Liquefaction-Induced Downdrag

This module presents a simplified practical analysis procedure for the geotechnical seismic design of a pile foundation subjected to the effects of soil liquefaction, including liquefaction-induced downdrag. Downdrag refers to the phenomenon in which a pile (or a pile-group) foundation is subjected to *negative (downward) side shear stresses* imposed by the grounds surrounding the pile length when settling (or moving downward) more than the pile. Liquefaction-induced downdrag occurs due to ground settlement caused by soil liquefaction and affects the axial behaviors of pile foundations immediately after the cessation of shaking.

Unless permitted otherwise in a Project Specific Seismic Design Criteria (PSDC), the scope of this module is limited to the bridges designed in accordance with the Caltrans Seismic Design Criteria (SDC). It is assumed that the reader is thoroughly familiar with the Caltrans current Load and Resistance Factor Design (LRFD) of bridge foundation, including AASHTO LRFD Bridge Design Specifications with California Amendments (Caltrans LRFD BDS), Memo to Designers (MTD 3-1) and Geotechnical Manual (GM).

Caltrans' Safety Evaluation Earthquake (SEE) design requires that axially loaded pile foundations are stable against bearing capacity type failure. Noting that for SEE seismic design of bridges Caltrans uses a value of 1.0 for all load and resistance factors, based on the schematic shown in Figure 1, the general bearing stability requirements for a single pile during seismic events can be expressed as follows:

$$\left[R_N = (R_s + R_p)\right] \geq Q_{EQ} \tag{1}$$

Where, R_s = Total pile nominal side resistance in compression from soil layer (kips)

R_p = Pile nominal tip (bearing) resistance in compression, (kips)

R_N = Total pile nominal (bearing) resistance in axial compression (kips)

Q_{EQ} = Factored seismic design load in axial compression (kips).

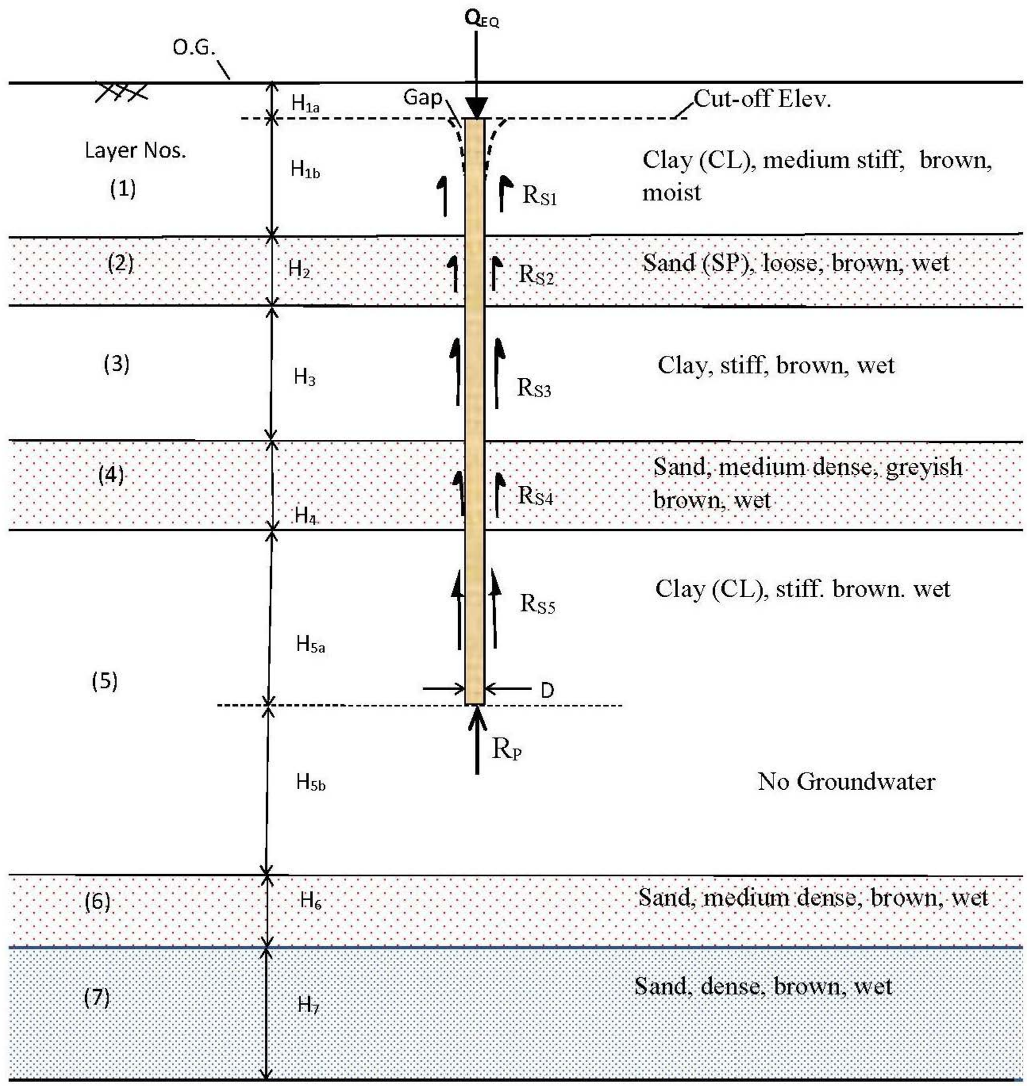
In general, the factored seismic design load in axial compression (QEQ) for a pile can be written as follows:

$$Q_{EQ} = (Q_{EQ})_{Perm} + (Q_{EQ})_I + W'p \tag{2}$$

Where, the component (QEQ)perm is due to the permanent gravity load from the superstructure. The component, (QEQ)I, is a live load resulting from the ground motion induced inertia of the superstructure. The parameter W'p is the effective weight of the pile, a permanent load.

Based on the foundation factored design load data table in MTD 3-1, Structure Designer (SD) provides two different seismic (Extreme Event) factored design loads: a "Max. Per Pile" Load and a "Max. Permanent Per Pile" Load. As designated above, QEQ = "Max. Per Pile" load (kips) and $(Q_{EQ})_{Perm}$ = "Max. Permanent Per Pile" load (kips).

In the absence of soil liquefaction, geotechnical seismic design of an axially loaded pile in compression involves determination of pile design tip elevation (DTE) that will satisfy the bearing stability criterion expressed by Equation 1, given the maximum per pile seismic factored design loads QEQ. It requires calculations of pile nominal side and tip bearing resistances (left side of Equation 1) that are available to resist QEQ (right side of Equation 1). In MTD 3-1 this pile tip elevation is designated as the DTE for Compression (Extreme Event).



Legends: H_i = Thickness of Soil Layer i (ft), D = Pile Diameter (ft), R_{Si} = Nominal Side Resistance from Soil Layer No. i (kips), R_p= Nominal Tip Resistance (kips)

Figure 1: Schematic of a Pile Foundation for Seismic Axial Bearing Stability Design

Soil liquefaction if predicted to occur as shown in Figure 2 for the same soil profile as in Figure 1 but includes shallow groundwater conditions, must be considered when determining the pile DTE based on Equation 1.

The potential effects of soil liquefaction on the pile includes reduction in the nominal side resistance from the (1) liquefied soil layers due to reduction in the shear strength, (2) disintegrated non-liquefied soil layers, if any, overlying liquefied soils when surface manifestation of liquefaction occurs, and (5) upper part of the pile due a gap that may form between the pile and the near surface ground due to cyclic loading. This usually occurs for cohesive soils.

A reduction in the nominal tip resistance can occur if liquefied soils are present within the zone of significant influence for pile tip resistance. A liquefied soil layer is considered a soft or weak soil layer characterized by a cohesion equal to its undrained residual or steady state shear strength (Sr). For seismic downdrag design, it can be assumed that this zone of influence extends from 1.5D above to 1.5D below the pile tip elevation.

One of the most significant effects of soil liquefaction on the pile bearing stability is the imposition liquefaction-induced downdrag load. Downdrag load occurs when liquefaction-induced soil settlements cause the grounds surrounding the pile length to settle more relative to the pile. Ground settlements occur when the excess porewater pressure generated in the liquefied soil during shaking dissipates.

However, dissipation of the excess porewater pressure generated in completely liquefied soils, and hence liquefaction-induced downdrag, does not occur until the cessation of ground shaking. Therefore, effects of liquefaction-induced downdrag on the pile need not be considered in combination with the inertial component (QEQ)I that occur during shaking. Thus, in liquefied soils a pile foundation needs to be designed to satisfy the seismic axial bearing stability requirements in compression (Equation 1), for the following two different factored design load (demand) and factored pile axial resistance (capacity) combinations:

1) ***Inertial (Extreme Event-I Compression) Loading Combination:*** This load combination occurs during ground shaking when the pile is subjected to the design load component $(Q_{EQ})_I$ due to the inertia of the superstructure in addition to the ever-present design permanent load component $(Q_{EQ})_{Perm}$. The concurrently available total pile nominal bearing resistance (R_N) corresponds to the completely liquefied soil conditions. Reductions in the pile nominal side and tip resistances as discussed above need to be considered in determining R_N. The required pile tip elevation for this loading and resistance case is designated as the DTE for Compression (Extreme Event) in MTD 3-1.

2) ***Downdrag (Extreme Event -I Downdrag) Load Combination:*** This load combination occurs soon after the cessation of ground shaking when the pile is subjected to the maximum possible liquefaction-induced downdrag load, DD_{max}, in addition to the ever-present permanent load $(Q_{EQ})_{Perm}$. That is, Q_{EQ} =[$(Q_{EQ})_{Perm}$ + DD_{max} +W'p] in Equation 1. The available total pile nominal bearing resistance (R_N) corresponding to the soil conditions that exists at the time the maximum downdrag load occurs. This resistance will depend on the magnitude of the total ground settlement relative to the pile that causes downdrag, and the depths and extent of soil liquefaction. The pile tip elevation necessary to satisfy the axial bearing stability

requirement corresponding to this load and resistance combination is designated as the DTE for Compression (Extreme Event) in MTD 3-1.

A simplified step by step procedure is presented below to determine the above pile DTEs. Due to the complexities involved, the procedure is presented with the aid of a generalized schematic shown in Figure 2. It is assumed that:

- The necessary general pile design information and all LRFD factored design loads, as per MTD 3-1 are available.
- Geotechnical subsurface information necessary to perform the LRFD Service State and Strength Limit State designs and soil liquefaction hazard analysis has been obtained.
- The ground surface and groundwater elevations appropriate for seismic analysis and design have been established.
- A representative subsurface soil profile, such as that shown in Figure 2 as an example, has been developed for the pile support location.
- The pile DTEs corresponding to the LRFD Service and Strength Limit States have been evaluated and are available.

Step by Step Procedure

Step 1: Perform Soil Liquefaction Hazard Analysis

Perform soil liquefaction hazard analysis in accordance with the procedure contained in the “Liquefaction Evaluation” module of the Caltrans GM. Identify the liquefied soil layers as shown in Figure 2(a).

Step 2: Evaluate and Summarize Design Soil Parameters

The following empirical correlations proposed by Kramer and Wang (2015) can be used to evaluate the undrained residual shear strengths (S_r) of liquefied soils:

$$S_r(\text{psf}) = 2116 \exp\left\{-8.444 + 0.109(N_1)_{60} + 5.379\left(\frac{\sigma'_{vo}}{2116}\right)^{0.1}\right\} \qquad (3)$$

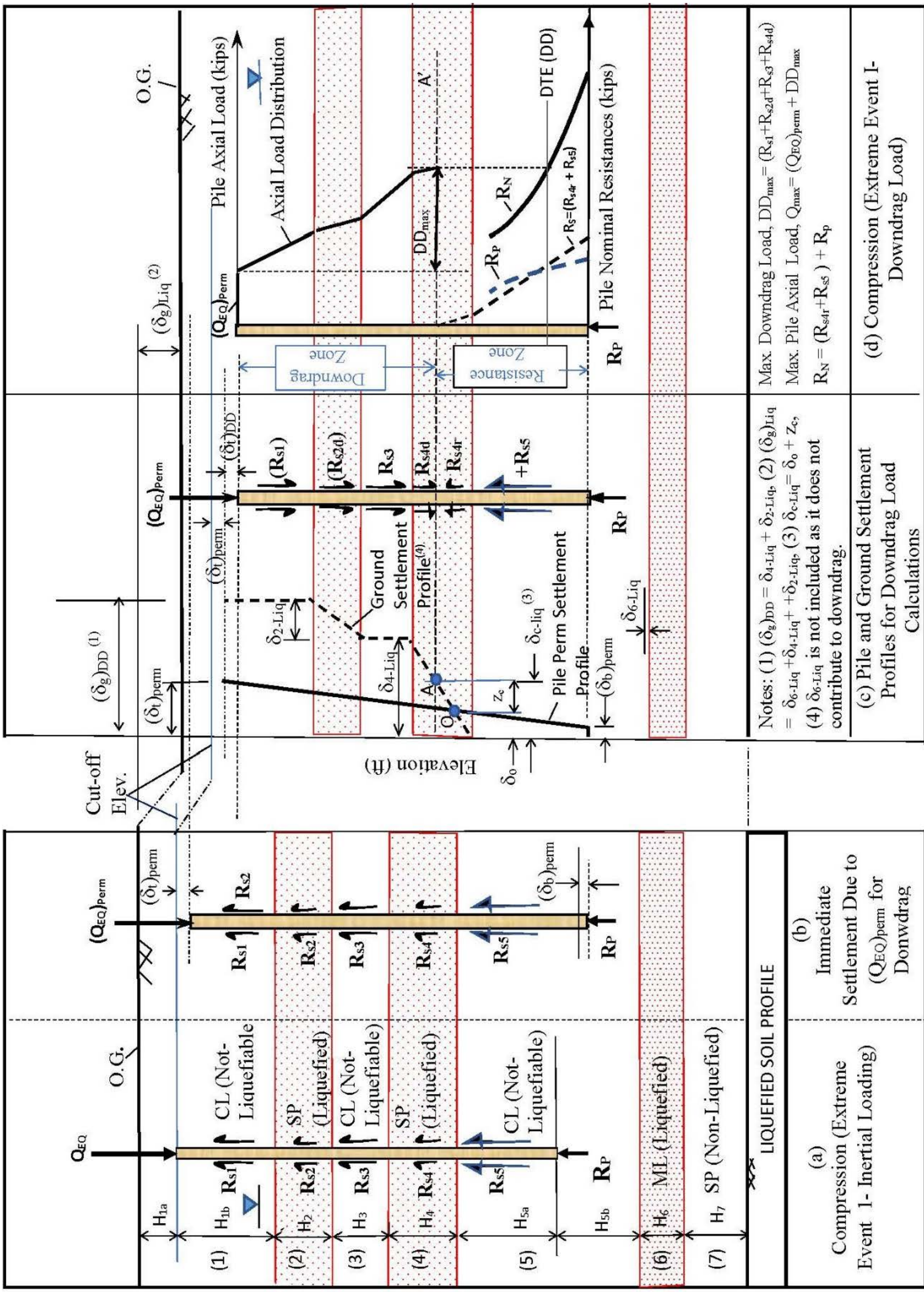


Figure 2. Seismic Design of a Pile Foundation at Liquefied Sites (Schematic).

Where, $(N_1)_{60}$ = energy corrected and overburden pressure normailzed SPT blow count at the depth under consideration. The field measured SPT blow count (N_m) at a given depth is corrected to standard 60% hammer engery and normailzed to an initial effective overburden stress equal to one (1.0) atmohspheric pressure ($\cong$ 2116 psf). See "Liquefaction Module" of the Geotechncial Manual for a detailed calculation procedure for $(N_1)_{60}$. Note that no fine content correction is used in Eqaution 3. And, σ'_{vo} = initial in-situ effective overburden stress at the depth under consideration evaluated based on the ground surface and groundwater elevations applicable to seismic design (Extreme Event-I Limit State).

The empirical correlation proposed by Robertson (2010) can be used to evaluate the undrained residual shear strengths (Sr) of liquefied soils using CPT data.

For non-liquefiable clayey soils, assume ϕ=0 condition and use undrained shear strength (S_u). For non-liquefied coarse-grained and fine-grained soils with plasticity index (PI)<7, both cohesion (c) and friction angle (ϕ) may be used for seismic design when these parameters are measured directly for the in-situ state conditions using appropriate field or laboratory test. Cohesion should not be used for these soils if the friction angle is estimated indirectly, such as using empirical correlations with SPT or CPT data. For non-liquefied soils, the current practice is to use the same shear strength parameters in the seismic analysis as those used in the analyses performed to determine the pile DTEs for the LRFD Strength Limit State.

Summarize the assigned design soil parameters, including soil total/effective unit weight and the shear strength parameters, in a table.

Step 3: Designing for the Inertial Load Combination, QEQ = (QEQ)Perm + (QEQ)I +W'P

Step 3-1: Calculate Pile Nominal Side and Tip Resistances

Calculate pile nominal side and tip bearing resistances for the liquefied soil profile, such as the one shown in Figure 2(a), based on the design soil parameters determined and assigned in Step 2. See the deep foundation modules in the Caltrans' Geotechnical Manual for details on how to calculate pile nominal side and tip bearing resistances for a given soil profile once the appropriate soil design parameters are assigned to the soil layers.

Step 3-2: Evaluate Potential for Surface Manifestation of Liquefaction

For the soil layers identified as liquefiable, evaluate starting from the top whether the combined thickness of the overlying soil layers is adequate to prevent surface manifestation of liquefaction based on Figure 3 (Modified after Ishihara,1985). See Figure 4 for the definitions and symbols to be used for the overlying soil layer thickness (Hs) and the liquefied soil layer thickness (HL) in Figure 3. Identify the lowest liquefied soil layer that is likely to cause surface manifestation of liquefaction.

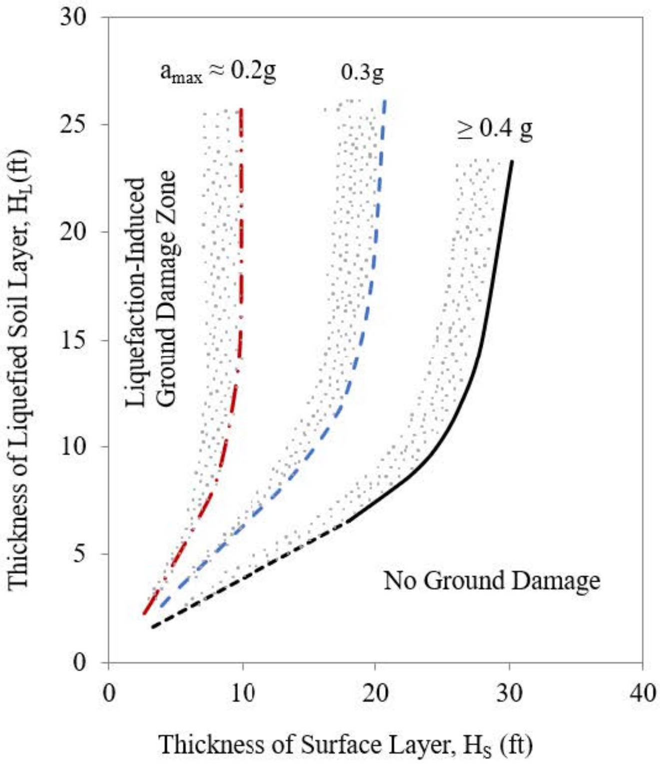


Figure 3. Boundaries for Surface Manifestation of Soil Liquefaction (Modified After Ishihara, 1985)

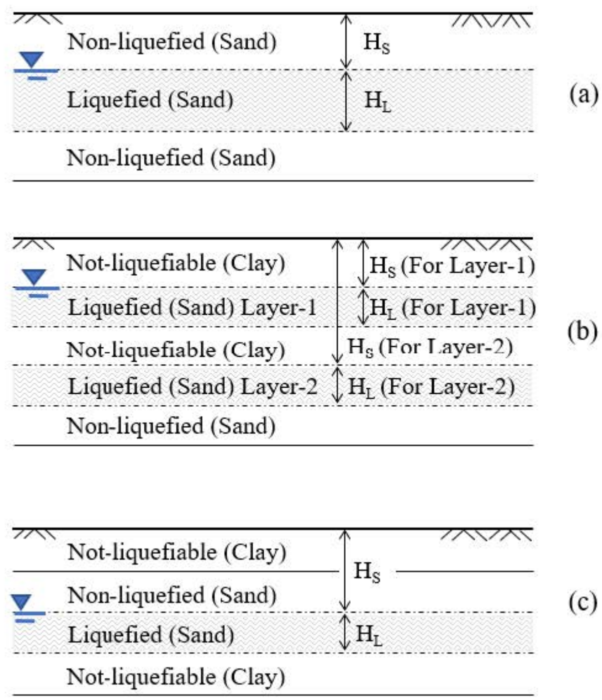


Figure 4. Definitions for Thickness of Surface Layer (H_s) and Thickness of Liquefiable Sand Layer (H_L) in Figure 3

Step 3-3: Determine Pile DTE for Compression (Extreme Event I - Compression)

Determine the pile DTE for Compression (Extreme Event) or the "Inertial Loading Combination", based on the seismic factored design load Q_{EQ} and the available pile total nominal resistance (R_N) calculated using the pile nominal side and tip bearing resistances from Step 3-1, except:

1) Ignore the pile nominal resistances calculated for the upper one pile diameter length of the pile if located in cohesive soils.
2) If surface manifestation of liquefaction is predicted in Step 3-2, ignore all pile nominal resistances from the lowest causative liquefied soil layer and the overlying soil layers, if any.

Step 4: Designing for Downdrag Load Combination, $Q_{EQ} = (Q_{EQ})_{Perm} + DD_{max} + W'_p$

The purpose of this step is to determine the pile DTE for Compression (Extreme Event-Downdrag), as designated in MTD 3-1. This DTE will be referred to hereafter simply as DTE(DD).

Settlements of the soil layers above the pile cut-off elevation or below the pile tip settlement zone of influence do not affect downdrag and need not be considered in the following analysis. Such ground settlements, if any occurs, will contribute to the total ground surface and bridge support settlement, and may also contribute to the differential settlements between adjacent supports. The objective of the following analysis, however, is to analyze and design the pile foundation for axial bearing stability in compression only.

It is not necessary to consider live load in downdrag analysis. Once downdrag load occurs, it is considered a permanent axial load on the pile. Use a load factor of 1.0 for liquefaction-induced downdrag load.

Step 4-1: Select the Preliminary Pile Tip Elevation for Downdrag Load Analysis

Determine the pile DTE that would be the minimum (i.e., shall not be raised above) tip elevation based on all other applicable design requirements (i.e., Service, Strength and Extreme Event -Compression), as per MTD 3-1, except, if not available, the DTE for lateral load. Use this minimum pile DTE as the preliminary pile tip elevation to evaluate the maximum downdrag load (DD_{max}) and the DTE(DD) as per the following steps.

Step 4-2: Determine Pile Settlement at the Onset of Ground Settlement

With the pile tipped at the lowest DTE as determined in Step 4-1, and for the same liquefied soil profile and assigned soil parameters as used in Step 3, estimate the pile immediate settlements (pseudo-static) of the pile top and tip, $(\delta_t)_{perm}$ and $(\delta_b)_{perm}$, respectively, due to the pile axial load $(Q_{EQ})_{perm}$ applied at the pile top, as shown in Figure 2(b). In determining these pile settlements, ignore any resistances to pile settlement from the soil layers that were ignored in determining the pile DTE for Compression (Extreme Event - I) in Step 3.

Pseudo-static pile load-immediate settlement curves can be developed using available computer programs for soil conditions represented by Figure 2(b) to estimate $(\delta_t)_{perm}$ and $(\delta_b)_{perm}$. For additional information, see specifications and commentaries related to the immediate settlement of bridge foundations in Section 10 of Caltrans LRFD BDS.

Step 4-3: Estimate Liquefaction-Induced Ground Settlements

Estimate liquefaction-induced settlement of the identified liquefied soil layers in accordance with the procedures recommended by Tokimatsu and Seed (1987). Zhang et al (2002) may be used to estimate liquefaction-induced ground settlement using CPT data. In Figure 2(c), $\delta_{2\text{-}Liq}$, $\delta_{4\text{-}Liq}$ and $\delta_{6\text{-}Liq}$ represents the estimated total settlements of the liquefied soil layers 2, 4 and 6, respectively.

Liquefaction-induced total ground settlement, $(\delta_g)_{Liq}$, at the ground surface elevation is equal to the sum of the total settlements of all the liquefied soil layers. In Figure 2(c), $(\delta_g)_{Liq} = [\delta_{2\text{-}Liq}) + \delta_{4\text{-}Liq} + \delta_{6\text{-}Liq}]$. However, settlement of the liquefied soil layer 6 does not contribute to the development of downdrag load on the pile. As shown in Figure 2(c), the settlements of the soil layers 2 and 4 only can cause downdrag load on the pile. Thus, the total ground settlement at the pile cut-off elevation that contributes to the development of downdrag load on the pile, $(\delta_g)_{DD} = (\delta_{2\text{-}Liq} + \delta_{4\text{-}Liq})$.

Step 4-4: Plot Pile and Ground Settlement Profiles

Plot pile settlement profile as shown in Figure 2(c) based on $(\delta_t)_{perm}$ and $(\delta_b)_{perm}$ determined in Step 4-2. Also plot the ground settlement profile based on the settlements evaluated in Step 4-3 based on the settlement of the liquefied soil layers located within the pile embedment length (i.e., Layers 2 and 4 only in Figure 2(c). Unless noted otherwise, ground settlements in the following steps refer to this ground settlement profile.

Step 4-5: Determine the Location of the Maximum Downdrag Load (DDmax)

Calculate the pile-ground relative settlement z_{max} at which the maximum negative side shear stresses will be developed on the pile. The parameter z_{max} depends on several factors, including the pile type, diameter, length, axial stiffness, and the soil type and state (e.g., API, 2014, O'Neil and Reese, 1999; Karlsrud, 2014, Coyle and Sullivan, 1967). For evaluating the maximum downdrag load on the pile, calculate the parameter z_{max} based on the simplified correlations in Table 1.

As shown in Figure 2(c), determine the ground settlement, δ_o, at Point O, where the pile settlement and the ground settlement profiles intersect each other. Determine the critical ground settlement, $\delta_{c\text{-}Liq}$, by adding δ_o and z_{max}. This is the total ground settlement above which the negative skin friction will be mobilized on the pile. If the total ground settlement at the surface elevation, $(\delta_g)_{DD}$, is less than or equal to $\delta_{c\text{-}Liq}$, it can be assumed that the pile will not experience any downdrag load, and no further analysis is necessary.

Draw a horizontal line AA' passing through the Point A. For design, assumed that the portion of the pile embedded in soil above the elevation of the line AA' will be subjected to the maximum negative side shear stresses or downdrag load.

Table 1: Parameter z_{max} for Various Pile and Soil Types

Pile Type	Soil Type	$\frac{z_{max}}{D}$
Driven-Closed End	Sand or Clay	0.03
Driven – Open Ended	Clay	0.02
	Sand	0.03
Drilled Shafts	Clay	0.008
	Sand	0.01

Step 4-6: Calculate Maximum Downdrag Load (DDmax)

Calculate the maximum negative side shear stresses that can be mobilized along the pile embedment length above the elevation of line AA'. Use the same shear strength parameters as those assigned in Step 2, except for the liquefied soil layers.

The maximum negative side shear stress from liquefied soils will not occur until the excess porewater pressures generated during ground shaking completely dissipate.

Thus, downdrag loads from the liquefied layers should be calculated based on the shear strengths of the resettled liquefied soils. In the absence of measured shear strength parameters, the maximum downdrag load from the resettled loose liquefied soils can be calculated using:

- $c=0$ and $\phi=30^o$ for coarse-grained liquefied soil (e.g., SW, SP, SM) or,
- $c=0$ and $\phi=25^o$ for fine-grained liquefied soils (e.g., ML)

Determine the nominal downdrag loads from each of the soil layers located above line AA' based on the calculated maximum (or nominal) negative side shear stresses.

Calculate the total axial load on the pile at any depth (or elevation) below the pile cut-off elevation as the sum of $(Q_{EQ})_{perm}$ and the total downdrag load at the same depth (or elevation).

Plot the total pile axial load as a function of the elevation and calculate the maximum total downdrag load, DD_{max}, on the pile which occurs at the elevation of the line AA'.

In Figure 2(d), $DD_{max} = R_{s1}+R_{s2d}+R_{s3}+R_{s4d}$. Here the subscript "d" is used to denote the downdrag loads from the liquefied soil layer.

The maximum axial load on pile is equal to $[(Q_{EQ})_{perm} + DD_{max}+W'_p)$, which occurs at the same elevation as the maximum downdrag load, DD_{max}.

Step 4-7: Calculate Pile Nominal Resistances (RN)

Calculate the available pile nominal side and tip bearing resistances from the soil layers below line AA' based on the soil shear strength parameters assigned in Step 2, except for liquefied soils. For the resettled liquefied soils, use the shear strength parameters recommended in Step 4-6.

Plot the total pile nominal side resistance, Rs), and the total pile nominal tip resistances, R_P, as a function of elevation as shown in Figure 2(d).

Calculate the available total pile nominal bearing resistance, R_N, in axial compression by adding R_S and R_P. Plot R_N as a function of elevation, as shown in Figure 2(d).

Step 4-8. Determine pile DTE for Compression (Extreme Event -Downdrag)

Determine the pile DTE(DD), as the elevation at which $R_N = [(Q_{EQ})_{perm} + DD_{max} + W'_p]$. If the DTE (DD) is located at or above the preliminary tip elevation used in the above downdrag load calculations, no further analysis is necessary.

If R_N at the pile preliminary tip elevation is less than the factored load, $[(Q_{EQ})_{perm} + DD_{max}$ $W'_p]$, extend calculations of R_S and R_P from underlying the preliminary tip the calculated R_N value exceeds the total factored seismic design load. Determine the DTE(DD) based on $R_N = [(Q_{EQ})_{perm} + DD_{max}+ W'_p]$.

Reporting

As per MTD 3-1, the following information need to be included in the Foundation Report for Extreme Event-I Limit State design:

- Extreme Event (Compression) case: Geotechnical Nominal Resistance Per Pile and Nominal Tip Resistance (Compression), and the corresponding DTE (Extreme Event)
- Extreme Event (Downdrag) case: Geotechnical Nominal Resistance Per Pile and Nominal Tip Resistance (Compression with Downdrag), and the corresponding DTE (Extreme Event-Downdrag)
- Maximum downdrag load per pile (DD_{max})
- The downdrag zone, i.e., the top and bottom elevations of the downdrag zone
- Ground settlement $(\delta_g)_{DD}$ at the pile cut-off elevation due to the settlement of liquefied soil layers located within the pile embedment length
- The total ground settlements $(\delta_g)_{Liq}$ at the ground surface elevation due to all liquefied soil layers

If requested by the Structure Designer (SD), provide data for the unit side resistance (t) versus pile-soil relative settlement (z) or t-z curves, and the unit tip bearing resistance (q) versus tip settlement (w) or q-w curves for both the inertial loading (Step 3) and liquefaction-induced downdrag loading (Step 4) combinations. Figures 2(a) and 2(d) represent the corresponding soil profiles and conditions, respectively.

References

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11. Tokimatsu, K. and Seed, H.B. (1987), Evaluation of the Settlement of Sand Due to Earthquakes, Journal of Geotechnical and Environmental Engineering, ASCE, 113(8), pp. 861-878.
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Appendix A

Liquefaction Induced Downdrag Example Problem

Limit State: Extreme Event -I (Seismic)

Appendix A presents an example geotechnical analysis and design of a pile foundation subjected to liquefaction-induced downdrag. The example involves designing a Cast-in-Drilled Hole (CIDH) pile or drilled shaft supporting a bridge bent. Figure A1 presents a schematic illustration of the example.

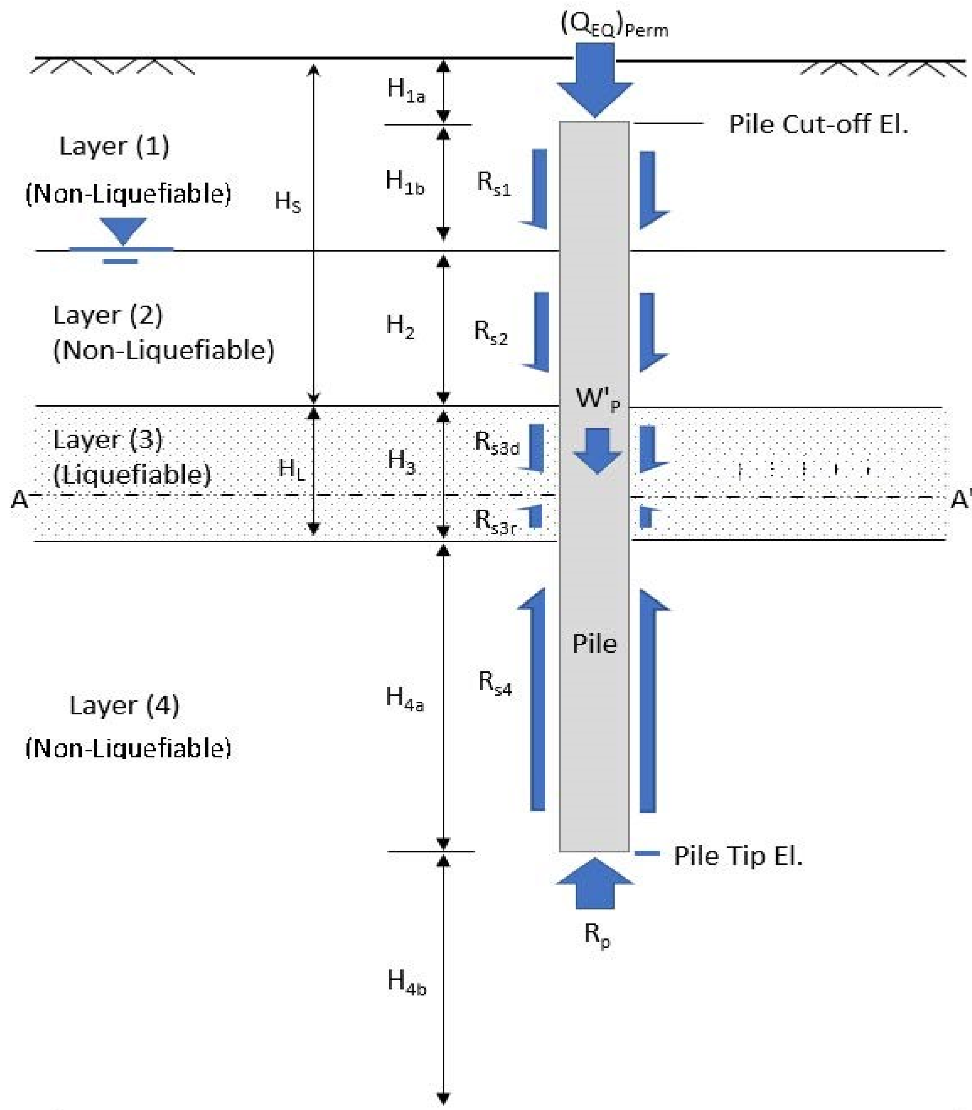


Figure A1: Components of Pile Foundation Design for Liquefaction-Induced Downdrag

Notations and definitions are:

H_i : thickness of the soil layer no. i
H_S : overlying soil layer thickness for surface manifestation of liquefaction
H_L : liquefied soil layer thickness for surface manifestation of liquefaction
A-A′: maximum downdrag load elevation due to liquefaction-induced ground settlement
R_{si} : nominal side resistance along the pile within the soil layer i.
R_{sid} : nominal side resistance along the pile above A-A' line within the liquefied soil layer i.
R_{sir} : nominal side resistance along the pile below A-A' line within the liquefied soil layer i.
R_p : nominal tip resistance
W'_P: effective weight of the pile
$(Q_{EQ})_{Perm}$: maximum permanent load per pile.

The pile properties, the general foundation design information and the factored design loads provided by the Structure Designer (SD) are:

- Pile type: CIDH
- Pile Diameter: 66-inch
- Unit weight of the CIDH pile: 150 pcf
- Elastic Modulus for Concrete: $\mathrm{E} = 3.6 \times 10^6$ psi

Table A1: General Foundation Design Information

Support Location	Pile Type	FG Elev. (ft)	Pile Cut-off Elevation (ft)	Pile Cap Size		Permissible Settlement under Service Load (in)	Number of Piles per Support
				B	L		
Bent	66-inch dia. CIDH	+0.0	- 5.0	N/A	N/A	1.0	1

Table A2: Foundation Factored Design Loads

Support Location	Service-1 Limit State (kips)		Strength Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)				
	Total Load Per Support	Perm. Load Per Support	Compression		Tension		Compression			Tension	
			Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Max Perm Per Pile	Per Support	Max Per Pile
Bent	1115	886	1860	1860	0	0	1135	1135	886	0	0

Step by Step Procedure

This example problem demonstrates the step-by-step procedure for the design of pile foundations subject to liquefaction-induced downdrag.

Step 1: Perform Soil Liquefaction Hazard Analysis

The design ground motion parameters corresponding to a 975-year return period for soil liquefaction evaluation are as follows:

- Design horizontal Peak Ground Acceleration (PGA) = 0.7g
- Moment magnitudue of the design (mean) earthquake event, M =7.3

Based on the procedure in the *Liquefaction Evaluation* module, it is indicated the soil layer from 20 feet to 30 feet below the ground surface is liquefiable during the design ground motion event (Figure A2).

Step 2: Evaluate and Summarize Design Soil Parameters

The soil profile and the relevant engineering soil parameters are summarized in Table A3 based on available existing geotechnical data, a current site investigation performed and interpretation of field and laboratory test data. A representative idealized cross section of the design soil profile including the pile is shown in Figure A2.

Table A3: Design Soil Parameters for the Example Downdrag Analysis

Layer No.	Elevation Range (feet)	Soil Description	$(N_1)_{60}$	Total Unit Weight, γ_t (pcf)	Shear Strength Parameters	
					Friction Angle, Φ (degrees)	Cohesion (c) or Undrained Shear Strength (S_u or S_r) (psf)
1	0 to -10	Silty Sand (SM), Medium dense, fine-to coarse-grained sands	20	120	34	c = 100
2	-10 to -20	Silty Clay (CL), Soft to medium stiff	-	108	0	S_u = 1500
3	-20 to -30	Sand (SP), fine to medium-grained, loose to low medium dense, 9% fines, non-plastic, wet, (Liquefied Layer)	12	110	32	c = 0
					0	S_r = 340
4	-30 to -80	Silty Sand (SM) with gravel, dense, fine to coarse-grained sand, fine gravel	36	130	38	c = 0

Note: S_r represents undrained residual or steady state shear strength of liquefied soil.

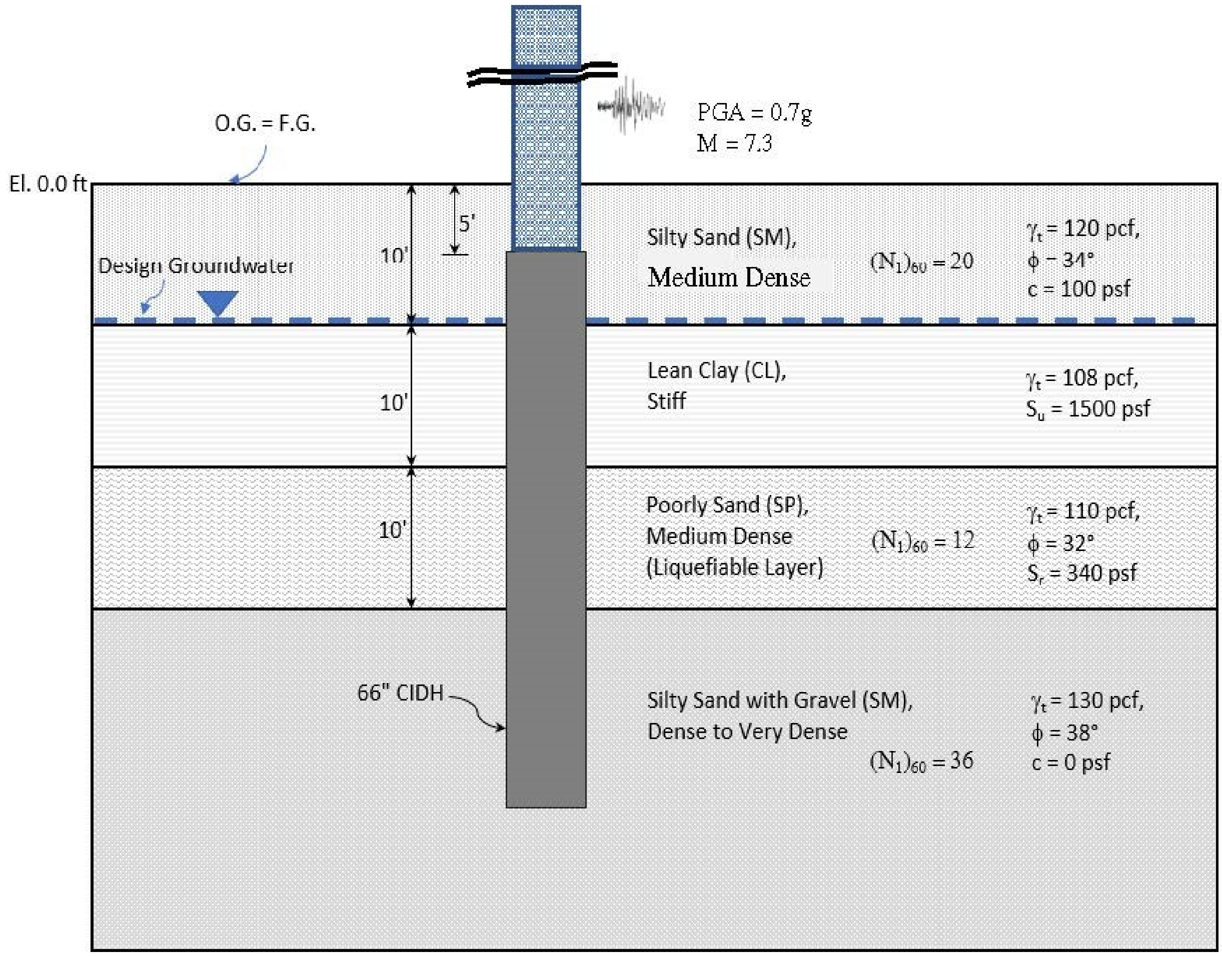


Figure A2: Soil Profile and Pile Model for Liquefaction-Induced Downdrag Analysis

Step 3: Designing for the Inertial Loading Combination (Extreme Event-I Compression)

Step 3-1: Evaluate Potential for Surface Manifestation of Liquefaction

From the soil profile in Figure A2, the overlying non-liquefiable soil layers (H_S) is 20 feet thick, and the liquefiable soil layer (H_L) is 10 feet thick. Based on Figure A3, the two overlying non-liquefiable layers are not thick enough to prevent surface manifestation of liquefaction. Therefore, pile nominal resistances from these two overlying layers and the liquefiable layer are ignored in the pile axial capacity analysis under this loading case.

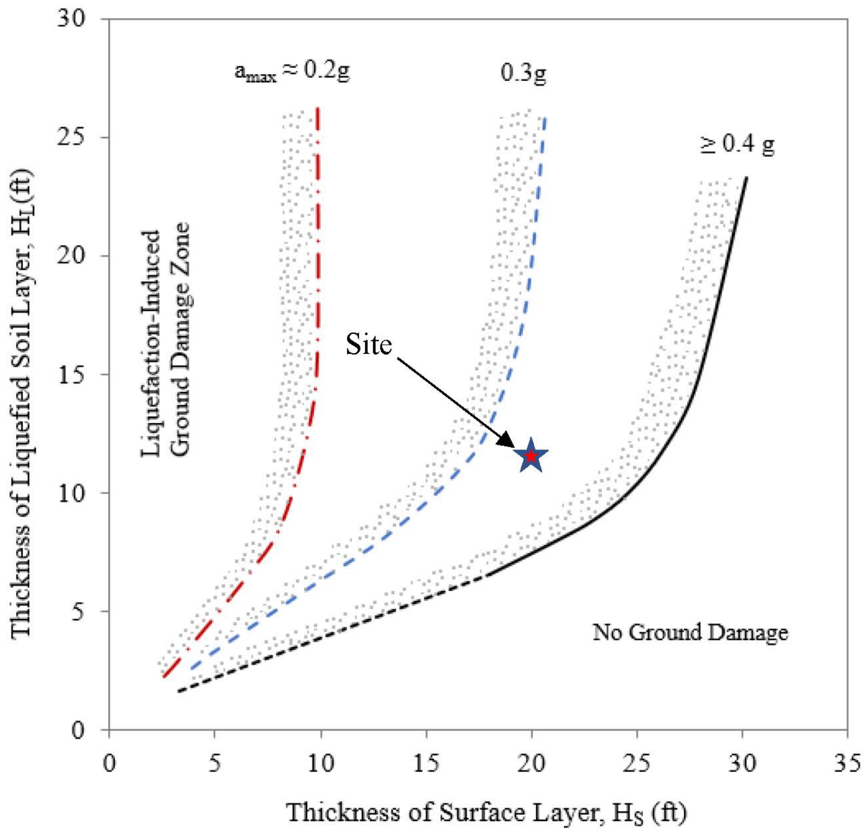


Figure A3: Evaluation of Surface Manifestation (Modified After Ishihara 1985)

Step 3-2: Calculate Pile Nominal Side and Tip Resistances

Based on the design soil parameters summarized in Table A3, the pile nominal side and tip resistances under the extreme event limit state are calculated using SHAFT software (Ensoft, 2012). The outputs using SHAFT are summarized in Table A4 starting from the pile embedment of length of 26 feet (= 31 feet – Depth to pile cut-off elevation 5.0 feet). Note that “LENGTH” in the Table A4 refers to pile length from the finish grade.

Table A4: Summary of Predicted Outputs from SHAFT (Extreme Event-I Limit State)

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	LRFD QS (TONS)	LRFD QB (TONS)	LRFD QU (TONS)
31.0	27.28	13.68	320.21	333.89	13.68	320.21	333.89
32.0	28.16	27.57	328.59	356.16	27.57	328.59	356.16
33.0	29.04	41.66	336.97	378.63	41.66	336.97	378.63
34.0	29.92	55.94	345.35	401.29	55.94	345.35	401.29
35.0	30.80	70.40	353.73	424.13	70.40	353.73	424.13
36.0	31.68	85.03	362.10	447.14	85.03	362.10	447.14
37.0	32.56	99.82	370.48	470.31	99.82	370.48	470.31
38.0	33.44	114.76	378.86	493.62	114.76	378.86	493.62
39.0	34.32	129.84	387.24	517.08	129.84	387.24	517.08
40.0	35.20	145.05	395.62	540.67	145.05	395.62	540.67
41.0	36.08	160.38	404.00	564.38	160.38	404.00	564.38
42.0	36.96	175.83	412.38	588.20	175.83	412.38	588.20
43.0	37.84	191.37	420.75	612.13	191.37	420.75	612.13
44.0	38.72	207.02	429.13	636.15	207.02	429.13	636.15
45.0	39.60	222.74	436.64	659.38	222.74	436.64	659.38
46.0	40.48	238.55	443.20	681.74	238.55	443.20	681.74
47.0	41.36	254.42	448.73	703.15	254.42	448.73	703.15
48.0	42.24	270.35	453.18	723.52	270.35	453.18	723.52
49.0	43.12	286.33	456.46	742.78	286.33	456.46	742.78
50.0	44.00	302.35	458.64	760.99	302.35	458.64	760.99
51.0	44.88	318.41	459.95	778.36	318.41	459.95	778.36
52.0	45.76	334.49	460.61	795.10	334.49	460.61	795.10
53.0	46.64	350.60	460.83	811.42	350.60	460.83	811.42
54.0	47.52	366.71	460.83	827.53	366.71	460.83	827.53
55.0	48.40	382.82	460.83	843.64	382.82	460.83	843.64
56.0	49.28	398.92	460.83	859.75	398.92	460.83	859.75
57.0	50.16	415.01	460.83	875.84	415.01	460.83	875.84
58.0	51.04	431.08	460.83	891.91	431.08	460.83	891.91
59.0	51.92	447.12	460.83	907.95	447.12	460.83	907.95
60.0	52.80	463.12	460.83	923.95	463.12	460.83	923.95

Step 3-3: Determine Pile Design Tip Elevation (DTE) for Extreme Event-I (Compression)

In this example the upper 25 feet of the CIDH pile has no nominal side resistances because of the surface manifestation of liquefaction (Step 3-1) and the 10 feet thick liquefied soil layer. When surface manifestation occurs, it is assumed that the overlying non-liquefied soils are disintegrated along the embedded pile length to provide any side resistance. Plots of the pile side, tip and total factored nominal resistances versus lengths (from the cut-off) are shown in Figure A4.

Since the effective pile weight is considered in the load combination, use of a trial and error method is necessary to determine the DTE.

For first trial, the maximum factored design load per pile from Table A2 (1145 kips rounded to the nearest 10 kips) will be used to estimate the preliminary DTE. Based on Figure A4, for this factored design load, the preliminary DTE is about -42 feet (pile length = 37 feet).

The effective weight for the 37 feet of pile = 92 kips. The revised factored design load (1135 kips +92 kips) = 1227 feet. Rounded to the nearest 10 kips, the factored design load = 1230 kips. As shown in Figure A4, the estimated DTE for this factored load is -43.5 feet (pile length = 38.5 feet).

For the above pile DTE, the ratio of the pile embedment depth into the bearing strata (measured from the bottom of the liquefied soil layer) to the pile diameter is 3.0. This ratio, being greater than 1.5, indicate no reduction is required in the pile tip nominal resistance due to the liquefied soil layer. More accurately, based data in Table A4, for the pile tip elevation – 43.5 feet, the estimated available total geotechnical nominal resistance per pile, RN = 1250 kips, and the corresponding estimated pile tip nominal resistance component, Rp= 850 kips.

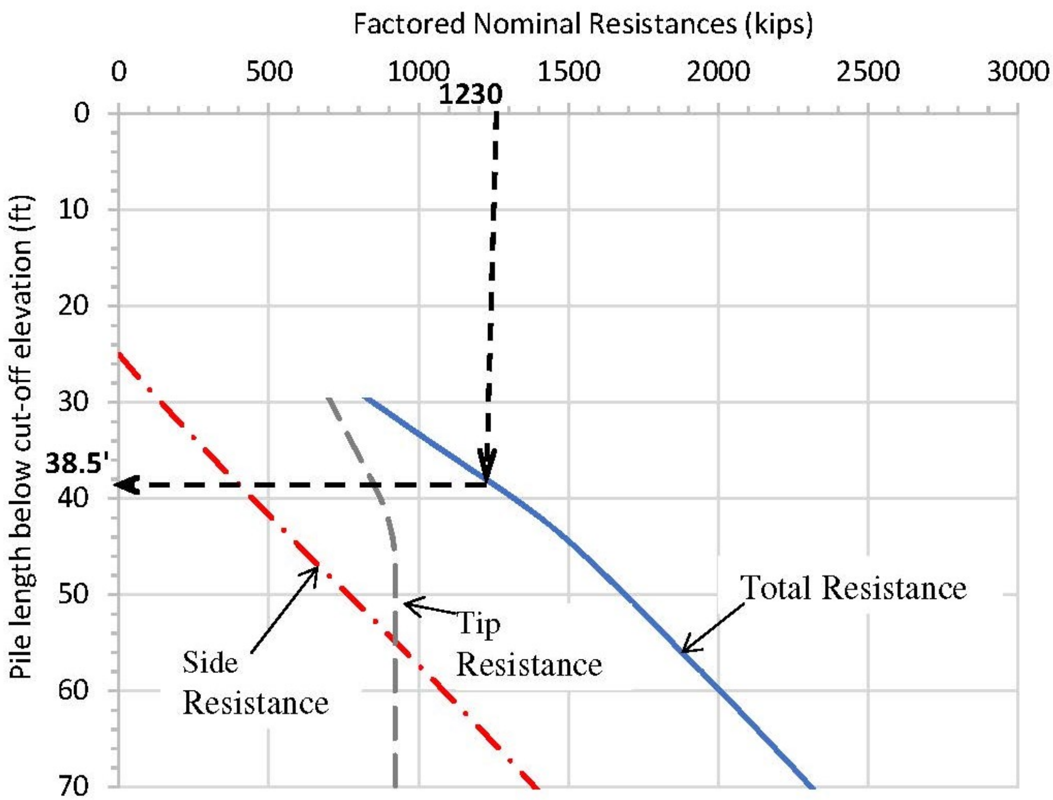


Figure A4: Factored Nominal Resistances with Pile Length for Extreme Limit State

Step 4: Designing for Downdrag Load Combination (Extreme Event I- Downdrag)

Step 4-1: Select the Preliminary Pile Tip Elevation for Downdrag Load Analysis

Based on the pile design tip elevations evaluated for Service and Strength limit states, and the above Extreme Event-I (Compression) design, the minimum (i.e., not to be raised above) tip elevation would be at -79 feet. (pile length = 74 feet. This design tip evaluation (-79 feet) is associated with the Strength Limit State. Note the DTE for lateral load should also be considered in this step, if available. However, in this example case, the lateral design tip elevation is not expected to be lower than -79 feet. This tip elevation will be used as the preliminary pile tip elevation for the subsequent downdrag analysis.

Step 4-2: Calculate Pile Settlements at Onset of Liquefaction-Induced Ground Settlement

Based on the preliminary tip elevation of -79 feet from Step 4-1, and the soil parameters used in Step 3, Table A.5 presents SHAFT output data for the pile axial load at the top versus pile top settlement, and the transferred axial load at the pile tip versus pile tip settlement. The corresponding plots are presented in Figure A5. Due to the seismic design permanent load of 890 kips per pile (886 kips in Table A2, rounded to the nearest 10 kips), the estimated pile top settlement $(\delta_t)_{perm}$ = 0.20 inches and the corresponding pile tip settlement $(\delta_b)_{perm}$ = 0.13 inches.

Table A5: Pile Axial Load-Settlement (Averaged) resulted from SHAFT

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.4315E-01	0.1455E-04	0.2444E-02	0.1000E-04
0.2157E+00	0.7277E-04	0.1222E-01	0.5000E-04
0.4315E+00	0.1455E-03	0.2444E-01	0.1000E-03
0.2157E+02	0.7277E-02	0.1222E+01	0.5000E-02
0.3236E+02	0.1092E-01	0.1833E+01	0.7500E-02
0.4315E+02	0.1455E-01	0.2444E+01	0.1000E-01
0.1081E+03	0.3640E-01	0.6109E+01	0.2500E-01
0.2164E+03	0.7282E-01	0.1222E+02	0.5000E-01
0.3081E+03	0.1077E+00	0.1833E+02	0.7500E-01
0.3863E+03	0.1411E+00	0.2444E+02	0.1000E+00
0.6456E+03	0.3203E+00	0.6109E+02	0.2500E+00
0.8206E+03	0.5917E+00	0.1210E+03	0.5000E+00
0.8672E+03	0.7585E+00	0.1590E+03	0.6600E+00
0.1001E+04	0.1769E+01	0.2949E+03	0.1650E+01
0.1172E+04	0.3445E+01	0.4677E+03	0.3300E+01

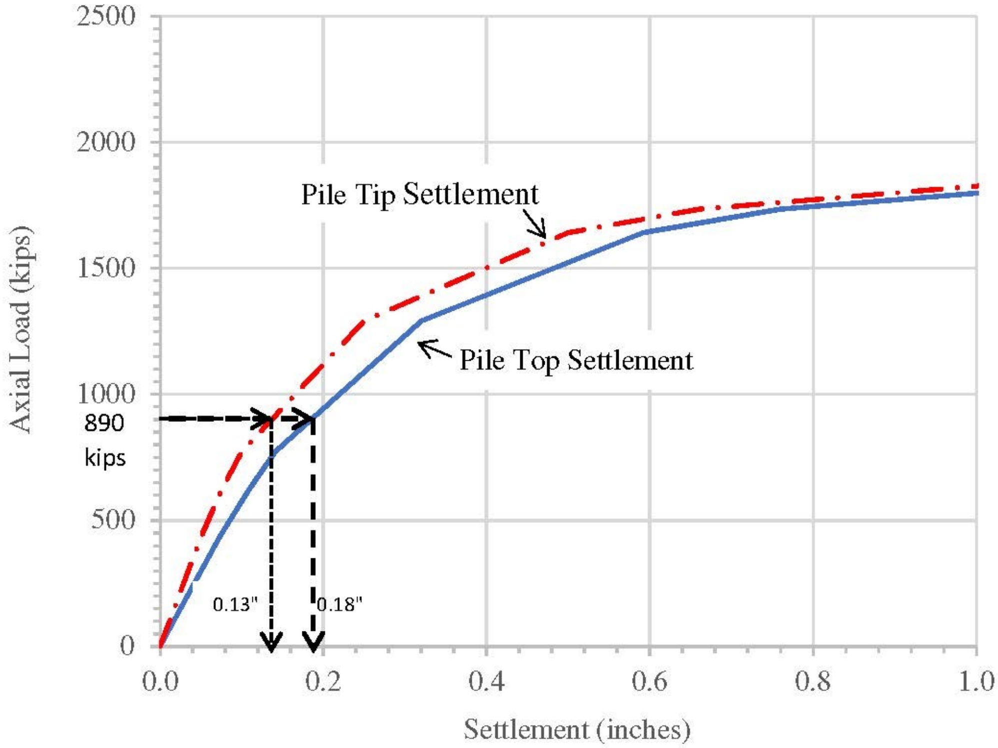


Figure A5: Pile Load-Settlement Curves

Step 4-3: Calculate Liquefaction-Induced Ground Settlements

Liquefaction-induced ground settlement for the liquefied soil layer (No. 3), $\delta_{3\text{-Liq}}$ was calculated using SPT blow counts and using the procedure in Tokimatsu and Seed (1987).

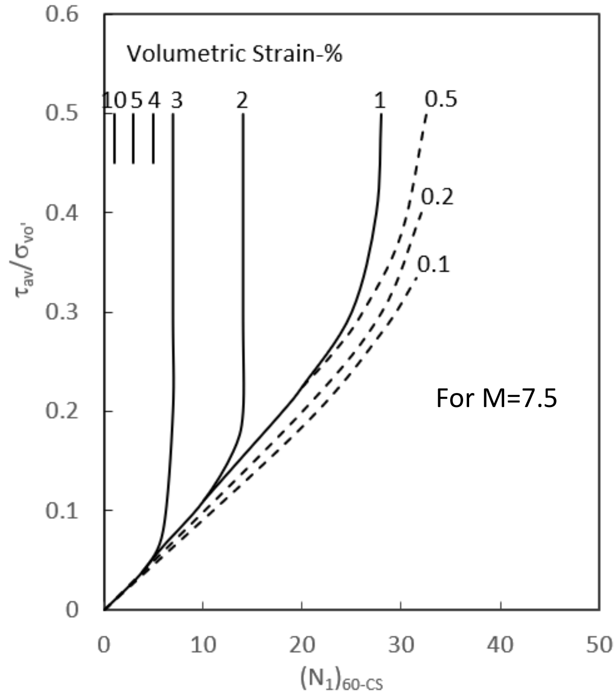
At the depth of 25 feet below the ground surface:

$$\text{Total vertical overburden stress, } \sigma_o(\text{psf}) = 120 \text{ pcf} \times 10 \text{ ft} + 108 \text{ pcf} \times 10 \text{ ft} + 110 \text{ pcf} \times 5 \text{ ft} = 2830 \text{ (psf)}$$

$$\text{Effective vertical overburden stress, } \sigma'_o(\text{psf}) = 120 \text{ pcf} \times 10 \text{ ft} + (108 - 62.4) \text{ pcf} \times 10 \text{ ft} + (110 - 62.4) \text{ pcf} \times 5 \text{ ft} = 1894 \text{ (psf)}$$

Stress reduction factor due to soil flexibility, r_d = 0.942 (Youd et. al, 2001)
Clean sand equivalent normalized SPT blow count, $(N_1)_{60\text{-CS}}$ = 13

$$\text{Cyclic Shear Stress Ratio, } \left(\frac{\tau_{av}}{\sigma'_o}\right)_{M=7.5} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} r_d \frac{1}{r_m} = 0.65 \times \frac{0.7\,g}{g} \times \frac{2830 \text{ psf}}{1894 \text{ psf}} \times 0.942 \times \frac{1}{r_m} = \frac{0.64}{r_m} 0.62$$



Earthquake magnitude, M	Scaling factor for stress ratio, r_m
8.50	0.89
7.50	1.00
6.75	1.13
6.00	1.32
5.25	1.50

Figure A6: Determination of Volumetric Strain of Liquefied Soils (Modified after Tokimatsu and Seed, 1987)

Step 4-4: Plot Pile and Ground Settlement Profiles

Figure A7 shows the pile settlement (Step 4-2) and liquefaction-induced ground settlement (Step 4-3) profiles.

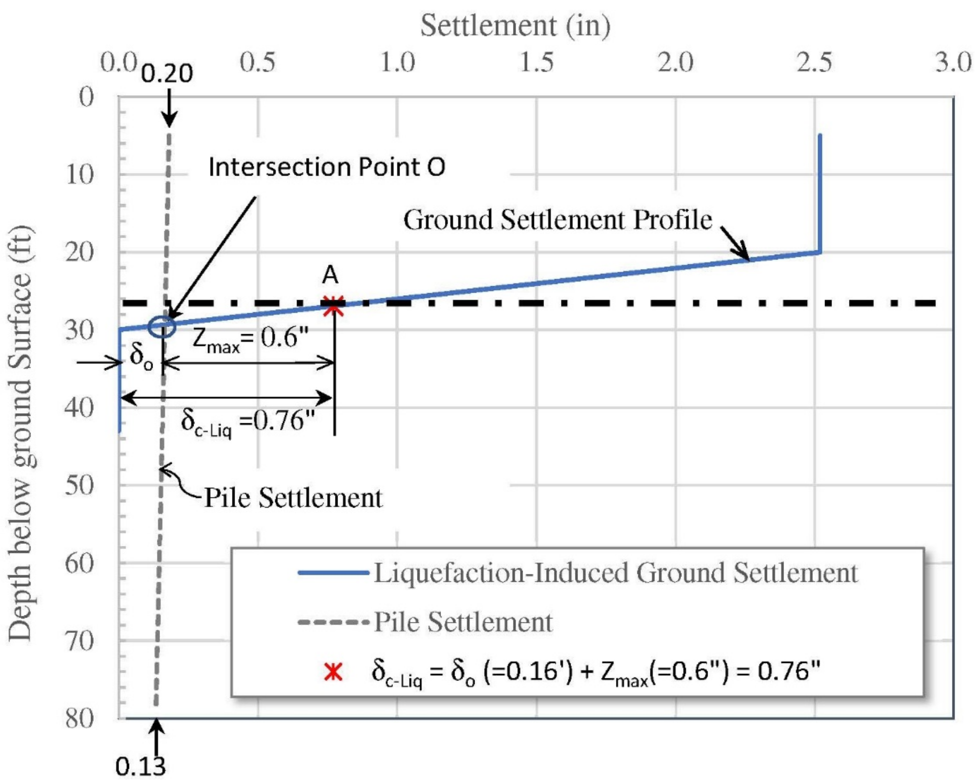


Figure A7: Plot of Pile and Liquefaction-Induced Ground Settlements

Step 4-5: Determine Location of the Maximum Downdrag Load (DDmax)

The relative pile-soil settlement, z_{max}, at which the unit side resistance is fully mobilized is calculated as about 0.9 percent of the pile diameter or 0.60 inches. The 0.9 percent was calculated by using an equivalent depth scheme for the clay and sand layers exerting downdrag load on the pile (i.e. 10 feet times 0.8 percent for the clay layer plus 10 feet times 1.0 percent for the sand layers, then divided by 20 feet).

The pile and ground settlement at the intersection point O (δ_o) is 0.16 inches. The critical ground settlement that will generate the maximum negative side resistance or downdrag load on the pile, $\delta_{c\text{-}Liq} = (\delta_o) + z_{max}$ = (0.16 inches + 0.60 inches) = 0.76 inches.

As shown in Figure A7, locate the Point A on the ground settlement profile representing a ground settlement equal to $\delta_{c\text{-}Liq}$ = 0.76 inches. Draw a horizontal line AA' passing through the Point A. The pile length at the elevation of the line AA' is about 22 feet, corresponding to the ground elevation of -27 feet. The soil layers above the A-A' line exert full downdrag loads on the pile and those below contribute to the pile axial nominal resistance.

Step 4-6: Calculate Maximum Downdrag Load (DDmax)

The nominal downdrag load due to liquefaction-induced settlement was calculated based on a soil friction angle of 30 degrees for the liquefiable soil layer (No. 3), and the soil shear strength parameters in Table A3 for the non-liquefiable soil layers (Nos. 1 and 2). Table A8 presents the output data from SHAFT.

Along the upper 22 feet pile length (from cut-off to A-A' line), the pile nominal side resistance for each soil layer are:

- Layer 1, R_{s1} = 43.3 tons x 2 = 86.6 kips
- Layer 2, R_{s2} = (114.6 tons – 43.3 tons) x 2 = 142.6 kips
- Layer 3 above A-A' line, R_{s3d} = [(206.92 tons – 193.36 tons) x 0.94 + 193.36 tons - 114.57 tons] x 2 = 183.07 kips.

Therefore, the maximum downdrag load, DD_{max} = (R_{s1} + R_{s2} + R_{s3d}) = 86.6 + 142.6+ 183.1 = 412 kips

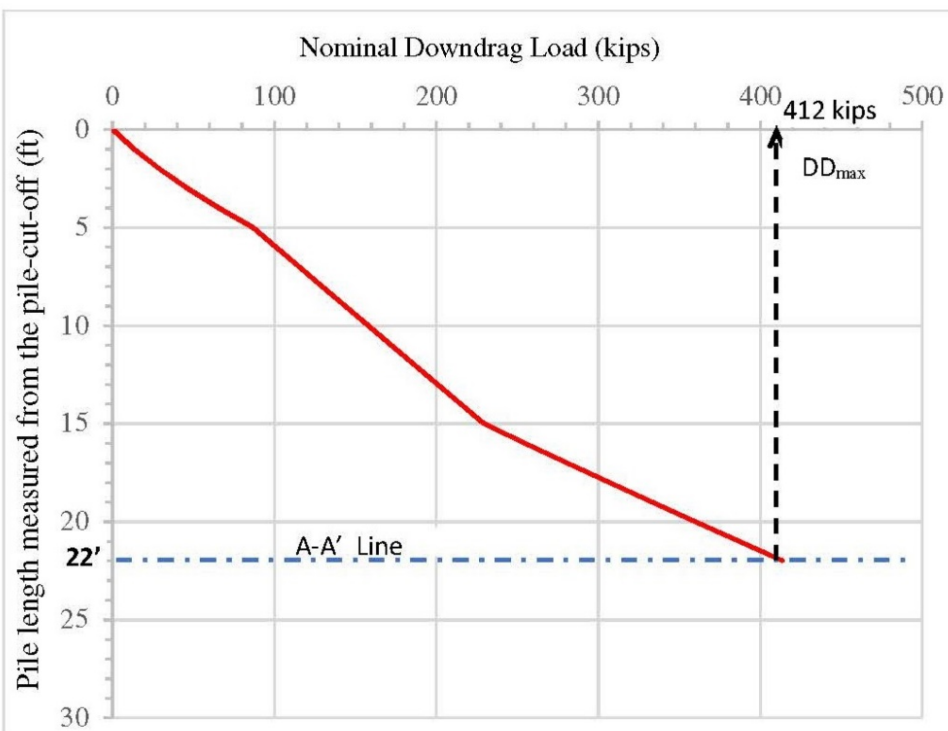


Figure A8: Nominal Downdrag Load vs. Pile Length

Table A8: Summary of Predicted Outputs from SHAFT (Extreme Event-I Downdrag)

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	LRFD QS (TONS)	LRFD QB (TONS)	LRFD QU (TONS)
6.0	5.28	6.67	132.93	139.60	6.67	132.93	139.60
7.0	6.16	14.37	143.23	157.60	14.37	143.23	157.60
8.0	7.04	23.07	150.52	173.59	23.07	150.52	173.59
9.0	7.92	32.72	154.87	187.59	32.72	154.87	187.59
10.0	8.80	43.29	139.44	182.73	43.29	139.44	182.73
11.0	9.68	50.41	122.45	172.87	50.41	122.45	172.87
12.0	10.56	57.54	103.89	161.44	57.54	103.89	161.44
13.0	11.44	64.67	83.74	148.42	64.67	83.74	148.42
14.0	12.32	71.80	62.00	133.80	71.80	62.00	133.80
15.0	13.20	78.93	41.46	120.39	78.93	41.46	120.39
16.0	14.08	86.06	24.95	111.01	86.06	24.95	111.01
17.0	14.96	93.19	12.51	105.70	93.19	12.51	105.70
18.0	15.84	100.31	4.18	104.50	100.31	4.18	104.50
19.0	16.72	107.44	0.00	107.44	107.44	0.00	107.44
20.0	17.60	114.57	27.10	141.67	114.57	27.10	141.67
21.0	18.48	127.36	57.34	184.70	127.36	57.34	184.70
22.0	19.36	140.30	90.78	231.08	140.30	90.78	231.08
23.0	20.24	153.38	127.50	280.88	153.38	127.50	280.88
24.0	21.12	166.58	167.57	334.16	166.58	167.57	334.16
25.0	22.00	179.91	206.55	386.47	179.91	206.55	386.47
26.0	22.88	193.36	239.85	433.21	193.36	239.85	433.21
27.0	23.76	206.92	267.24	474.17	206.92	267.24	474.17
28.0	24.64	220.59	288.52	509.11	220.59	288.52	509.11
29.0	25.52	234.35	303.45	537.81	234.35	303.45	537.81
30.0	26.40	248.21	311.83	560.04	248.21	311.83	560.04
31.0	27.28	262.21	320.21	582.42	262.21	320.21	582.42
32.0	28.16	276.42	328.59	605.01	276.42	328.59	605.01
33.0	29.04	290.82	336.97	627.79	290.82	336.97	627.79
34.0	29.92	305.41	345.35	650.76	305.41	345.35	650.76
35.0	30.80	320.18	353.73	673.90	320.18	353.73	673.90
36.0	31.68	335.10	362.10	697.21	335.10	362.10	697.21
37.0	32.56	350.19	370.48	720.67	350.19	370.48	720.67
38.0	33.44	365.41	378.86	744.27	365.41	378.86	744.27
39.0	34.32	380.78	387.24	768.02	380.78	387.24	768.02
40.0	35.20	396.27	395.62	791.88	396.27	395.62	791.88
41.0	36.08	411.87	404.00	815.87	411.87	404.00	815.87
42.0	36.96	427.59	412.38	839.96	427.59	412.38	839.96
43.0	37.84	443.40	420.75	864.15	443.40	420.75	864.15
44.0	38.72	459.30	429.13	888.44	459.30	429.13	888.44
45.0	39.60	475.29	436.64	911.92	475.29	436.64	911.92
46.0	40.48	491.34	443.20	934.54	491.34	443.20	934.54
47.0	41.36	507.46	448.73	956.19	507.46	448.73	956.19
48.0	42.24	523.64	453.18	976.81	523.64	453.18	976.81
49.0	43.12	539.86	456.46	996.31	539.86	456.46	996.31
50.0	44.00	556.12	458.64	1014.76	556.12	458.64	1014.76
51.0	44.88	572.41	459.95	1032.36	572.41	459.95	1032.36
52.0	45.76	588.72	460.61	1049.32	588.72	460.61	1049.32
53.0	46.64	605.04	460.83	1065.87	605.04	460.83	1065.87
54.0	47.52	621.37	460.83	1082.20	621.37	460.83	1082.20
55.0	48.40	637.70	460.83	1098.52	637.70	460.83	1098.52

Step 4-7: Calculate Pile Nominal Resistances in Compression (RN) below the A-A' Line

The total pile nominal side resistance below the A-A' line (for 22 feet pile length) is calculated utilizing the information in Table A8. The nominal resistance for any soil layer along the pile length can be obtained by subtracting the total nominal side resistance at the top of the layer from that at the bottom of the layer. In the example case, the total nominal side resistance of the pile located below the A-A' line includes the nominal side resistance from the portion of the soil layer no. 3 below the A-A' line (R_{s3r}) and the nominal side resistance from the soil layer no. 4 (R_{s4}). Figure A9 presents plots of the pile nominal side and tip resistances below the A-A' line.

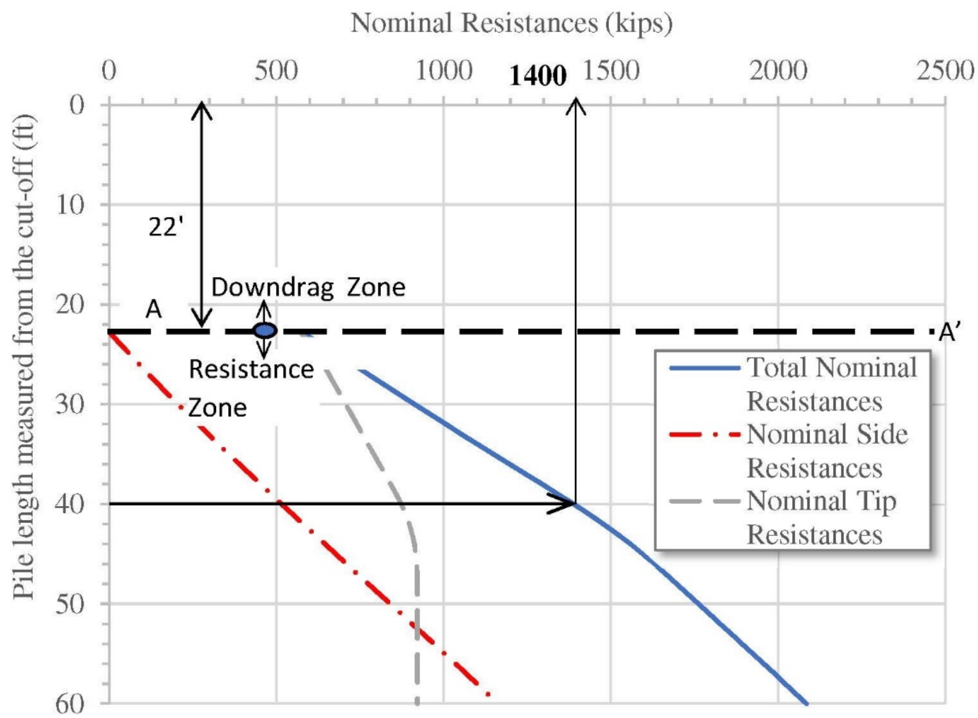


Figure A9: Nominal Resistances with Pile Length for Extreme Event-I Downdrag

Step 4-8: Determine Pile DTE for Compression (Extreme Event-I Downdrag)

For the Extreme Event Limit-I Downdrag case, the factored total seismic design load per pile = (factored max. permanent load per pile) + (factored DD_{max} per pile) + (Factored effective pile weight of the pile). = (886 kips x 1.0) + (412 kips x 1.0) + (W'p kips x 1.0) = 1298 kips + W'p (kips). Note all load and resistance factors are taken as equal to 1.0.

Due to the inclusion of the effective pile weight in the factored design, a trial and error method is necessary to determine the pile DTE from Figure A9. A preliminary estimate of the required pile length = 37 feet can be made based on a factored design load = 1298 kips (i.e. excluding effective weight of the pile).

Trial 1: Assuming the required preliminary pile length = 37 feet, the available total factored resistance can be estimated to about 1270 kips from Table A8 or Figure 9.

The total effective pile weight for the 37 feet long pile, W'p (kips) = 85 kips. In this case, the total factored design load on the pile for the downdrag case = 1298 kips + 85 kips = 1383 kips. This factored design load is greater than the available total factored resistance of 1270 kips for the 37 feet long pile. Therefore, additional pile length is required to support the factored design load.

Trial 2.: Assume pile length = 40 feet. From Figure A9 and Table A8, the available total factored nominal resistance in axial compression for 40 feet long pile is estimated to be about 1400 kips. This total factored nominal resistance consists of total factored nominal side resistance, R_s=530 kips and total nominal pile tip resistance R_p = 870 kips.

The revised total factored load, including effective weight if the pile, for the 40 feet long pile is estimated to be about 1380 kips which is slightly lower than the available total factored resistance of 1400 kips. This pile length is thus adequate to support the applicable factored design load for the Extreme Event (Downdrag) limit state.

Therefore, the design DTE for the Extreme Event (Downdrag) is -45 feet.

Reporting

Based on the above results, for the example case:

- Extreme Event (Compression) case: Geotechnical Nominal Resistance per Pile = 1250 kips, Nominal Tip Resistance = 850 kips and the design DTE = - 43.5 feet.
- Extreme Event (Downdrag) case: Geotechnical Nominal Resistance per Pile = 1400 kips, Nominal Tip Resistance = 873 kips and the design DTE =-42 feet.
- Maximum downdrag load on the pile, DD_{max}= 412 kips.
- The downdrag zone: Top Elevation = - 5 feet and Bottom Elevation = -27 feet.
- Seismic ground settlement at the cut-off elevation, $(\delta_g)_{DD}$ = 2.5 inches.
- Total Ground Settlement $(\delta_g)_{Liq}$ = 2.5 inches.
- No t-z and q-w curves are not requested.

Additional Information

Table A9 presents a partially completed foundation design recommendations table for the example.

Extreme Table A9: Foundation Design Recommendations

Support Location	Pile Type	Cut-off Elev. (feet)	Service-1 Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Geotechnical Nominal Resistance (kips)					Design Tip Elev. (feet)	Specified Design Tip Elev. (feet)
						Strength Limit		Extreme Event Limit (All Resistance Factors = 1.0)				
			Total	Perm.		Comp. (Φqs=0.7) (Φqp=0.5)	Tension (Φqs=0.7)	Comp.	Comp. with Downdrag	Tension		
Bent	66" dia. CIDH	-5.0	8916	7085	1.0	2920	0	1140	1390	0	-79 (a-I) -43.5 (a-II) -45 (a-III) -34.3 (c)	xx

Note: *Design tip elevations are controlled by, (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (a-III) Compression (Extreme Event-Downdrag), (c) Settlement.*

Additional References

Ensoft, Inc (2012), SHAFT A Program for the Study of Drilled Shafts Under Axial Load, Austin, Texas.