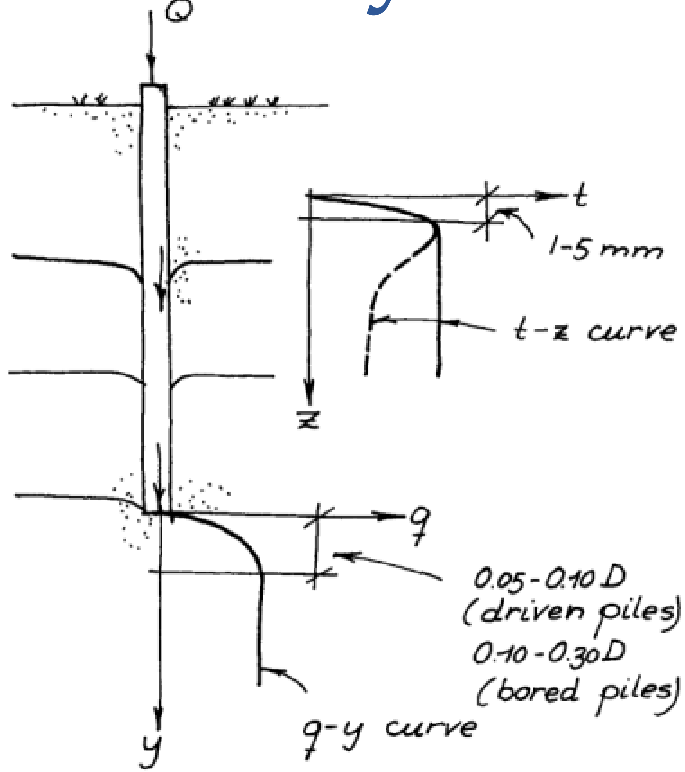
ENCE 4610
Foundation Analysis and Design



Lecture 14

Axial Load Capacity of Deep Foundations

Driven Pile Static Analytic Methods

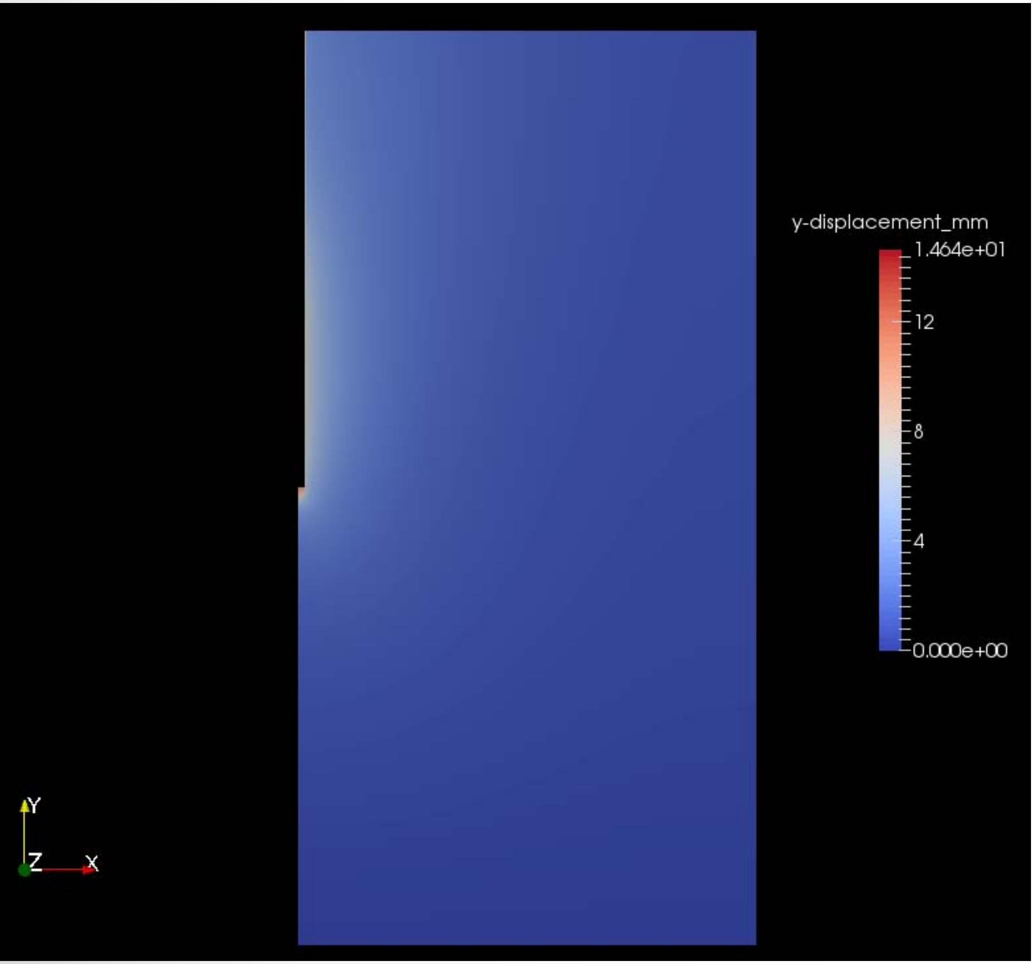
Failure Methods of Deep Foundations

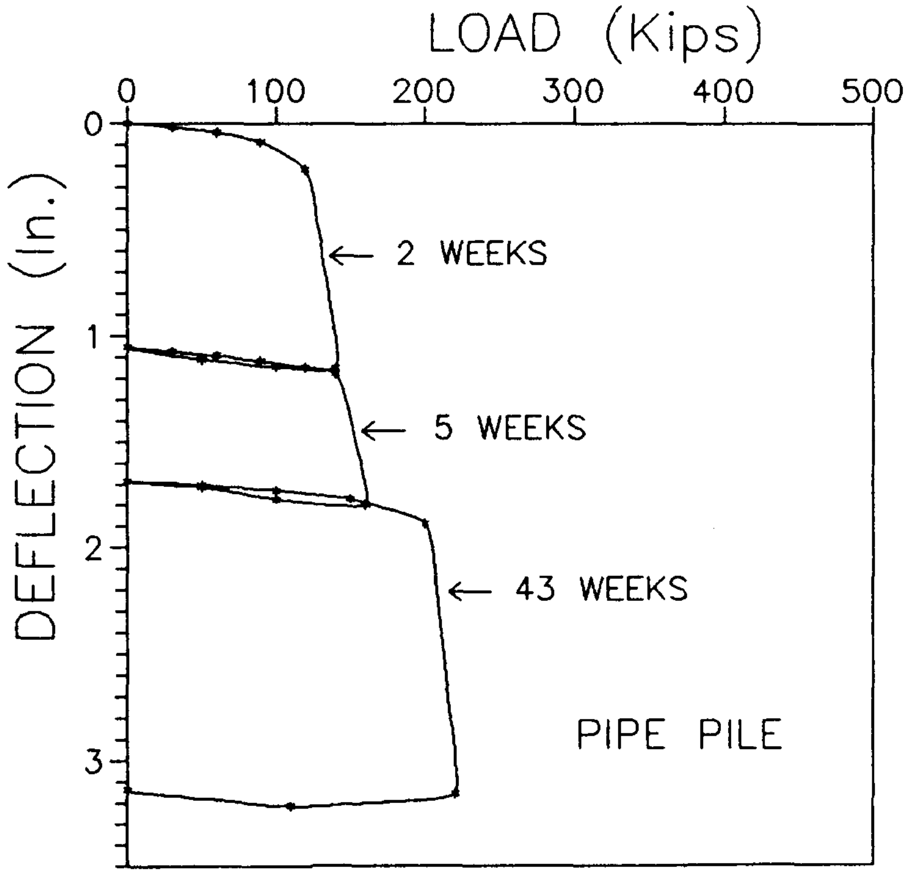
- Bearing Capacity
  - Catastrophic, limit-state failure of deep foundations (such as with slope stability and shallow foundations) is rare due to the difficulties of generating a "clean" failure surface
  - Nevertheless, we have an extensive collection of methods to estimate the "bearing capacity" of the pile
  - These can be used (with caution and generous safety/resistance factors) to estimate the point at which settlement is unacceptable

Settlement

- Excessive settlement is the normal way we expect failure in deep foundations
- Best way to anticipate excessive settlement is to model the load-deflection characteristics of the pile head
  - We can use information from "bearing capacity" methods to help this analysis
  - We can also model the soil mass around the pile as well
- Must be verified in the field, either with static or dynamic load testing

Soil Deflection and Static Load Test Results





From Finno (1989) and Warrington (2016)

Changes in Soil During Pile Installation

- Driven Piles
  - Interact with the soils differently than drilled piles (drilled shafts, auger cast piles, etc.)
  - This leads to different load transfer characteristics; the effect of driving must be understood
  - Methods are somewhat different, and will be considered separately
- Sands
  - Sands do not experiences the kinds of changes that clays do with driven piles
  - Sands (especially loose ones) experience localised compaction during driving; this increases the capacity
- Clays
  - Distortion – the movement of soil out of the way during driving remoulds the clay and reduces its strength
  - Compression and Excess Pore Water Pressures – these are raised temporarily during driving and thus decrease the driving resistance and the load bearing capacity immediately after driving
  - Loss of contact between pile and soils – also weaken the soil-pile bond along the shaft

Overview of Static Method of Pile Design

The general equation for the ultimate axial capacity of driven piles is

$$Q_u = R_s + R_p$$

Where

- Q_u = ultimate axial capacity of the pile, kN
- R_s = ultimate shaft capacity of the pile, kN
- R_t = ultimate toe capacity of the pile, kN
- W_p = weight of the pile, kN

This is valid for piles loaded in compression; for tension piles, the toe capacity is not included and (in some cases) the weight of the pile is added.

The shaft capacity is in turn estimated by the equation

$$R_s = \int f(z)_z \, dA_z = \sum_{i=1}^{n} \overline{f_{s_n}} A_{s_n}$$

Where

- $f(z)_s$ = unit shaft friction along the pile shaft as a function of depth, kPa
- A_s = shaft area of the pile which interfaces with the soil, m^2
- f_{sn} = average shaft friction along a portion n of the pile, kPa
- A_{sn} = shaft area of portion n of the pile, m^2

This equation is only solved in the integral form in theoretical considerations. For practical considerations, it is solved in the summation form. Piles are customarily divided up into regions with a reasonably uniform soil type and unit shaft resistance.

The capacity of the pile toe is computed by the equation

$$R_p = q_t A_t$$

Where

- q_t = unit pile toe capacity, kPa
- A_p = area of pile toe, m^2

Driven Pile Factors of Safety (ASD)

9.4.1 Factors of Safety

The results of static analyses yield a **geotechnical ultimate pile capacity**, Q_u. The **allowable geotechnical soil resistance** (**geotechnical pile design load**), Q_a, is selected by dividing the **geotechnical ultimate pile capacity**, Q_u, by a **factor of safety** as follows.

$$Q_a = \frac{Q_u}{\text{Factor of Safety}} \qquad 9\text{-}3$$

The range of the factor of safety, FS, has depended primarily upon the reliability of the particular method of static analysis with consideration of the following items:

1. The level of confidence in the input parameters. The level of confidence is a function of the type and extent of the subsurface exploration and laboratory testing of soil and rock materials.
2. Variability of the soil and rock.
3. Method of static analysis.
4. Effects of and consistency of the proposed pile installation method.
5. Level of construction control (static load test, dynamic analysis, wave equation analysis, Gates dynamic formula).

Construction Control Method	Factor of Safety
Static load test with wave equation analysis	2.00
Dynamic testing with wave equation analysis	2.25
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75
Gates dynamic formula	3.50

Driven Pile Capacity using Static Method

Consider a pile to be driven through the soil profile described in Figure 9-5. The proposed pile type penetrates through a sand layer subject to scour in the 100-year flood into an underlying very soft clay layer unsuitable for long term support and then into competent support materials. The soil resistances from the scour-susceptible sand layer and soft clay layer do not contribute to long term load support and should not be included in the soil resistance for support of the design load. In this example, static load testing with wave equation analysis will be used for construction control. Therefore, a factor of safety of 2.0 should be applied to the ultimate soil resistance calculated in suitable support layers in the static analysis. It should be noted that this approach is for scour conditions under the 100-year or overtopping flood events and that a different approach would apply for the superflood or 500-year event. For a superflood, a minimum factor of safety of 1.0 is used. This minimum factor of safety is determined by dividing the maximum pile load by the sum of the shaft and toe resistances available below scour depth.

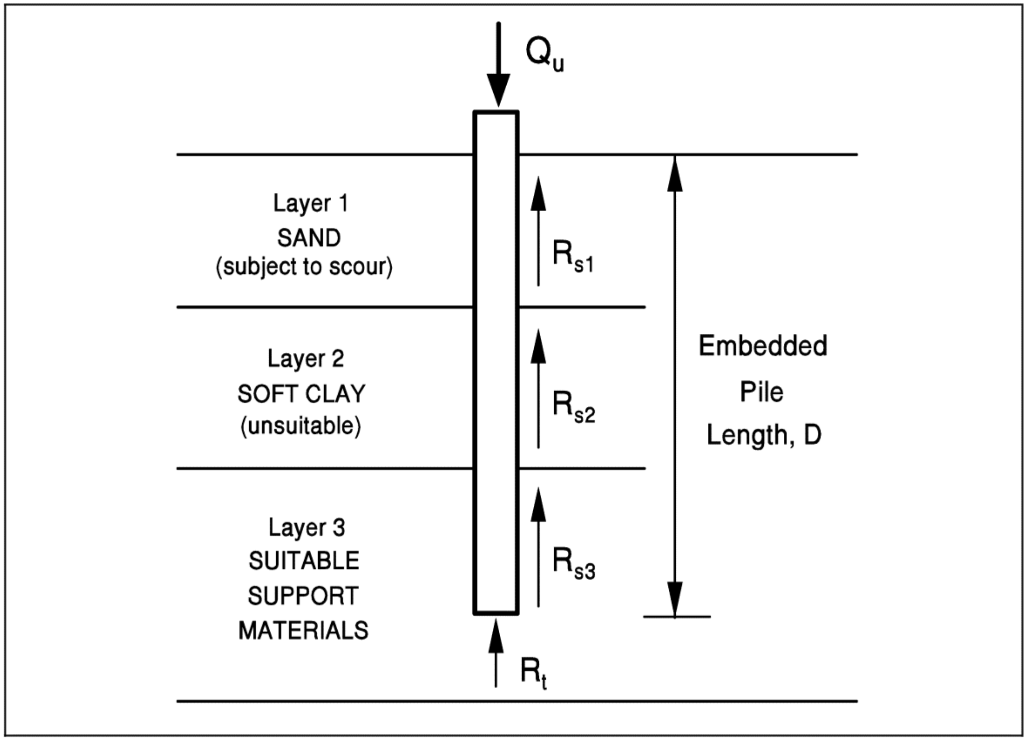


Figure 9-5. Soil profile for factor of safety discussion (FHWA, 2006a).

In the static analysis, a trial pile penetration depth is chosen and an ultimate pile capacity, Q_u, is calculated. This ultimate capacity includes the soil resistance calculated from all soil layers including the shaft resistance in the scour susceptible layer, R_{s1}, the shaft resistance in the unsuitable soft clay layer, R_{s2} as well as the resistance in suitable support materials along the pile shaft, R_{s3}, and at the pile toe resistance, R_t.

$$Q_u = R_{s1} + R_{s2} + R_{s3} + R_t$$

The design load, Q_a, is the sum of the soil resistances from the suitable support materials divided by a factor of safety, FS. As noted earlier, a factor of safety of 2.0 is used in the equation below because of the planned construction control with static load testing. Therefore,

$$Q_a = (R_{s3} + R_t) / (FS{=}2)$$

The design load may also be expressed as the sum of the ultimate capacity minus the calculated soil resistances from the scour susceptible and unsuitable layers divided by the factor of safety. In this alternative approach, the design load is expressed as follows:

$$Q_a = (Q_u - R_{s1} - R_{s2}) / (FS{=}2)$$

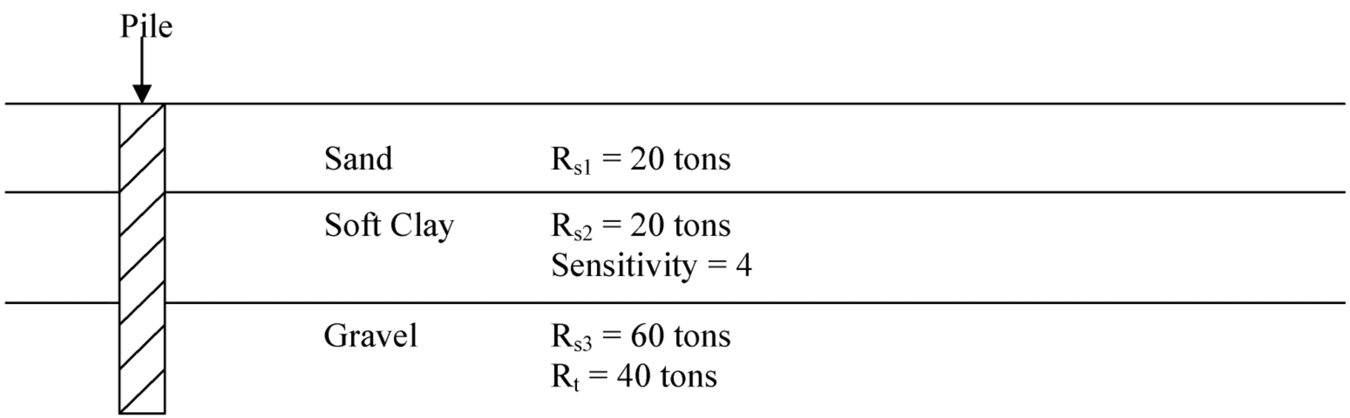
The result of the static analysis is then the estimated pile penetration depth, D, the design load for that penetration depth, Q_a, and the calculated ultimate capacity, Q_u.

For preparation of construction plans and specifications, the **calculated geotechnical ultimate capacity**, Q_u, is specified. Note that if the construction control method changes after the design stage, the required ultimate capacity and the required pile penetration depth for the ultimate capacity will also change. This is apparent when the previous equation for the design load is expressed in terms of the ultimate capacity as follows:

$$Q_u = R_{s1} + R_{s2} + (Q_a)(FS{=}2)$$

Example of Pile Capacity Using Static Method

Example 9-1: Find the ultimate capacity and driving capacity for the pile from the data listed in the profile. The hydraulic specialist determined that the sand layer is susceptible to scour. The geotechnical specialist determined that the soft clay layer is unsuitable for providing resistance.



Solution:

Ultimate capacity $= R_{s3} + R_t$

$= 60 \text{ tons} + 40 \text{ tons} = 100 \text{ tons}$

Driving capacity $= R_{s1} + (R_{s2}/\text{Sensitivity}) + R_{s3} + R_t$

$$= 20 \text{ tons} + \frac{20 \text{ tons}}{4} + 60 \text{ tons} + 40 \text{ tons} = 125 \text{ tons}$$

Fellenius Method

9.5.2.2 Effective Stress – β-method

Static capacity calculations in cohesionless, cohesive, and layered soils can also be performed by using an effective stress based method. Effective stress based methods were developed to model the long term drained shear strength conditions. Therefore, the effective soil friction angle, ϕ', should be used in parameter selection.

In an effective stress analysis, the unit shaft resistance is calculated from the following expression:

$$f_s = \beta\, p_o \qquad 9\text{-}12$$

where: β = Bjerrum-Burland beta coefficient = $K_s \tan \delta$.
p_o = average effective overburden pressure along the pile shaft, in ksf (kPa).
K_s = earth pressure coefficient.
δ = interface friction angle between pile and soil.

The unit toe resistance is calculated from:

$$q_t = N_t \;\; p_t \qquad 9\text{-}13$$

where: N_t = toe bearing capacity coefficient.
p_t = effective overburden pressure at the pile toe in ksf (kPa).

Recommended ranges of β and N_t coefficients as a function of soil type and ϕ' angle from Fellenius (1991) are presented in Table 9-7. Fellenius (1991) notes that factors affecting the β and N_t coefficients consist of the soil composition including the grain size distribution, angularity and mineralogical origin of the soil grains, the original soil density and density due to the pile installation technique, the soil strength, as well as other factors. Even so, β coefficients are generally within the ranges provided and seldom exceed 1.0.

Table 9-7
Approximate range of β and N_t coefficients (Fellenius, 1991)

Soil Type	ϕ'	β	N_t
Clay	25 – 30	0.23 - 0.40	3 - 30
Silt	28 – 34	0.27 - 0.50	20 - 40
Sand	32 – 40	0.30 - 0.60	30 - 150
Gravel	35 – 45	0.35 - 0.80	60 - 300

For sedimentary cohesionless deposits, Fellenius (1991) that states N_t ranges from about 30 to a high of 120. In very dense non-sedimentary deposits such as glacial tills, N_t can be much higher, but it can also approach the lower bound value of 30. In clays, Fellenius (1991) notes that the toe resistance calculated by using an N_t of 3 is similar to the toe resistance calculated from an analysis where undrained shear strength is used. Therefore, the use of a relatively low value of the N_t coefficient in clays is recommended unless local correlations suggest higher values are appropriate.

Graphs of the ranges in β and N_t coefficients versus the range in ϕ' angle as suggested by Fellenius are presented in Figure 9-17 and 9-18, respectively. These graphs may be helpful in selection of β or N_t. The inexperienced user should select conservative β and N_t coefficients. As with any design method, the user should also confirm the appropriateness of a selected β or N_t coefficient in a given soil condition with local correlations between static capacity calculations and static load tests results.

It should be noted that the effective stress method places no limiting values on either the shaft or toe resistance.

STEP BY STEP PROCEDURE FOR THE EFFECTIVE STRESS METHOD

STEP 1 Delineate the soil profile into layers and determine ϕ' angle for each layer.

a. Construct p_o diagram by using previously described procedures in Chapter 2.

b. Divide soil profile throughout the pile penetration depth into layers and determine the effective overburden pressure, p_o, in ksf (kPa) at the midpoint of each layer.

c. Determine the ϕ' angle for each soil layer from laboratory or in-situ test data.

Fellenius Method

d. In the absence of laboratory or in-situ test data for cohesionless layers, determine the average corrected SPT N1 value for each layer and estimate ϕ' angle from Table 8-1 in Chapter 8.

STEP 2 Select the β coefficient for each soil layer.

a. Use local experience to select β coefficient for each layer.

b. In the absence of local experience, use Table 9-7 or Figure 9-17 to estimate the β coefficient from the ϕ' angle for each layer.

STEP 3 For each soil layer compute the unit shaft resistance, f_s in ksf (kPa).

$$f_s = \beta \, p_o$$

STEP 4 Compute the shaft resistance in each soil layer and the ultimate shaft resistance, R_s in kips (kN) from the sum of the shaft resistance from each soil layer.

$$R_s = \Sigma f_s A_s$$

where: A_s = pile-soil surface area in ft^2 (m^2) = (pile perimeter) x (length).

STEP 5 Compute the unit toe resistance, q_t in ksf (kPa).

$$q_t = N_t \, p_t$$

a. Use local experience to select N_t coefficient.

b. In the absence of local experience, estimate N_t from Table 9-7 or Figure 9-18 based on ϕ' angle.

c. Calculate the effective overburden pressure at the pile toe, p_t in ksf (kPa).

STEP 6 Compute the ultimate toe resistance, R_t in kips (kN).

$$R_t = q_t \, A_t$$

where: A_t = area of the pile toe in m^2 (ft^2).

STEP 7 Compute the ultimate geotechnical pile capacity, Q_u in kips (kN).

$$Q_u = R_s + R_t$$

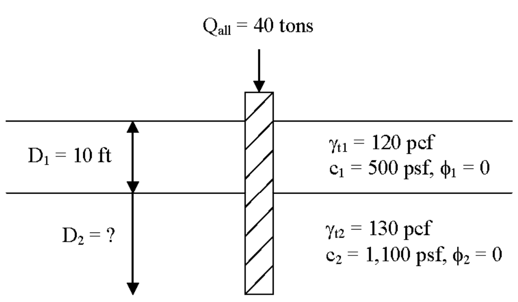
STEP 8 Compute the allowable geotechnical soil resistance, Q_a in kips (kN).

$$Q_a = \frac{Q_u}{\text{Factor of Safety}}$$

The factor of safety in this static calculation should be based on the specified construction control method as described in Section 9.4 of this chapter. Recommended factors of safety based on construction control methods are listed in Table 9-5

The concepts discussed above are illustrated numerically in Example 9-3.

Example 9-3: Determine the required pile length to resist a 40 tons load with a safety factor of 2. Assume no toe resistance for the 1 ft^2 precast concrete pile. Site specific tests have indicated that the adhesion may be assumed equal to cohesion.



Solution:

$Q_u = R_{s1} + R_{s2}$ (Note: No toe resistance, i.e. $9\, c_u A_t = 0$)

$Q_u = c_{a1} A_{s1} + c_{a2} A_{s2}$

$Q_u = c_{a1} C_{d1} D_1 + c_{a2} C_{d2} D_2$

where C_{d1} and C_{d2} are pile perimeters within depths D_1 and D_2

Fellenius Method

$C_{d1} = C_{d2} = 4 \times 1 \text{ ft} = 4 \text{ ft}$

From the problem statement, for site-specific conditions, adhesion = cohesion. Therefore,

$c_{a1} = c_1 = 500 \text{ psf}$

$c_{a2} = c_2 = 1{,}100 \text{ psf}$

$Q_u = 40 \text{ tons} \times FS = 40 \text{ tons x } 2 = 80 \text{ tons}$

$80 \text{ tons} = (500 \text{ psf})(4 \text{ ft})(10 \text{ ft}) + (1{,}100 \text{ psf})(4 \text{ ft})D_2$

$80 \text{ tons} = 20{,}000 \text{ lbs} + 4{,}400\ D_2 \text{ lbs/ft}$

$80 \text{ tons} = 10 \text{ tons} + 2.2\ D_2 \text{ tons/ft}$

Solve for D_2,

$$D_2 = \frac{80\,\text{tons} - 10\,\text{tons}}{2.2\,\text{tons}/\text{ft}} \approx 32\,\text{ft}$$

$\therefore$Total pile length required = 32 ft + 10 ft ≈ 42 ft

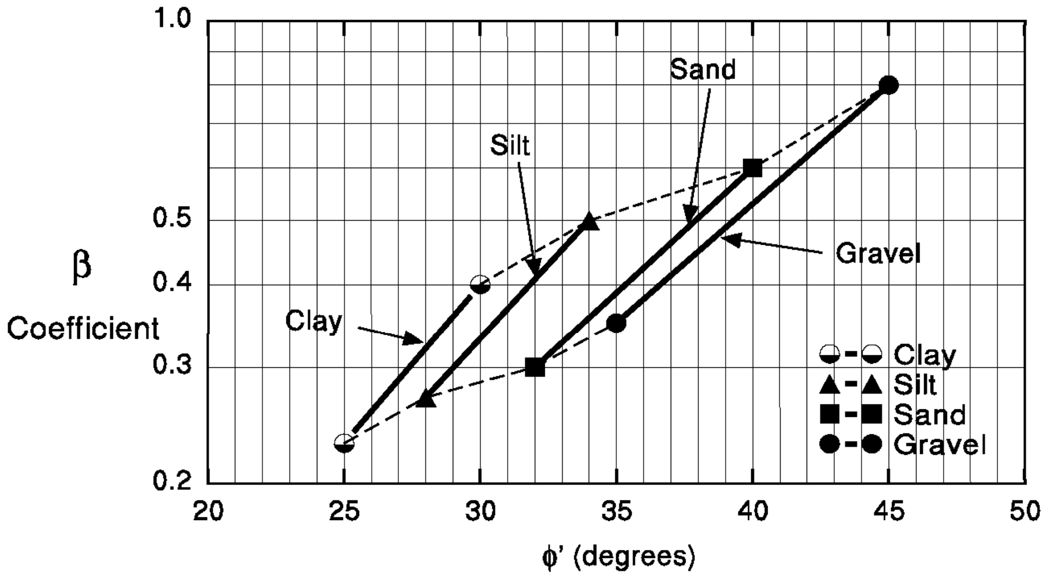


Figure 9-17. Chart for estimating β coefficient as a function of soil type ϕ' (after Fellenius, 1991).

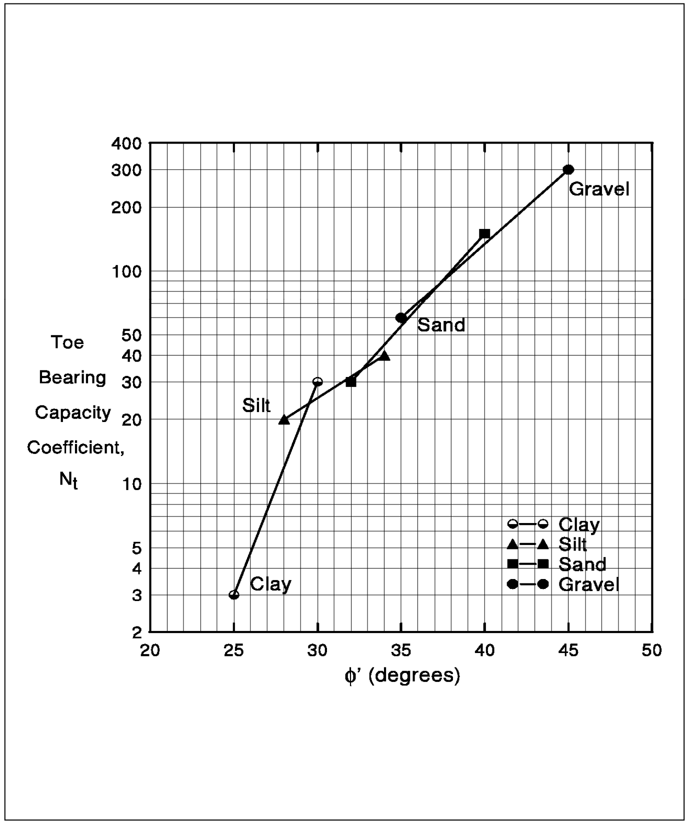


Figure 9-18. Chart for estimating N_t coefficients as a function of soil type ϕ' angle (after Fellenius, 1991).

Profiles and Contact Areas

- Contact Areas
  - Closed-section foundations
    - where foundation-soil contact takes place along a well defined border between pile and soil
    - Most deep foundations come under this designation
  - Open Section Foundations
    - Where foundation-soil contact is poorly defined and the soil will move as the pile penetrates
    - This includes open-ended pipe piles and (to some degree) H-beams
    - This phenomenon is referred to as plugging
- Displacement Characteristics

Table 9-3
Pile type selection pile shape effects

Shape Characteristics	Pile Types	Placement Effects
Displacement	Steel Pipe (Closed end), Precast Concrete	• Increase lateral ground stress • Densify cohesionless soils, remolds and weakens cohesive soils temporarily • Set-up time may be 6 months in clays for pile groups
Nondisplacement	Steel H, Steel Pipe (Open end)	• Minimal disturbance to soil • Not suited for friction piles in coarse granular soils. Piles often have low driving resistances in these deposits making field capacity verification difficult thereby often resulting in excessive pile lengths.
Tapered	Timber, Monotube, Tapertube, Thin-wall shell	• Increased densification of soils with less disturbance, high capacity for short length in granular soils

Plugging in Open-Section Foundations

- Plugging difficult to accurately predict
- For open-ended pipe pile piles where plugging is expected
  - Ignore plug for capacity calculations (esp. for toe area)
  - Assume closed ended condition for drivability purposes

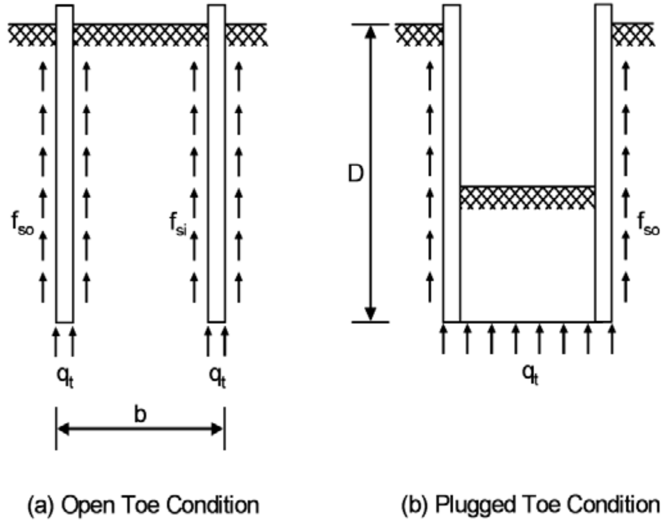


Figure 9-19. Plugging of open end pipe piles (after Paikowsky and Whitman, 1990).

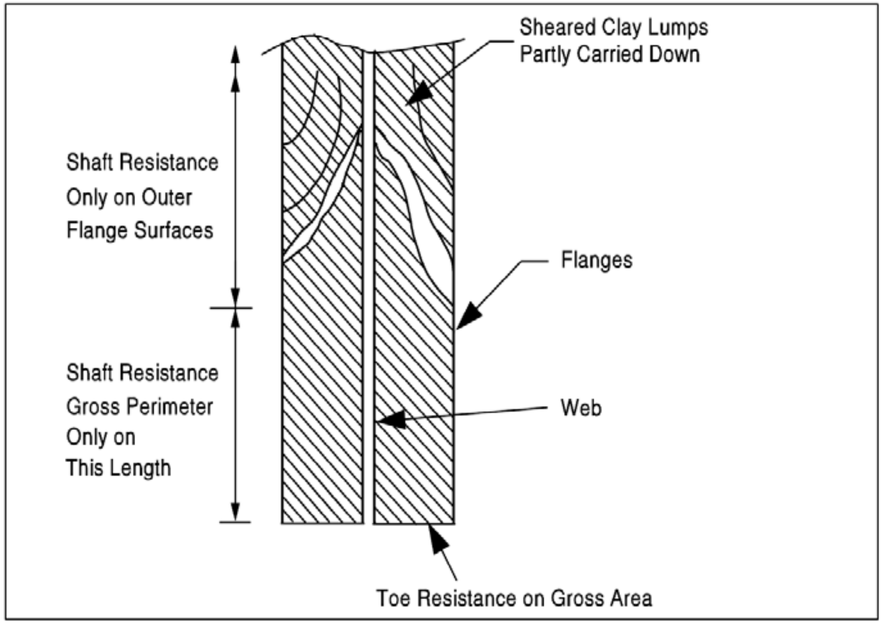


Figure 9-20. Plugging of H-piles (FHWA, 2006a).

Methods Based on SPT and CPT Data

- Most soil borings have stratigraphic profiles with either SPT data, CPT data, or both
- This eliminates the need for undisturbed samples deep in the earth
- Traditionally in the US SPT data have been standard, but CPT is becoming the standard method

- SPT Methods
  - Meyerhof’s Method, very crude, for preliminary analysis only
- CPT Methods
  - Schmertmann’s Method
  - The methods presented earlier can be adapted for CPT data and in fact this is the way they are normally used

Meyerhof's SPT Method

Meyerhof (1976) compiled and rationalized some of the wealth of experience then available and recommended that the capacity be a function of the N-index, as follows:

$$R = R_t + R_s = mN_t A_t + n\overline{N}_s A_s D \tag{7.6}$$

where	m	=	a toe coefficient
	n	=	a shaft coefficient
	N_t	=	N-index at the pile toe (taken as a pure number)
	A_s	=	N-index average along the pile shaft (taken as a pure number)
	A_t	=	pile toe area
	A_s	=	unit shaft area; circumferential area
	D	=	embedment depth

For values inserted into Eq. 7.6 using base SI-units, that is, **R** in newton, **D** in metre, and **A** in m^2/m, the toe and shaft coefficients, **m** and **n**, become:

m = $400 \cdot 10^3$ for driven piles and $120 \cdot 10^3$ for bored piles (N/m^2)

n = $2 \cdot 10^3$ for driven piles and $1 \cdot 10^3$ for bored piles (N/m^2)

For values inserted into Eq. 7.6 using English units with **R** in kips, **D** in feet, and **A** in ft^2/ft, the toe and shaft coefficients, **m** and **n**, become:

m = 8 for driven piles and 2.4 for bored piles (ksf)

n = 0.04 for driven piles and 0.02 for bored piles (ksf)

Schmertmann's CPT Method

7.10 Schmertmann and Nottingham

Toe resistance

The **Schmertmann and Nottingham** method is based on a summary of the work on model and full-scale piles presented by Nottingham (1975) and Schmertmann (1978). The unit toe resistance, r_t, is a "minimum path" average obtained from the cone stress values in an influence zone extending from 8b above the pile toe (b is the pile diameter) and 0.7b or 4b, as indicated in Fig. 7.9.

The procedure consists of five steps of filtering the q_c data to "minimum path" values. Step 1 is determining two averages of cone stress within the zone below the pile toe, one for a zone depth of 0.7b and one for 4b along the path "a" through "b". The smaller of the two is retained. (The zone height 0.7b applies to where the cone stress increases with depth below the pile toe). Step 2 is determining the smallest cone stress within the zone used for the Step 1. Step 3 consists of determining the average of the two values per Steps 1 and 2. Step 4 is determining the average cone stress in the zone above the pile toe according to the minimum path shown in Fig. 7.9. (Usually, just the average of the cone stress within the zone is good enough). Step 5, finally, is determining the average of the Step 3 and Step 4 values. This value is denoted q_{ca}.

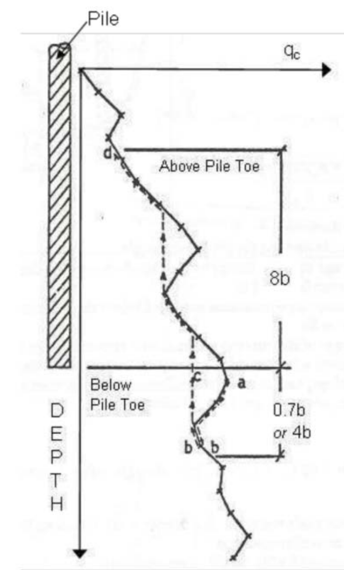


Fig. 7.9 Determining the influence zone for toe resistance (Schmertmann, 1978)

The pile toe resistance is then determined according to Eq. 7.8.

$$r_t = C\,q_{ca} \tag{7.8}$$

where r_t = pile unit toe resistance; an upper limit of 15 MPa is imposed
C = correlation coefficient governed by the overconsolidation ratio, OCR
q_{ca} = the cone stress filtered in the influence zone per the above procedure

The correlation coefficient, C, ranges from 0.5 through 1.0 depending on overconsolidation ratio, OCR, according to one of the "1" through" 3" slopes between the toe resistance, r_t, and the minimum-path average of the cone stress ("filtered in the influence zone"), as indicated in Fig. 7.10. For simplicity, the relations are usually also applied to a pile toe located in clay.

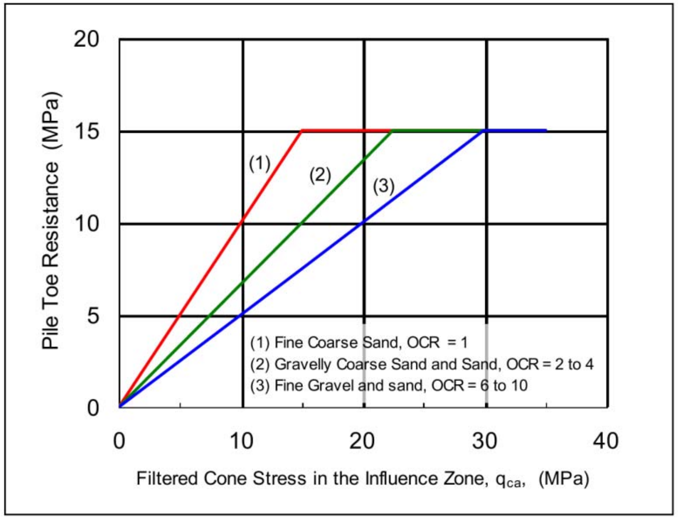


Fig. 7.10 Adjustment of unit toe resistance to OCR

Schmertmann's CPT Method

Unit shaft resistance

The unit shaft resistance, r_s, may be determined from the sleeve friction as expressed by Eq. 7.8.

(7.9) $$r_s = K_f f_s$$

where
r_s = pile unit shaft resistance; an upper limit of 120 kPa is imposed
K_f = a dimensionless coefficient
f_s = sleeve friction

In sand, K_f is assumed to be a function of the pile embedment ratio, D/b. Within a depth of the first eight pile diameters below the ground surface (D/b = 8), the K_f-coefficient is linearly interpolated from zero at the ground surface to 2.5. Hereunder, the value reduces from 2.5 to 0.891 at an embedment of 20 D/b. Simply applying K_f = 0.9 straight out is usually satisfactory.

In clay, K_f is a function of the sleeve friction and ranges from 0.2 through 1.25 as indicated in Fig. 7.11.

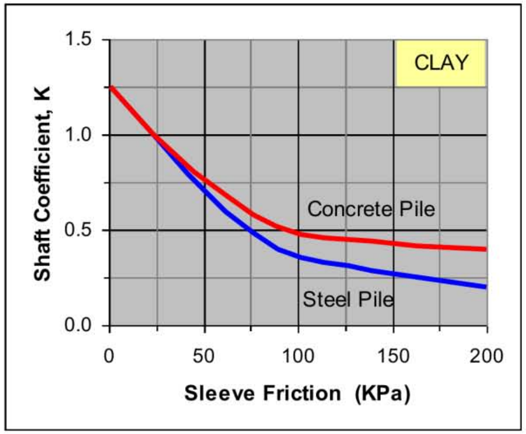


Fig. 7.11 Shaft coefficients for use in Eq. 7.8

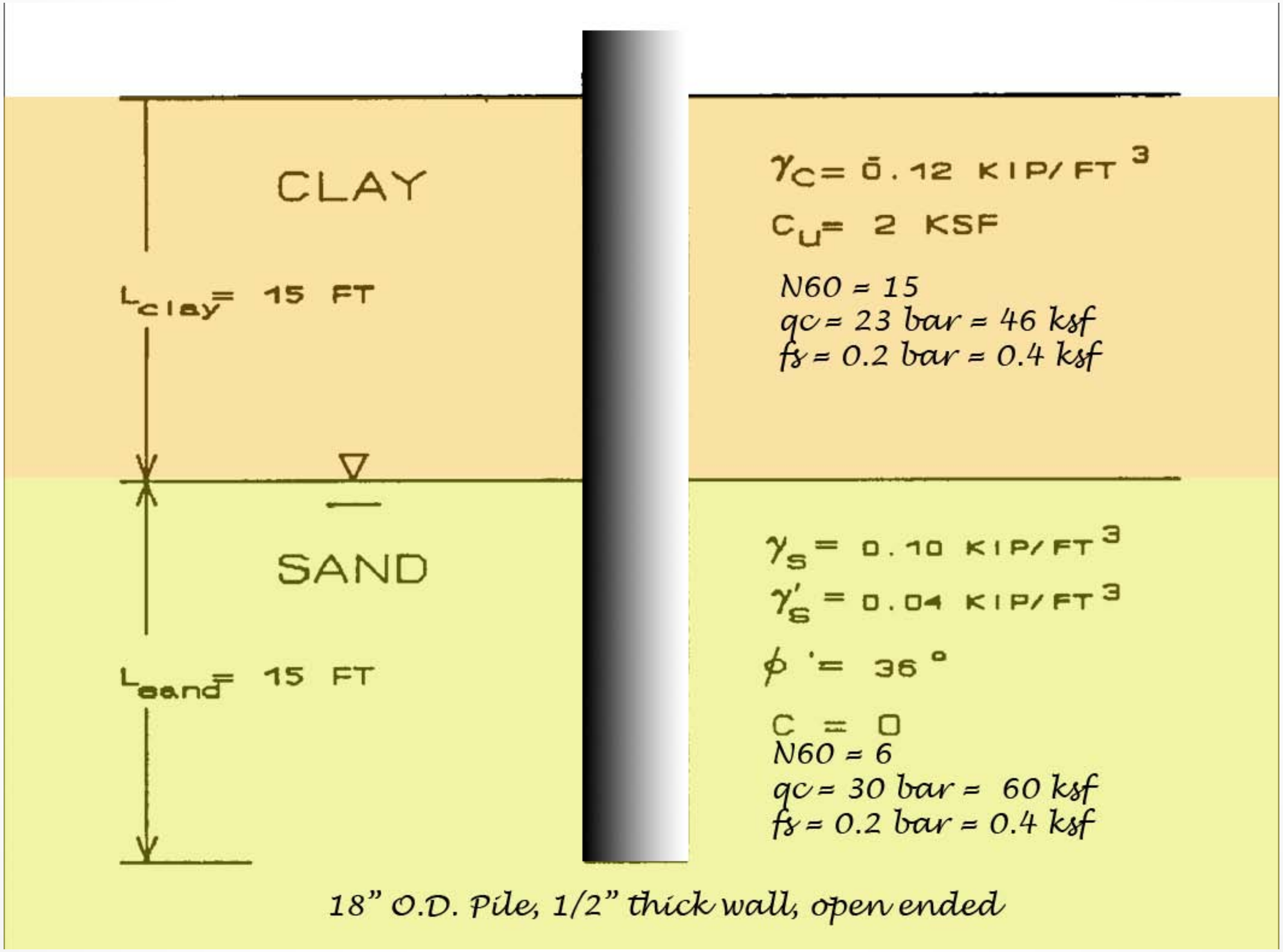
Alternatively, in sand, but not in clay, the shaft resistance may be determined from the cone stress, q_c, according to Eq. 7.10.

(7.10) $$r_s = K_c q_c$$

where
r_s = unit shaft resistance; an upper limit of 120 kPa is imposed
K_c = a dimensionless coefficient; a function of the pile type.
- for open toe, steel piles K_c = 0.8 %
- for closed-toe pipe piles K_c = 1.8 %
- for concrete piles K_c = 1.2 %

q_c = cone stress

SPT/CPT Driven Pile Example



SPT/CPT Driven Pile Example (Meyerhof and Schmertmann)

Layer	Layer Thickness	Start Elevation, ft	End Elevation, ft	Area for Shaft Layer, ft^2	Unit Weight, kcf	Effective Stress at Bottom of Layer, ksf	Effective Stress at Centre of Layer, ksf	N60	(N60)1 at Layer Bottom	(N60)1 at Layer Centre	Meyerhof Unit Shaft Resistance, ksf (Driven Piles)	Meyerhof Shaft Resistance per Layer, kips	fs, ksf	qc, ksf	Rs, kips
1a	7.50	0.00	7.50	35.34	0.12	0.90	0.45	15	22.36	30.00	1.20	42.41	0.4	46	12.72
1b	7.50	7.50	15.00	35.34	0.12	1.80	1.35	15	15.81	18.26	0.73	25.81	0.4	46	12.72
2a	7.50	15.00	22.50	35.34	0.04	2.10	1.95	6	5.86	6.08	0.24	8.59	0.4	60	14.14
2b	7.50	22.50	30.00	35.34	0.04	2.40	2.25	6	5.48	5.66	0.23	8.00	0.4	60	14.14
Toe		30.00			0.04	2.40		6	5.48		Total Shaft Resistance, kips, SPT Method	84.81		Total Shaft Resistance, kN, CPT Method	53.72
Length of Pile, ft	30										Unit Toe Resistance, ksf, SPT Method	Total Toe Resistance, kips,SPT Method		Unit Toe Resistance, ksf, CPT Method	Total Toe Resistance, ksf,CPT Method
Diameter of Pile, ft	1.50	D/b	20.00								43.82	2.60		60	3.56
Wall thickness, m	0.01	K for cone sleeve friction (Electrical Penetrometer)	0.80								Total Resistance/Ultimate Capacity, kips	87.41		Total Resistance/Ultimate Capacity, kips	57.28
Toe Area (no plugging), ft^2	0.06	8b	12.00	ft											
Pile Length, ft	98.40														
Pile Cross-Sectional Area, ft^2	0.638														
Pile Diameter, ft.	4.92														

Questions