upper limit on the design shear force, recognizing the general problem of unrealistic shear magnification due to abrupt changes in stiffness, and discretization of distributed soil reaction at nodal points in rock.

Replace the article and title for Article 10.9.1.2 with the following:

10.9.1.2—Maximum Micropile Diameter and Minimum Micropile Spacing, Clearance, and Embedment into Cap

Center-to-center spacing of micropiles shall 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. The diameter of the micropile drilled hole shall not be greater than 13 inches.

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10.9.3.5.4—Micropile Load Test

Delete the article in its entirety.

C10.9.3.5.4

Delete the article in its entirety.

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10.10—REFERENCES

Add the following reference:

Gu. R. X., et. al. 2004 “Deep Foundation Challenges At The New Benicia-Martinez Bridge.” In *Geotechnical Engineering for Transportation Projects,* Geotechnical Special Publication No. 126. American Society of Civil Engineers. pp. 1183-1191.

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