10.5.1 General

Revise as follows:

The limit states shall be as specified in Article 1.3.2; geotechnical foundation design ~~specific~~ provisions are contained in this section.

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

Foundations for end supports shall be designed using the SERVICE-I LIMIT STATE loads, as provided in these Specifications, and the Working Stress Design (WSD) method provided in the *Caltrans Bridge Design Specifications (2000)*, dated November 2003.

C10.5.1

Add a 1st Paragraph as follows:

In general, “intermediate supports” and “end supports” refer to bents/piers and abutments, respectively.

10.5.2.1 General

Revise the 1st Paragraph as follows:

Foundation design at the service limit state shall include:

- Settlements
- Horizontal movements
- Overall stability, and
- Total s~~S~~cour at the base ~~design~~ flood

10.5.2.2

Add paragraph as follows:

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.

Limit eccentricity under Service Limit State loading to B/6 and B/4 when spread footings are founded on soil and rock, respectively.

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.

C10.5.2.1

Revise the 3rd Paragraph as follows:

The base ~~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.

10.5.3.1 General

Revise the 2nd Paragraph as follows:

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

- structural resistance and
- loss of lateral and <u>axial</u> ~~vertical~~ support due to scour at the <u>base</u> ~~design~~ flood event.

C10.5.3.1

Revise the 4th Paragraph as follows:

The <u>base flood</u> ~~design event~~ for scour is defined in <u>Article 2.6</u> ~~Section 2~~ and is specified in Article 3.7.5 as applicable at the strength limit state.

C10.5.4

Revise the 2nd Paragraph as follows:

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. Scour should be considered with extreme events as per Article 3.4.1

10.5.5.1 Service Limit States

Revise the 2nd Paragraph as follows:

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 base ~~design~~ flood.

10.5.5.2.1 General

Revise the 1st Paragraph as follows:

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, ~~and~~ 10.5.5.2.4, and 10.5.5.2.5. ~~unless regionally specific values or substantial successful experience if available to justify higher values.~~

C10.5.5.2.1

Revise as follows:

~~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. When a single pile or drilled shaft supports a bridge column, reduction of the resistance factors in Articles 10.5.5.2.3, and 10.5.5.2.4 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 ~~sand or clay~~. Many n~~N~~aturally occurring soils do not fall neatly into these two classifications. In general, the terms ~~“sand” and~~ “cohesionless soil” or “sand” may be connoted to mean drained conditions during loading, while ~~“clay” or~~ “cohesive soil” or “clay” implies undrained conditions in the short term. For other or intermediate soil classifications, such as clayed sand or silts ~~or gravels~~, 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 ~~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.~~

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

10.5.5.2.2 Spread Footings

Revise the 1st Paragraph as follows:

The resistance factors provided in Table 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.~~

C10.5.5.2.1

Revise as follows:

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. ~~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, and 10.5.5.2.4. 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 base ~~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.

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

Revise as follows:

Nominal Resistance	Resistance Determination Method/~~Soil/~~Conditions		Resistance Factor
Bearing ~~Resistance~~ in Compression	φ_b	Theoretical method (*Munfakh et al., 2001*), in ~~clay~~ cohesive soils	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

10.5.5.2.3 Driven Piles

Delete the entire Article 10.5.5.2.3 and replace with the following:

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

C10.5.5.2.3

Delete the entire Commentary to Article 10.5.5.2.3 and replace with the following:

The resistance factors in Table 10.5.5.2.3-1 are based on engineering judgment, and past WSD and Load Factored Design (LFD) practices.

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Table 10.5.5.2.3-1 Resistance Factors for Driven Piles

Replace Table 10.5.5.2.3-1 with the following:

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 5.5.4.2.1
	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 5.5.4.2.1	
	Timber Piles	See the provisions of Article 8.5.2.2 and 8.5.2.3	

10.5.5.2.4 Drilled Shafts

Delete the entire Article 10.5.5.2.4 and replace with the following:

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

C10.5.5.2.4

Delete the entire Commentary to Article 10.5.5.2.4 and replace with the following:

The resistance factors in Table 10.5.5.2.4-1 are based on engineering judgment, and past WSD and LFD practice.

The maximum value of the resistance factors in Table 10.5.5.2.4-1 are based on an assumed normal level of field quality control during shaft construction. If a normal level of quality control cannot 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.

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

Replace Table 10.5.5.2.4-1 with the following:

Nominal Resistance	Resistance Determination Method/Conditions	Resistance Factor	
Axial Compression and Tension or U~~u~~plift	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 10.5.5.3.2

C10.5.5.3.2

Revise the 1st Paragraph as follows:

~~The axial nominal strength after scour due to the check flood must be greater than the unfactored pile or shaft load for the Strength Limit State loads. 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.~~ See Commentary to Article 3.4.1, Extreme Events, and Article 3.7.5.

10.5.5.3.3 Other Extreme Event Limit States

Revise the 1st Paragraph as follows:

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

C10.5.5.3.3

Delete the entire Commentary to Article 10.5.5.3.3

10.6.1.1 General

Revise the 1st Paragraph as follows:

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

Revise the 2nd Paragraph as follows:

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. Alternatively, the unsuitable material can be removed and replaced with suitable and properly compacted engineered fill material, or improved in place, at reasonable coast as compared to other foundation support alternatives.

10.6.1.3 Effective Footing Dimension

Revise as follows:

For eccentrically loaded footings, a reduced effective area, B' x L', within the confines of the physical footing shall be used in geotechnical design for settlement and ~~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 = M_L/V = eccentricity parallel to dimension B (ft.)

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

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

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

V = vertical load (kips)

C10.6.1.3

Add a 3rd Paragraph as follows:

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

10.6.1.4 Bearing Stress Distributions

Revise the 1st Paragraph as follows:

When proportioning footings 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 as:

- Uniform over the effective area for footing on soils, or

10.6.1.6 Groundwater

Revise the 2nd Paragraph as follows:

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

10.6.2.4.1 General

Revise the 6th Paragraph as follows:

The distribution of vertical stress increase below ~~circular or~~ square and long rectangular footings, i.e., where $L > 5B$, may be estimated using Figure 1.

C10.6.2.4.1

Add a 7th Paragraph as follows:

<u>For eccentrically loaded footings, replace *L* and *B* in these specifications with the effective dimensions *L'* and *B'* respectively.</u>

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10.6.2.4.2 Settlement of Footings on Cohesionless Soils

Revise the 3rd Paragraph as follows:

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

C10.6.2.4.2

Revise the 6th Paragraph as follows:

~~The stress distributions used to calculate elastic settlement assume the footing is flexible and supported on a homogeneous soil of infinite depth.~~ In Table 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 footings, 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.

Modify the last sentence in the 8th Paragraph as follows:

Therefore, in selecting an appropriate value for soil modulus, considerations should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent foundations ~~footings~~.

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10.6.2.4.2 Settlement of Footings on Cohesionless Soils

Revise the Last Paragraph as follows:

In Figure 1, N1 ~~N'~~ shall be taken as $(N1)_{60}$, 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 Footings on Cohesive Soils

Insert the following after the 1st Paragraph:

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

10.6.2.4.3 Settlement of Footings on Cohesive Soils

Insert the following under Figure 10.6.2.4.3-3:

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

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C10.6.3.1.2e

Revise equations 5 and 6 as follows:

- For circular or square footings:

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

$$N_c^* = 6.17$$

- For strip footings:

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

$$N_c^* = 5.14$$

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10.6.3.1.2e Two-Layered Soil System in Undrained Loading

Revise Figure 10.6.3.1.2e-2 as follows:

Replace H with H_{s2}.

10.6.3.1.2f Two-Layered Soil System in Drained Loading

Revise equation 1 as follows:

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

(10.6.3.1.2f-1)

C10.6.3.1.2f

Revise equation 1 as follows:

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

10.6.3.1.3 Semiempirical Procedures

Revise as follows:

10.6.3.1.3 Semiempirical Procedures for Cohesionless Soils

C10.6.3.1.3

Add the following to the end:

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

C10.6.3.2.1

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 strength limit state before checking the movements ~~nominal bearing resistance~~ at ~~both~~ the service ~~and strength~~ limit state~~s~~.

10.6.3.2.4 Load Test

Revise as follows:

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

Revise as follows:

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

- ~~One-fourth of the corresponding footing dimension, *B* or *L*, for footings on soils, or~~
- ~~Three-eighths of the corresponding footing dimensions *B* or *L*, for footings on rock.~~

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

C10.6.3.3

Revise as follows:

~~A comprehensive parametric study was conducted for cantilevered retaining walls of various heights and soil conditions. The base widths obtained using the LRFD load factors and eccentricity of *B*/4 were comparable to those of ASD with an eccentricity of *B*/6.~~

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.

10.6.3.4 Failure by Sliding

Revise Figure 10.6.3.4-1 as follows:

Replace Q_τ with R_τ

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10.7.1.2 Minimum Pile Spacing, Clearance, and Embedment into Cap

Revise the 1st and 2nd Paragraphs as follows:

Center-to-center pile spacing should not be less than 36.0 ~~30.0~~ in. or 2.0 ~~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 or 0.5 pile diameters.

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 3.0 ~~6.0~~ in. into the cap.

10.7.1.5 Pile Design Requirements

Revise as follows:

Pile design shall address the following issues as appropriate:

- Pile cut off elevation, ~~nominal axial resistance to be specified in the contract,~~ type of pile, and size and layout of pile group required to provide adequate support, with considerations of subsurface conditions, loading, constructability and how nominal axial pile 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 u~~ Uplift, lateral loads, scour, downdrag, ~~settlement,~~ liquefaction, lateral spreading ~~loads~~, 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.
- Verification of pile drivability to confirm that acceptable driving stresses and blow counts can be achieved with an available driving system to meet all contract acceptance criteria.
- Long-term durability of the pile in service, i.e. corrosion and deterioration.

C10.7.1.6.2

Revise the 1st two Paragraphs as follows:

Static downdrag does not affect the ultimate geotechnical capacity or nominal resistance of the pile foundations. It acts to increase pile settlement, and load on the pile or pile group and the cap. ~~Downdrag occurs when settlement of soils along the side of the piles results in downward movement of the soil relative to the pile. See commentary to Article C3.11.8~~

In the case of friction piles with limited tip resistance, the downdrag ~~load~~ can ~~exceed the geotechnical resistance of the pile,~~ caus~~ing~~e 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 pile bearing below the downdrag zone, service limit state tolerances ~~will~~ may govern the geotechnical design of piles subjected to downdrag.

10.7.2.2 Tolerable Movements

Revise as follows:

The provisions of Article 10.5.2.~~1~~2 shall apply.

C10.7.2.2

Revise as follows:

See Article C10.5.2.~~1~~2.

C10.7.2.3

Add the following:

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.

10.7.2.3.2 Pile Groups in Cohesive Soil

Revise as follows:

10.7.2.3.2 Pile Group~~s~~ Settlement ~~in Cohesive Soil~~

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~~.1~~ or Figure 10.7.2.3.1-2.

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

where:

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

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10.7.2.3.2 Pile Groups in Cohesive Soil

Revise as follows:

D_b = depth of embedment of piles in the cohesionless layer that provides support~~, as specified in Figure 10.7.2.3.1-1~~ (ft.)

Revise the 4th Paragraph as follows:

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. 1 and 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

Revise as follows:

Table 10.7.2.4-1 Pile *P*-Multipliers, P_m for Multiple Row Shading (~~averaged from *Hannigan et al., 2005*~~).

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
2.0*B*	0.60	0.35	0.25
3.0*B*	0.75	0.55	0.40 ~~0.35~~
5.0*B*	1.0	0.85	0.7
7.0*B*	1.0	1.0	0.90

Revise the 7th Paragraph as follows:

Loading direction and spacing shall be taken as defined in Figure 1. A *P*-multiplier of 1.0 shall be used for pile CTC spacing of 8B 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 ~~group reduction factor~~ of less than 1.0 should only be used if the pile spacing is 4~~5~~*B* or less~~, i.e., a P_m of 0.7 for a spacing of 3*B*, as shown in Figure 1.~~ 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

Revise the 8th Paragraph as follows:

The multipliers on the pile rows are a topic of current research and may change in the future. Values from recent research have been ~~tabulated by~~ presented in Reese and Van Impe (*2000*), Caltrans (*2003*), Hannigan et al. (*2005*), and Rollins et al. (*2006*). ~~Averaged values are provided in Table 1.~~

10.7.2.5 Settlement Due to Downdrag

Delete the entire Article and replace with:

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

C10.7.2.5

Delete the entire Commentary and replace with:

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

Revise as follows:

For strength limit state design, the following shall be determined:

- Loads and performance requirements;
- Pile type, dimensions, and nominal axial pile resistance in compression;
- Size and configuration of the pile group to provide adequate foundation support;
- The specified pile tip elevation ~~Estimated pile length~~ to be used in the construction contract documents to provide a basis for bidding;

10.7.3.1 General

Revise as follows:

- 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 ~~minimum pile penetration required, if applicable,~~ including any soil/pile skin friction that will not contribute to the long-term nominal axial resistance of the pile, e.g., surficial soft or loose soil layers, soil contributing to downdrag, or soil that will be removed by scour~~ed away~~;

- The drivability of the selected pile to the specified tip elevation ~~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.

C10.7.3.1

Revise the 1st Paragraph as follows:

A minimum pile penetration should only be specified if needed to ensure 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 10.7.6 for additional details. ~~Assuming dynamic methods e.g., wave equation calibrated to dynamic measurements with signal matching analysis, pile formulae, etc., are used during pile installation to establish when the 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.~~

Revise as follows:

10.7.3.3 Pile Length Estimates ~~for Contract Documents~~

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 for establishment of contract pile quantities. Local experience shall also be considered when making pile quantity estimates, both to select an estimation method and to assess the potential prediction bias for the method used to account for any tendency to over-predict or under-predict pile compressive resistance. 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

Revise the 1st Paragraph as follows:

The estimated pile length required to support the required nominal resistance is determined using a static analysis; knowledge of the site subsurface conditions, and/or results from a pile load test. The specified pile tip elevation or 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.

Delete the entire 3rd Paragraph.

C10.7.3.3

Revise the 4th Paragraph as follows:

~~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.~~ <u>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.</u>

Delete the last Paragraph.

C10.7.3.4.3

Revise the 3rd Paragraph as follows:

If a dynamic formula is used to evaluate pile axial resistance on re-strike, care should be used as these formulae may not be as effective at beginning of redrive (BOR)~~, and furthermore, the resistance factors provided in Table 10.5.5.2.3-1 for driving formulae were developed for end of driving conditions. See Article C10.5.5.2.3 for additional discussion on this issue~~. Higher degrees of confidence are provided by dynamic measurements of pile driving with signal matching analyses or static load tests.

C10.7.3.6

Revise the 2nd Paragraph as follows:

The piles will need to be driven to the specified tip elevation and the required nominal axial resistance plus the side resistance that will be lost due to scour. The resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal axial resistance plus the magnitude of the skin friction lost as a result of scour, considering the prediction method bias.

Revise the 3rd paragraph as follows:

The magnitude of skin friction that will be lost due to scour may be estimated by static analysis. 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. D~~ during the driving of ~~test~~ piles, the skin friction component of the axial resistance of pile in the scourable material may be determined by a signal matching analysis of the dynamic measurements obtained when the pile is tipped below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

C10.7.3.7

Add the following to the end of the Article:

Additional guidance to estimate downdrag on single pile and pile groups are provided in ASCE (*1993*), Briaud and Tucker (*1997*), and Hennigan et al. (*2005*).

10.7.3.8.1 General

Revise as follows:

Pile nominal axial resistance should be field verified during pile installation using load tests, dynamic tests, wave equation or dynamic formula. The resistance factor selected for design shall be ~~based on the method used to verify pile axial resistance~~ 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 formula used. ~~unless a deeper penetrations is required due to uplift, scour, lateral resistance, or other requirements as specified in Article 10.7.6. If it is determined that dynamic methods are unsuitable for field verification of nominal axial resistance, and a static analysis method is used without verification of axial resistance during pile driving by static load test, dynamic test or formula, 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.~~

C10.7.3.8.1

Revise as follows:

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 resistance needed to resist the factored loads applied to the foundation are 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.

10.7.3.8.2 Static Load Test

Revise the 1st Paragraph as follows:

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 ~~less than 5 days after the test pile was driven unless approved~~ 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 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 as:

10.7.3.8.2 Static Load Test

Revise the 2nd Paragraph as follows:

Driving criteria should be established from the pile load test results using one of the following approaches:

1. Use dynamic measurements with signal matching analysis calibrated to match the pile load test results; a dynamic test shall be performed on the static test pile at the end of driving and again as soon as possible after completion of the static load test by re-strike testing. The signal matching analysis of the re-strike dynamic test should then be used to produce a calibrated signal matching analysis that matches the static load test result. Perform additional production pile dynamic tests with calibrated signal matching analysis (~~see Table 10.5.5.2.3-3 for~~ the number of tests required shall be determined by the Engineer) to develop the final driving criteria.

C10.7.3.8.2

Modify Figure C10.7.3.8.2-1 as follows:

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

$$\delta = \frac{QL_{\cancel{p}}}{\underline{144}\,AE}$$

Revise the 4th Paragraph as follows:

The specific application of the four driving criteria development approaches provided herein may be site specific, and may also depend on the degree of scatter in the pile load test and dynamic test results. If multiple load tests and dynamic tests with signal matching are conducted at a given site ~~as defined in Article 10.5.5.2.3~~, the Engineer will need to decide how to "average" the results to establish the final driving criteria for the site, and if local experience is available, in consideration of that local experience. Furthermore, if one or more of the pile load tests yield significantly higher or lower nominal resistance values than the other load tests at a given project site, the reason for the differences should be thoroughly investigated before simply averaging the results together or treating the result(s) as anomalous.

C10.7.3.8.2

Revise the 6th Paragraph as follows:

Regarding the fourth driving criteria development approach, it is very important to have the bearing zone well defined at each specific location within the site where piles are to be driven. Additional test borings beyond the minimums specified in Table 10.4.2-1 will likely be necessary to obtain an adequately reliable foundation when using this driving criteria development approach. ~~Note that a specific resistance factor for this approach to using load test data to establish the driving criteria is not provided. While some improvement in the reliability of the static analysis method calibrated for the site in this manner is likely, no statistical data are currently available from which to fully assess reliability and establish a resistance factor. Therefore, the resistance factor for the static analysis method used should be used for the pile foundation design.~~

10.7.3.8.3 Dynamic Testing

Revise the 1st Paragraph as follows:

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 <u>soil</u> strength change. The pile nominal axial 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

Revise the 1st Paragraph as follows:

The dynamic test may be used to establish the driving criteria at the beginning of production driving. The minimum number of piles that should be tested are as specified <u>by the Engineer</u> ~~in Table 10.5.5.2.3-3~~. A signal matching analysis (*Rausche et al., 1972*) of the dynamic test data should always be used to determine axial resistance if a static load test is not performed. See Hannigan et al. (*2005*) for a description of and procedures to conduct a signal matching analysis. Re-strike testing should be performed if setup or relaxation is anticipated.

10.7.3.8.3 Dynamic Testing

Add the following to the end of the Article:

<u>Dynamic testing shall not be used without calibrating to static load testing to determine the nominal bearing resistance of piles larger than 36-in. in diameter.</u>

10.7.3.8.4 Wave Equation Analysis

Add the following to the end of the Article:

<u>The wave equation shall not be used without correlating to static load testing to determine the nominal bearing resistance of piles larger than 36 in. in diameter.</u>

C10.7.3.8.4

Revise the 1st Paragraph as follows:

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), considerable judgment is required to use the wave equation to predict the pile bearing resistance. Key soil input values that affect the predicted resistance include the soil damping and quake values, the skin friction distribution, e.g., such as could be obtained from a pile bearing static analysis, and the anticipated amount of soil setup or relaxation. Furthermore, the actual hammer performance is a variable that can only be accurately assessed through dynamic measurements, though “standard” input values are available. <u>The reliability of the predicted pile axial nominal resistance can be improved by selecting the key input parameters based on local experience.</u> ~~The resistance factor of 0.40 provided in Article 10.5.5.2.3 for the wave equation was developed from calibrations performed by Paikowsky et al. (*2004*), in which default wave equation hammer and soil input values were used. Therefore, their wave equation calibrations did not consider the potential improved pile resistance prediction reliability that could result from measurement of at least some of these key input values. It is for these reasons that the resistance factor specified in Article 10.5.5.2.3 is relatively low (see *Paikowsky et al., 2004*, for additional information regarding the development of the resistance factor for the wave equation). If additional local experience or site-specific test results are available to allow the wave equation soil or hammer input values to be refined and made more accurate, a higher resistance factor may be used.~~

10.7.3.8.5 Dynamic Formula

Revise the 1st Paragraph as follows:

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

$$R_{ndr} = \left[1.83 * (E_r)^{1/2} * \log_{10}(0.83 * N_b)\right] - 124$$

$$\cancel{R_{ndr} = 1.75\sqrt{E_d}\log_{10}(10N_b) - 100} \qquad (10.7.3.8.5\text{-}1)$$

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.)

~~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.-lbs.)~~

N_b = Number of hammer blows in the last foot, (maximum value to be used for N is 96) ~~for 1.0 in. of pile permanent set~~ (blows/ft.~~in.~~)

Delete the 2nd and 3rd Paragraphs.

C10.7.3.8.5

Revise the 1st Paragraph as follows:

~~Two dynamic formulas are provided here for the Engineer. If a dynamic formula is used, 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., 2005*).~~ Note that the units in the modified ~~FHWA~~ Gates formula are not consistent. The specified units in Eq.1 must be used.

Delete the 2nd Paragraph.

Delete the 4th Paragraph.

Revise the 5th Paragraph as follows:

Dynamic Formulas should not be used when the required nominal resistance exceeds 600 kips <u>or the pile diameter is greater than or equal to 18-inch.</u>

Revise the 5th Paragraph as follows:

As the required nominal axial compression resistance increases, the reliability of dynamic formulae tends to decrease. The <u>modified</u> ~~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 formulae 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.~~

C10.7.3.8.6a

Revise as follows:

<u>The static analysis methods presented in this article should 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 should be used.</u>

<u>For open ended pipe piles, the nominal axial resistances should be calculated for both plugged and unplugged conditions. The lower of the two nominal resistances should be used for design.</u>

~~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 axial 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 re-strike 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.~~

10.7.3.10 Uplift Resistance of Single Piles

Revise the 1st and 2nd Paragraphs as follows:

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.

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.

Revise the 5th Paragraph as follows:

The pile 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 pile load test results when available.

C10.7.3.10

Add before the 1st Paragraph as follows:

In general, piles may be considered to resist an intermittent or temporary, but not sustained, uplift by skin friction.

Revise the 2nd Paragraph as follows:

See Hannigan et al. (2005) for guidance on the reduction of skin friction due to the effects of uplift. ~~Note that the resistance factor for uplift already is reduced to 80 percent of the resistance factor for static skin friction resistance. Therefore, the skin friction resistance estimated based on Article 10.7.3.8.6 does not need to be reduced to account for uplift effects on skin friction.~~

10.7.3.11 Uplift Resistance of Pile Groups

Revise the 4th Paragraph as follows:

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 1. The nominal uplift resistance of the pile group when considered as a block is 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. 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:

In cohesionless soil, 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.

10.8.1.2 Shaft Spacing, Clearance, and Embedment into Cap

Revise the 1st Paragraph as follows:

The center-to-center spacing of drilled shafts in 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 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.~~

C10.8.1.2

Revise the 1st Paragraph as follows:

Larger spacing may be required to preserve shaft excavation stability or to prevent interaction ~~communication~~ between shafts during excavation and concrete placement. If the center-to-center spacing of drilled shafts is less than 3.0 diameters, the sequence of shaft installation should be specified in the contract documents.

10.8.1.3

Add as follows:

10.8.1.3 Shaft Diameter, Concrete Cover, and Enlarged Bases

Add as follows:

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 CIDH piles or drilled shafts, the minimum concrete cover to reinforcement (including epoxy coated rebar) will be as specified in Table 10.8.1.3-1.

Table 10.8.1.3-1 Recommended Concrete Cover for CIDH Piles or Drilled Shafts

Diameter of the CIDH Pile or Drilled Shaft “D”	Concrete Cover[a]
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”

[a] For shaft capacity calculations, only 3” of cover is assumed effective and shall be used in calculations.

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.

10.8.3.1 General

Add the following bullet:

- group effects

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10.8.3.5.1c Tip Resistance

Add the following after the 1st Paragraph:

S_u = undrained shear strength (ksf)

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 O'Neill and Reese (*1999*) 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:

$N_c = 9$ for $S_u \geq 2$ ksf

$N_c = \left(\frac{4}{3}\right)\left[\ln\left(I_r + 1\right)\right]$ for $S_u \geq 2$ ksf

If $Z < 3D$,

$$q_p = \left(\frac{2}{3}\right)\left[1 + \left(\frac{1}{6}\right)\left(\frac{D}{B}\right)\right] N_c * S_u \qquad (10.8.3.5.1c\text{-}2)$$

where:

D = diameter of drilled shaft (ft.)

Z = depth of drilled shaft base (ft.)

S_u = design undrained shear strength (ksf)

I_r = rigidity index = $\left(\frac{E_s}{2S_u}\right)$

E_s = Young's modulus of soil (ksf)

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10.8.3.7.2 Uplift Resistance of Single Drilled Shaft

Revise the 1st Paragraph as follows:

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~~3.~~, and, when appropriate, by considering reduction due to effects of uplift.

C10.8.7.3.2

Revise the 1st Paragraph as follows:

The side resistance ~~factors~~ for uplift is ~~are~~ lower than that ~~those~~ for axial compression. One reason for this is that drilled shafts in tension unload ~~the~~ soils, 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*).~~

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