Driven Pile Foundations

This document presents the design methods and communication steps between Structure Design (SD) and Geotechnical Services (GS) for the load and resistance factor design (LRFD) of driven pile foundations used for support of bridges, retaining walls, non-standard walls, signs, and other structures. This module discusses design of driven Standard Plan Class piles (Alternative V, W, X, and Y), Cast-In-Steel-Shell (CISS), and H-piles. The Appendices include three design calculation examples.

Standards relating to driven pile foundation investigations, design, and reporting are:

- Seismic Design Criteria (SDC)
- AASHTO LRFD Bridge Design Specifications with CA Amendments (AASHTO)
- American Petroleum Institute (API, 2000) publication RP 2A
- Standard Specifications, Standard Plans, Bridge Standard Detail Sheets (XS Sheets)
- Bridge Memos to Designers (MTD) 3-1, *Deep Foundations*
- Bridge Design Aids
- Bridge Construction Records and Procedures Manual, Volume II
- Geotechnical Manual
    - *Foundation Reports for Bridges*
    - *Geotechnical Investigations*

Geotechnical Service's role in driven pile foundation design is to provide the Structure Designer with a foundation report addressing the following:

- Design Tip Elevations for piles for Service, Strength and Extreme Event Limit State.
- The Controlling Design Tip Elevation.
- The estimated Nominal Driving Resistance.
- Pile acceptance criteria.
- Recommendations relating to specifications and construction.

The Structure Designer's role in deep foundation design includes:

- Providing GS with the foundation design data and factored design load information.
- Providing GS with the latest plan sheets pertinent to foundation design (e.g. General Plan, Foundation Plan, Foundation Detail Sheets, etc.).

Investigations

The goal of the geotechnical investigation for a driven pile foundation is to determine the properties and behaviors of the soil and/or rock, and the groundwater condition that can affect foundation design and construction. All subsurface conditions that might influence the foundation design and performance should be investigated.

Perform a literature search (see *Geotechnical Investigations*) to gather all relevant information related to site geology, strength of soil and rock, and geologic hazards. Then develop a prudent exploration plan considering site constraints, geologic variability, and available resources. Borings should be located as close as possible to the proposed foundation.

The exploration plan should include:

- An appropriate number of exploratory borings and/or cone penetration tests (CPT) to develop the design soil profile (AASHTO Table 10.4.2-1).
- An appropriate depth of exploration for the borings or CPT. The depth of exploration should generally extend below the anticipated pile tip elevation a minimum of 20 feet, or a minimum of two times the maximum pile group dimension, whichever is deeper (AASHTO Table 10.4.2-1).
- Standard penetration tests (SPT). When SPTs are to be performed, sampling intervals should be limited to no more than 5 feet.
- Groundwater measurements.
- Soil and water samples for corrosion testing in accordance with current *Caltrans Corrosion Guidelines*.
- Adequate samples for laboratory testing (e.g. classification tests, consolidation test, soil strength parameters required for design).

Design

The following provides design methodologies used for calculating settlement (Service-I Limit State) and pile resistance (Strength and Extreme Event Limit States) in accordance with AASHTO. For appropriate resistance factors refer to AASHTO Table 10.5.5.2.3-1.

Design methods for calculation of axial pile resistance for various pile and soil types are presented in AASHTO 10.7.3.8.6.

For steel pipe and CISS piles larger than 18" in diameter, the static analysis methods from the American Petroleum Institute (API, 2000) publication RP 2A must be used. It is important to note, the CISS pile tip resistance is the sum of tip resistance of steel shell section and the lesser of (1) Soil plug tip resistance, or (2) Soil plug side resistance based on the final plug length after the pile concrete placement. The plug length is limited to 4 times the pile diameter.

The soil properties used in design should come from: (1) SPT correlations (see *Soil Correlations* module) and/or (2) results from laboratory tests under similar field conditions.

The design must also account for geologic hazards such as:

- Liquefaction (see *Liquefaction* module)
- Lateral spreading (see *Lateral Spreading* module)
- Scour: Foundations that are constructed in a watercourse must meet AASHTO guidelines regarding scour depths (AASHTO C2.6.4.4.2). The top of the pile cap must be below the degradation plus contraction scour depth. The bottom of the pile cap must be below the degradation plus contraction plus local pier scour depth.

Design Information and Communication (Preliminary Foundation Report)

After the field investigation and testing has been completed, the Geoprofessional must review the design information provided by the Structure Designer which should include:

- General Plan
- Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Design Process (Preliminary Foundation Report)

Complete the driven pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Calculate the Preliminary Tip Elevations

- Calculate the preliminary tip elevations meeting the controlling compression and tension requirements for the Strength Limit State at each support location.

Step 3: Complete Preliminary Foundation Design Recommendations table.

- Present the tip elevations for compression in the Preliminary Foundation Design Recommendations table under the Preliminary Tip Elevation column in the Preliminary Foundation Report.

Step 4: Reporting

- Complete the Preliminary Foundation Report according to the *Foundation Reports for Bridges* module.

Design Information and Communication (Foundation Report)

The Geoprofessional must review the design information provided by the Structure Designer which should include:

- General Plan
- Foundation Plan
- Scour Data Table (MTD 3-1, Attachment 1) or Hydraulics Report (if scour potential exists)
- Foundation Design Data Sheet (MTD 3-1, Attachment 1)
- Foundation Factored Design Loads information (MTD 3-1, Attachment 1)

Table X: Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abut 1	Class 140 Alt “V”						
Pier 2	24” diam. CISS						
Abut 3	Class 140 Alt “V”						

Table X: Foundation Factored Design Loads (MTD 3-1, Attachment 1)

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load per Support	Permanent Load per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1							N/A	N/A	N/A	N/A
Pier 2										
Abut 3							N/A	N/A	N/A	N/A

Design Process (Foundation Report)

Complete the driven pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Calculate the Design Tip Elevations for the Piles

- Calculate the design tip elevations meeting the controlling compression and tension requirements for the Strength Limit State and the Extreme Event Limit State at each support location.
- Calculate the design tip elevations meeting the permissible settlement criteria for Service-I Limit State.

  (Commentary: Pile design should ensure that strength limit state considerations are satisfied before checking service limit state considerations. For piles embedded adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived. If the design tip for service limit state is waived, then a note should be placed at the bottom of the pile data table).
- Using the design tip elevation from Strength or Extreme Limit State, verify the settlement under the Service-I Limit State load is less than the permissible settlement.

Step 3: *Calculate the Required Nominal Driving Resistance for the Piles*

- The *Required Nominal Driving Resistance* is calculated for the specified tip elevation, which might include driving through soil not utilized to determine the specified tip elevation (e.g., scourable or liquefiable layers).

Step 4: Complete the Tables

- Present the tip elevations for compression, tension, and settlement in the Foundation Design Recommendations and Pile Data Tables.

Step 5: Prepare and Send Draft Foundation Report

- Complete the Draft Foundation Report according to the *Foundation Reports for Bridges* module.

Step 6: Determine the Specified Tip Elevation (Lateral Tip Considerations)

- Obtain the lateral tip elevation from the Structure Designer
  - If the lateral tip is higher than or equal to the specified tip elevation then there is no action required by the GP.
  - If the lateral tip is lower, the GP must re-calculate the Required Nominal Driving Resistance, verify that the pile can be installed to the lateral tip elevation, and that all other recommendations in the report are correct (e.g., pile tip is now below groundwater, pile tip is now in rock).

Step 7: Reporting

- Complete the Foundation Report according to the *Foundation Reports for Bridges* module.

Driven Pile Acceptance Criteria

The following information relates to driving system submittals, dynamic monitoring, and Wave Equation Analysis Program (WEAP) bearing acceptance criteria (AASHTO LRFD BDS, CA Amendment 10.7.3.8.4), and pile load testing.

- Require a driving system submittal (see *Notes for Specifications*) for all non-standard plan piles, except H-Piles. For all other piles, the need for a driving system submittal should be evaluated based upon anticipated pile installation difficulties and/or concern about pile damage during installation. The Foundation Testing and Instrumentation Branch must review and approve all driving system submittals submitted to GS by Structure Construction.
- For driven piles with a nominal resistance greater than 600 kips, dynamic monitoring and analyses are required to develop the bearing acceptance criteria.
- For driven piles with a diameter, or side dimension, of 18 inches to 36 inches, dynamic monitoring and analyses are required to develop the bearing acceptance criteria.
- For driven piles that are larger than 36 inches, select dynamic monitoring and a pile load test is required to measure the nominal resistance. Exceptions to the requirement to perform an axial load test may include situations where geologic conditions (e.g., bearing in rock or very dense soil) provide confidence in the nominal resistance development. These cases must be reviewed by Foundation Testing and Instrumentation Branch and require approval of a design exception from the Office of Geotechnical Design Policies and Practices (OGDPP) and Structures Policy and Innovation (SP&I).

The Geoprofessional must consult with the Foundation Testing and Instrumentation (FTI) branch for all language and requirements included in the Pile Load Testing and Dynamic Monitoring section of the Foundation Report. The FTI branch will provide technical assistance in determining dynamic monitoring details, order of work, and layout of the load test piles. In addition, FTI can assist with contractual details and information for the special provisions. If a foundation type that would require a pile load test is being considered, the Geoprofessional must have a meeting with the FTI branch and the Structure Designer to review proposed pile load test loading schedule, layout, and to determine if any test or anchor piles may be incorporated in the proposed bridge foundation.

Attachments

- Appendix A: Example Design Calculations for Concrete Pile
- Appendix B: Example Design Calculations for H-Pile
- Appendix C: Example Calculations for CISS Pile

Appendix A: Nordlund Method and Alpha (α) Method for Concrete Pile

This design example presents calculations for the Strength Limit State only. A typical pile design includes evaluation of Service Limit State, Strength Limit State, and Extreme Event Limit State. The Extreme Event Limit State analysis is the same as this example, except for designing for the Extreme Event Limit State required nominal resistance and scour percentages.

For pile foundations in cohesive soils or layers and loose granular soils, pile group settlement must be evaluated by using the equivalent footing method for Service Limit State design. Figures that show soil layers and stress distribution for various cases can be found in AASHTO LRFD Bridge Design Specifications Section 10.7.2.3.

In conditions where the piles are driven into very dense layers or rock, and settlement is anticipated to be minimal, Service Limit State calculation may not be needed. However, If the pile is driven into a very dense layer that is underlain by a loose granular or cohesive layer, then settlement should be evaluated.

Lateral design of piles is usually performed by the structure designer, however the Geoprofessional typically provides the structure designer the soil parameters for the L-Pile analysis in Class S2 soils per Seismic Design Criteria 6.1.2.

Information provided by Structure Design

- 12” square concrete pile is used for this example.
- Foundation information and loads provided in the tables below.

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Pile Type	Finished Grade Elevation (feet)	Pile Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement Under Service Load (inches)	Number of Piles per Support
				B	L		
Pier 2	Class 140 Alt. X	100.0	90.0	14	126	2	82

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Service I Limit State (kips)		Strength/Construction Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Pier 2	5370	4730	8770	150	0	0	1100	120	0	0

Geotechnical Design Considerations

- The Geoprofessional will take the factored load and apply the resistance factor (0.7 for both side and tip resistance for Strength Limit State, and 1.0 for both side and tip resistance for Extreme Event Limit State) and then round up to the nearest 10 kips. For this example, the Required Nominal Resistance for the Strength Limit State and Extreme Event Limit State are 220 kips and 120 kips, respectively. The Foundation Design Recommendations Table and the Pile Data Table examples are presented at the end of this example.
- Pile Perimeter (C_d) = 4.0'
- Pile width: 12" = 1.0'
- Area of Concrete pile end: A_p = 1.0 ft^2
- Pile taper angle, $\omega = 0°$
- Scour information for this example is presented below:

Scour Data Table

Support No.	Long-Term (Degradation and Contraction) Scour Elevation (ft)	Short-Term (Local) Scour Depth (ft)
Pier 2	95.0	5.0

- AASHTO LRFD Bridge Design Specifications – California Amendments, Table 3.7.5-1, provides the percentages of Long-Term (Degradation + Contraction) and Short-Term (Local) scour to be used for each limit state.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

- When calculating nominal resistance:
  1. The vertical effective stress is calculated using 100% of Long-Term (Degradation + Contraction) scour and 0% Short-Term (Local) scour for Strength Limit and Extreme Event Limit states.
  2. Side resistance pile-soil contact is calculated using 100% Long-Term and 50% Short-Term (Local) scour for Strength Limit State, and 100% Long-Term and 0% Short-Term scour for Extreme Event Limit State. See the table below for information.

Effect of Scour on Pile Design				
	Vertical Effective Stress		Side Resistance	
	Long-Term	Short-Term	Long-Term	Short-Term
Service	0%	0%	N/A	N/A
Strength	100%	0%	100%	50%
Extreme	100%	0%	100%	0%

- For this example, the Strength Limit State pile tip elevation is based on 100% Long-Term (Degradation + Contraction) scour + 50% Short-Term (Local) scour. For vertical effective stress calculations, only Long-Term scour is considered (down to elevation 95 ft), therefore only five feet of Layer #1 contributes to vertical effective stress. Short-Term scour does not affect vertical effective stress. In this example 50% of Short-Term scour is at elevation 92.5 ft. If the Geoprofessional is performing analysis for the Extreme Event Limit State, 0% of Short-Term scour would be used for both vertical effective stress and pile-soil contact area calculations.

- AASHTO LRFD Bridge Design Specifications – California Amendments, Figure C2.6.4.4.2-2, shows that the bottom of pile cap must be at or below the total scour elevation. The cutoff elevation is at elevation 90.0 feet. This is the total scour elevation and below the calculated elevation for pile-soil contact. Therefore, elevation 90.0 feet is the top of the pile-soil contact in the nominal resistance calculations.

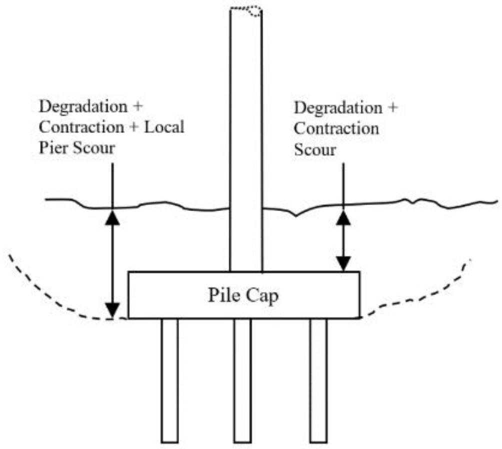
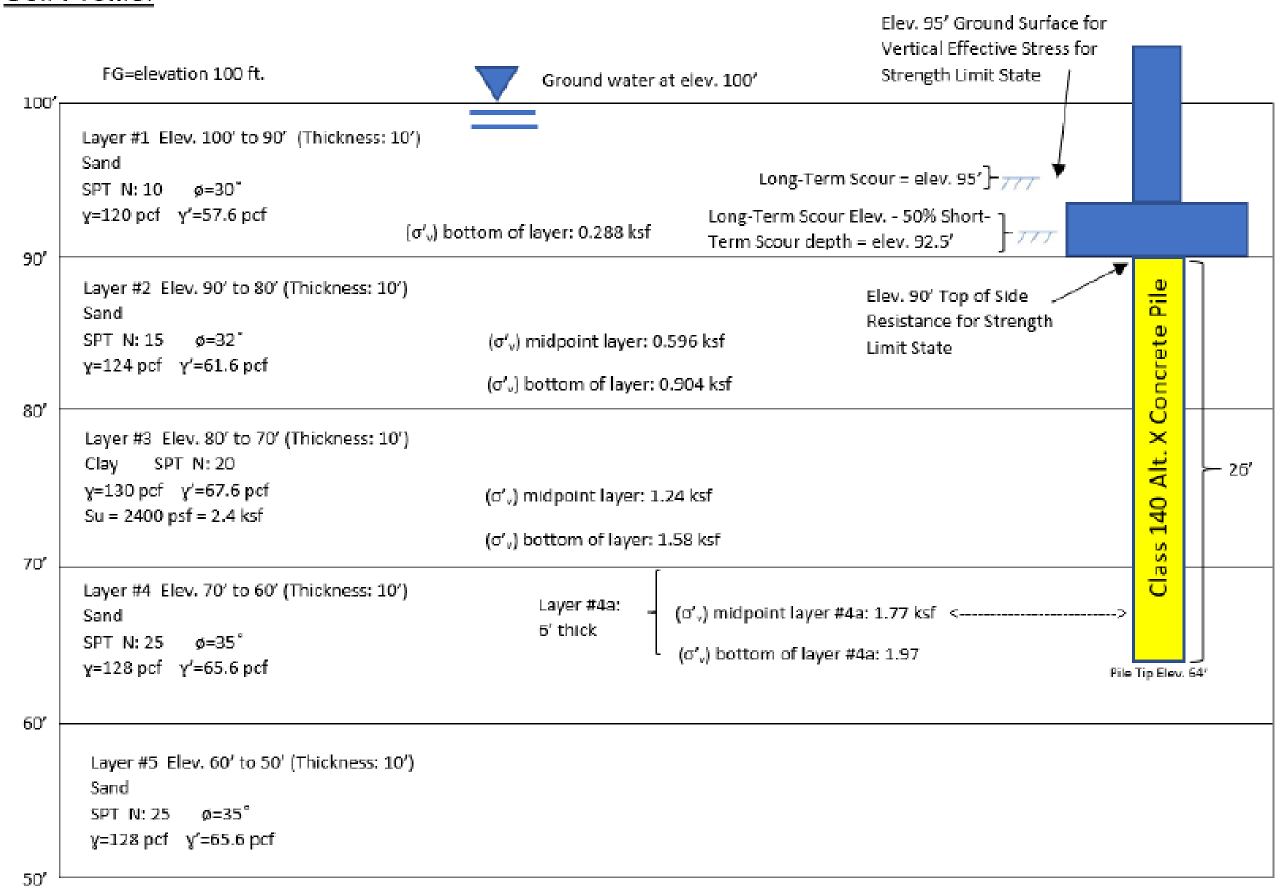


Figure C2.6.4.4.2-2 Deep Foundation Location

Soil Profile:



For cohesionless soil layers, use the Nordlund method as shown in Section 10.7.3.8.6f of AASHTO LRFD Bridge Design Specifications. For cohesive soil layers, use the Alpha (α) method as shown in Section 10.7.3.8.6b of AASHTO LRFD Bridge Design Specifications. The following steps should be used to calculate the nominal pile resistance (R_n), which is the sum of the pile side resistance (R_s) and the pile tip resistance (R_p).

Step 1: Divide the soils into layers and determine the γ for all soil layers, determine the ø angle for each cohesionless soil layer, and the undrained shear strength S_u for each cohesive soil layer. (See Soil Profile above).

Step 2: Determine δ, the friction angle between the pile and soil based on displaced soil volume, V, and the soil friction angle, ø.

Step 2a: Compute volume of soil displaced per unit length of pile, V.

Area of pile tip: 12 in^2 = 1.0 ft^2, therefore: **V = 1.0 ft^3/ft**

Step 2b:

Using Figure 10.7.3.8.6f-6 from AASHTO LRFD Bridge Design Specifications, using V = 1.0 ft^3/ft, determine **δ/ø$_f$ = 0.77**

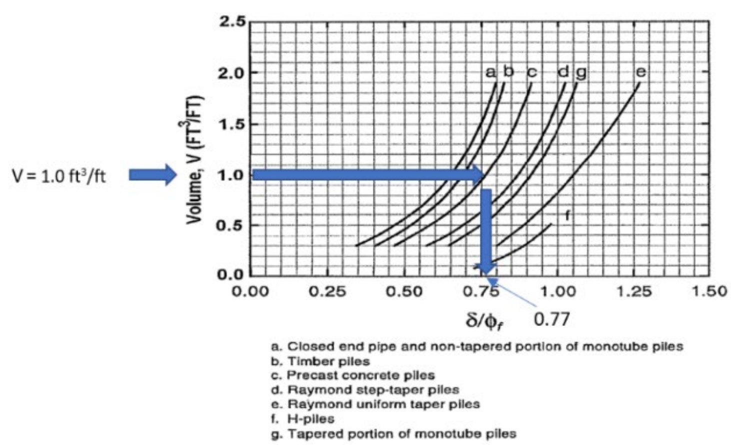


Figure 10.7.3.8.6f-6—Relation of δ/ϕ_f and Pile Displacement, V, for Various Types of Piles (Hannigan et al., 2006 after Nordlund, 1979)

Step 2c: For cohesionless layers, calculate δ from δ/ø$_f$ ratio:

Soil layer #1: (0.77) * (30°) = δ = 23.1°
Soil layer #2: (0.77) * (32°) = δ = 24.6°
Soil layer #3: Cohesive
Soil layer #4: (0.77) * (35°) = δ = 27.0°

Step 3: For cohesionless layers, determine the coefficient of lateral earth pressure, K_δ, based on the ø angle of the soil layer.

Determine K_δ for ø angle based on displaced volume, V= 1.0 ft^3/ft, and pile taper angle, $\omega = 0^\circ$, using AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, interpolating where necessary. Figure 10.7.3.8.6f-2 below is an example for soil layer #1 with ø=30° and Figure 10.7.3.8.6f-3 below is an example for soil layer #4 with ø=35° using note "a" below. Soil layer #2 was determined by using the process in note "b", below). Figures 10.7.3.8.6f-1 and 4 are presented for reference.

Soil layer #1: ø=30° K_δ = 1.15
Soil layer #2: ø=32° K_δ = 1.39
Soil layer #3: Cohesive
Soil layer #4: ø=35° K_δ = 1.75

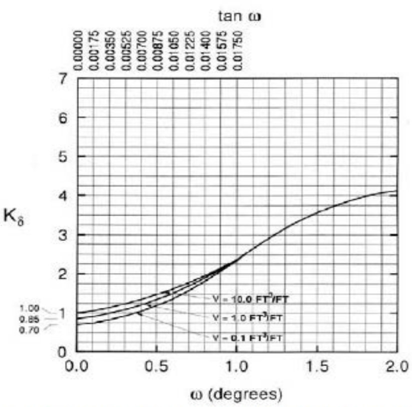


Figure 10.7.3.8.6f-1—Design Curve for Evaluating K_δ for Piles where ϕ_f = 25 degrees (Hannigan et al., 2006 after Nordlund, 1979)

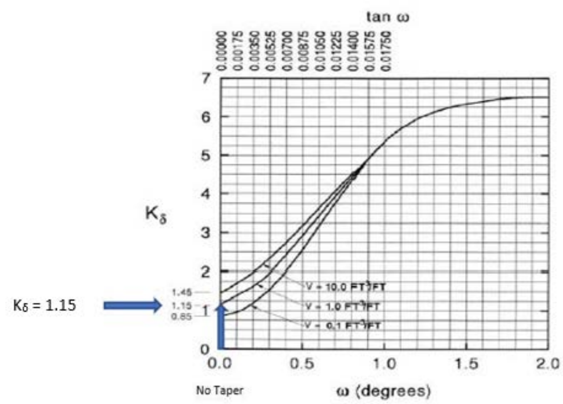


Figure 10.7.3.8.6f-2—Design Curve for Evaluating K_δ for Piles where ϕ_f = 30 degrees (Hannigan et al., 2006 after Nordlund, 1979)

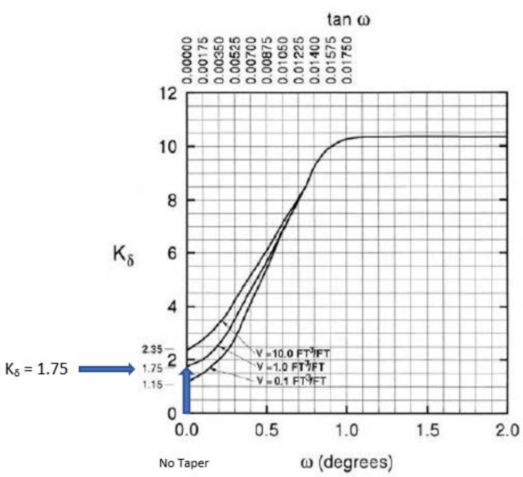


Figure 10.7.3.8.6f-3—Design Curve for Evaluating K_δ for Piles where ϕ_f = 35 degrees (Hannigan et al., 2006 after Nordlund, 1979)

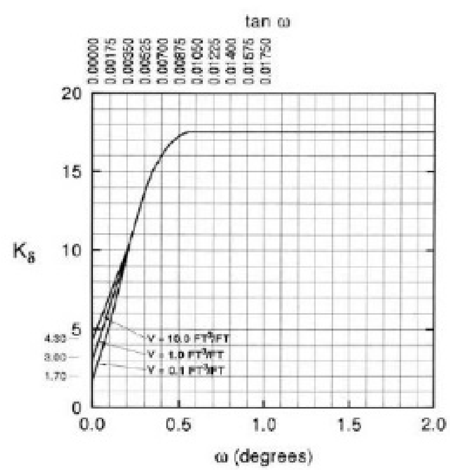


Figure 10.7.3.8.6f-4—Design Curve for Evaluating K_δ for Piles where ϕ_f = 40 degrees (Hannigan et al., 2006 after Nordlund, 1979)

Notes:

a) If the displaced volume is 0.1, 1.0 or 10.0 ft^3/ft, which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle is one of those provided, K_δ can be determined directly from the appropriate figure.

b) If the displaced volume is 0.1, 1.0, or 10.0 ft^3/ft which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle is different from those provided, use linear interpolation to determine K_δ for the required ø angle.

c) If the displaced volume is other than 0.1, 1.0 or 10.0 ft^3/ft which corresponds to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle corresponds to one of those provided, use log linear interpolation to determine K_δ for the required displaced volume.

d) If the displaced volume is other than 0.1, 1.0, or 10.0 ft^3/ft which correspond to one of the curves provided in in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle does not correspond to one of those provided, first use linear interpolation to determine K_δ for the required ø angle at the displaced volume curves provided for 0.1, 1.0 and 10.0 ft^3/ft. Then use log linear interpolation to determine K_δ for the required displaced volume.

Step 4: For cohesionless layers, determine the correction factor, C_F, to be applied to K_δ, if δ ≠ ø angle using AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-5. (Example below is for ø=30°). $\delta/ø_f = 0.77$ is from Step 2b, above.

Soil layer #1: ø=30°	$\delta/ø_f = 0.77$	therefore $C_F = 0.92$
Soil layer #2: ø=32°	$\delta/ø_f = 0.77$	therefore $C_F = 0.91$
Soil layer #3: Cohesive		
Soil layer #4: ø=35°	$\delta/ø_f = 0.77$	therefore $C_F = 0.89$

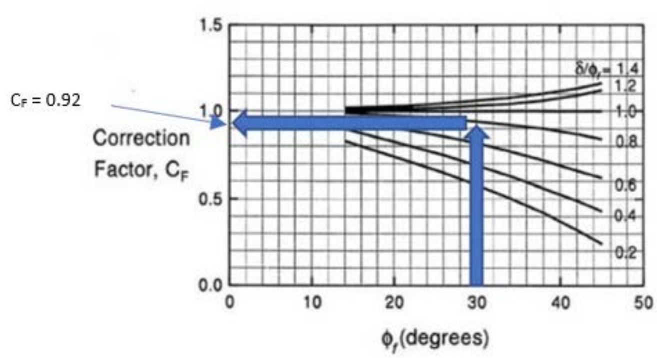


Figure 10.7.3.8.6f-5—Correction Factor for K_δ where $\delta \neq \phi_f$ (Hannigan et al., 2006 after Nordlund, 1979)

Step 5: Compute the vertical effective stress (σ'_v) at the midpoint and bottom of each relevant soil layer. (See Soil Profile above).

Layer #1:
Midpoint of Layer #1: Not relevant for this example
Bottom of Layer #1: 5'(γ'=57.6 pcf) = 288 psf = 0.288 ksf

Layer #2:
Midpoint of Layer #2: 5'(γ'=61.6 pcf) + 288 psf = 596 psf = 0.596 ksf
Bottom of Layer #2: 5'(γ'=61.6 pcf) + 596 psf = 904 psf = 0.904 ksf

Layer #3:
Midpoint of Layer #3: 5'(γ'=67.6 pcf) + 904 psf = 1242 psf = 1.24 ksf
Bottom of Layer #3: 5'(γ'=67.6 pcf) + 1242 psf = 1580 psf = 1.58 ksf

Layer #4:
Midpoint of Layer #4a: 3'(γ'=65.6 pcf) + 1580 psf = 1777 psf = 1.77 ksf
Bottom of Layer #4a: 3'(γ'=65.6 pcf) + 1777 psf = 1974 psf = 1.97 ksf

Step 6: For cohesive layers, determine the adhesion factor, α, using the applicable AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6b-1.

In this example, Layer #3 is below two sand layers, so the top diagram best fits the example.
Using an undrained shear strength (S_u) of 2.4 ksf for Layer #3, using a D_b=less than 10D, α = 1.0.

Where:

- D_b = Pile embedment into the clay layer (10' for this example).
- D = Pile width (1.0' for this example).

Layer # 3: **α = 1.0**

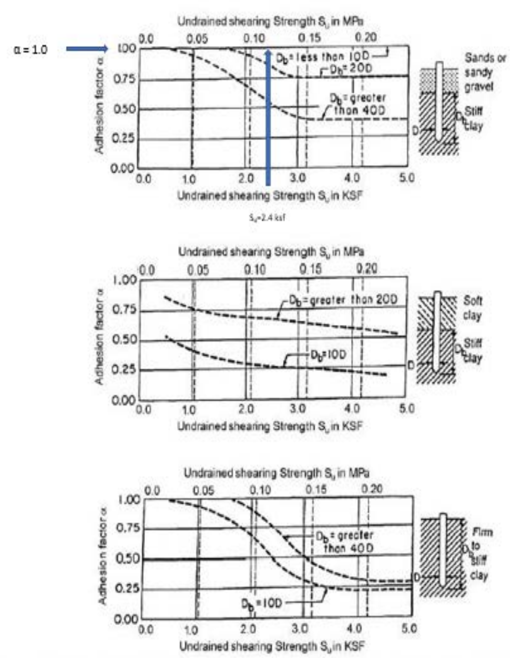


Figure 10.7.3.8.6b-1—Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (1980)

Compute the Strength Limit State Compression Design Tip Elevation

Step 7: Compute the pile side resistance (R_s).

Step 7a: Compute the pile side resistance for each soil layer.

For cohesionless layers:

$q_s = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})$ (AASHTO 10.7.3.8.6f-1)

$A_s = (C_d)(\Delta d)$

$R_s = (q_s)(A_s) = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2)
- K_δ = Coefficient of lateral earth pressure (from Step 3)
- C_F = Correction factor to be applied to K_δ (from Step 4)
- σ'_v = Vertical effective stress at midpoint of each relevant soil layer (from Step 5) (ksf)
- δ = Friction angle between the pile and soil (from δ in Step 2c)
- ω = Pile taper angle. (In this example no taper, so $\omega = 0°$)
- C_d = Pile perimeter = 4.0' for this example
- Δd = Thickness of soil layer (ft)

For cohesive layers:

$q_s = (\alpha)(S_u)$ (AASHTO 10.7.3.8.6b-1)

$A_s = (C_d)(\Delta d)$

$R_s = (q_s)(A_s) = (\alpha)(S_u)(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2)
- α = Adhesion factor (from Step 6)
- S_u = Undrained shear strength of the soil layer (see soil profile) (ksf)
- C_d = Pile perimeter = 4.0' for this example
- Δd = Thickness of soil layer (ft)

Layer #1: No side resistance contribution from Layer #1

Layer #2: (cohesionless)

- $(K_\delta{=}1.39)(C_F{=}0.91)(\sigma'_v{=}0.596 \text{ ksf})(\frac{\sin(24.6^\circ+0^\circ)}{\cos 0^\circ})(C_d{=}4.0')(\Delta d{=}10.0') = 12.6$ kips

Layer #3 (cohesive):

- $(\alpha{=}1.0)(S_u{=}2.4 \text{ ksf})(C_d{=}4.0')(\Delta d{=}10.0') = 96.0$ kips

Layer #4 (cohesionless):

- $(K_\delta{=}1.75)(C_F{=}0.89)(\sigma'_v{=}1.77 \text{ ksf})(\frac{\sin(27.0^\circ+0^\circ)}{\cos 0^\circ})(C_d{=}4.0')(\Delta d{=}6.0') = 30.0$ kips

Step 7b: After computing the side resistance in each soil layer, sum the side resistances from each soil layer to obtain the total pile side resistance (R_s).

- $\Sigma R_s = 12.6 + 96.0 + 30 = 138$ kips

Step 8: Determine the pile tip resistance (R_p):

$q_p = (\alpha_t)(N'_q)(\sigma'_v)$

$R_p = (q_p)(A_p) = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

Where:

- q_p = Nominal unit tip resistance (ksf)
- α_t = Coefficient from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7
- N'_q = Bearing capacity factor from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8
- σ'_v = Vertical effective stress at bottom of Layer #4a (at pile tip) (ksf)
- A_p = Area of the concrete pile end = 1.0 ft^2

Step 8a: Determine α_t coefficient by entering ø angle of the soil at the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7 intersecting the D/b ratio line. For this example, ø=35°; D/b ratio is: Pile Length/Pile Width= 26'/1.0' = 26.

Therefore, $\boldsymbol{\alpha_t}$ **= 0.67**

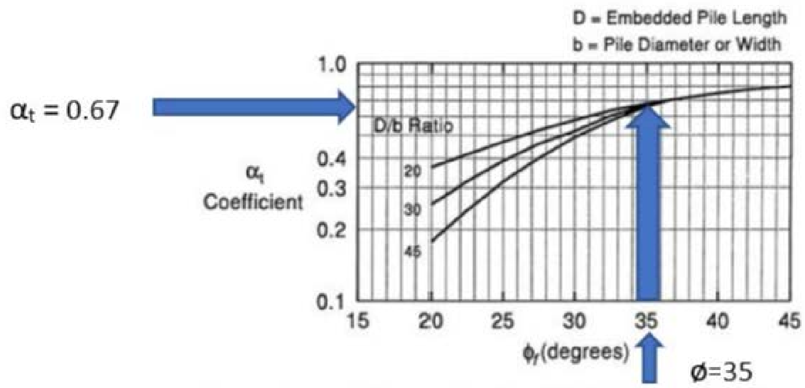


Figure 10.7.3.8.6f-7—α_t Coefficient (Hannigan et al., 2006 modified after Bowles, 1977)

Step 8b: Determine bearing capacity factor, N'_q , by entering ø angle of the soil at the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8. For this example, ø=35°.

Therefore, $\mathbf{N'_q}$ **= 65**

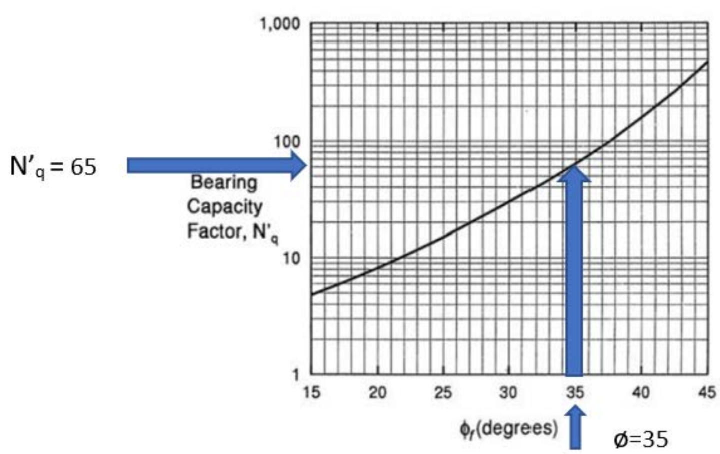


Figure 10.7.3.8.6f-8—Bearing Capacity Factor, N'_q (Hannigan et al., 2006 modified after Bowles, 1977)

Step 8c: Determine the vertical effective stress (σ'_v) at the pile tip.

The vertical effective stress (σ'_v) at the bottom of Layer #4a = 1.97 ksf. AASHTO LRFD Bridge Design Specifications section 10.7.3.8.6f states it must be limited to ≤ 3.2 ksf.

Therefore, σ'_v = 1.97 ksf.

Step 9: Compute the pile tip resistance (R_p) .

$R_p = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

$R_p = (0.67)(65)(1.97\text{ ksf})(1.0\text{ ft}^2)$ = **86 kips**

Step 9a: Compare pile tip resistance (R_p) to the limiting unit pile tip resistance (q_L)

The calculated pile tip resistance must be compared to the limiting unit pile tip resistance (q_L). The limiting q_L value is obtained from entering the ø angle of soil at the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-9. For this example, ø=35°, therefore: **q_L = 100 ksf**

$R_{p\ limited} = (q_L)\,(A_p) = (100\text{ ksf})\,(1.0\text{ ft}^2)$ = **100 kips**

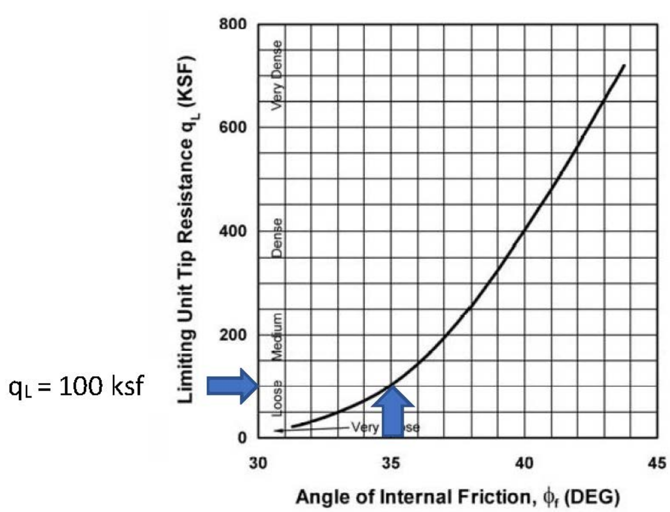


Figure 10.7.3.8.6f-9—Limiting Unit Pile Tip Resistance (Hannigan et al., 2006 after Meyerhof, 1976)

Step 9b: Use the lesser of either the R_p or $R_{p\ limited}$ value. Therefore, use **R_p = 86 kips**

Note: If the pile tip is in a cohesive soil layer, the pile tip resistance (R_p) is determined from AASHTO LRFD Bridge Design Specifications equation 10.7.3.8.6e-1:

$$\mathbf{R_p=(9S_u)\ (A_p)}$$

Step 10: Compute the Nominal Pile Resistance (R_n):

$\mathbf{R_n}$ = R_s + R_p = 138 kips + 86 kips = **224 kips** (rounded to **230 kips**) at the design tip elevation of 64.0 feet.

Once the pile design tip elevation is calculated, it must be presented in both the Foundation Design Recommendations Table and the Pile Data Table in the Foundation Report. Example tables are shown below.

Foundation Design Recommendations

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (in)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	Comp. (φ_{qs}=1.0) (φ_{qp}=1.0)	Tension (φ_{qs}=1.0)			
Pier 2	Class 140 Alt. X	90.0	5370	4730	2	220	0	120	0	64.0 (a-I)	64.0	230

Note: Design tip elevations are controlled by (a-I) Compression (Strength Limit)

Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Pier 2	Class 140 Alt. X	220	0	64.0 (a)	64.0	230

Note: Design tip elevations are controlled by (a) Compression

Appendix B: Nordlund Method for H-pile

This design example is for Strength Limit State only. A typical pile design includes evaluation of Service Limit State, Strength Limit State, and Extreme Event Limit State. The Extreme Event Limit State analysis is the same as this example, except for designing for the Extreme Event Limit State required nominal resistance and scour percentages.

For pile foundations in cohesive soils or layers and loose granular soils, pile group settlement must be evaluated by using the equivalent footing method for Service Limit State design. Figures that show soil layers and stress distribution for various cases can be found in AASHTO LRFD Bridge Design Specifications Section 10.7.2.3.

In conditions where the piles are driven into very dense layers or rock, and settlement is anticipated to be minimal, Service Limit State calculation may not be needed. However, If the pile is driven into a very dense layer that is underlain by a loose granular or cohesive layer, then settlement should be evaluated.

Lateral design of piles is usually performed by the structure designer, however the Geoprofessional typically provides the structure designer the soil parameters for the L-Pile analysis in Class S2 soils per Seismic Design Criteria 6.1.2.

Information provided by Structure Design

- HP 10X57 pile is used for this example.
- Foundation information and loads will be provided from the Structure Designer, as shown in the example tables below.

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Pile Type	Finished Grade Elevation (feet)	Pile Cut-off Elevation (feet)	Pile Cap Size (ft)		Permissible Settlement Under Service Load (inches)	Number of Piles per Support
				B	L		
Pier 2	HP 10X57 H-pile	100.0	90.0	6	50	1	14

Foundation Factored Design Loads Provided by Structure Designer

Support No.	Service I Limit State (kips)		Strength/Construction Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Pier 2	760	550	1090	150	0	0	1020	120	0	0

Geotechnical Design Considerations

- The Geoprofessional will take the factored load and apply the resistance factor (0.7 for both side and tip resistance for Strength Limit State, and 1.0 for both side and tip resistance for Extreme Event Limit State) and then round up to the nearest 10 kips. For this example, the Required Nominal Resistance for the Strength Limit State and Extreme Event Limit State are 220 kips and 120 kips, respectively. The Foundation Design Recommendations Table and the Pile Data Table examples are presented at the end of this example.
- Pile Perimeter (Cd) = 3.37' (Using "box" area of pile side)
- H-pile width: 10.225" = 0.85'
- Area of H-pile steel section: 16.8 in2 = 0.117 ft2
- Pile taper angle, $\omega = 0^{\circ}$
- Scour information for this example is presented below:

Scour Data Table

Support No.	Long-Term (Degradation and Contraction) Scour Elevation (ft)	Short-Term (Local) Scour Depth (ft)
Pier 2	95.0	5.0

- AASHTO LRFD Bridge Design Specifications – California Amendments, Table 3.7.5-1, provides the percentages of Long-Term (Degradation + Contraction) and short-term (Local) scour to be used for each limit state.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

- When calculating nominal resistance:
    1) The vertical effective stress is calculated using 100% of Long-Term (Degradation + Contraction) scour and 0% Short-Term (Local) scour for Strength Limit and Extreme Event Limit states.
    2) Side resistance pile-soil contact is calculated using 100% Long-Term and 50% Short-Term (Local) scour for Strength Limit State, and 100% Long-Term and 0% Short-Term scour for Extreme Event Limit State. See the table below for information.

Effect of Scour on Pile Design				
	Vertical Effective Stress		Side Resistance	
	Long-Term	Short-Term	Long-Term	Short-Term
Service	0%	0%	N/A	N/A
Strength	100%	0%	100%	50%
Extreme	100%	0%	100%	0%

- The Strength Limit State pile nominal resistance calculation is based on 100% long-term (Degradation + Contraction) scour + 50% short-term (local) scour. For vertical effective stress calculations, only long-term scour is considered (down to elevation 95 ft), therefore only five feet of Layer #1 contributes to vertical effective stress. Short-term scour does not affect vertical effective stress. In this example 50% of short-term scour is at elevation 92.5 ft. If the Geoprofessional is performing analysis for the Extreme Event Limit State, 0% of short-term scour would be used for both vertical effective stress and pile-soil contact area calculations.

- AASHTO LRFD Bridge Design Specifications – California Amendments, Figure C2.6.4.4.2-2, shows that the bottom of pile cap must be at or below the total scour elevation. The cutoff elevation is at elevation 90.0 feet. This is the total scour elevation and below the calculated elevation for pile-soil contact. Therefore, elevation 90.0 feet is the top of the pile-soil contact in the nominal resistance calculations.

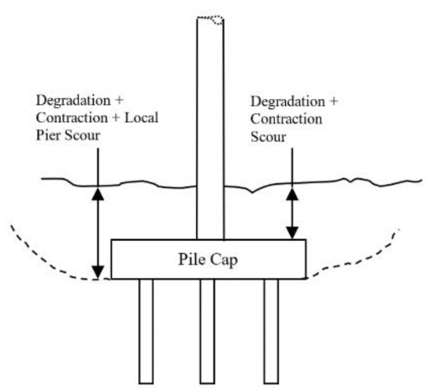
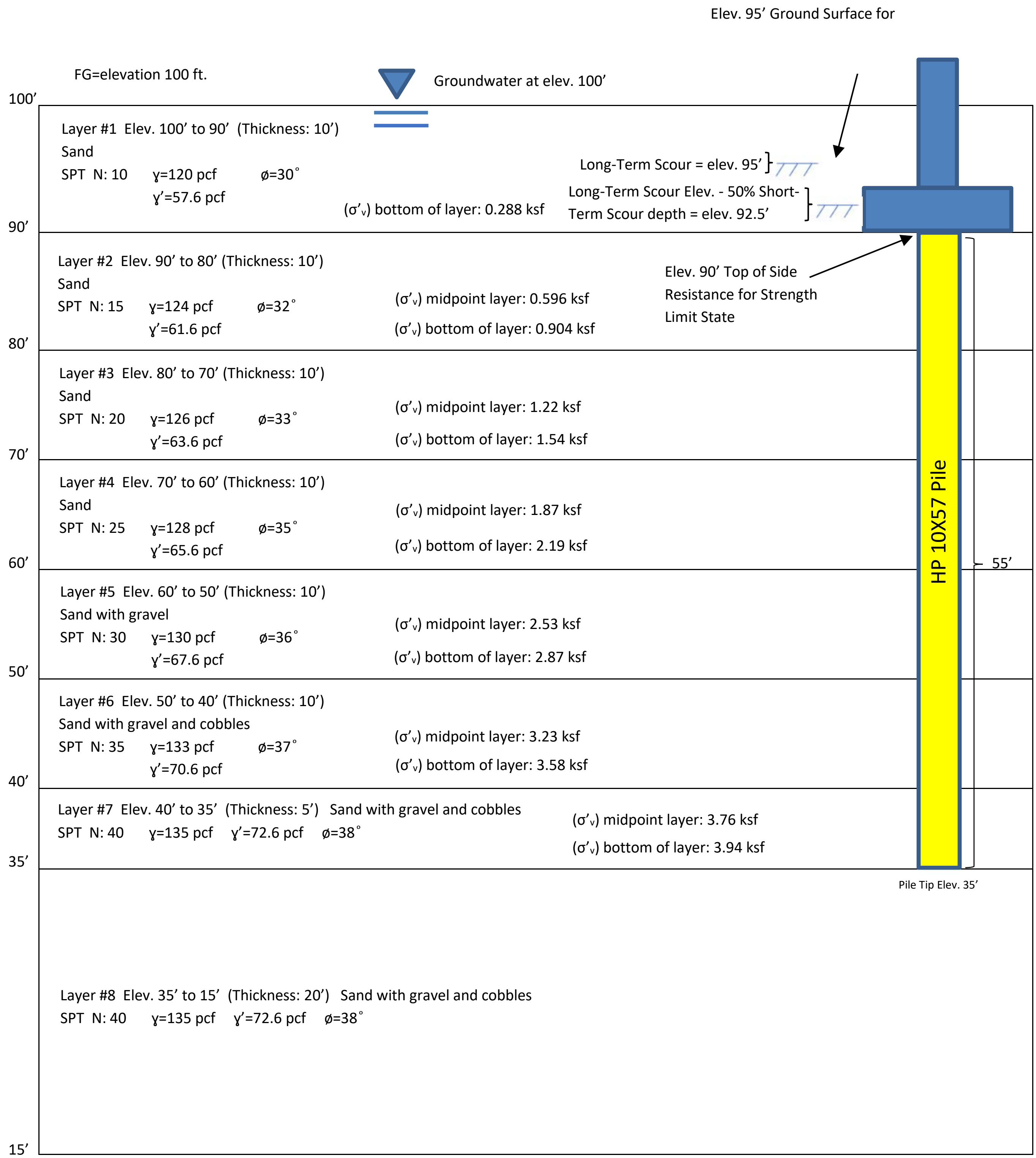


Figure C2.6.4.4.2-2 Deep Foundation Location

Soil Profile:



Using the Nordlund method as shown in Section 10.7.3.8.6f of AASHTO LRFD Bridge Design Specifications, the following steps should be used to calculate the nominal pile resistance (R_n), which is the sum of the pile side resistance (R_s) and the pile tip resistance (R_p).

Step 1: Divide the soils into layers and determine the γ and ø angle for each layer. (See Soil Profile above).

Step 2: Determine δ, the friction angle between the pile and soil based on displaced soil volume, V, and the soil friction angle, ø.

Step 2a: Compute volume of soil displaced per unit length of H-pile, V.

Area of H-pile steel section: 16.8 in^2 = 0.117 ft^2, therefore: **V = 0.117 ft^3/ft**

Step 2b: Using Figure 10.7.3.8.6f-6 from AASHTO LRFD Bridge Design Specifications, using V = 0.117 ft^3/ft, determine **δ/ø$_f$ = 0.761**

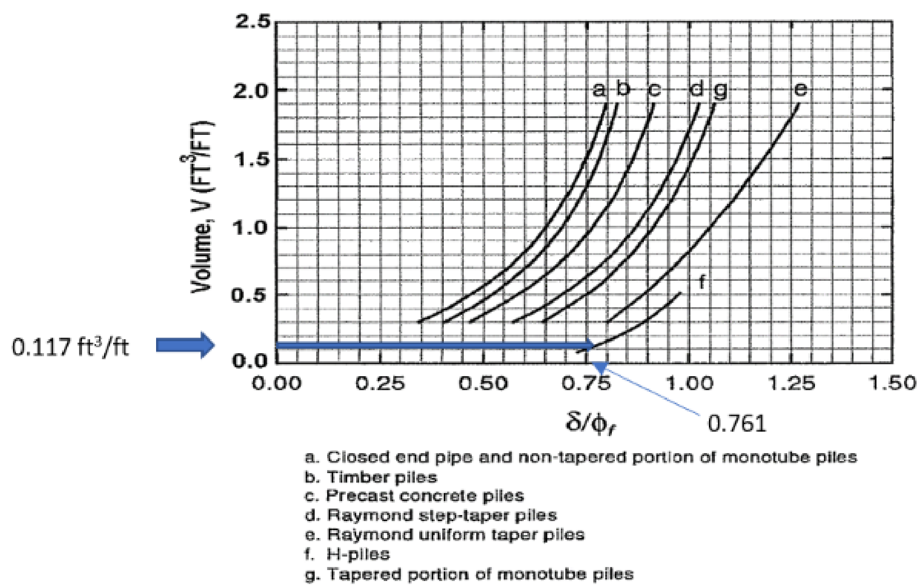


Figure 10.7.3.8.6f-6—Relation of δ/ϕ_f and Pile Displacement, V, for Various Types of Piles (Hannigan et al., 2006 after Nordlund, 1979)

Step 2c: Calculate δ from δ/ϕ_f ratio for each soil layer:

Soil layer #1: (0.761) * (30°) = δ = 22.8°
Soil layer #2: (0.761) * (32°) = δ = 24.4°
Soil layer #3: (0.761) * (33°) = δ = 25.1°
Soil layer #4: (0.761) * (35°) = δ = 26.6°
Soil layer #5: (0.761) * (36°) = δ = 27.4°
Soil layer #6: (0.761) * (37°) = δ = 28.2°
Soil layer #7: (0.761) * (38°) = δ = 28.9°

Step 3: Determine the coefficient of lateral earth pressure, K_δ, based on the ø angle of the soil layer.

Determine K_δ for ø angle based on displaced volume, V= 0.117 ft^3/ft, and pile taper angle, *ω = 0*°, using AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, interpolating where necessary. (Figure 10.7.3.8.6f-2 below is an example for soil layer #1 with ø=30° using the process in note "a" below. Figure 10.7.3.8.6f-3 is an example for soil layer #4 with ø=35° using the process in note "c" below. Other soil layers were determined by using the process in note "d", below). Figures 10.7.3.8.6f-1 and 4 are presented for reference.

Soil layer #1: ø=30° K_δ = 0.87
Soil layer #2: ø=32° K_δ = 0.99
Soil layer #3: ø=33° K_δ = 1.05
Soil layer #4: ø=35° K_δ = 1.18
Soil layer #5: ø=36° K_δ = 1.30
Soil layer #6: ø=37° K_δ = 1.41
Soil layer #7: ø=38° K_δ = 1.53

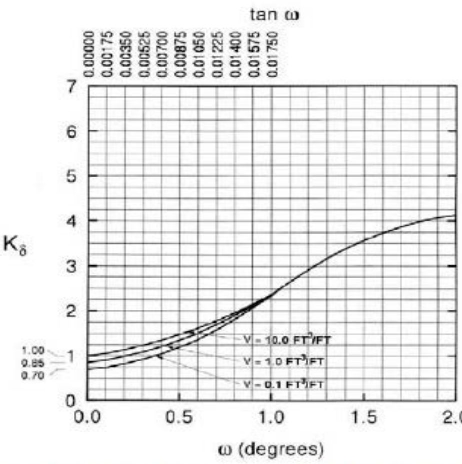


Figure 10.7.3.8.6f-1—Design Curve for Evaluating K_δ for Piles where ϕ_f = 25 degrees (Hannigan et al., 2006 after Nordlund, 1979)

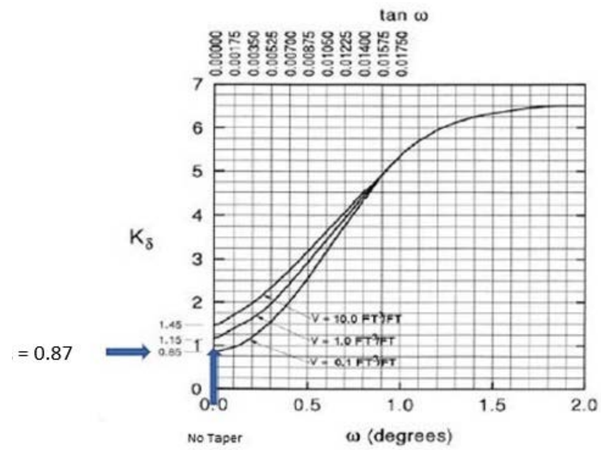


Figure 10.7.3.8.6f-2—Design Curve for Evaluating K_δ for Piles where ϕ_f = 30 degrees (Hannigan et al., 2006 after Nordlund, 1979)

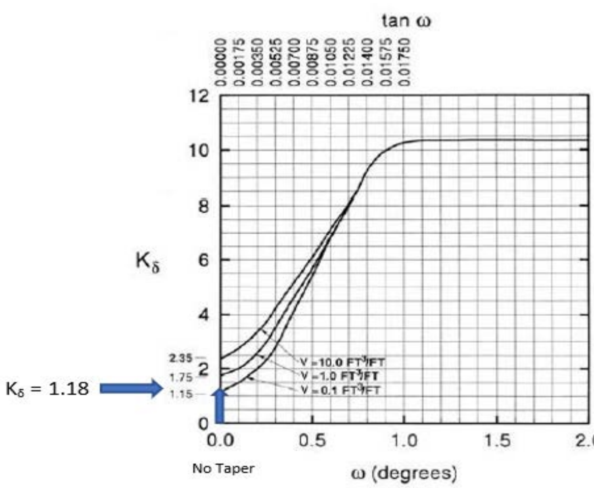


Figure 10.7.3.8.6f-3—Design Curve for Evaluating K_δ for Piles where ϕ_f = 35 degrees (Hannigan et al., 2006 after Nordlund, 1979)

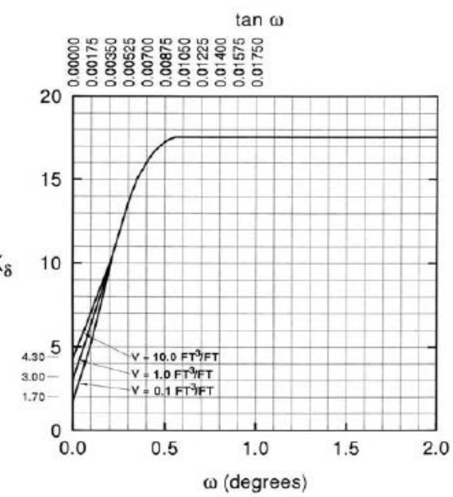


Figure 10.7.3.8.6f-4—Design Curve for Evaluating K_δ for Piles where ϕ_f = 40 degrees (Hannigan et al., 2006 after Nordlund, 1979)

Notes:

e) If the displaced volume is 0.1, 1.0 or 10.0 ft^3/ft, which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle is one of those provided, K_δ can be determined directly from the appropriate figure.

f) If the displaced volume is 0.1, 1.0, or 10.0 ft^3/ft which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle is different from those provided, use linear interpolation to determine K_δ for the required ø angle.

g) If the displaced volume is other than 0.1, 1.0 or 10.0 ft^3/ft which corresponds to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle corresponds to one of those provided, use log linear interpolation to determine K_δ for the required displaced volume.

h) If the displaced volume is other than 0.1, 1.0, or 10.0 ft^3/ft which correspond to one of the curves provided in in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle does not correspond to one of those provided, first use linear interpolation to determine K_δ for the required ø angle at the displaced volume curves provided for 0.1, 1.0 and 10.0 ft^3/ft. Then use log linear interpolation to determine K_δ for the required displaced volume.

Step 4: Determine the correction factor, C_F, to be applied to K_δ, if $\delta \neq ø$ angle for each soil layer using AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-5. (Example below is for ø=30°). $\delta/ø_f = 0.761$ is from Step 2b, above.

Soil layer #1: ø=30°	$\delta/ø_f = 0.761$	therefore $C_F = 0.91$
Soil layer #2: ø=32°	$\delta/ø_f = 0.761$	therefore $C_F = 0.90$
Soil layer #3: ø=33°	$\delta/ø_f = 0.761$	therefore $C_F = 0.89$
Soil layer #4: ø=35°	$\delta/ø_f = 0.761$	therefore $C_F = 0.88$
Soil layer #5: ø=36°	$\delta/ø_f = 0.761$	therefore $C_F = 0.87$
Soil layer #6: ø=37°	$\delta/ø_f = 0.761$	therefore $C_F = 0.86$
Soil layer #7: ø=38°	$\delta/ø_f = 0.761$	therefore $C_F = 0.85$

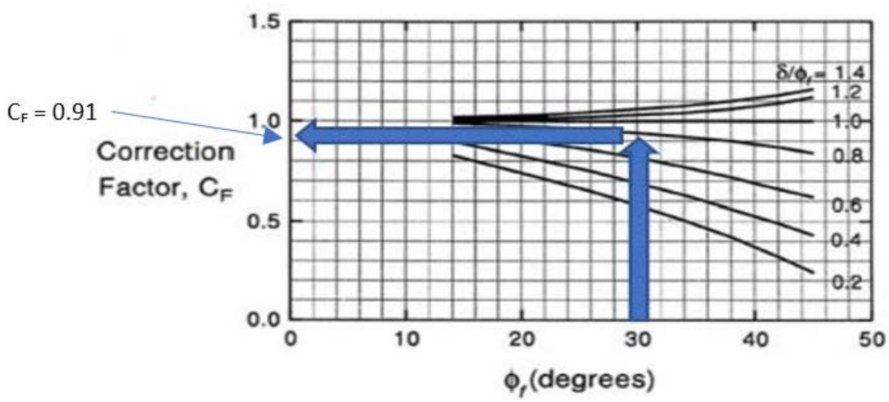


Figure 10.7.3.8.6f-5—Correction Factor for K_δ where $\delta \neq \phi_f$ (Hannigan et al., 2006 after Nordlund, 1979)

Step 5: Compute the vertical effective stress (σ'_v) at the midpoint and bottom of each relevant soil layer. (See Soil Profile above).

Layer #1:
Midpoint of Layer #1: Not relevant for this example
Bottom of Layer #1: 5’(γ'=57.6 pcf) = 288 psf = 0.288 ksf

Layer #2:
Midpoint of Layer #2: 5’(γ'=61.6 pcf) + 288 psf = 596 psf = 0.596 ksf
Bottom of Layer #2: 5’(γ'=61.6 pcf) + 596 psf = 904 psf = 0.904 ksf

Layer #3:
Midpoint of Layer #3: 5’(γ'=63.6 pcf) + 904 psf = 1222 psf = 1.22 ksf
Bottom of Layer #3: 5’(γ'=63.6 pcf) + 1222 psf = 1540 psf = 1.54 ksf

Layer #4:
Midpoint of Layer #4: 5’(γ'=65.6 pcf) + 1540 psf = 1868 psf = 1.87 ksf
Bottom of Layer #4: 5’(γ'=65.6 pcf) + 1868 psf = 2196 psf = 2.19 ksf

Layer #5:
Midpoint of Layer #5: 5’(γ'=67.6 pcf) + 2196 psf = 2534 psf = 2.53 ksf
Bottom of Layer #5: 5’(γ'=67.6 pcf) + 2534 psf = 2872 psf = 2.87 ksf

Layer #6:
Midpoint of Layer #6: 5’(γ'=70.6 pcf) + 2872 psf = 3225 psf = 3.23 ksf
Bottom of Layer #6: 5’(γ'=70.6 pcf) + 3225 psf = 3578 psf = 3.58 ksf

Layer #7:
Midpoint of Layer #7: 2.5’(γ'=72.6 pcf) + 3578 psf = 3760 psf = 3.76 ksf
Bottom of Layer #7: 2.5’(γ'=72.6 pcf) + 3760 psf = 3941 psf = 3.94 ksf

Compute the Strength Limit State Compression Design Tip Elevation

Step 6: Compute the pile side resistance (R_s).

Step 6a: Compute the pile side resistance for each soil layer. (For H-piles in cohesionless soils, the “box” area should generally be used to compute the surface area of the pile side).

$q_s = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})$ (AASHTO 10.7.3.8.6f-1)

$A_s = (C_d)(\Delta d)$

$R_s = (q_s)(A_s) = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2)
- K_δ = Coefficient of lateral earth pressure (from Step 3)
- C_F = Correction factor to be applied to K_δ (from Step 4)
- σ'_v = Vertical effective stress at midpoint of each relevant soil layer (from Step 5) (ksf)
- δ = Friction angle between the pile and soil (from δ in Step 2c)
- ω = Pile taper angle. (In this example no taper, so $\omega = 0°$)
- C_d = Pile perimeter (using "box" H-pile area) = 3.37' for this example
- Δd = Thickness of soil layer (ft)

Layer #1: No side resistance contribution from Layer #1

Layer #2: $(K_\delta=0.99)(C_F=0.90)(\sigma'_v=0.596 \text{ ksf})\ (\frac{\sin(24.4°+0°)}{\cos 0°})\ (C_d=3.37')(\Delta d=10.0') = 7.3$ kips

Layer #3: $(K_\delta=1.05)(C_F=0.89)(\sigma'_v=1.22 \text{ ksf})\ (\frac{\sin(25.1°+0°)}{\cos 0°})\ (C_d=3.37')(\Delta d=10.0') = 16.1$ kips

Layer #4: $(K_\delta=1.18)(C_F=0.88)(\sigma'_v=1.87 \text{ ksf})\ (\frac{\sin(26.6°+0°)}{\cos 0°})\ (C_d=3.37')(\Delta d=10.0') = 29.4$ kips

Layer #5: $(K_\delta=1.30)(C_F=0.87)(\sigma'_v=2.53 \text{ ksf})\ (\frac{\sin(27.4°+0°)}{\cos 0°})\ (C_d=3.37')(\Delta d=10.0') = 44.3$ kips

Layer #6: $(K_\delta=1.41)(C_F=0.86)(\sigma'_v=3.23 \text{ ksf})\ (\frac{\sin(28.2°+0°)}{\cos 0°})\ (C_d=3.37')(\Delta d=10.0') = 62.0$ kips

Layer #7: $(K_\delta=1.53)(C_F=0.85)(\sigma'_v=3.76 \text{ ksf})\ (\frac{\sin(28.9°+0°)}{\cos 0°})\ (C_d=3.37')(\Delta d=5.0') = 39.5$ kips

Step 6b: After computing the side resistance in each soil layer, sum the side resistances from each soil layer to obtain the total pile side resistance (R_s).

ΣR_s = 199 kips

Step 7: Determine the pile tip resistance (Rp):

$q_p = (\alpha_t)(N'_q)(\sigma'_v)$

$R_p = (q_p)(A_p) = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

Where:

- q_p = Nominal unit tip resistance (ksf)
- α_t = Coefficient from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7
- N'_q = Bearing capacity factor from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8
- σ'_v = Vertical effective stress at the bottom of Layer #7 (at pile tip) (ksf)
- A_p = Area of the H-pile steel section = 16.8 in^2 = 0.117 ft^2

Step 7a: Determine the α_t coefficient by entering the ø angle of soil at the pile tip in Layer #7 into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7 intersecting the D/b ratio line. For this example, ø=38°; D/b ratio is: Pile Length/Pile Width = 55'/0.85' = 64.71

Therefore, $\boldsymbol{\alpha_t = 0.72}$

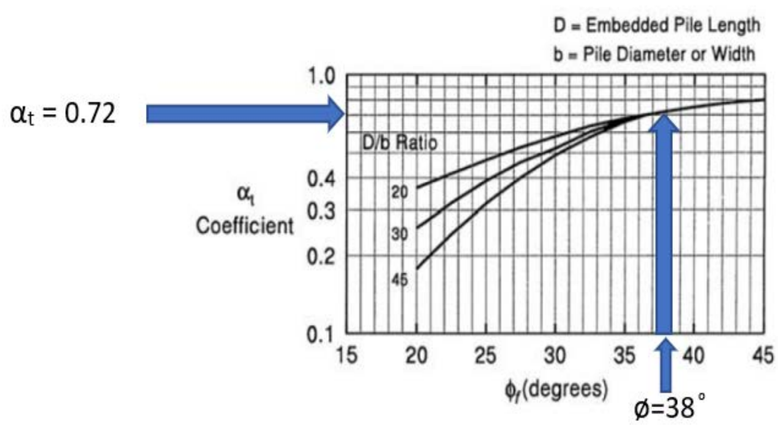


Figure 10.7.3.8.6f-7—α_t Coefficient (Hannigan et al., 2006 modified after Bowles, 1977)

Step 7b: Determine the bearing capacity factor, N'_q , by entering ø angle of soil at the pile tip in Layer #7 into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8. For this example, ø=38°.

Therefore, $\mathbf{N'_q = 105}$

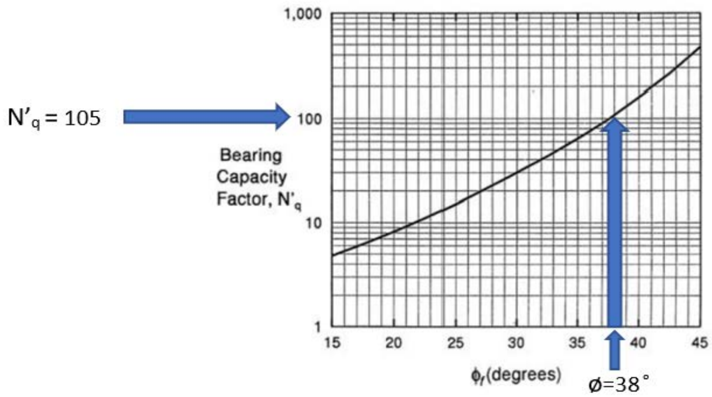


Figure 10.7.3.8.6f-8—Bearing Capacity Factor, N'_q
(Hannigan et al., 2006 modified after Bowles, 1977)

Step 7c: Determine the vertical effective stress (σ'_v) at the pile tip.

The vertical effective stress (σ'_v) at the bottom of Layer #7 = 3.94 ksf. However, AASHTO LRFD Bridge Design Specifications section 10.7.3.8.6f states it must be limited to ≤ 3.2 ksf, therefore:

$\mathbf{\sigma'_v}$ **= 3.2 ksf**.

Step 8: Compute the pile tip resistance (R_p).

$R_p = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

$\mathbf{R_p =}$ (0.72)(105)(3.2 ksf)(0.117 ft^2) = **28 kips**

Step 8a: Compare the pile tip resistance (R_p) to the limiting unit tip resistance (q_L)

The calculated pile tip resistance must be compared to the limiting unit tip resistance (q_L). The limiting q_L value is obtained from entering the ø angle of soil near the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-9. For this example, ø=38°, therefore $\mathbf{q_L = 260}$ **ksf**.

$R_{p\ limited} = (q_L)\ (A_p)$

$R_{p\ limited}$ = (260 ksf) (0.117 ft^2) = **30.4 kips**

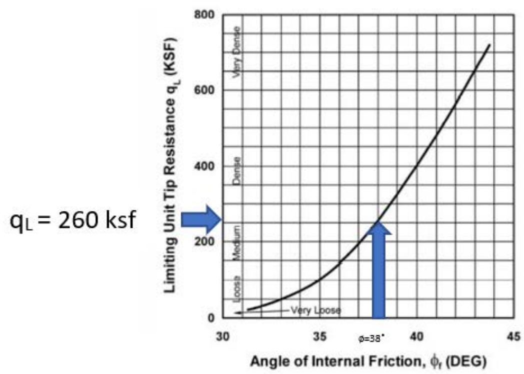


Figure 10.7.3.8.6f-9—Limiting Unit Pile Tip Resistance (Hannigan et al., 2006 after Meyerhof, 1976)

Step 8b: Use the lesser of either the R_p or $R_{p\ limited}$ value: Therefore, use $\mathbf{R_p}$ **= 28 kips**

Step 9: Compute the Nominal Pile Resistance (R_n)

$\mathbf{R_n}$ = R_s + R_p = 199 kips + 28 kips = **227 kips** (rounded to **230 kips**) at design tip elevation 35.0 feet.

Once the pile design tip elevation is calculated, it must be presented in both the Foundation Design Recommendations Table and the Pile Data Table in the Foundation Report. Example tables are shown below.

Foundation Design Recommendations

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inch)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (ft)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	Comp (φ_{qs}=1.0) (φ_{qp}=1.0)	Tension (φ_{qs}=1.0)			
Pier 2	HP 10X57 H-Pile	90.0	760	550	1	220	0	120	0	35.0 (a-I)	35.0	230

Note: Design tip elevations are controlled by (a-I) Compression (Strength Limit)

Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Pier 2	HP 10X57 H-Pile	220	0	35.0 (a)	35.0	230

Note: Design tip elevations are controlled by (a) Compression

Appendix C: 2000 American Petroleum Institute (API) Method - Driven CISS Pile

This design example is for Strength Limit State only. A typical pile design includes evaluation of Service Limit State, Strength Limit State, and Extreme Event Limit State. The Extreme Event Limit State analysis is the same as this example, except that it would be designed for the Extreme Event Limit State required nominal resistance and scour percentages.

For pile foundations in cohesive soils or layers and loose granular soils, pile group settlement must be evaluated by using the equivalent footing method for Service Limit State design. Figures that show soil layers and stress distribution for various cases can be found in AASHTO LRFD Bridge Design Specifications Section 10.7.2.3.

In conditions where the piles are driven into very dense layers or rock, and settlement is anticipated to be minimal, Service Limit State calculation may not be needed. However, If the pile is driven into a very dense layer that is underlain by a loose granular or cohesive layer, then settlement should be evaluated.

Lateral design of piles is usually performed by the structure designer, however the Geoprofessional typically provides the structure designer the soil parameters for the L-Pile analysis in Class S2 soils per Seismic Design Criteria 6.1.2.

Information provided by Structure Design

- 24-inch diameter CISS pile is required.
- The structure designer determines the minimum length of the rebar cage. For this example, the structure designer needs 25 feet of concrete and reinforcement from the pile cutoff.
- A ½-inch thick steel section is required.
- Foundation information and loads will be provided from the Structure Designer, as shown in the tables below.

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Pile Type	Finished Grade Elevation (feet)	Pile Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement Under Service Load (inch)	Number of Piles per Support
				B	L		
Pier 2	24” diam. CISS Piles	100.0	90.0	N/A	N/A	1	5

Foundation Factored Design Loads Provided by Structure Designer

Support No.	Service I Limit State (kips)		Strength/Construction Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Pier 2	1154	730	1765	383	0	0	750	187	0	0

Geotechnical Design Considerations

- The Geoprofessional will take the factored load and apply the resistance factor (0.7 for both side and tip resistance for Strength Limit State, and 1.0 for both side and tip resistance for Extreme Event Limit State) and then round up to the nearest 10 kips. For this example, the Required Nominal Resistance for the Strength Limit State and Extreme Event Limit State are 510 kips and 190 kips, respectively. The Foundation Design Recommendations Table and the Pile Data Table examples are presented at the end of this example.
- If it is anticipated that the contractor will request to use the vibratory method to "seat" the pile prior to impact driving, the design should ignore side resistance throughout that zone of vibratory driving. The depth of "seating" will depend on the pile size and length, and is typically limited to a maximum length of 20% of the pile length or 20 feet. Usage of the vibratory method and "seating" depth used in the pile design should be disclosed in the "Notes for Construction" section of the Foundation Report.
- Due to the CISS pile tip below the groundwater table, a seal course will be required to place the concrete and reinforcement in a dry condition.
- A drivability analysis by Foundation Testing and Instrumentation will determine what minimum thickness of shell is needed based on the subsurface conditions at the site. The structure designer must also be consulted as to what minimum thickness of shell is required structurally for their design. The shell thickness that satisfies both Geotechnical Services and Structure Design requirements must be included in the foundation report.
- Scour information for this example is presented below:

Scour Data Table

Support No.	Long-Term (Degradation and Contraction) Scour Elevation (feet)	Short-Term (Local) Scour Depth (feet)
Pier 2	95.0	20.0

- AASHTO LRFD Bridge Design Specifications – California Amendments, Table 3.7.5-1, provides the percentages of Long-Term (Degradation + Contraction) and Short-Term (Local) scour to be used for each limit state.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

- When calculating nominal resistance:
    1. The vertical effective stress is calculated using 100% of Long-Term (Degradation + Contraction) scour and 0% Short-Term (Local) scour for Strength Limit and Extreme Event Limit states.
    2. Side resistance pile-soil contact is calculated using 100% Long-Term and 50% Short-Term (Local) scour for Strength Limit State, and 100% Long-Term and 0% Short-Term scour for Extreme Event Limit State. See the table below for information.

Effect of Scour on Pile Design				
	Vertical Effective Stress		Side Resistance	
	Long-Term	Short-Term	Long-Term	Short-Term
Service	0%	0%	N/A	N/A
Strength	100%	0%	100%	50%
Extreme	100%	0%	100%	0%

- For this example, the following elevations are used to calculate the Strength Limit State pile design tip elevation:
    1) Elev. 85’ is the top of side resistance zone.
    2) Elev. 95’ is the ground surface for vertical effective stress calculation for both side and tip resistance.
- This example does not calculate the Extreme Event Limit State pile design tip elevation. However, if it did, the following would be used:
    1) Elev. 90’ would be the top of side resistance zone.
    2) Elev. 95’ would be the ground surface for vertical effective stress calculation for both side and tip resistance.

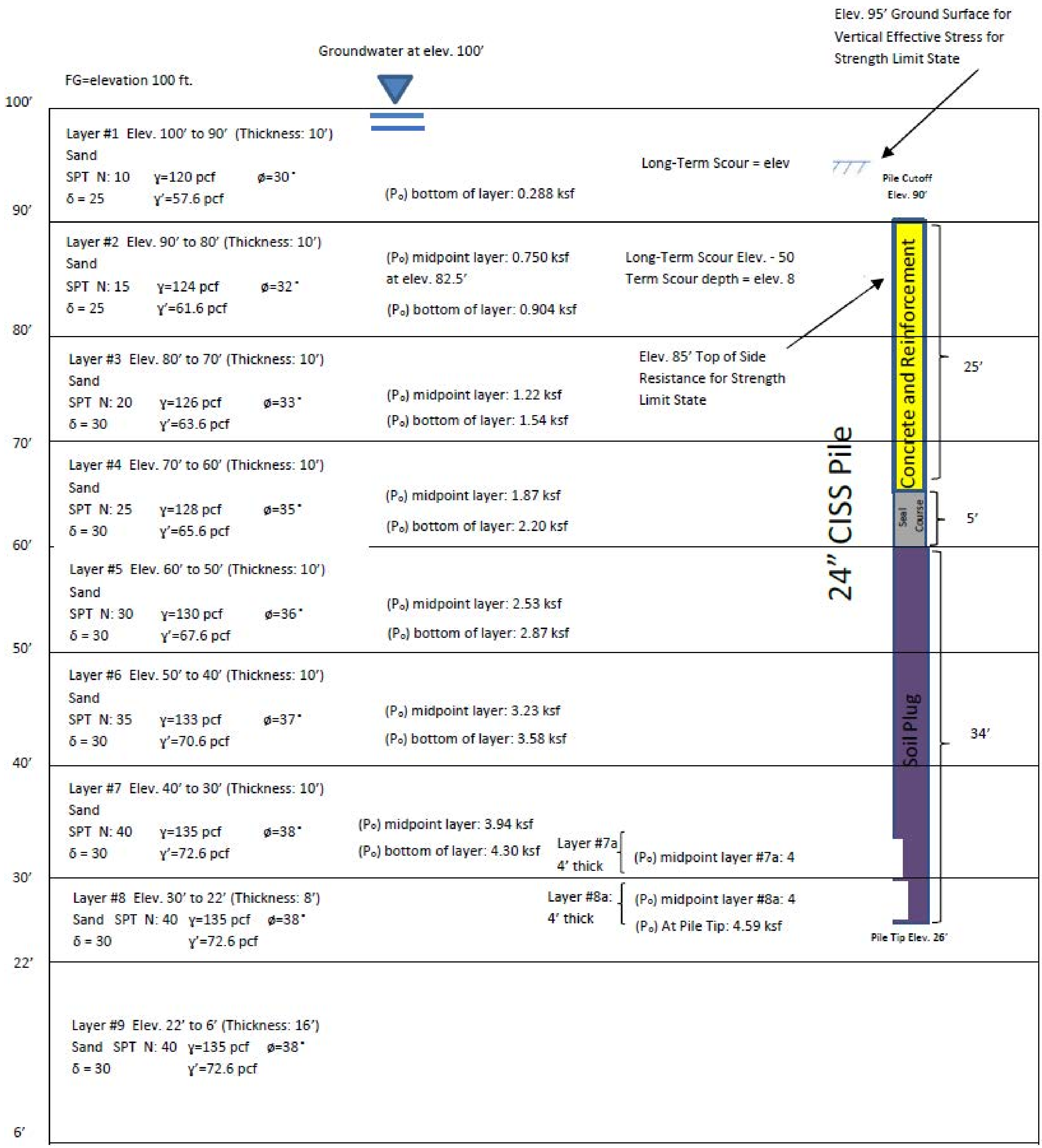
- Table 6.4.3-1 in the API Method is used for determining the Soil-Pile Friction Angle (δ), the limiting skin friction values, Nq, and the limiting unit end bearing values in the design example.

Table 6.4.3-1—Design Parameters for Cohesionless Siliceous Soil*

Density	Soil Description	Soil-Pile Friction Angle, δ Degrees	Limiting Skin Friction Values kips/ft2 (kPa)	N_q	Limiting Unit End Bearing Values kips/ft2 (MPa)
Very Loose Loose Medium	Sand Sand-Silt** Silt	15	1.0 (47.8)	8	40 (1.9)
Loose Medium Dense	Sand Sand-Silt** Silt	20	1.4 (67.0)	12	60 (2.9)
Medium Dense	Sand Sand-Silt**	25	1.7 (81.3)	20	100 (4.8)
Dense Very Dense	Sand Sand-Silt**	30	2.0 (95.7)	40	200 (9.6)
Dense Very Dense	Gravel Sand	35	2.4 (114.8)	50	250 (12.0)

*The parameters listed in this table are intended as guidelines only. Where detailed information such as in situ cone tests, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.
**Sand-Silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

Soil Profile:



California Amendments to AASHTO LRFD Bridge Design Specifications, Section C10.7.3.8.6a, states that the 2000 American Petroleum Institute method should be used to determine the nominal pile resistance (R_n) for steel pipe piles and CISS piles over 18 inches in diameter. The following steps should be used to calculate the nominal pile resistance of an open-ended pile, which is the sum of the outside pile side resistance ($R_{s\ outside}$), the steel shell end area resistance ($R_{p\ steel}$), and the lesser of either the soil plug end area resistance ($R_{p\ soil}$) or internal soil plug side resistance ($R_{s\ inside}$).

Step 1: Divide the soils into layers and determine the ɣ and ø angle for each layer. (See Soil Profile above).

Step 2: Compute the vertical effective stress (P_o) at the midpoint and bottom of each relevant soil layer. (See Soil Profile above).

Layer #1:
Elevation 90’: 5’(ɣ’=57.6 pcf) = 288 psf = 0.288 ksf

Layer #2:
Elevation 82.5’: 7.5’(ɣ’=61.6 pcf) + 288 psf = 750 psf = 0.75 ksf
Elevation 80’: 2.5’(ɣ’=61.6 pcf) + 750 psf = 904 psf = 0.90 ksf

Layer #3:
Elevation 75’: 5’(ɣ’=63.6 pcf) + 904 psf= 1222 psf = 1.22 ksf
Elevation 70’: 5’(ɣ’=63.6 pcf) + 1222 psf = 1540 psf = 1.54 ksf

Layer #4:
Elevation 65’: 5’(ɣ’=65.6 pcf) + 1540 psf = 1868 psf = 1.87 ksf
Elevation 60’: 5’(ɣ’=65.6 pcf) + 1868 psf = 2196 psf = 2.20 ksf

Layer #5:
Elevation 55’: 5’(ɣ’=67.6 pcf) + 2196 psf = 2534 psf = 2.53 ksf
Elevation 50’: 5’(ɣ’=67.6 pcf) + 2534 psf = 2872 psf = 2.87 ksf

Layer #6:
Elevation 45’: 5’(ɣ’=70.6 pcf) + 2872 psf = 3225 psf = 3.23 ksf
Elevation 40’: 5’(ɣ’=70.6 pcf) + 3225 psf = 3578 psf = 3.58 ksf

Layer #7:
Elevation 35’: 5’(ɣ’=72.6 pcf) + 3578 psf = 3941 psf = 3.94 ksf
Elevation 30’: 5’(ɣ’=72.6 pcf) + 3941 psf = 4304 psf = 4.30 ksf

Layer #8a:
Elevation 28’: 2’(ɣ’=72.6 pcf) + 4304 psf = 4449 psf = 4.45 ksf
Elevation 26’: 2’(ɣ’=72.6 pcf) + 4449 psf = 4594 psf = 4.59 ksf

Compute the Strength Limit State Compression Design Tip Elevation

Step 3: Compute the outside pile side resistance ($R_{s\ outside}$) in each soil layer.

$q_s = (K)(P_o)(\tan\delta)$

$A_s = (C_d)(\Delta d)$

$R_{s\ outside} = (q_s)(A_s) = (K)(P_o)(\tan\delta)(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2).
- K = Coefficient of lateral earth pressure. The API method states it is usually appropriate to assume K= 0.8 for both compression and tension loadings for piles driven unplugged. Values of K for full displacement piles (plugged or closed end) may be assumed to be 1.0. The API method states that a pile may be considered plugged or unplugged based on static calculations. It also states that a pile could be driven in an unplugged condition, but act plugged under static loading. Due to the uncertainty of where plugging may occur, a value of K=0.8 was conservatively selected for this example.
- P_o = Vertical effective stress at midpoint of each relevant soil layer (from Step 2).
- δ = Friction angle between steel and soil (from Table 6.4.3-1 API Method). For this example, a δ of 25 was selected for Layer #2, and a δ of 30 was selected for Layers #3 through #8.
- C_d = Outside Pile circumference for 24” dia. CISS pile: (π)(24”) = 75.4” = 6.28 ft. for this example.
- Δd = Thickness of soil layer (ft)

Layer #1: (No side resistance contribution from Layer #1 due to cut-off)

Layer #2: (K=0.8)(P_o=0.75 ksf)(tanδ=tan25=0.466) (C_d=6.28’)(Δd=5’) = 8.8 kips

Layer #3: (K=0.8)(P_o=1.22 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 35.4 kips

Layer #4: (K=0.8)(P_o=1.87 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 54.2 kips

Layer #5: (K=0.8)(P_o=2.53 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 73.3 kips

Layer #6: (K=0.8)(P_o=3.23 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 93.6 kips

Layer #7: (K=0.8)(P_o=3.94 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 114.2 kips

Layer #8: (2.0 ksf) (C_d=6.28’)(Δd=4’) = 50.2 kips

For Layer #8, $q_s = (K)(P_o)(\tan\delta)$ = 2.05 ksf. However, Table 6.4.3-1 in the API method limits the unit skin friction value to 2.0 ksf for dense sand and very dense sand-silt. Therefore, for Layer #8 q_s is limited at 2.0 ksf.

After computing the side resistance in each soil layer, sum the side resistances from each soil layer to obtain the outside pile side resistance.

$\Sigma R_{s\ outside}$ = 430 kips

<u>Pile Tip Resistance (R_p)</u>

The pile tip resistance has two components: the steel shell end area resistance ($R_{p\ steel}$), and the lesser of either the soil plug end area resistance ($R_{p\ soil}$) or the internal soil plug side resistance ($R_{s\ inside}$). The sum of these two components is the total pile tip resistance of the pile.

- $R_p = (R_{p\ steel})$ + [Lesser of $(R_{p\ soil})$ or $(R_{s\ inside})$].

Steps 4 and 5, below, provide examples for calculating the steel shell end area resistance and the soil plug end area resistance, respectively. Step 6 provides an example for calculating the internal soil plug side resistance ($R_{s\ inside}$). Step 7 compares the value of the soil plug end area resistance ($R_{p\ soil}$) to the value of the internal soil plug side resistance ($R_{s\ inside}$) and the lesser of the two values is used.

Step 4: Calculate the steel shell end area resistance.

Where:
- 24” CISS pile outside diameter
- 23” CISS pile inside diameter
- ½” thick steel shell

Step 4a: Calculate the steel shell end area:

Area$_1$ (A_1) : Using 24” CISS outside diameter: $\pi D^2/4 = 3.14(24")^2/4 = 452.16\ in^2 = 3.14\ ft^2$

Area$_2$ (A_2): Using 23” CISS inside diameter: $\pi D^2/4 = 3.14(23")^2/4 = 415.27\ in^2 = 2.89\ ft^2$

Steel shell end area: $\mathbf{A_{p\ steel}} = A_1 - A_2 = 3.14\ ft^2 - 2.89\ ft^2 = \mathbf{0.25\ ft^2}$

Step 4b: Calculate the steel shell end area resistance: $R_{p\ steel} = (q_{p\ steel})(A_{p\ steel})$

$q_{p\ steel} = (P_o)(Nq)$

Where:
- P_o = Vertical effective stress at bottom of Layer #8a = 4.59 ksf
- Nq = Dimensionless bearing capacity factor (from Table 6.4.3-1 API Method)= 40

$q_{p\ steel}$ = (4.59 ksf) (40) = 183.6 ksf
$A_{p\ steel} = 0.25\ ft^2$

$\mathbf{R_{p\ steel}} = (q_{p\ steel})(A_{p\ steel})$ = (183.6 ksf) (0.25 ft^2) = **46 kips**

Internal Soil Plug

For steel pipe piles that are installed open ended, during pile driving the soil enters the pile and as pile penetration increases the pile fills with soil. If the interior soil column does not equal the pile penetration, then this is called the plugging effect and the pile may behave during driving as if it was closed-ended where soil does not move into the pipe/shell.

The API method provides minimal information on plugging and unplugging conditions. The API method states it is usually appropriate to assume K=0.8 for both compression and tension loadings for piles driven unplugged. Values of K for full displacement piles (plugged or closed end) may be assumed to be 1.0. The API method states that a pile may be considered plugged or unplugged based on static calculations. It also states that a pile could be driven in an unplugged condition, but act plugged under static loading. Due to the uncertainty of where plugging may occur, a value of K=0.8 was conservatively selected for this example.

The API method compares the soil plug end area resistance and the internal soil plug side resistance. It states that the soil plug end area resistance should not exceed the internal soil plug side resistance, and the lesser of the two values should be used.

Studies have shown that the calculated internal soil plug may be overestimated, so consideration should be given to reducing the side resistance of the internal soil plug. If reduction is utilized, it is recommended that studies with similar site conditions and design methodologies be utilized when adopting such reductions.

Step 5: Calculate the seal course thickness:

For this example, the structure designer needs 25 feet of concrete and reinforcement from the pile cutoff. The Geoprofessional must make an assumed seal course thickness (typically 3-5 feet). For this example, an assumed value of 5 feet is used. A hydrostatic head of 40 feet is used in the calculation for this example, which is from the groundwater surface down to the bottom of the assumed seal course thickness (elev. 100 ft to elev. 60 ft). The bond strength (Q_s) was assumed to be 300 psf for the seal course concrete placed below groundwater.

The seal course thickness (t) is calculated by the following equation: $t = \frac{\gamma_w Z r}{\gamma_c r + 2Q_s}$

Where:

- γ_w = Unit weight of water: 62.4 pcf
- Z = Hydrostatic head from the groundwater surface to bottom of seal course: 40 ft this example
- r = Inside pile radius. For this example: r = (12” – ½”)/12 = 0.96 ft.
- γ_c = Unit weight of concrete. For this example: 145 pcf
- Q_s = Bond strength of steel to concrete: 300 psf

Seal Course Thickness: $$t = \frac{(62.4\text{ pcf})(40')(0.96')}{(145\text{ pcf})(0.96') + 2(300\text{ psf})} = 3.2\,feet$$

Due to variable conditions at the site (variations in groundwater, soil type, etc.), it was decided to round up the calculated seal course thickness to 5 feet in this example.

Therefore, the seal course thickness t = 5.0 feet

- The seal course thickness must be reported in the foundation report.

Step 6: Calculate the internal soil plug side resistance ($R_{s\ inside}$).

The soil plug length was determined by taking the required length of the rebar cage (25 feet in this example), then adding the seal course thickness (5 feet in this example), then subtracting that length from the total length of the pile (64 feet). 64 ft – 30 ft = 34 feet. However, the contributing plug length must be limited to 4 times the pile diameter, which is 8 feet for this example. The vertical effective stress (P_o) for the internal soil plug is calculated from the long-term scour elevation (elev. 95').

$q_{s\ inside} = (K)(P_o)(\tan\delta)$

$A_{s\ inside} = (C_d)(\Delta d)$

$R_{s\ inside} = (q_{s\ inside})(A_{s\ inside}) = (K)(P_o)(\tan\delta)(C_d)(\Delta d)$

Where:

- $q_{s\ inside}$ = Internal soil plug nominal unit side resistance
- $A_{s\ inside}$ = Surface area of the inside of the pile (ft^2).
- K = Coefficient of lateral earth pressure. Always use K=0.8 for the internal pile unit side resistance.
- P_o = Vertical effective stress from the long-term scour elevation to midpoint of soil layer being analyzed.
- δ = Friction angle between steel and soil (from Table 6.4.3-1 API Method). δ=30 degrees for this example.
- C_d = Inside Pile circumference for 23" dia.: (π)(23") = 72.2" = 6.02 ft. for this example.
- Δd = Thickness of internal soil plug. 34 feet for this example.

Step 6a: Calculate Side Resistance for Internal Soil Plug, which is limited to 4D, or 8 feet (Elev. 34' to 26')

Internal Soil Plug Side Resistance for Layer #7a (Elev. 34' – 30'):

$P_{o\ Elev\ 32'}$ = (8')(72.6 pcf) + 3580 psf = 4161 psf = 4.16 ksf

$P_{o\ Bottom\ Layer\ \#7}$ = (2')(72.6 pcf) + 4161 psf = 4306 psf = 4.31 ksf

$q_{s\ Elev\ 32'}$ = (K=0.8) (P_o=4.16 ksf) (tanδ=tan30=0.577) = 1.92 ksf

$A_{s\ Inside\ Layer\ \#7}$ = Surface area of the pile side (ft^2): $A_{s\ Inside\ Layer\ \#7}$ = (6.02 ft)(4 ft) = 24.1 ft^2

$\mathbf{R_{s\ Inside\ Layer\ \#7a}}$ = $(q_{s\ inside})(A_{s\ inside})$ = (1.92 ksf) (24.1 ft^2) = **46.3 kips**

Internal Soil Plug Side Resistance for Layer #8a (Elev. 30' – 26'):

$P_{o\ Elev\ 28'}$ = (2')(72.6 pcf) + 4306 psf = 4451 psf = 4.45 ksf

$P_{o\ Bottom\ Layer\ \#8a}$ = (2')(72.6 pcf) + 4451 psf = 4596 psf = 4.60 ksf

$q_{s\ Elev\ 28'}$ = (K=0.8) (P_o=4.45 ksf) (tanδ=tan30=0.577) = 2.05 ksf (not to exceed 2.0 ksf)

$A_{s\ Inside\ Layer\ \#8a}$ = Surface area of the pile side (ft^2): $A_{s\ Inside\ Layer\ \#8a}$= (6.02 ft)(4 ft) = 24.1 ft^2

$\mathbf{R_{s\ Inside\ Layer\ \#8a}}$ = $(q_{s\ inside})(A_{s\ inside})$ = (2.0 ksf) (24.1 ft^2) = **48.2 kips**

$\mathbf{R_{s\ Inside\ Soil\ Plug}}$ = 46.3 kips + 48.2 kips = **94.5 kips**

Step 7: Calculate the internal soil plug end area resistance: $R_{p\ soil} = (q_{p\ soil})(A_{p\ soil})$

$q_{p\ soil} = (P_o)(Nq)$

Where:

- P_o = Vertical effective stress at the bottom of Layer #8a = 4.6 ksf
- Nq = Dimensionless bearing capacity factor (from Table 6.4.3-1 API Method)= 40

$q_{p\ soil}$ = (4.6 ksf) (40) = 184.0 ksf

(Note: Table 6.4.3-1 in the API method limits the unit end bearing value to 200 ksf for dense sand and very dense sand-silt).

$A_{p\ soil}$ = 23" circular end area (using inside diameter of pile): $\pi D^2/4 = 3.14(23^2)/4$ = 415.27 in^2 = 2.89 ft^2

$\mathbf{R_{p\ soil}}$ = $(q_{p\ soil})(A_{p\ soil})$ = (184.0 kips/ft^2) (2.89 ft^2) = **532 kips**

Step 8: Determining the lesser of either ($R_{s\ inside}$) or ($R_{p\ soil}$)

Based on the 2000 API method, the soil plug end area resistance ($R_{p\ soil}$) should not exceed the internal soil plug side resistance ($R_{s\ inside}$). Therefore, the two values are compared, and the lesser of the two values is used.

Internal soil plug side resistance for 8 feet long soil plug: $R_{s\ inside}$ = 94.5 kips

Soil plug end area resistance = $R_{p\ soil}$ = 532 kips

Therefore, using the lesser of the two values: **94.5 kips ($\mathbf{R_{s\ inside}}$)**

Step 9: Compute the Nominal Pile Resistance (R_n):

$\mathbf{R_n} = R_{s\ outside} + R_{p\ steel} + R_{s\ inside}$ = 430 kips + 46 kips + 95 kips = **571 kips** at tip elevation 26.0 feet.

Step 10: Compute the Nominal Driving Resistance:

The calculation of Required Nominal Driving Resistance differs from the Required Nominal Resistance calculation as follows:

- For stress calculations, the overburden for the Required Nominal Driving Resistance is calculated using ground elevation of 100', instead of 95' (long-term scour elevation).
- The side resistance is calculated from elevation 90', instead of 85' (long-term and short-term scour).
- The soil plug resistance is calculated from elevation 60' (the bottom of seal course), which accounts for center-relief drilling. The Required Nominal Driving Resistance calculation limits the soil plug resistance to 4D, which is elevation 34'.

Nominal Driving Resistance = **874 kips**

Once the pile design tip elevation is calculated, it must be presented in both the Foundation Design Recommendations Table and the Pile Data Table in the Foundation Report. Example tables are shown below.

Foundation Design Recommendations

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (ϕqs=0.7) (ϕqp=0.7)	Tension (ϕqs=0.7)	Comp. (ϕqs=1.0) (ϕqp=1.0)	Tension (ϕqs=1.0)			
Pier 2	24" diam. CISS Pile	90.0	1160	730	1	550	0	190	0	26.0 (a-l)	26.0	874

Note: Design tip elevations are controlled by (a-l) Compression (Strength Limit)

Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Pier 2	24" diam. CISS Pile	550	0	26.0 (a)	26.0	874

Note: Design tip elevations are controlled by (a) Compression