Chapter 3
Vertical Loads

1. Design Philosophy

Analyses are performed to determine the diameter or cross section, length and number of driven piles or drilled shafts required to support the structure, and for procuring the correct materials and equipment to construct the foundation.

a. Type of loads. Loads applied to deep foundations consist of vertical forces and horizontal forces. These forces are resisted by the soil through bearing and friction. Therefore, the pile capacity analysis should be performed to determine that foundation failure by bearing or friction will be avoided, and load-displacement analysis performed to determine that foundation movements will be within acceptable limits.

(1) Load distribution. Loads on a deep foundation are simulated by a vertical force Q and a lateral force T, Figure 3-1. These vertical and horizontal forces are considered separately and their individual effects are superimposed. Unusual cross sections should be converted to a circular cross section for analysis when using computer programs such as CAXPILE (WES Instruction Report K-84-4) or AXILTR (Appendix C). Analysis for lateral loads is treated separately and given in Chapter 4.

(2) Construction influence. Construction methods, whether for driven piles or drilled shafts, influence pile capacity for vertical loads through soil disturbance and pore pressure changes.

(a) Driving resistance. A wave equation analysis shall be performed for driven piles to estimate the total driving resistance that will be encountered by the pile to assist in determining the required capability of the driving equipment. Refer to Chapter 6 for further details.

(b) Structural capacity. Total stresses that will be generated in the deep foundation during driving or by vertical and lateral loads will be compared with the structural capacity of the foundation. Structural capacity may be calculated by procedures in Chapter 2.

b. Analysis of vertical loads. The design philosophy for resisting vertical load is accomplished by calculating the ultimate pile capacity Q_u to determine the load to cause a bearing failure, then using FS to estimate the allowable pile capacity Q_a that can limit the settlement to permissible levels. Settlement of the individual piles or drilled shafts shall be calculated as presented later in this chapter, however settlement of a group of piles or drilled shafts shall be evaluated as given in Chapter 5. Table 3-1 illustrates this procedure.

(1) Ultimate pile capacity. Applied vertical loads Q (Figure 3-1) are supported by a base resisting force Q_b and soil-shaft skin resisting force Q_s. The approximate static load capacity Q_u resisting the applied vertical compressive forces on a single driven pile or drilled shaft is:

$$Q_u \approx Q_{bu} + Q_{su} \tag{3-1a}$$

$$Q_u \approx q_{bu} A_b + \sum_{i=1}^{n} Q_{sui}$$

where

Q_u = ultimate pile capacity, kips

Q_{bu} = ultimate end-bearing resisting force, kips

Q_{su} = ultimate skin resisting force, kips

q_{bu} = ultimate end-bearing resistance, ksf

A_b = area of tip or base, feet^2

Q_{sui} = ultimate skin resistance of pile element (or increment) i at depth z, kips

n = number of pile elements in pile length, L

Pile weight is negligible for deep foundations and neglected in practice. A drilled shaft or driven pile may be visualized to consist of a number of elements (as illustrated in Figure C-1, Appendix C), for calculation of ultimate pile capacity. The vertical pile resistance is a combination of the following:

(a) End-bearing resistance. Failure in end bearing is normally by punching shear with compression of the underlying supporting soil beneath the pile tip. Applied vertical compressive loads may lead to several inches of compression prior to plunging failure. Ultimate end-bearing resistance is

$$q_{bu} = cN_c\zeta_c + \sigma_L' N_q \zeta_q + \frac{B_b}{2}\gamma_b' N_\gamma \zeta_\gamma \tag{3-2}$$

where

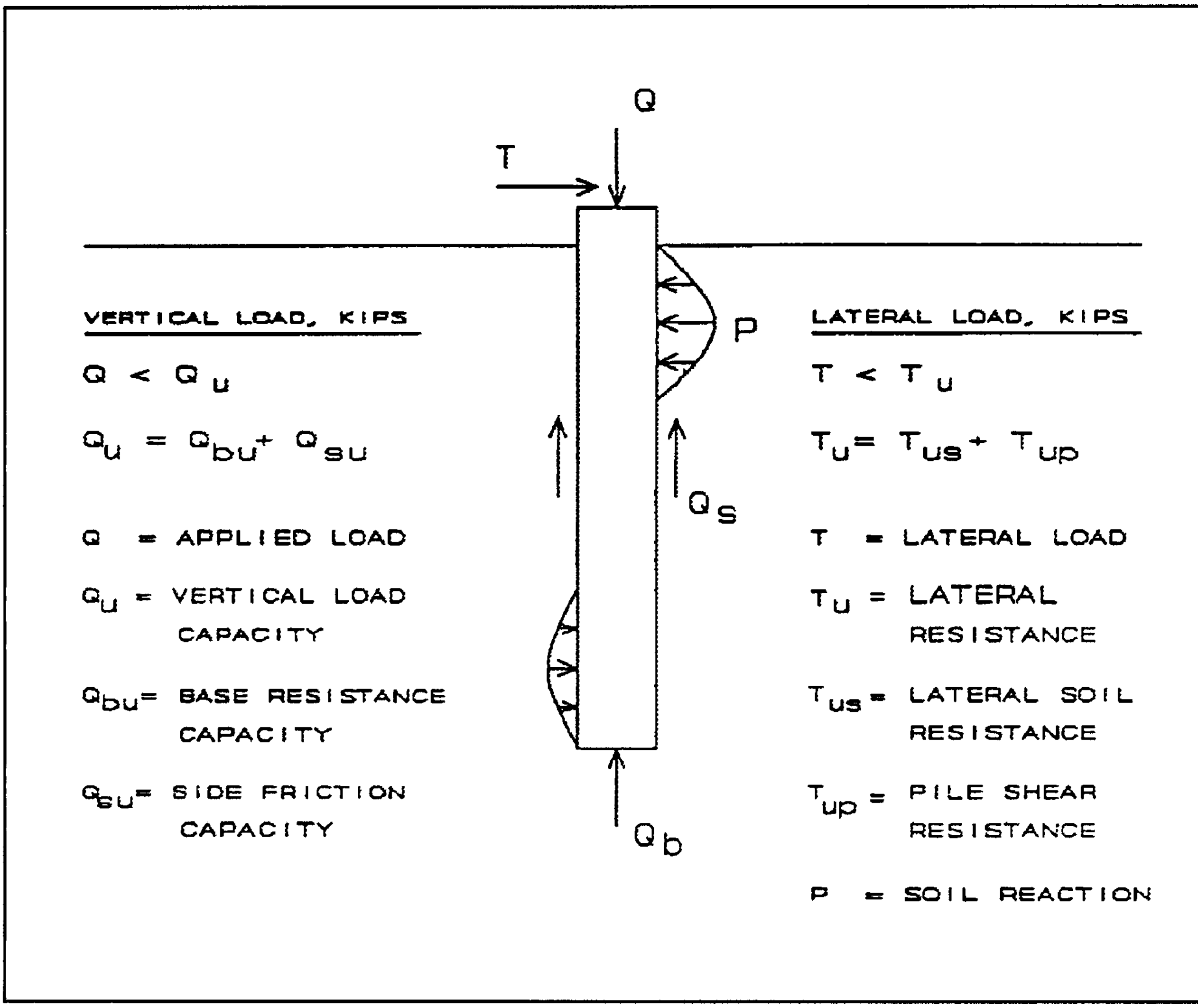


Figure 3-1. Loading support of deep foundation

c = cohesion of soil beneath the tip, ksf

σL = effective soil vertical overburden pressure at pile base, $\approx \gamma'_L L$, ksf

γ'_L = effective unit weight of soil along shaft length L, γ_{sat} - γ_w, kips/feet^3

γ_{sat} = saturated unit weight of soil, kips/feet^3

γ_w = unit weight of water, 0.064 kip/feet^3

γ_{sat} = saturated unit weight of soil, kips/feet^3

γ_w = unit weight of water, 0.064 kip/feet^3

B_b = base diameter, feet

γ'_b = effective unit weight of soil beneath base, kips/feet^3

$N_c N_q N_\gamma$ = cohesion, surcharge, and wedge-bearing capacity factors

$\zeta_c, \zeta_q, \zeta_\gamma$ = cohesion, surcharge, and wedge geometry correction factors

The submerged unit weight of soil below the phreatic surface is γ_{sat} - γ_w. The wet unit weight γ is used instead of the effective unit weight if the soil is above the water table. The bearing capacity N_c, N_q, N_γ and geometry correction ζ_c, ζ_q, ζ_γ factors are given with the methods recommended below for calculating end bearing resistance q_{bu}.

Table 3-1
Vertical Load Analysis

Step	Procedure
1	Evaluate the ultimate bearing capacity Q_u using guidelines in this manual and equation 3-1.
2	Determine a reasonable *FS* based on sub-surface information, soil variability, soil strength, type and importance of the structure, and past experience. The *FS* recommended for normal design will typically be between 2 and 4, Table 3-2a. Variations in *FS* are permitted depending on how critical the foundation is to structural performance, Table 3-2b. Allowable loads may be increased when the soil performance investigation is thorough, settlements will remain tolerable, and performance will not be affected.
3	Evaluate allowable bearing capacity Q_a by dividing Q_u by *FS*, $Q_a = Q_u/FS$, equation 3-4.
4	Perform settlement analysis of driven pile groups and drilled shafts and adjust the bearing pressure on the top (head or butt) of the deep foundation until settlement is within permitted limits. The resulting design load Q_d should be $\leq Q_a$. Settlement analysis is particularly needed when compressible layers are present beneath the potential bearing stratum. Settlement analysis will be performed on important structures and those sensitive to settlement. Settlement analysis of individual piles or drilled shafts is presented in Chapter 3-3 and for pile groups is presented in Chapter 5.
5	Conduct a load test when economically feasible because bearing capacity and settlement calculations are, at most, approximate. However, load tests of normal duration will not reflect the true behavior of saturated compressible layers below the bearing stratum. Load tests permit a reduced *FS* = 2 in most situations, which can reduce the cost of the foundation. Refer to Chapter 6 for information on conducting load tests.

(b) Side friction resistance. Soil-shaft side friction develops from relatively small movements between the soil and shaft, and it is limited by the shear strength of the adjacent soil. Side friction often contributes the most bearing capacity in practical situations unless the base is located on stiff soil or rock. The maximum skin resistance that may be mobilized along an element *i* of pile at depth *z* may be estimated by

$$Q_{sui} = f_{sui} C_z \Delta L \tag{3-3}$$

where

Q_{sui} = maximum load transferred to pile element *i* at depth *z*, kips

f_{sui} = maximum skin friction of pile element *i* at depth *z*, ksf

C_z = shaft circumference of pile element *i* at depth *z*, feet

ΔL = length of pile element *i*, feet

Ignoring effects due to the self-weight of the pile and residual stresses from pile driving, Figure 3-2 shows the distribution of skin friction and the associated load on a pile, where load is shown by the abscissa and depth is shown by the ordinate. The load carried in end bearing Q_b is shown in the sketch and the remainder Q is carried by skin friction. The slope of the curve in Figure 3-2c yields the rate that the skin friction f_s is transferred from the pile to the soil as shown in Figure 3-2b. Near the ground surface, f_s is usually small probably because vibrations from driving a pile form a gap near the ground surface and because of the low lateral effective stress near the top of the pile or drilled shaft. The relatively low values of f_s near the tip of a pile or drilled shaft in cohesive soils has been observed in experiments because of the decreasing soil movement against the pile as moving toward the tip. Therefore, the skin friction f_s, as a function of depth, frequently assumes a shape similar to a parabola (Figure 3-2b).

(2) Critical depth. The Meyerhof (1976) and Nordlund (1963) methods for driven piles assume that the effective vertical stress reaches a constant value after some critical depth D_c, perhaps from arching of soil adjacent to the shaft length. The critical depth ratio D_c/B, where B is the shaft diameter, is found in Figure 3-3a. For example, if the effective friction angle $\phi' = 35°$, then $D_c = 10B$, and end-bearing capacity will not increase below depth D_c, Figure 3-3b. End-bearing resistance q_{bu} will not exceed q given by Figure 3-4. Analysis of deep foundations using the pile driving analyzer has not supported this concept.

(3) Load Limits. Applied loads should be sufficiently less than the ultimate capacity to avoid excessive pile vertical and lateral displacements; e.g., $\leq$ 0.5 inch.

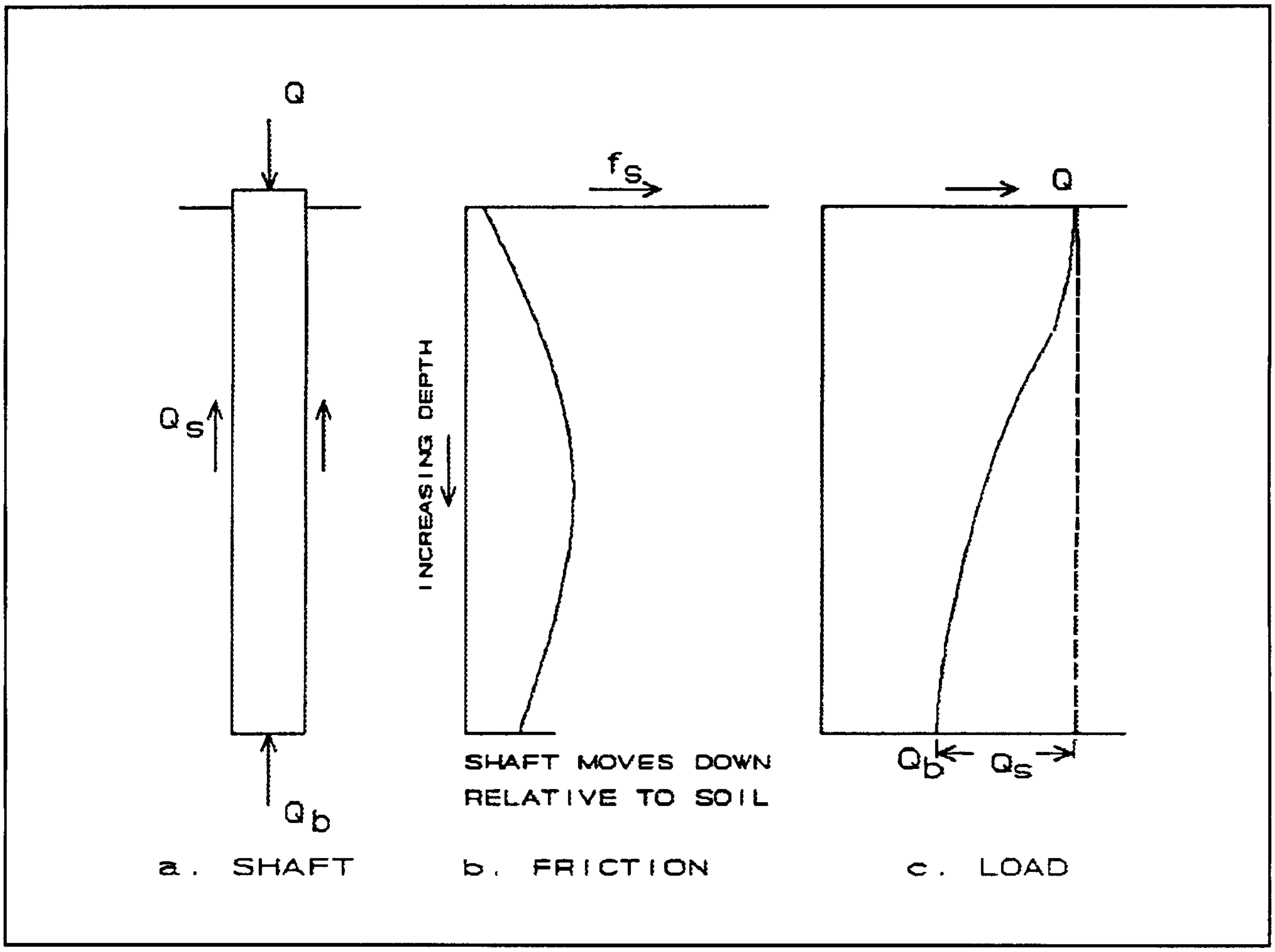


Figure 3-2. Distribution of skin friction and the associated load resistance

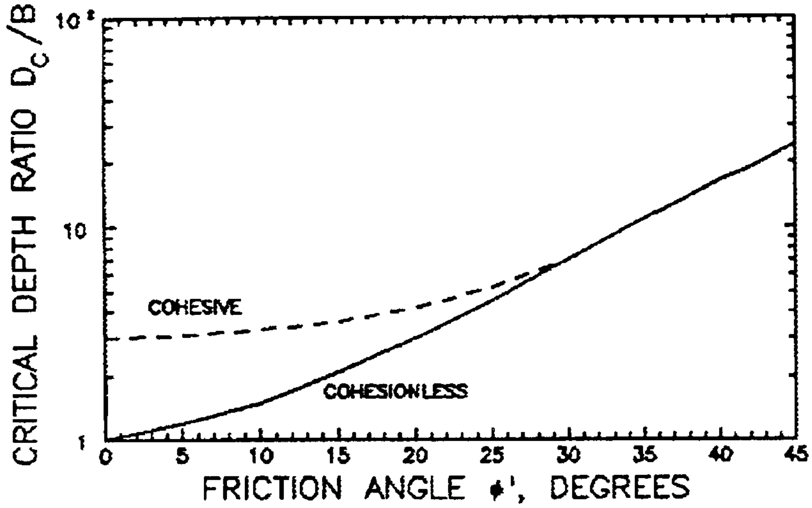
Applied loads one-half to one-fourth of the ultimate load capacity are often specified for design.

(a) Allowable pile capacity. The allowable pile capacity Q_a is estimated from the ultimate pile capacity using *FS*

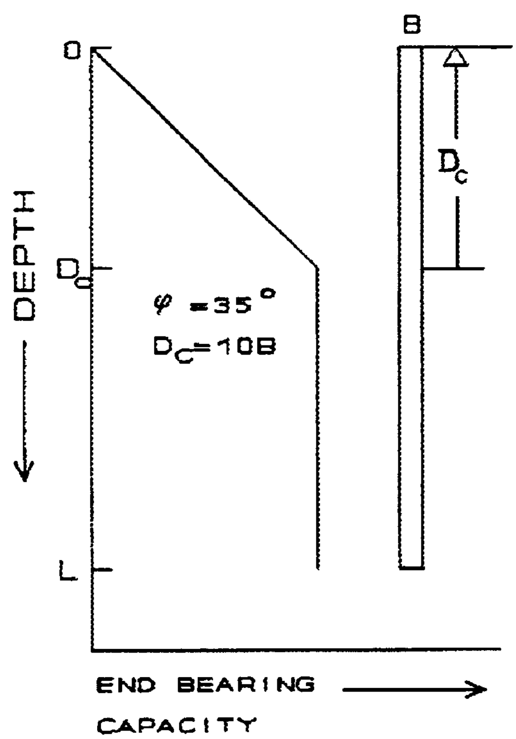
$$Q_a = \frac{Q_u}{FS} \tag{3-4}$$

The design load $Q_d \leq Q_a$, depending on results of settlement analysis.

(b) Typical factors of safety. Table 3-2a provides typical *FS* for vertical load behavior. Typical or usual loads refer to conditions which are a primary function of a structure and can be reasonably expected to occur during the service life. Such loads may be long-term, constant, intermittent, or repetitive nature. Deviations from these minimum values may be justified by extensive foundation investigations and testing to reduce uncertainties related to the variability of the foundation soils and strength parameters. Load tests allow *FS* to be 2 for usual design and may lead to substantial savings in foundation costs for economically significant projects.



a. D_c/B VERSUS FRICTION ANGLE



b. EXAMPLE CRITICAL DEPTH RATIO

Figure 3-3. Critical depth ratio (Meyerhof 1976) (Copyright permission, American Society of Civil Engineers

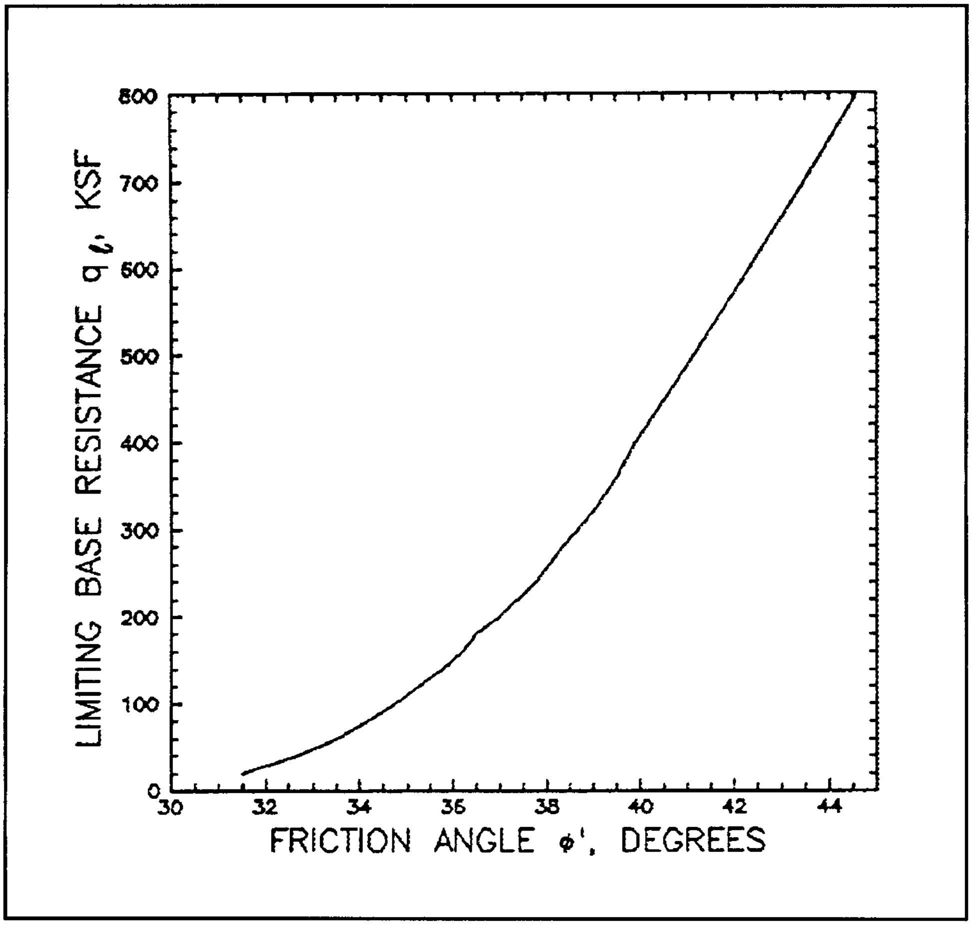


Figure 3-4. Limiting base resistance for Meyerhof and Nordlund methods

(c) Other factors of safety. Lower *FS* are possible for unusual or extreme loads, Table 3-2b, provided soil investigation is thorough and settlement will be within a tolerable range. Unusual loads refer to construction, operation, or maintenance conditions which are of relatively short duration or infrequent occurrence. Risks associated with injuries or property losses can be controlled by specifying the sequence or duration of activities and/or by monitoring performance. Extreme loads refer to events which are highly improbable and occur only during an emergency. Such events may be associated with major accidents involving impacts or explosions and natural disasters due to hurricanes. Extreme loads may also occur from a combination of unusual loads. The basic design for typical loads should be efficiently adapted to accommodate extreme loads without experiencing a catastrophic failure; however, structural damage which partially impairs the operational functions and requires major rehabilitation or replacement of the structure is possible. Caution is required to achieve an efficient design that will avoid unacceptable injuries or property losses.

(d) Group performance. Pile group analyses should be conducted as discussed in Chapter 5 to be sure that a state of ductile, stable equilibrium is attained even if individual piles will be loaded to or beyond their peak capacities.

(e) Field verification. Field instrumentation, frequent or continuous field monitoring of performance, engineering studies and analyses, and constraints on operational or rehabilitation activities may be required to ensure that the structure will not fail catastrophically during or after extreme loading. Deviations from these criteria for extreme loads should be formulated in consultation with and approved by CEMP-ET.

2. Driven Piles

The general procedure for calculating vertical loads of driven piles is given in Table 3-3. The total vertical capacity Q_u is calculated by equation 3-1 where methods for determining end-bearing Q_{bu} and skin friction Q_{su} resistance

Table 3-2
Factors of Safety for Bearing Capacity (Pile Buck, Inc. 1992)

Usual Loads			
Condition			**Factor of Safety**
With load test			2.0
Base on bedrock			2.0
Driven piles with wave equation analysis calibrated to results of dynamic pile tests			
Compression			2.5
Tension			3.0
Resistance to uplift			2.5
Resistance to downdrag			3.0
Without load tests			3.0
Groups			3.0
Soil profile containing more than one type of soil or stratum			4.0
Influence of Loading Condition			
		Minimum Factor of Safety	
Method of Capacity Calculation	**Loading Condition[1]**	**Compression**	**Tension**
Verified by pile load test	Usual	2.0	2.0
	Unusual	1.5	1.5
	Extreme	1.15	1.15
Verified by pile driving analyzer, Chapter 6	Usual	2.5	3.0
	Unusual	1.9	2.25
	Extreme	1.4	1.7
Not verified by load test	Usual	3.0	3.0
	Unusual	2.25	2.25
	Extreme	1.7	1.7

[1] Defined in paragraph 3-1.b (3)(c)

are given below. In addition, a wave equation and pile driving analysis should be performed to estimate bearing capacity, maximum allowable driving forces to prevent pile damage during driving, and total driving resistance that will be encountered by the pile. These calculations assist in determining the required capability of the driving equipment and to establish pile driving criteria.

a. End-bearing resistance. Ultimate end-bearing resistance is given by equation 3-2 neglecting the N_γ term

Table 3-3
General Design Procedure of a Driven Pile

Step	Procedure	Description
1	Select potentially suitable pile dimensions	Select several potentially suitable dimensions; final design selected to economize materials and while maintaining performance.
2	Evaluate end-bearing capacity Q_{bu}	Use equation 3-6 to compute end-bearing capacity q_{bu} for clay and equations 3-7 to 3-10 for sands. Use equations 3-11 to 3-13 to compute q_{bu} from in situ tests. $Q_{bu} = q_{bu} A_b$ from equation 3-1b.
3	Evaluate skin resisting force Q_{sui}	Use equation 3-3 to compute skin resisting force Q_{sui} for each element *i*. For clays, skin friction f_{sui} is found from equation 3-16 using α_a from Table 3-5 or equation 3-17 with Figure 3-11. For sands, f_{sui} is found from equation 3-20 using Figure 3-13 or Nordlund method in Table 3-4b. The Q_{su} for clays or sands is found from CPT data from equation 3-19 and Figures 3-12 and 3-14.
4	Compute ultimate pile capacity Q_u	Add Q_{bu} and Q_{su} to determine Q_u using equation 3-1.
5	Check that design load $Q_d \le Q_a$	Calculate Q_a from equation 3-4 using factors of safety from Table 3-2 and compare with Q_d.

$$q_{bu} = cN_c\zeta_c + \sigma_L'(N_q - 1)\zeta_q \qquad (3\text{-}5a)$$

or

$$q_{bu} = cN_c\zeta_c + \sigma_L' N_q\zeta_q \qquad (3\text{-}5b)$$

Equation 3-5b is often used because omitting the "1" usually has negligible effect. The N_γ term is negligible for driven piles.

(1) Cohesive soil. The shear strength of cohesive soil is $c = C_u$, the undrained strength, the effective friction angle $\phi' = 0$ and $N_q = 1$. Thus, equation 3-5a may be reduced to

$$q_{bu} = N_c \times C_u = 9 \times C_u \qquad (3\text{-}6)$$

where shape factor $\zeta_c = 1$ and $N_c = 9$. Undrained shear strength C_u may be taken as the mean value within $2B_b$ beneath the pile tip.

(2) Cohesionless soil. Several of the methods using equation 3-5 and in the following subparagraphs should be used for each design problem to provide a reasonable range of bearing capacity.

(a) Nordlund method. This semiempirical method (Nordlund 1963) taken from FHWA-DP-66-1 (Revision 1), "Manual on Design and Construction of Driven Pile Foundations," considers the shape of the pile taper and the influence of soil displacement on skin friction. Equations for calculating ultimate capacity are based on load test results that include timber, steel H, pipe, monotube, and Raymond step-taper piles. Ultimate capacity is

$$Q_u = \alpha_f N_q A_b \sigma_L' + \sum_{z=0}^{z=L} K C_f \sigma_z' \frac{\sin(\delta + \omega)}{\cos\omega} C_z \Delta L \qquad (3\text{-}7a)$$

where

α_f	=	dimensionless pile depth-width relationship factor
A_b	=	pile point area, ft^2
σ_L'	=	effective overburden pressure at pile point, ksf
K	=	coefficient of lateral earth pressure at depth z
C_f	=	correction factor for K when $\delta \neq \phi'$
ϕ'	=	effective angle of internal friction for soil, degrees
δ	=	friction angle between pile and soil, degrees
ω	=	angle of pile taper from vertical, degrees

σ'_z = effective overburden pressure at the center of depth increment ΔL, $0 < z \leq L$, ksf

C_z = pile perimeter at depth z, feet

Δ = L
length of pile increment, feet

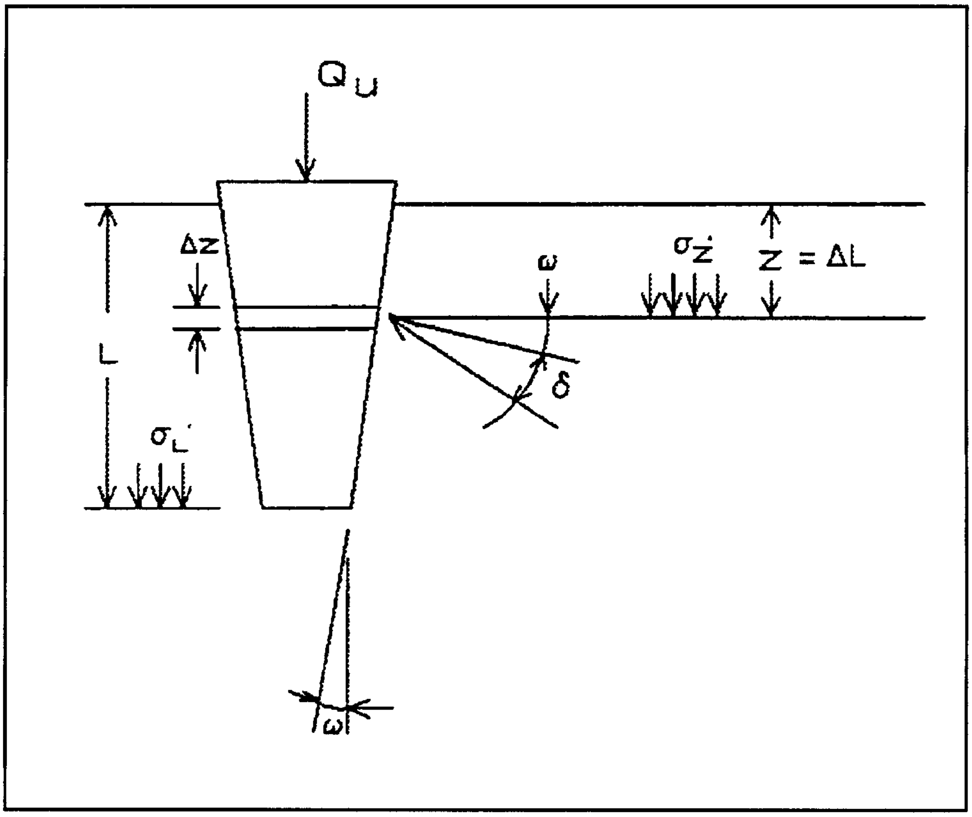


Figure 3-5. Illustration of input parameters for equation 3-7a

L = length of pile, feet

Some of these parameters are illustrated in Figure 3-5. End-bearing resistance $q_{bu} = \alpha_f N_q \sigma'_L A$ from equation 3-7a. As shown in Figure 3-4, q_{bu} should not exceed q_l where q is given. Other parameters can be determined as follows: α_f and N_q are found from Figure 3-6, K from Figure 3-7, δ from Figure 3-8 for a given ϕ' and V, and C_f from Figure 3-9. The volume V is displacement by the pile per given penetration length. The Q_u for a pile of uniform cross section ($\omega = 0$) and length L driven in a homogeneous soil with a single friction angle ϕ and single effective unit weight is

$$Q_u = \alpha_f N_q A \sigma'_L + K C_f \sigma'_m \sin \delta \, C_s L \qquad (3\text{-}7b)$$

where

A = pile cross section area

C_s = is the pile perimeter

σ'_m = mean effective vertical stress between the ground surface and pile tip, ksf.

The procedure for evaluating Q_{bu} by the Nordlund method is given in Table 3-4.

(b) Vesic method. Bearing capacity factors of equation 3-5b are estimated by (Vesic 1977)

$$N_c = (N_q - 1) \cot \phi' \qquad (3\text{-}8a)$$

$$N_q = \frac{3}{3 - \sin\phi'} e^{\frac{(90-\phi')\pi}{180} \tan \phi'} \tan^2\left[45 + \frac{\phi'}{2}\right] I_{rr}^{\frac{4\sin\phi'}{3(1+\sin\phi')}} \qquad (3\text{-}8b)$$

$$I_{rr} = \frac{I_r}{1 + \epsilon_v \times I_r} \qquad (3\text{-}8c)$$

$$I_r = \frac{G}{C_u + \sigma_L' \tan\phi'} \qquad (3\text{-}8d)$$

$$\epsilon_v = \frac{1 - 2\nu_s}{2(1 - \nu_s)} \times \frac{\sigma_L'}{G} \qquad (3\text{-}8e)$$

where

- ϵ_v = volumetric strain, fraction
- υ_s = soil Poisson's ratio
- G = soil shear modulus, ksf
- C_u = undrained shear strength, ksf
- ϕ' = effective friction angle, degrees
- σ_L' = effective soil overburden pressure at pile base, ksf

The reduced rigidity index $I_{rr} \approx$ rigidity index I_r for undrained or dense soil where $\nu_s = 0.5$. $G = E/[2(1 + \upsilon_s)]$ where E is the soil elastic modulus. Shape factor $\zeta_c = 1.00$ and

$$\zeta_q = \frac{1 + 2K_o}{3} \qquad (3\text{-}9a)$$

$$K_o = (1 - \sin\phi') \cdot OCR^{\sin\phi'} \qquad (3\text{-}9b)$$

where

- K_o = coefficient of earth pressure at rest
- OCR = overconsolidation ratio

The OCR is the ratio of the preconsolidation pressure p_c to the vertical effective soil pressure. If the OCR is not known, then K_o can be estimated from the Jaky equation as follows

$$K_o = 1 - \sin\phi' \qquad (3\text{-}9c)$$

(c) General shear method. The bearing capacity factors of equation 3-5b may be estimated, assuming the Terzaghi general shear failure (Bowles 1968), as

$$N_q = \frac{a^2}{2\cos^2(45 + \frac{\phi'}{2})}, \quad a = e^{(\frac{3\pi}{4} - \frac{\phi}{2}) \tan\phi} \qquad (3\text{-}10)$$

Shape factor $\zeta_q = 1.00$. $N_c = (N_q - 1) \cot \phi'$.

(d) SPT Meyerhof Method. End-bearing capacity may be estimated from penetration resistance data of the SPT by (Meyerhof 1976)

$$q_{bu} = 0.8 \times N_{SPT} \times \frac{L_b}{B} < 8 \times N_{SPT}, \quad \frac{L}{B} \le 10 \qquad (3\text{-}11)$$

where

- N_{SPT} = average uncorrected blow count within $8B$ above and $3B_b$ below the pile tip
- L_b = depth of penetration of the pile tip into the bearing stratum
- q_{bu} = is in units of ksf.

(e) CPT Meyerhof method. End-bearing capacity may be estimated from cone penetration resistance data by (Meyerhof 1976)

$$q_{bu} = \frac{q_c}{10} \times \frac{L_b}{B} < q_\ell \qquad (3\text{-}12)$$

based on numerous load tests of piles driven to a firm cohesionless stratum not underlain by a weak deposit. The limiting static point resistance given by Figure 3-4 is q_ℓ. q_{bu} and q_ℓ are in units of ksf.

(f) CPT 1978 FHWA-Schmertmann method. End bearing capacity may be estimated by (FHWA-TS-78-209)

$$q_{bu} = \frac{q_{c1} + q_{c2}}{2} \qquad (3\text{-}13)$$

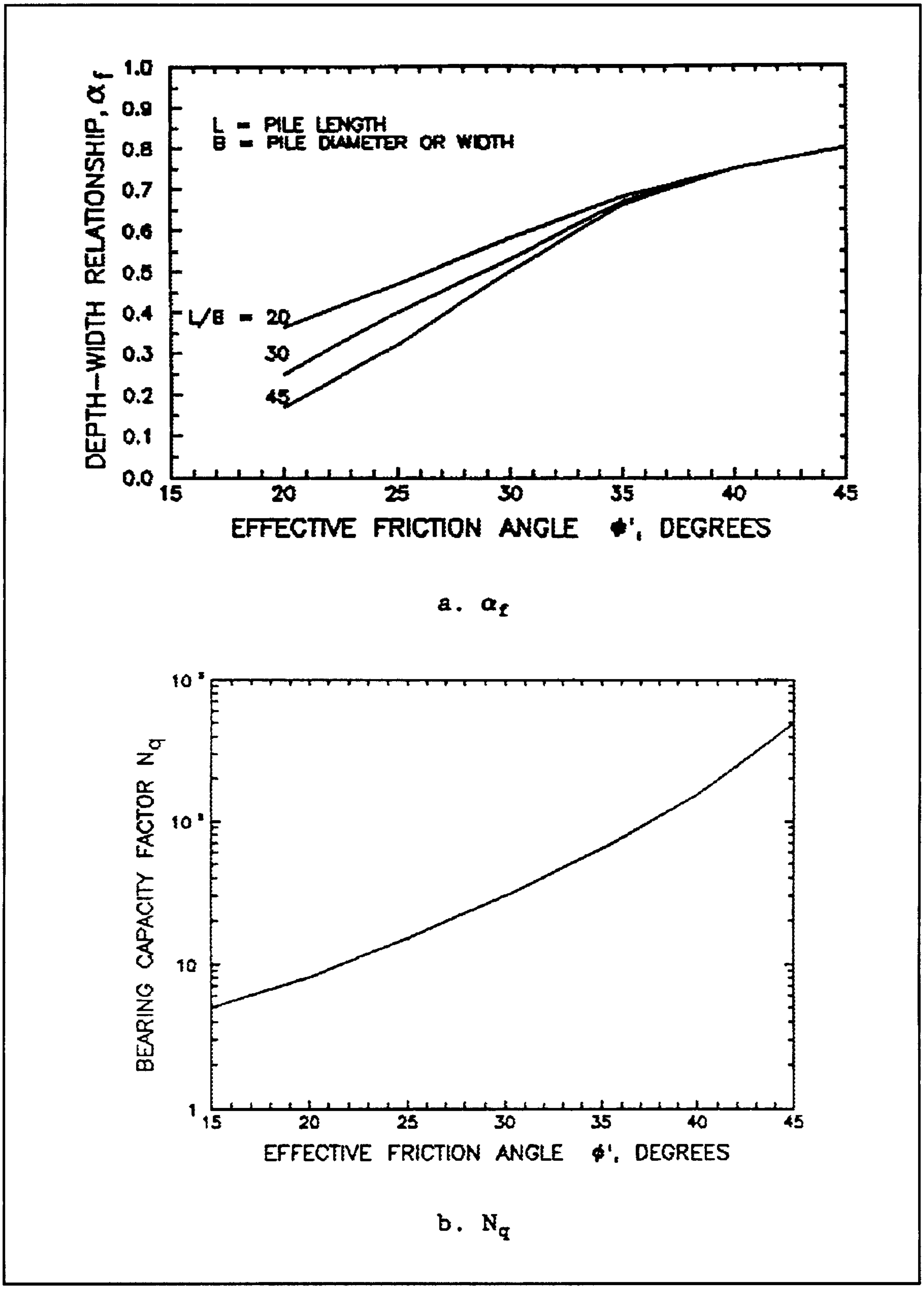


Figure 3-6. Variation of α_f and bearing capacity factor N_q with respect to ϕ' (FHWA-DP-66-1)

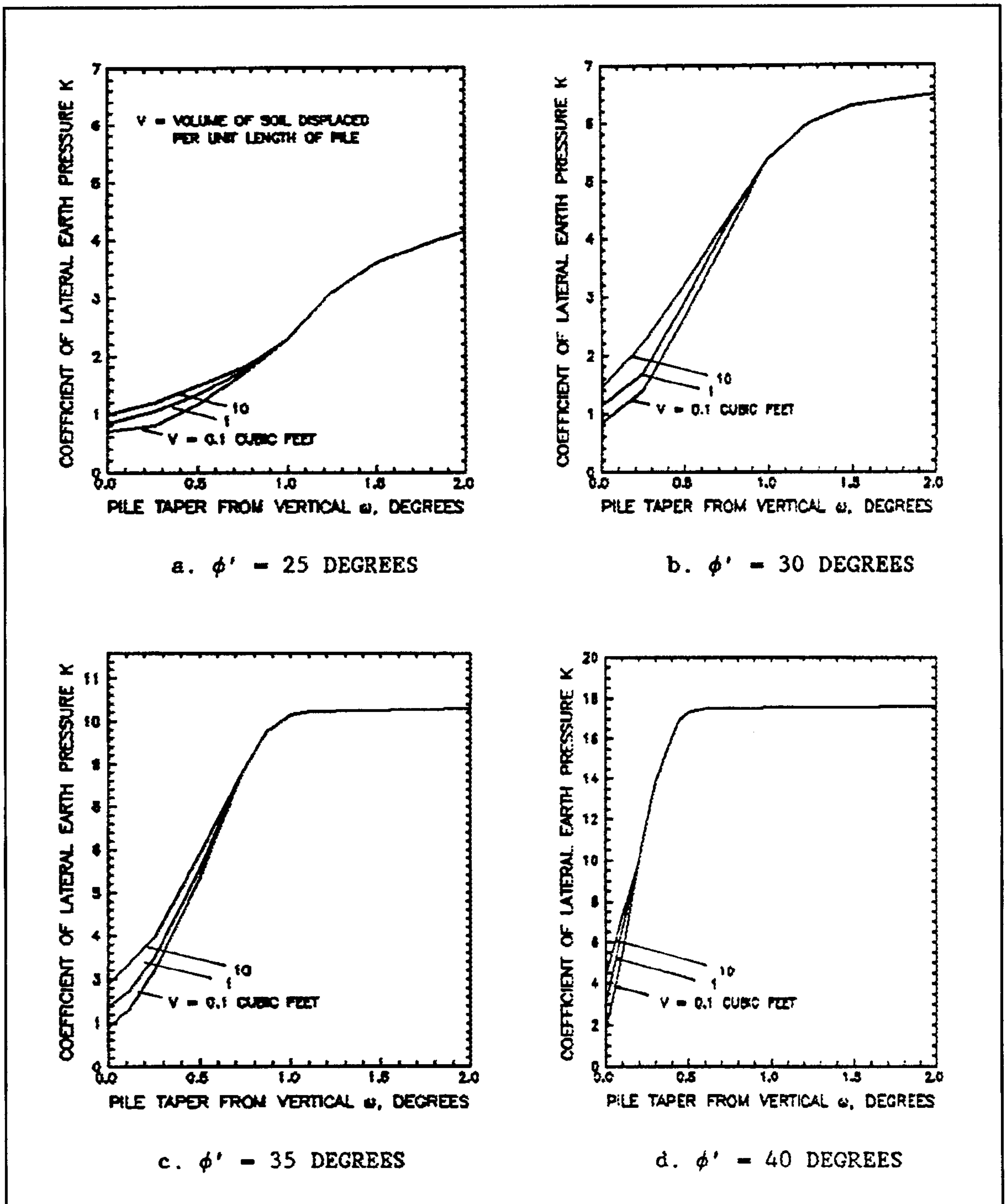
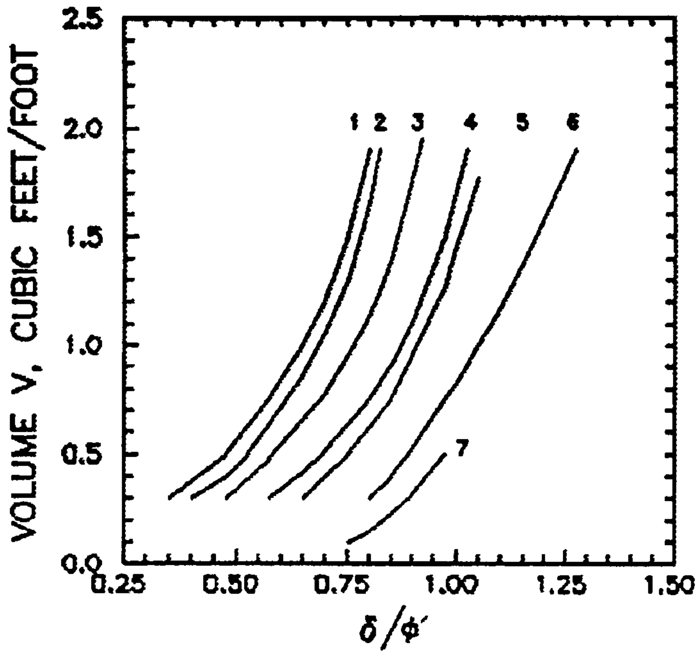


Figure 3-7. Variation of the coefficient *K* with respect to φ' (FHWA-DP-66-1)



1. Pipe and nontapered portions of monotube piles
2. Timber piles
3. Precast concrete piles
4. Raymond step-taper piles
5. Tapered portion of monotube piles
6. Raymond uniform taper piles
7. H- and augercast piles

Figure 3-8. Ratio δ/φ′ for given displacement Volume *V*

where q_{c1} and q_{c2} are unit cone point resistances determined as given in Figure 3-10.

For example, q_{c1} calculated over the minimum path is as follows:

$$q_{c1} = \frac{180 + 170 + 170 + 170 + 170}{5}$$

$$= 172 \text{ ksf}$$

q_{c2} over the minimum path is:

$$q_{c2} = \frac{120 + 150 + 160 + 160 + 160 + 160 + 160 + 160}{8}$$

$$= 153.75 \text{ ksf}$$

From equation 3-13,

$$q_{bu} = (172 + 153.75)/2 = 162.9 \text{ ksf}$$

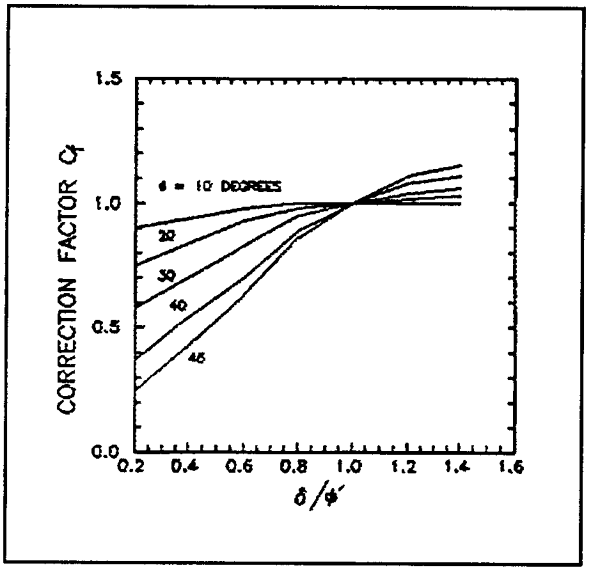


Figure 3-9. Correction factor C_f with respect to δ/ϕ' (FHWA-DP-66-1)

(3) Scale effects. Ultimate end-bearing capacity q_{bu} tends to be less for larger diameter driven piles and drilled shafts than that indicated by equation 3-11 or 3-12 or equation 3-5. Skin friction per unit circumferential area f_s is assumed to be independent of scale effects.

(a) Sands. The reduction in end-bearing capacity has been related with a reduction of the effective angle in internal friction ϕ' with larger diameter deep foundations. End-bearing capacity q_{bu} from equation 3-5 should be multiplied by a reduction factor (Meyerhof 1983) R_{bs}

$$R_{bs} = \left[\frac{B + 1.64}{2B}\right]^m \leq 1 \qquad (3\text{-}14a)$$

for $B > 1.64$ feet. The exponent $m = 1$ for loose sand, 2 for medium dense, and 3 for dense sand.

(b) Clays. A reduction in end-bearing capacity q_{bu} in clays appears to be related with soil structure and fissures. Equation 3-5 should be multiplied by the reduction factor R_{bc}. For driven piles in stiff fissured clay, $R_{bc} = R_{bs}$ from equation 3-14a where $m = 1$. For drilled shafts

$$R_{bc} = \left[\frac{B + 3.3}{2B + 3.3}\right] \leq 1 \qquad (3\text{-}14b)$$

for B from 0 to 5.75 ft.

(4) Base resistance of piles driven to rock. The ultimate end-bearing resistance may be estimated from the uniaxial compression strength of the rock by (Canadian Geotechnical Society 1985)

$$q_{bu} = 3\,\sigma_c K_{\text{rock}} f_d \qquad (3\text{-}15a)$$

$$K_{\text{rock}} = \frac{3 + \dfrac{s_d}{B_{\text{sock}}}}{10\left[1 + 300\,\dfrac{w_d}{s_d}\right]^{0.5}} \qquad (3\text{-}15b)$$

where

σ_c = uniaxial compressive strength of rock, ksi

f_d = $1 + 0.4D_{sock}/B_{sock}$

w_d = width of discontinuities in rock, inches

s_d = spacing of discontinuities in rock, inches

B_{sock} = socket diameter, inches

D_{sock} = depth of embedment of pile socketed into rock, inches

The rock quality designation (RQD) should be greater than 50 percent, s_d should be greater than 12 inches, w_d should be less than 0.25 inch for unfilled discontinuities or w_d should be less than 1.0 inch for discontinuities filled with soil or rock debris, and B should be greater than 12 inches. Rocks are sufficiently strong that the structural capacity of the piles will govern the design. This method is not applicable to soft, stratified rocks such as shale or limestone. Piles supported on these rocks should be designed from the results of pile load tests.

b. Skin friction resistance. The maximum skin resistance between the soil and the shaft is $Q_{sui} = A_i\, f_{ui}$, equation 3-3.

(1) Cohesive soil. Skin friction resisting applied loads are influenced by the soil shear strength, soil disturbance, and changes in pore pressure and lateral earth pressure. The mean undrained shear strength should be used to estimate skin friction by the alpha and Lambda methods (Barker et al. 1991).

Table 3-4
Q_u by the Nordlund Method

Step	Procedure
	a. End-Bearing Capacity
1	Determine friction angle ϕ' for each soil layer. Assume $\phi = \phi'$.
2	Determine α_f using ϕ for the soil layer in which the tip is embedded and the pile L/B ratio from Figure 3-6a.
3	Determine N_q using ϕ for the soil layer in which the tip is embedded from Figure 3-6b.
4	Determine effective overburden pressure at the pile tip σ'_L and limiting stress q_l according to Figure 3-4.
5	Determine the pile point area, A_b.
6	Determine end-bearing resistance pressure $q_{bu} = \alpha_f N_q \sigma'_L$. Check $q_{bu} \le q_l$. Calculate end-bearing capacity $Q_{bu} = q_{bu} A_b$.
	b. Skin Friction Capacity
7	Compute volume of soil displaced per unit length of pile.
8	Compute coefficient of lateral earth pressure K for ϕ' and ω using Figure 3-7; use linear interpolation.
9	Determine δ/ϕ' for the given pile and volume of displaced soil V from Figure 3-8. Calculate δ for friction angle ϕ'.
10	Determine correction factor C_f from Figure 3-9 for ϕ and the δ/ϕ' ratio.
11	Calculate the average effective overburden pressure σ'_z of each soil layer.
12	Calculate pile perimeter at center of each soil layer C_z.
13	Calculate the skin friction capacity of the pile in each soil layer i from $Q_{sui} = K C_f \sigma'_z \dfrac{\sin(\delta + \omega)}{\cos \omega} C_z \Delta L$ Add Q_{sui} of each soil layer to obtain Q_{su}, $Q_{su} = \sum Q_{sui}$ of each layer.
14	Compute ultimate total capacity, $Q_u = Q_{bu} + Q_{su}$.

(a) Alpha method. The soil-shaft skin friction of a length of a pile element at depth z may be estimated by

$$f_{sui} = \alpha_a \times C_u \tag{3-16}$$

where

α_a = adhesion factor

C_u = undrained shear strength, ksf

Local experience with existing soils and load test results should be used to estimate appropriate α_a. Estimates of α_a may be made from Table 3-5 in the absence of load test data and for preliminary design.

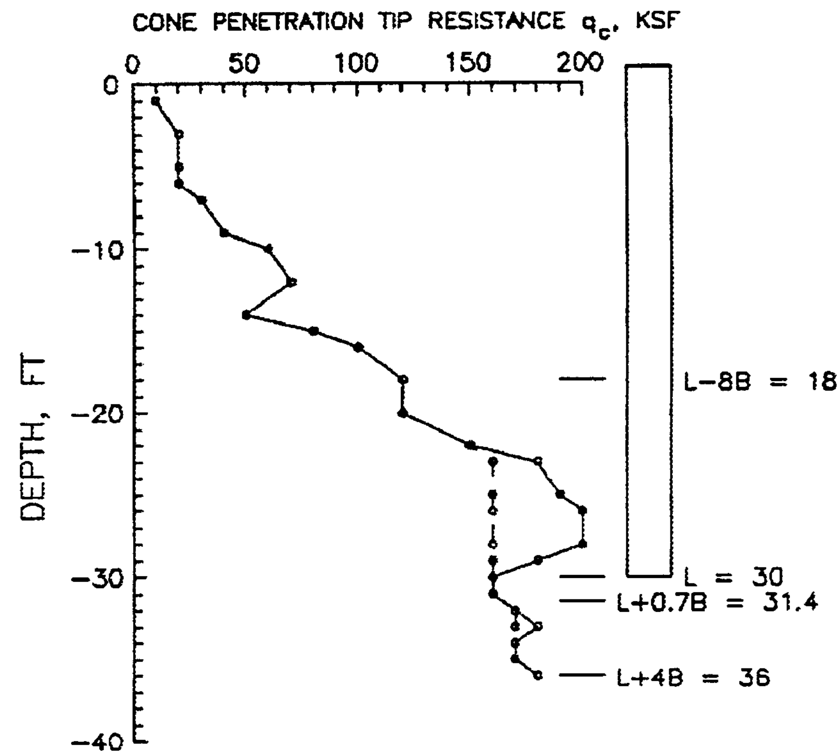
(b) Lambda method. This semiempirical method is based on numerous load test data of driven pipe piles embedded in clay assuming that end-bearing resistance was evaluated from equation 3-6. Skin friction is (Vijayvergiya and Focht 1972)

$$f_{sui} = \lambda \times (\sigma'_m + 2C_{um}) \tag{3-17}$$

where

λ = correlation factor, Figure 3-11

σ'_m = mean effective vertical stress between the ground surface and pile tip, ksf



q_{bu} = $\dfrac{q_{c1} + q_{c2}}{2}$

q_{c1} = Average q_c over a distance L + 0.7B to L + 4B below the pile tip; use the minimum path, which is a vertical line spanning the minimum measured q_c between L + 0.7B and L + 4B, ksf

q_{c2} = Average q_c over a distance of L to L - 8B above the pile tip; use the minimum path as above

B = Pile diameter, ft

Figure 3-10. Estimating pile tip capacity from CPT data (FHWA-TS-78-209)

C_{um} = mean undrained shear strength along pile length, ksf

λ may also be given approximately by

$$\lambda = L^{-0.42} \qquad L \geq 10 \text{ ft} \tag{3-18a}$$

$$\lambda = 0.5 - 0.01L \qquad L < 10 \text{ ft} \tag{3-18b}$$

where L is the pile length, feet, λ may also be estimated as follows (Kraft, Focht, and Amarasinghe 1981)

Normally consolidated:

$$\lambda = 0.296 - 0.032 \ln L \tag{3-18c}$$

Overconsolidated:

$$\lambda = 0.488 - 0.078 \ln L \tag{3-18d}$$

The ratio of the mean undrained shear strength to the effective overburden pressure should be greater than 0.4 for overconsolidated soil.

(c) CPT field estimate. The cone penetration test provides a sleeve friction f_{sl} which can be used to estimate the ultimate skin resistance Q_{su} (Nottingham and Schmertmann 1975)

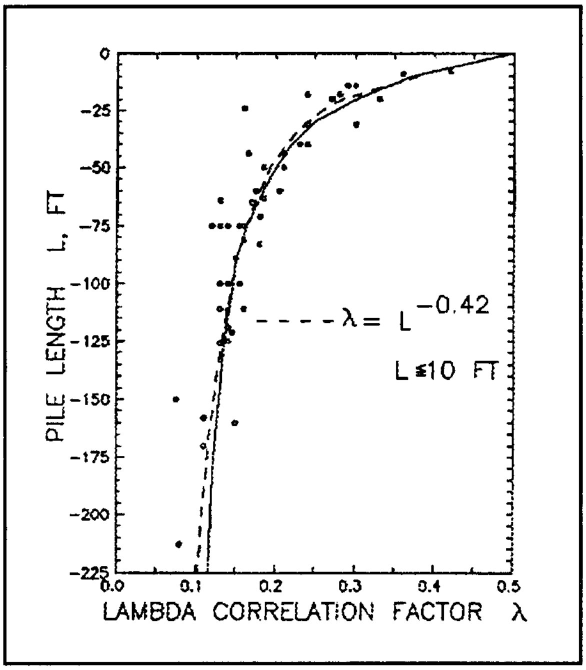


Figure 3-11. Lambda correlation factor for clay (Copyright permission, Offshore Technology Conference, Society of Petroleum Engineers)

$$Q_{su} = k_{sl} \left[\sum_{z_L=0}^{8B} \frac{z_L}{8B} f_{slz} C_z + \sum_{z_L=8B}^{L} f_{slz} C_z \right] \tag{3-19}$$

where

k_{sl} = sleeve friction factor, Figure 3-12

f_{slz} = cone sleeve friction at depth z, ksf

C_z = pile circumference at depth z, feet

B = pile diameter or width, feet

z_L = depth to point considered, feet

L = length of embedded pile, feet

Equation 3-19 corrects for the cone (mechanical or electrical), pile material (steel, concrete, or wood), type of soil through sleeve friction f_{sl} , and corrects for the depth of the pile embedment. f_{sl} for high *OCR* clays is 0.8 times f_{sl} measured by the mechanical cone. The cone penetration test procedure is given in ASTM D 3441.

(2) Cohesionless soil. The soil-shaft friction may be estimated using effective stresses

$$f_{sui} = \beta_f \times \sigma_i' \tag{3-20a}$$

$$\beta_f = K \times \tan \delta_a \tag{3-20b}$$

where

f_{sui} = soil shaft skin friction

β_f = lateral earth pressure and friction angle factor

K = lateral earth pressure coefficient

δ_a = soil-shaft effective friction angle, $\leq \phi'$, degrees

σ_i' = effective vertical stress in soil adjacent to pile element i, ksf

Cohesion c is zero. The σ_i' is limited to the effective overburden pressure calculated at the critical depth D_c of Figure 3-3.

(a) Values of β_f as a function of the effective friction angle ϕ' of the soil prior to installation of the deep foundation are shown in Figure 3-13. Values in Figure 3-13 are lower bound estimates.

(b) The Nordlund method in Table 3-4b provides an alternative method of estimating skin resistance.

Table 3-5
Adhesion Factors for Cohesive Soil

Length/Width Ratio $\frac{L}{B}$	Undrained Shear Strength C_u, ksf	Adhesion Factor α_a
< 20	< 3	$1.2 - 0.3C_u$
	> 3	0.25
> 20	0.0 - 1.5	1.0
	72 - 4.0	$1.25 - 0.24C_u$
	> 4	0.3

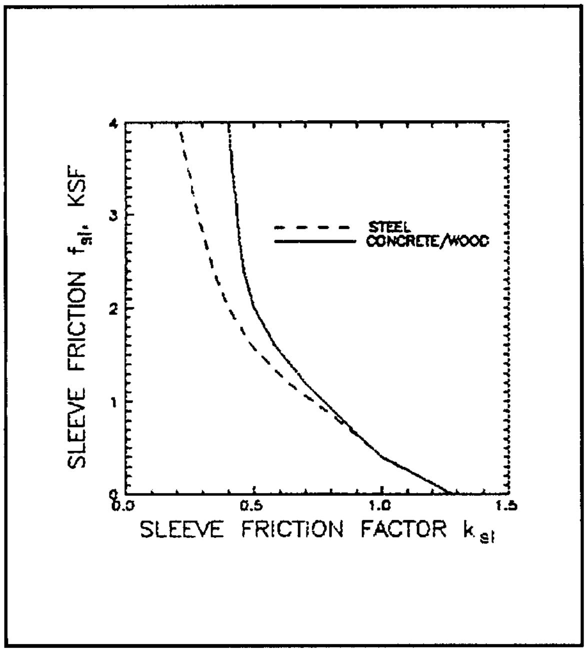


Figure 3-12. Sleeve friction factor for clays (Copyright permission, Florida Department of Transportation)

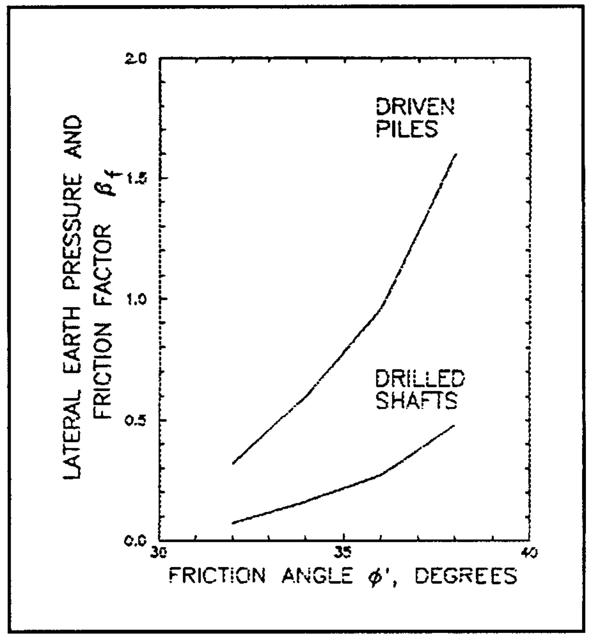


Figure 3-13. Lateral earth pressure and friction angle factor β_f (Copyright permission, American Society of (Civil Engineers)

(3) CPT field estimate. The ultimate skin resistance may be estimated from the cone sleeve friction similar to that for clays from equation 3-19 where the sleeve friction factor k_{sl} is estimated for sands from Figure 3-14 (Nottingham and Schmertmann 1975). The factor k_{sl} for wood piles is 1.25 times the k_{sl} for steel piles.

c. *Computer programs.* Pile capacity can be calculated using computer programs CAXPILE (WES IR-K-84-4), AXILTR (Appendix C), and GRLWEAP (Goble et al. 1988). CAXPILE AND AXILTR solve for axial load-displacement behavior of single piles by load transfer curves. Several base and shaft load transfer curves applicable to different types of soils are available in these programs. Other curves may be input if data are available. Refer to Chapter 6 for further details on wave equation program GRLWEAP.

d. *Load-displacement relationships.* The settlement of a pile group is of more interest than that of a single pile because most piles are placed as groups, Chapter 5. If required, the settlement of single piles can be estimated using methods in paragraph 3-3 for drilled shafts.

e. *Application.* Each pile for a certain project is required to support Q_d = 100 kips. Steel circular, 1.5-foot-diameter, closed-end pipe piles are tentatively selected, and they are to be driven 30 feet through a two-layer soil of clay over fine uniform sand, Figure 3-15. The water level (phreatic surface) is 15 feet below ground surface at the clay-sand interface. The pile will be filled with concrete with density γ_{conc} = 150 pounds per cubic foot. The strength and density of the soils are given in Figure 3-15. The friction angle ϕ of 36 degrees for the lower sand layer given in Figure 3-15 is an average value. ϕ increases from 34 degrees at the top to 38 degrees at the base of the pile to be consistent with the cone penetration data given in Fig. 3-10.

(1) Soil parameters

(a) Mean effective vertical stress. The mean effective vertical stress σ'_s in the sand layer below the surface clay layer may be estimated by

$$\sigma'_s = L_{clay} \times \gamma_c + \frac{L_{sand}}{2} \times \gamma'_s \qquad \text{(3-21a)}$$

where

L_{clay} = thickness of a surface clay layer, feet

γ_c = wet unit weight of surface clay layer above the phreatic surface, kips/cubic foot

L_{sand} = thickness of an underlying sand clay layer, feet

γ'_s = submerged unit weight of underlying sand layer below the phreatic surface, kips/cubic feet

The mean effective vertical stress in the sand layer adjacent to the embedded pile from equation 3-21a is

$$\sigma'_s = L_{clay} \times \gamma_c + \frac{L_{sand}}{2} \times \gamma'_s = 15 \times 0.12 + \frac{15}{2} \times 0.04 = 1.8 + 0.3$$

$$\sigma'_s = 2.1 \text{ ksf}$$

The effective vertical soil stress at the pile tip is

$$\begin{aligned} \sigma'_L &= L_{clay} \times \lambda_c + L_{sand} \times \lambda'_s \\ &= 1.5 \times 0.12 + 15 \times 0.04 \\ &= 1.8 + 0.6 = 2.4 \text{ ksf} \end{aligned} \qquad \text{(3-21b)}$$

Figure 3-3 indicates that the D_c/B ratio is 10 for an average ϕ' = 36 degrees. Therefore, D_c = 10 · 1.5 = 15 feet. The effective stress is limited to σ'_s = 1.8 ksf below 15 feet and the effective stress at the pile tip is σ'_L = 1.8 ksf for the Meyerhof and Nordlund methods. The remaining methods use σ'_L = 2.4 ksf.

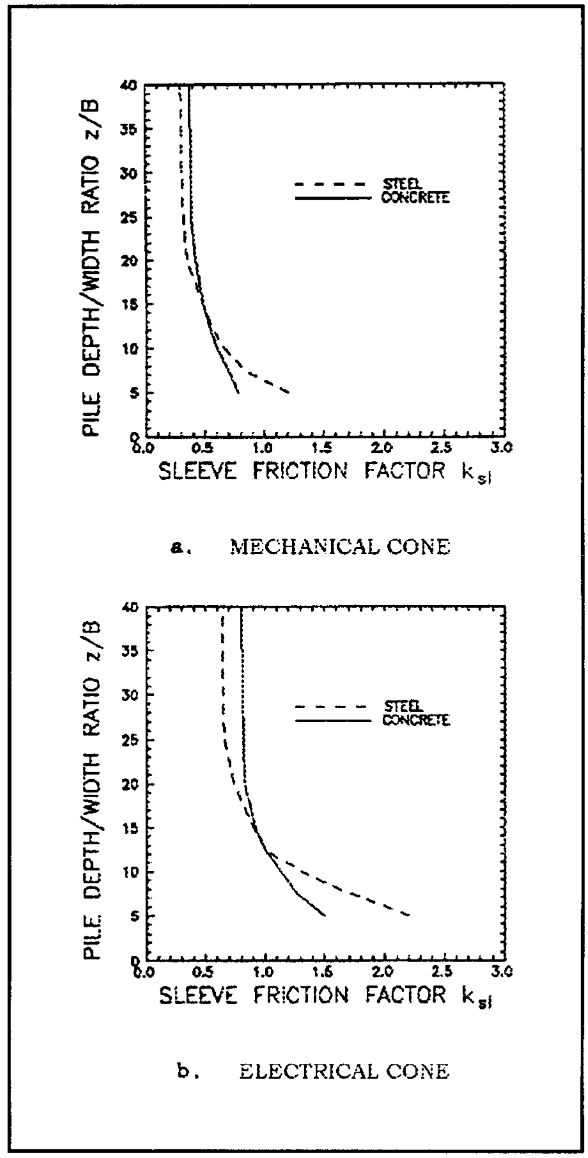


Figure 3-14. Sleeve friction factors for sands (Copyright permission, Florida Department of Transportation)

(b) Cone penetration resistance. Penetration tests using an electrical cone indicate that an average cone tip resistance q_c in the clay is 40 ksf and in the sand it is 160 ksf. The shear modulus $G = E_s / [\, 2\,(1 + \nu_s\,)\,] = 250 / [\, 2\,(1 + 0.3\,)\,] = 96$ ksf or about 100 ksf using an assumed elastic soil modulus E_s = 250 ksf and Poisson's ratio υ_s = 0.3. These E_s and υ_s values are typical of soft to medium stiff clay or loose to medium dense sands. E_s is consistent with that calculated for sands from equation 1-3a. Sleeve friction f_{sl} in the clay is 1.0 ksf and in the sand it is 1.5 ksf.

(c) Coefficient of earth pressure. Coefficient of earth pressure at rest from the Jaky equation is $K_o = 1 - \sin \phi = 1 - \sin 36$ degrees = 0.42.

(2) Vertical load capacity. Solution of the vertical load capacity of a single pile using Table 3-3 is given in Table 3-6.

3. Drilled Shafts

The general procedure for design of a single drilled shaft is given in Table 3-7. The vertical capacity Q_u is given by equation 3-1 where the end bearing Q_{bu} and skin friction Q_{su} capacities are calculated by methods given below. Load tests to confirm the design should be performed where economically feasible. Refer to Chapter 6 for further information on load tests.

a. End-bearing resistance. Ultimate end bearing resistance for single drilled shafts with enlarged bases should be evaluated using equation 3-2. Equation 3-2 may be simplified for shafts without enlarged tips by eliminating N_γ

$$q_{bu} = cN_c\zeta_c + \sigma_L'(N_q - 1)\zeta_q \qquad \text{(3-5a, bis)}$$

or

$$q_{bu} = cN_c\zeta_c + \sigma_L' N_q \zeta_q \qquad \text{(3-5b, bis)}$$

Equations 3-5 also adjust for pile weight W_p assuming $\gamma_p \approx \gamma_L'$.

(1) Cohesive soil. The undrained shear strength of saturated cohesive soil for deep foundations in saturated clay subjected to a rapidly applied load is $c = C_u$ and the friction angle $\phi = 0$. Equations 3-5 simplifies to (FHWA-HI-88-042)

$$q_{bu} = F_r N_c C_u, \quad q_{bu} \le 80 \text{ ksf} \qquad \text{(3-22)}$$

where the shape factor $\zeta_c = 1$ and $N_c = 6\,[\,1 + 0.2\,(L/B_b)\,] \le 9$. The limiting q_{bu} of 80 ksf is the largest value that has so far been measured for clays. The undrained shear strength C_u may be reduced by about one-third in cases where the clay at the base has been softened and could cause local bearing failure due to high strain. F_r should be 1.0, except when B_b exceeds about 6 feet. For $B_b > 6$ feet

$$F_r = \frac{2.5}{aB_b + 2.5b}, \; F_r \le 1.0 \qquad \text{(3-23)}$$

where

a = $0.0852 + 0.0252\,(L/B_b)$, $a \le 0.18$

b = $0.45C_u^{0.5}$, $0.5 \le b \le 1.5$, where C_u is in units of ksf

Equation 3-22 limits q_{bu} to bearing pressures for a base settlement of 2.5 inches. C_u should be the average shear strength within $2B_b$ beneath the tip of the shaft.

(2) Cohesionless soil. Vesic method and the general shear methods discussed for driven piles in paragraph 2a, Chapter 3, and the Vesic Alternate Method are recommended for solution of ultimate end bearing capacity using equation 3-5 (Vesic 1977).

(a) Vesic Alternate Method. This method assumes a local shear failure and provides a lower bound estimate of bearing capacity

$$N_q = e^{\pi \tan\phi'}\left[\tan^2\left(45 + \frac{\phi'}{2}\right)\right] \qquad \text{(3-24)}$$

The shape factor may be estimated by equation 3-9. A local shear failure occurs at the base of deep foundations only in poor soils such as loose silty sands or weak clays or in soils subject to disturbance due to the construction of drilled shafts. The Vesic Alternate Method may be more appropriate for deep foundations constructed under difficult conditions, for drilled shafts placed in soil subject to disturbance, and when a bentonite-water slurry is used to keep the hole open during drilled shaft construction.

(b) SPT field estimate. The end bearing resistance q_{bu} in units of ksf may be estimated from standard penetration data (Reese and Wright 1977) by

$$q_{bu} = \frac{4}{3}N_{SPT} \qquad N_{SPT} \le 60 \qquad \text{(3-25a)}$$

$$q_{bu} = 80 \text{ ksf} \qquad N_{SPT} > 60 \qquad \text{(3-25b)}$$

where N_{SPT} is the uncorrected standard penetration resistance in blows per foot.

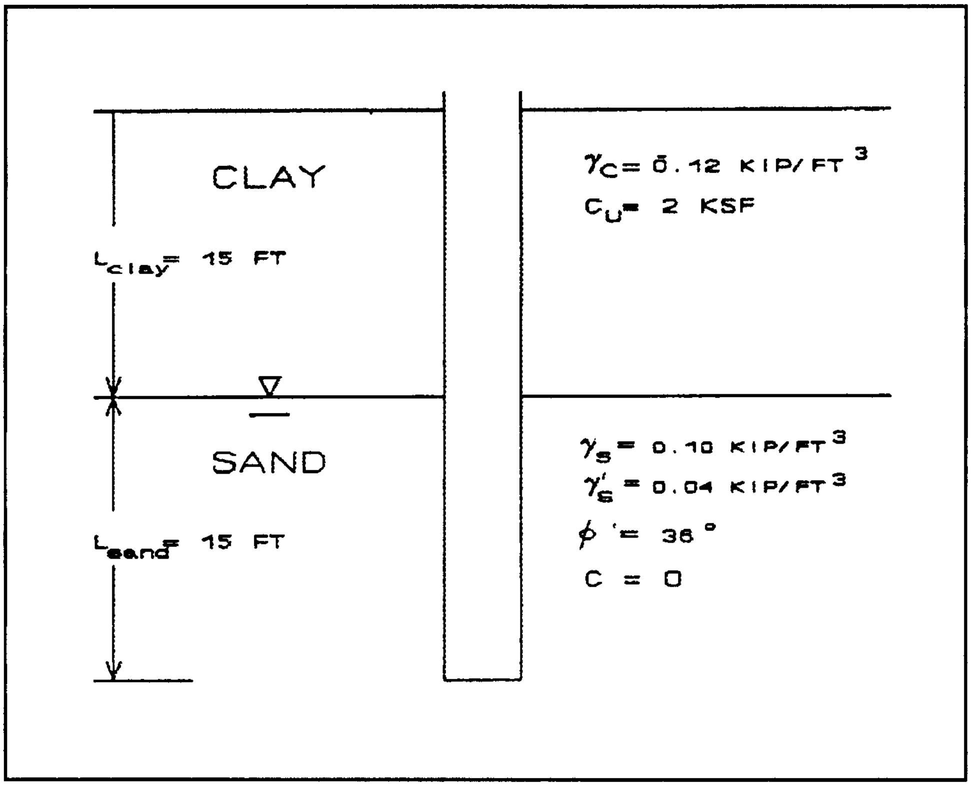


Figure 3-15. Driven steel pipe pile

b. Skin friction resistance. The maximum skin resistance that may be mobilized between the soil and shaft perimeter is $Q_{sui} = A_{si} f_{sui}$, equation 3-3, where A_{si} is the perimeter area of element *i*. Several methods of estimating skin friction f_{sui}, based on past experience and the results of load tests, are described below.

(1) Cohesive soil. Skin friction between the soil and shaft is estimated by using the average undrained shear strength and an empirical adhesion factor α_a.

(a) Alpha method. The soil-shaft skin friction f_{sui} of a length of shaft (or pile) element may be estimated by

$$f_{sui} = \alpha_a C_u \qquad \text{(3-16 bis)}$$

where

α_a = adhesion factor

C_u = undrained shear strength, ksf

Local experience with existing soils and load test results should be used to estimate appropriate α_a. Estimates of α_a may be made from Table 3-8 in the absence of load test data and for preliminary design.

(b) Adhesion factor. The adhesion factor may also be related to the plasticity index PI for drilled shafts constructed dry. For cohesive soil, the following expression (Stewart and Kulhawy 1981) may be used

Overconsolidated:

$$\alpha_a = 0.7 - 0.01 \times PI \qquad \text{(3-26a)}$$

Slightly over-consolidated ($OCR \leq 2$):

$$\alpha_a = 0.9 - 0.01 \times PI \qquad \text{(3-26b)}$$

Normally consolidated:

$$\alpha_a = 0.9 - 0.004 \times PI \qquad \text{(3-26c)}$$

where $15 < PI < 80$. Drilled shafts constructed using the bentonite-water slurry should use α_a of about 1/ 2 to 2/3 of those given by equation 3-26.

Table 3-6
Calculations of Vertical Loads in a Single Pile

Step	Procedure	Description
1	Select suitable dimensions	Select a trial 1.5-ft-diameter by 30-ft-long steel closed-end pipe pile. Pile circumference C_z = 4.71 ft and area of base A_b = 1.77 ft^2
2	Evaluate end bearing capacity Q_{bu}	$Q_{bu} = q_{bu} A_b$ from equation 3-1b; q_{bu} is found using several methods in the sand:

(a) Nordlund method: Use Table 3-4a

α_f = 0.72 for ϕ' = 38 deg, Figure 3-6a
N_q = 105 ϕ' = 38 deg, Figure 3-6b
σ'_L = 1.8 ksf
$q_{bu} = \alpha_f N_q \sigma'_L$ = 0.72 × 105 × 1.8 = 136.1 ksf
q_l = 150 ksf from Figure 3-4
Therefore, q_{bu} = 136.1 ksf $\leq q_l$

(b) Vesic method: Reduced rigidity index from equation 3-8c

$$I_{rr} = \frac{I_r}{1 + \epsilon_v \times I_r} = \frac{53.3}{1 + 0.006 \times 53.3} = 40.4$$

$$\epsilon_v = \frac{1 - 2\nu_s}{2(1 - \nu_s)} \times \frac{\sigma_L'}{G_s} = \frac{1 - 2 \times 0.3}{2(1 - 0.3)} \times \frac{2.4}{100}$$
$$= 0.006$$

$$I_r = \frac{G_s}{\sigma_L' \times \tan\phi'} = \frac{100}{2.4 \times \tan 38} = 53.3$$

From equation 3-8b

$$= \frac{3}{3 - \sin\phi}\, e^{\frac{(90 - \phi)}{180}\pi \tan\phi} \tan^2\left[45 + \frac{\phi}{2}\right] I_{rr}^{\frac{4\sin\phi}{3(1 + \sin\phi)}}$$

$$= \frac{3}{3 - \sin 38}\, e^{\frac{(90 - 38)}{180}\pi \tan 38} \tan^2\left[45 + \frac{38}{2}\right] I_{rr}^{\frac{4\sin 3}{3(1 + \mathrm{si}}}$$
$$= 1.258 \times 2.032 \times 4.023 \times 6.549$$
$$= 70.4$$

Shape factor equation 3-9a

$$\zeta_q = \frac{1 + 2K_o}{3} = \frac{1 + 2 \times 0.42}{3} = 0.61$$

where K_o was found from equation 3-9c

(Sheet 1 of 5)

Table 3-6 (Continued)

Step	Procedure	Description
		From equation 3-5b, $$q_{bu} = \sigma_L' \times N_q \times \zeta_q = 2.4 \times 70.4 \times 0.61 = 103 \text{ ksf}$$ (c) <u>General Shear (Bowles method (Bowles 1968)):</u> From equation 3-10 $$N_q = \frac{e^{\frac{270-\phi}{180}\pi \tan\phi}}{2\cos^2\left[45 + \frac{\phi}{2}\right]} = \frac{e^{\frac{270-38}{180}\pi \tan 38}}{2\cos^2\left[45 + \frac{38}{2}\right]} = \frac{e^{3.164}}{2 \times 0.192} = \frac{23.65}{0.384} = 61.5$$ The shape factor ζ_q = 1.00 when using equation 3-10; from equation 3-5b, $$q_{bu} = \sigma_L' \times N_q \times \zeta_q = 2.4 \times 61.5 \times 1.00 = 147.7 \text{ ksf}$$ (d) <u>CPT Meyerhof method:</u> From equation 3-12, $$q_{bu} = \frac{q_c}{10} \times \frac{L_{\text{sand}}}{B} < q_\ell = \frac{160}{10} \times \frac{15}{1.5} = 160 \text{ ksf}$$ q_ℓ = 150 ksf from Figure 3-4; therefore, q_{bu} = 150 ksf (e) <u>CPT FHWA & Schmertmann</u>: Data in Figure 3-10 are used to give q_{bu} = 163 ksf as illustrated in paragraph 2a, Chapter 3 (f) Comparison:

(Sheet 2 of 5)

Table 3-6 (Continued)

Step	Procedure	Description

Method	q_{bu}, ksf
Friction Angle ϕ = 38 deg	
Nordlund	136
Vesic	103
General Shear	148
Cone Penetration Test	
CPT Meyerhof	150
CPT FHWA & Schmertmann	163

q_{bu} varies from 103 to 148 ksf for ϕ' = 38 deg and 150 to 163 ksf for the cone data. Select lower bound $q_{bu,1}$ = 103 ksf and upper bound $q_{bu,u}$ = 163 ksf. Scale effects of equation 3-14 are not significant because B < 1.64 ft

$Q_{bu,1} = q_{bu,1} \times A_b$ = (103) (1.77) = 182 kips
$Q_{bu,u} = q_{bu,u} \times A_b$ = (163) (1.77) = 289 kips

Step	Procedure	Description
3	Evaluate skin resistance Q_{su}	<u>ToTop Layer:</u> Cohesive soil; average skin friction using the alpha method, equation 3-16 is

$$f_{su} = \alpha_a \times C_u = 0.6 \times 2.0 = 1.2 \text{ ksf}$$

where $\alpha_a = 1.2 - 0.3C_u = 0.6$ for L/B = 20 from Table 3-5

Q_{su} from equation 3-3 is

$$Q_{su} = f_{su} \times C_z \times L_{\text{clay}} = (1.2) \times (4.71) \times (15)$$
$$= 84.8 \text{ kips}$$

Average skin friction using the lambda method and equation 3-17 is

$$f_{su} = \lambda\,(\sigma_m + 2C_{um}) = 0.32\,(0.9 + 2.2)$$
$$= 1.57 \text{ ksf}$$

where $\lambda = L_{clay}^{-0.42} = 15^{-0.42} = 0.32$ from equation 3-18a; σ'_m is found from

$$\sigma'_m = \frac{L_{\text{clay}}}{2} \times \gamma'_{\text{clay}} = \frac{15}{2} \times 0.12 = 0.9 \text{ ksf}$$

Q_{su} from equation 3-3 is

$$Q_{su} = f_{su} \times C_z \times L_{\text{clay}} = 1.57 \times 4.71 \times 15$$
$$= 110.9 \text{ kips}$$

(Sheet 3 of 5)

Table 3-6 (Continued)

Step	Procedure	Description
		Q_{su} using the CPT field estimate method is found from equation 3-19 where k_{sl} = 0.75 for f_{sl} = 1.0 ksf, Figure 3-12

$$Q_{su} = 0.75\ [12 \times 1.0 \times 1.0 \times 4.71 + 3 \times 1.0 \times 4.71]$$
$$= 0.75\ [56.5 + 14.1] = 53.0 \text{ kips}$$
$$Q_{su} = k_{sl}\,[8B \times \frac{8B}{8B} \times f_{sl} C_z + \sum_{8B}^{L_{clay}} f_{sl} C_z]$$

Lower bound $Q_{su,1}$ = 53 kips and upper bound $Q_{su,u}$ = 111 kips

<u>Bottom Layer:</u> Cohesionless soil; average skin friction from equation 3-20a using $\sigma'_s \le$ limiting stress 1.8 ksf is

$$f_{su} = \beta_f \times \sigma_s' = 0.96 \times 1.8 = 1.7 \text{ ksf}$$

where β_f is from Figure 3-13 for average ϕ' = 36 deg

Q_{su} from equation 3-3 is

$$Q_{su} = f_{su} \times C_z \times L_{clay} = 1.7 \times 4.71 \times 15$$
$$= 120 \text{ kips}$$

An alternative estimate from the Nordlund method, Table 3-4b, is

$V = \pi \times (1.5^2/2) \times 1 = 1.77\ ft^3/ft$
K = 2.1 from Figure 3-7 for ω = 0 deg
δ/ϕ = 0.78 for V = 1.77 and pile type 1 from Figure 3-8
$\delta = 0.78 \cdot 36$ = 28 deg
C_f = 0.91 for δ/ϕ = 0.78, ϕ = 36 deg from Figure 3-9
$C_z = \pi \times B_s = \pi \times 1.5$ = 4.71 ft

$$Q_{su} = KC_f \sigma_s' \sin\delta \times C_z L_{sand}$$
$$= 2.1 \times 0.91 \times 1.8 \times \sin 28 \times 4.71 \times 15$$
$$= 114 \text{ kips}$$

Q_{su} using the CPT field estimate method is found from equation 3-19 where k_{sl} varies from 1.3 to 0.7, Figure 3-14b, for $z/B = L_{clay}/B$ = 15/1.5 = 10 to $z/B = (L_{clay} + L_{sand})/B$ = 30/1.5 = 20

$$Q_{su} = k_{sl}\ [\sum_{L_{clay}}^{L_{clay}+L_{sand}} f_{sl} C_z]$$

(Sheet 4 of 5)

Table 3-6 (Concluded)

Step	Procedure	Description
		$Q_{su} = (1.3 + 0.7)/2[15 \times 1.5 \times 4.71]$ $= 106$ kips Lower bound $Q_{su,1}$ in sand is 106 kips and upper bound $Q_{su,u}$ = 120 kips Total Q_{su} in both clay and sand is: Lower bound: $Q_{su,1}$ = 53 + 106 = 159 kips Upper bound: $Q_{su,u}$ = 111 + 120 = 231 kips
4	Compute ultimate capacity Q_u	The total bearing capacity from equation 3-1a is $Q_u = Q_{bu} + Q_{su}$ Lower bound: $Q_{u,1} = Q_{bu,1} + Q_{su,1}$ $= 182 + 159 = 341$ kips Upper bound: $Q_{u,u} = Q_{bu,u} + Q_{su,u}$ $= 289 + 231 = 520$ kips Q_u ranges from a low of 341 to a high of 520 kips for a difference of 179 kips or 42 percent of the mean (341 + 520) /2 = 430 kips. This difference is reasonable because of assumptions used by various methods
5	Check $Q_d \le Q_a$	Q_d = 100 kips; for *FS* = 3 and using $Q_{u,1}$ lower bound $Q_a = \frac{Q_u}{FS} = \frac{341}{3} = 114$ kips Therefore, Q_d is less than the lower bound estimate. A load test should be performed to failure to assure that the pile has adequate capacity. The *FS* may also be reduced to 2.0 and permit the design load Q_d to be increased leading to fewer piles and a more economical foundation when load tests are performed as a part of the design

(Sheet 5 of 5)

(2) Cohesionless soil. Skin friction is estimated using effective stresses, the soil friction angle, and empirical correlations.

(a) The soil-shaft skin friction of a length of pile element is estimated by

$$f_{sui} = \beta_f \sigma_i' \tag{3-20a, bis}$$

$$\beta_f = K \tan \delta_a \tag{3-20b, bis}$$

where

β_f = lateral earth pressure and friction angle factor

K = lateral earth pressure coefficient

Table 3-7
Design of a Drilled Shaft

Step	Procedure	Description
1	Select shaft length	Length depends on location of a bearing stratum of sufficient strength and load bearing requirements for the foundation.
2	Evaluate ultimate base resistance q_{bu}	Use equation 3-22 to compute end bearing in clay (total stress analysis $\phi = 0$); $N_c = 9$ or 7 with hammer grab or bucket auger. Use equations 3-8, 3-9, and 3-10 with equations 3-5 for sands setting cohesion c to zero.
3	Evaluate maximum mobilized skin friction f_{sui}	f_{sui} is estimated from equation 3-16 and adhesion factors from equations 3-26 and Table 3-8 for clays. Q_{su} is estimated from equation 3-19 and Figures 3-12 or 3-14, then dividing by $C_s\ \Delta L$ where C_s is pile circumference and ΔL is length in sand or clay.
4	Evaluate Q_{bu} and Q_{su} for several shaft and base diameters	Select several shaft and base diameters; $Q_{bu} = q_{bu}\,A_b$, equation 3-1b; Q_{sui} is found from equation 3-3 and adding increments of Q_{sui} over shaft length L less top and bottom 5 ft or from Table 3-8.
5	Check that design load $Q_d \le Q_a$	Q_a is evaluated from equation 3-4 using FS in Table 3-2.
6	Evaluate shaft resistance to other loads	If pullout, uplift thrust, or downdrag is significant, use program AXILTR, Appendix C.
7	Evaluate maximum settlement from design load Q_d	Estimate settlement for design load Q_d using equations 3-36 to 3-38, load transfer functions, or program CAXPILE or AXILTR.
8	Check computed $\le$ specified settlement or heave	Adjust design load or shaft dimensions.

δ_a = soil-shaft effective friction angle, $\le \phi'$, degree

σ'_i = effective vertical stress in soil at shaft element i, ksf

The cohesion c is taken as zero.

(b) Figure 3-13 indicates values of β_f as a function of the effective friction angle ϕ' of the soil prior to installation of the deep foundation. σ'_i is limited to the effective overburden pressure calculated at the critical depth D_c in Figure 3-3.

(c) SPT field estimate. The skin friction f_s in units of ksf may be estimated for drilled shafts in sand (Reese and Wright 1977) by

$$f_s = \frac{N_{SPT}}{17} \qquad \text{for } N_{SPT} \le 53 \tag{3-27a}$$

$$f_s = \frac{N_{SPT} - 53}{225} \qquad \text{for } 53 < N_{SPT} \le 100 \tag{3-27b}$$

c. Drilled shafts socketed in rock. This calculation of pile capacity of drilled shafts socketed in rock assumes that the load is carried either entirely by skin resistance or by end-bearing resistance depending on the value of the estimated settlement of the shaft in the socket (FHWA-HI-88-042). If the settlement is greater than 0.4 inch, loads are assumed to be carried by base resistance. Loads are carried by skin friction if settlement is less than 0.4 inch. This assumption is conservative because no allowance is provided for loads carried by a combination of both skin and end-bearing resistances.

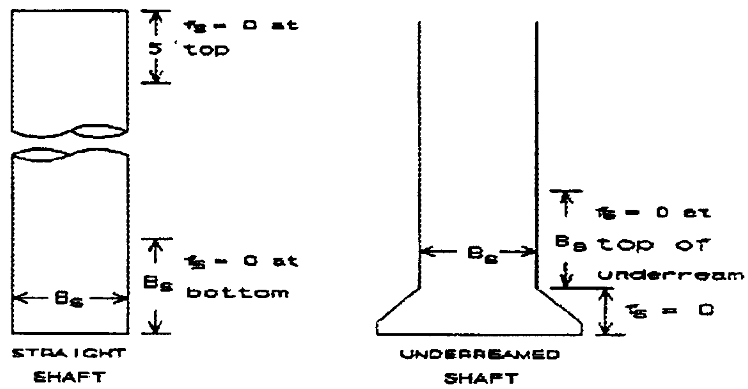
(1) Calculation of socket settlement. Settlement of the portion of the drilled shaft socketed in the rock is

$$\rho_{sock} = \rho_{e,\,sock} + \rho_{b,\,sock} \tag{3-28a}$$

Table 3-8
Adhesion Factors for Drilled Shafts in Cohesive Soil

Shaft Depth, ft	Adhesion Factor α_a
0 – 5	0.0
diameter of shaft from bottom of straight or from top of underream	0.0
All Other Points	0.55

Note: skin friction f_{si} should be limited to 5.5 ksf



$$\rho_{e,\,sock} = \frac{Q_{sock} D_{Sock}}{A_{sock} E_p} \qquad (3\text{-}28b)$$

$$\rho_{b,\,sock} = \frac{Q_{sock} I_{sock}}{B_{sock}} E_{mass} \qquad (3\text{-}28c)$$

where

ρ_{sock} = settlement in socket, inches

$\rho_{e,\,sock}$ = elastic shortening of drilled shaft in socket, mm (inches)

$\rho_{b,\,sock}$ = settlement of base of drilled shaft in socket, mm (inches)

Q_{sock} = load at top of socket, kips

D_{sock} = depth of embedment in socket, inches

A_{sock} = cross section area of socket, inches^2

E_p = Young's modulus of concrete in socket, ksi

B_{sock} = socket diameter, inches

I_{sock} = settlement influence factor, Figure 3-16

E_{mass} = Young's modulus of the mass rock, ksi

Elastic shortening of the shaft not in the socket should also be calculated to determine the total elastic settlement

$$\rho_e = \rho_{sock} + \frac{Q + Q_{sock}}{2} \, \frac{(L - D_{sock})}{A E_p} \qquad (3\text{-}28d)$$

where

Q = load at shaft top, kips

L = embedded shaft length, inches

A = cross section area of shaft, inches^2

Further information for the derivation of Figures 3-16, 3-17, and 3-18 is available from FHWA-HI-88-042, "Drilled Shafts: Construction Procedures and Design Methods." Young's modulus of the mass rock is estimated from the Young's modulus of the intact (core) rock by

$$E_{mass} = K_e E_{core} \tag{3-29}$$

where

K_e = modulus reduction ratio, E_{mass}/E_{core}, Figure 3-17

E_{core} = Young's modulus of the intact rock, ksi

E_{core} is given as a function of the uniaxial compressive strength σ_c in Figure 3-18.

(2) Skin resistance. The capacity of the drilled shaft in the rock socket is determined by skin resistance if $\rho_{sock} < 0.4$ inch. Ultimate skin resistance Q_{su} is (Barker et al. 1991)

$$Q_{su} = 0.15\sigma_c C_z D_{sock} \qquad \sigma_c \le 0.28 \text{ ksi} \tag{3-30a}$$

$$Q_{su} = 2.5\sqrt{\sigma_c}\, C_z D_{sock} \qquad \sigma_c > 0.28 \text{ ksi} \tag{3-30b}$$

where

Q_{su} = ultimate skin resistance of drilled shaft in socket, kips

σ_c = uniaxial compressive strength of the rock (or concrete, whichever is less), ksi

C_z = circumference of socket, inches

D_{sock} = depth of embedment of socket, inches

(3) Base resistance. The capacity of the drilled shaft in the rock socket is determined by base resistance if $\rho_{sock} >$ 0.4 inch.

(a) Base resistance is computed the same as that for driven piles on rock by equation 3-15 in paragraph 2a, Chapter 3.

(b) The base resistance q_{bu} in units of KN/M2 (ksf) of drilled shafts socketed in rock may also be estimated from pressuremeter data (Canadian Geotechnical Society 1985) by

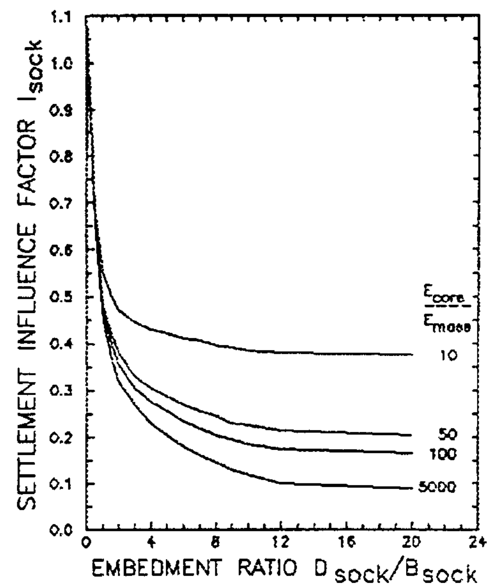


Figure 3-16. Settlement influence factor, I_{sock}

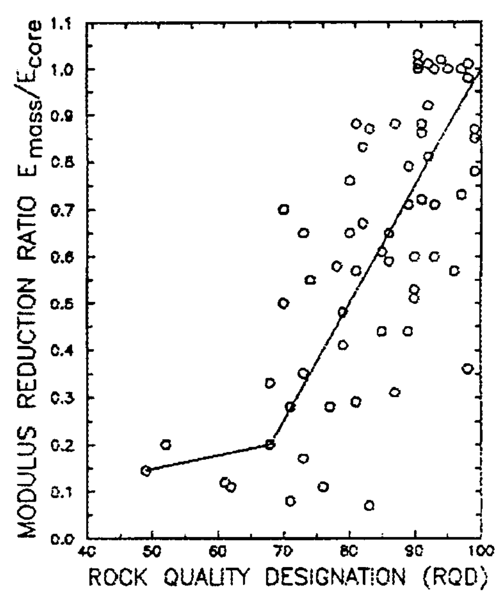


Figure 3-17. Modulus reduction ratio E_{mass} / E_{core}

$$q_{bu} = K_b (P_l - P_o) + \sigma_v \tag{3-31}$$

where

K_b = pressuremeter coefficient, dimensionless, Table 3-9

P_l = pressuremeter limit pressure, ksf

P_o = pressuremeter at rest, horizontal pressure measured at the base elevation, ksf

σ_v = vertical pressure, kfs

Table 3-9
Dimensionless Pressuremeter Coefficient (from Canadian Geotechnical Society 1985, BiTech Publishers Ltd.)

D_{sock} / B_{sock}	k_b
0	0.8
1	2.8
2	3.6
3	4.2
5	4.9
7	5.2

(4) Limitations for analysis of the socket capacity.

(a) The strength of the rock will not deteriorate during construction from values measured during the site investigation.

(b) The drilling fluid will not form a lubricated film on the sides of the excavation.

(c) The bottom of the rock socket is properly cleaned out. This limitation is important if pile capacity is based on the end-bearing resistance. Depth of the rock socket is typically one to three times the diameter of the socket.

(d) Shaft load tests are required if the RQD is less than 50 percent.

d. Vertical capacity to resist other loads. Deep foundations may be subject to other vertical loads such as uplift and downdrag forces. Uplift forces are caused by pullout loads from structures or heave of expansive soils surrounding the shaft tending to drag the shaft up. Downdrag forces are caused by settlement of soil surrounding the shaft that exceeds the downward displacement of the shaft and increases the downward load on the shaft. A common cause of settlement is a lowering of the water table. These forces influence the skin friction that is developed between the soil and the shaft perimeter and influence bearing capacity.

(1) Method. Analysis of bearing capacity with respect to these vertical forces requires an estimate of the relative movement between the soil and the shaft perimeter and the location of neutral point *n*, the position along the shaft length where there is no relative movement between the soil and the shaft. In addition, tension or compression stresses in the shaft or pile caused by uplift or downdrag shall be considered to properly design the shaft. These shaft movements are time-dependent and complicated by soil movement. Background theory for analysis of pullout, uplift, and downdrag forces of single circular drilled shafts and a method for computer analysis of these forces are provided.

(2) Pullout. Deep foundations are frequently used as anchors to resist pullout forces. Pullout forces are caused by overturning moments such as from wind loads on tall structures, utility poles, or communication towers.

(a) Force distribution. Deep foundations may resist pullout forces by shaft skin resistance and resistance mobilized at the tip contributed by enlarged bases illustrated in Figure 3-19. The shaft resistance is defined in terms of negative skin friction f_n to indicate that the shaft is moving up relative to the soil. This is in contrast to compressive loads that are resisted by positive skin friction where the shaft moves down relative to the soil, Figure 3-2. The shaft develops a tensile stress from pullout forces. Bearing capacity resisting pullout may be estimated by

$$P_u = Q_{bu} + P_{nu} \tag{3-32a}$$

$$P_u = q_{bu}A_{bp} + \sum_{i=1}^{n} P_{nui} \tag{3-32b}$$

$$P_{ni} = \sum_{i=1}^{n} P_{nui}C_z \Delta L \tag{3-32c}$$

where

P_u = ultimate pullout resistance , kips

Q_{bu} = ultimate end-bearing force available to resist pullout force *P*, kips

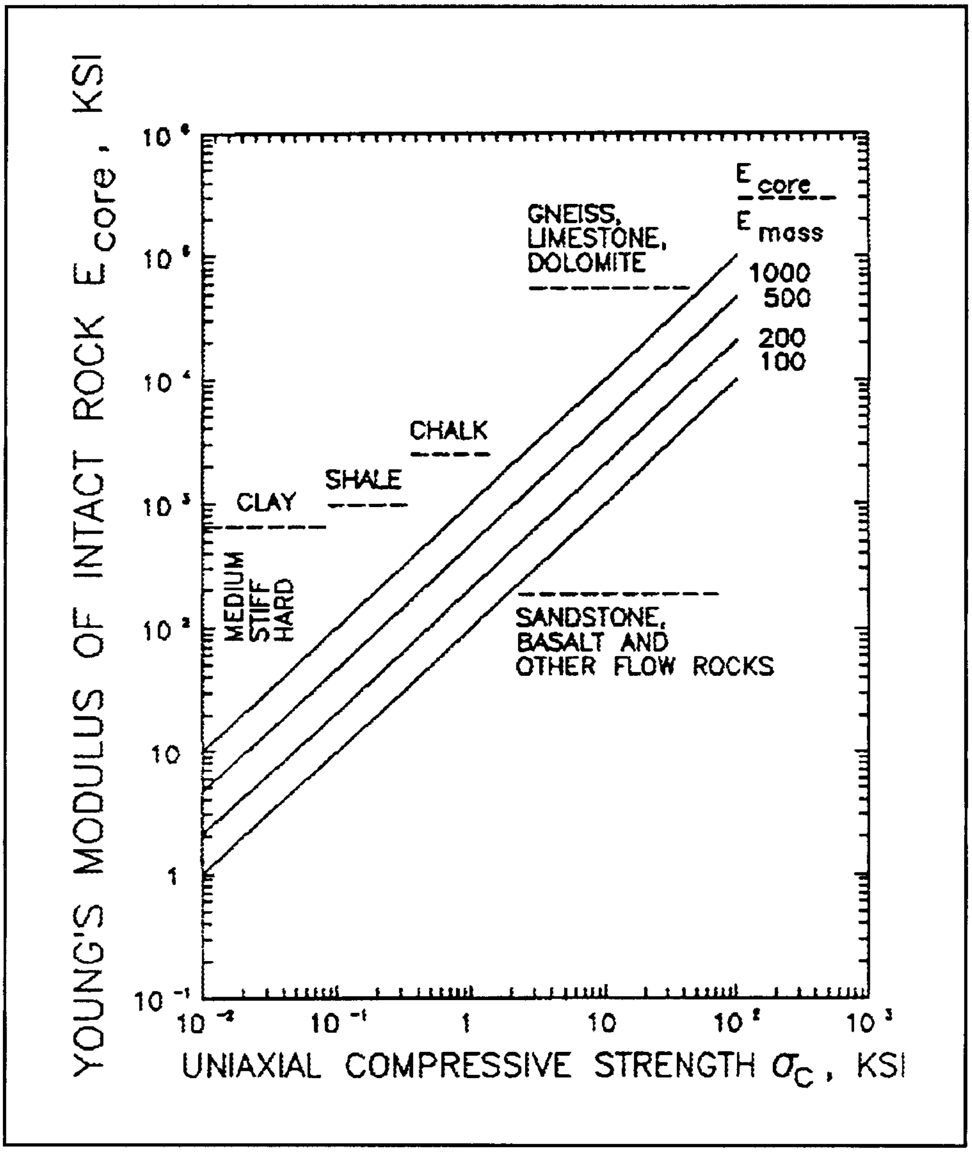


Figure 3-18. Elastic modulus of intact rock

P_{nu} = ultimate skin resistance available to resist pullout force P, kips

q_{bu} = ultimate end-bearing resistance available to resist pullout force P, kips

A_{bp} = area of base resisting pullout force P, ft^2

P_{nui} = pullout skin resistance for pile element i, kips

f_{ni} = negative skin friction resisting pullout force P at element i, ksf

C_z = circumference of shaft, feet

ΔL = length of pile element i, feet

(b) P_n in Figure 3-19 is the skin resistance force that is resisting pullout force P.

(3) Uplift. Deep foundations constructed in expansive soil are subject to uplift forces caused by swelling of expansive soil adjacent to the shaft. These uplift forces cause a friction on the upper length of the shaft perimeter tending to move the shaft up. The portion of the shaft perimeter subject to uplift thrust is in the soil subject to heave. This soil is often within the top 7 to 20 feet of the soil profile

referred to as the depth of the active zone for heave Z_a. The shaft located within Z_a is sometimes constructed in such a manner that isolates the shaft perimeter from the expansive soil to reduce uplift thrust.

(a) Stiffened and ribbed mats as well as drilled shafts are frequently used to support structures in expansive soil areas. Uplift forces may be controlled by minimizing the shaft diameter consistent with that required for downloads and to counter the uplift thrust, by extending the shaft length into nonswelling soil to depths of twice the depth of the active zone for heave. Such force can be reduced by the construction of widely spaced shafts to reduce differential movement, and by making shafts vertically plumb (maximum variation of 1 inch in 6 feet) and smooth to reduce adhesion between the swelling soil and the shaft.

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(b) End-bearing resistance. The q_{bu} of enlarged bases may be estimated by equation 3-5b. For sands, cohesion c is set to zero and N_q is calculated by the Nordlund (1963), Vesic (1977), general shear, and Vesic Alternate Methods (1977). For clays, the friction angle is set to zero and N_c varies from zero at the ground surface to a maximum of 9 at a depth of $2.5B_b$ below the ground surface where B_b is the diameter of the base of the shaft (Vesic 1971). The undrained shear strength C_u is the average strength from the base to a distance $2B_b$ above the base. Base area A_b, resisting pullout to be used in equation 3-1b for underreamed drilled shafts, is

$$A_{bp} = \frac{\pi}{4} \times (B_b^2 - B_s^2) \qquad (3\text{-}33)$$

where

B_b = diameter of base, feet

B_s = diameter of shaft, feet

The soil above the underream is assumed to shear as a cylinder of diameter B_b.

(c) Skin resistance. The shaft diameter may be slightly reduced from pullout forces by a Poisson effect that reduces lateral earth pressure on the shaft perimeter. Thus, skin resistance may be less than that developed for shafts subject to compression loads because horizontal stress is slightly reduced (Stewart and Kulhawy 1980).

(d) Force distribution. During uplift, the shaft moves down relative to the soil above neutral point n, figure 3-20, and moves up relative to the soil below point n. The negative skin friction f_n below point n and enlarged bases of drilled shafts resist the uplift thrust of expansive soil. The positive skin friction f_s above point n contributes to uplift thrust from heaving soil and puts the shaft in tension. End-bearing and skin friction capacity resisting uplift thrust may be estimated by equations 3-32.

(e) End bearing. End-bearing resistance may be estimated similar to that for pullout forces. Bearing capacity factor for pullout in clays N_{cp} should be assumed to vary from 0 at the depth of the active zone of heaving soil to 9 at a depth $2.5B_b$ below the depth of the active zone of heave. The depth of heaving soil may be at the bottom of the expansive soil layer or it may be estimated by guidelines provided in TM 5-818-7.

(f) Skin friction. Skin friction from the top of the shaft to the neutral point n contributes to uplift thrust, while skin friction from point n to the base contributes to skin friction that resists the uplift thrust. The magnitude of skin friction f_s above point n that contributes to uplift thrust will be as much or greater than that estimated for compression loads. Skin friction f_n that resists uplift thrust should be estimated similar to that for pullout loads because uplift thrust places the shaft in tension tending to pull the shaft out of the ground and slightly reduces lateral pressures below point n.

(4) Downdrag. Deep foundations constructed through compressible soils and fills can be subject to an additional downdrag force. This downdrag force is caused by the soil surrounding the drilled shaft or pile settling downward more than the deep foundation. The deep foundation is dragged downward as the soil compresses. The downward load applied to the shaft is significantly increased and can even cause a structural failure of the shaft as well as excessive settlement of the foundation. Settlement of the loose soil after installation of the deep foundation can be caused by the weight of overlying fill, compaction of the fill, and lowering of the groundwater level. The effects of downdrag can be reduced by isolating the shaft from the soil using a bituminous coating or by allowing the consolidating soil to settle before construction. Downdrag loads can be considered in the design by adding them to column loads.

(a) Force distribution. The shaft moves up relative to the soil above point n, Figure 3-21, and moves down relative to the soil below point n. The positive skin friction f_s below point n and end bearing capacity resists the downward loads applied to the shaft by the settling soil and the structural loads. Negative skin friction f_n above the neutral point contributes to the downdrag load and increases the compressive stress in the shaft.

(b) End bearing. End-bearing capacity may be estimated similar to methods for compressive loads given by equation 3-5.

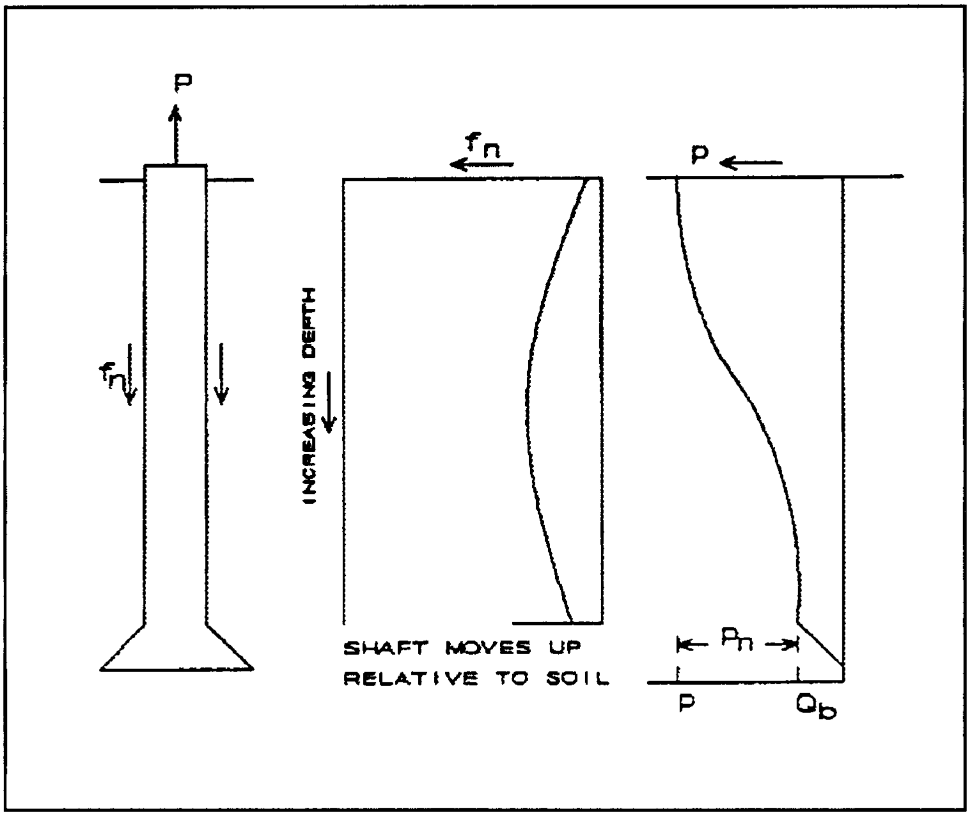


Figure 3-19. Pullout force in underreamed shaft

(c) Skin friction. Skin friction may be estimated by equation 3-3 where the positive skin friction is given by equations 3-16 and 3-20.

(5) Computer analysis. Program AXLITR (Axial Load-Transfer), Appendix C, computes the vertical shaft and soil displacements for axial down-directed structural, axial pullout, uplift and downdrag forces as described above using load-transfer functions to relate base pressures and skin friction with displacements. Some load-transfer functions available in program AXILTR are presented in Figure 3-22. AXILTR also calculates the load and displacement distribution with depth permitting evaluation of the load distribution illustrated in Figures 3-19 to 3-21. Refer to Appendix C for example applications of AXILTR for pullout, uplift, and downdrag loads.

(a) Load-transfer principle. Vertical loads are transferred from the top of the shaft to the supporting soil adjacent to the shaft using skin friction-load transfer functions and to soil beneath the base using base load-transfer functions or consolidation theory. The total bearing capacity of the shaft $Q_u = Q_{su} + Q_{bu}$ is given by equation 3-1. The program should be used to provide a minimum and maximum range for the load-displacement behavior for given soil conditions.

(b) Base resistance. The maximum base resistance q_{bu} in equation 3-1b is computed by AXILTR from equation 3-5b. Correction factors ζ are considered equal to unity. Program AXILTR does not set a limit for σ'_L. For effective stress analysis, N_q is evaluated by equation 3-24 for local shear and by equation 3-10 for general shear. For effective stress analysis, N_c is given by equation 3-8a. For total stress analysis, N_c is equal to 9 when general shear is specified and 7 when local shear is specified. In total stress analysis, the angle of internal friction ϕ is zero. Additional resistance provided by an underream to pullout loads or uplift thrust is seven-ninths (7/9) of the end-bearing resistance.

(c) Base displacement. Base displacement is computed using the Reese and Wright (1977) or Vijayvergiya (1977) base load-transfer functions (Figure 3-22a) or consolidation theory. Ultimate base displacement for the Reese and Wright model is

$$\rho_{bu} = 2B_b \cdot \epsilon_{50} \tag{3-34}$$

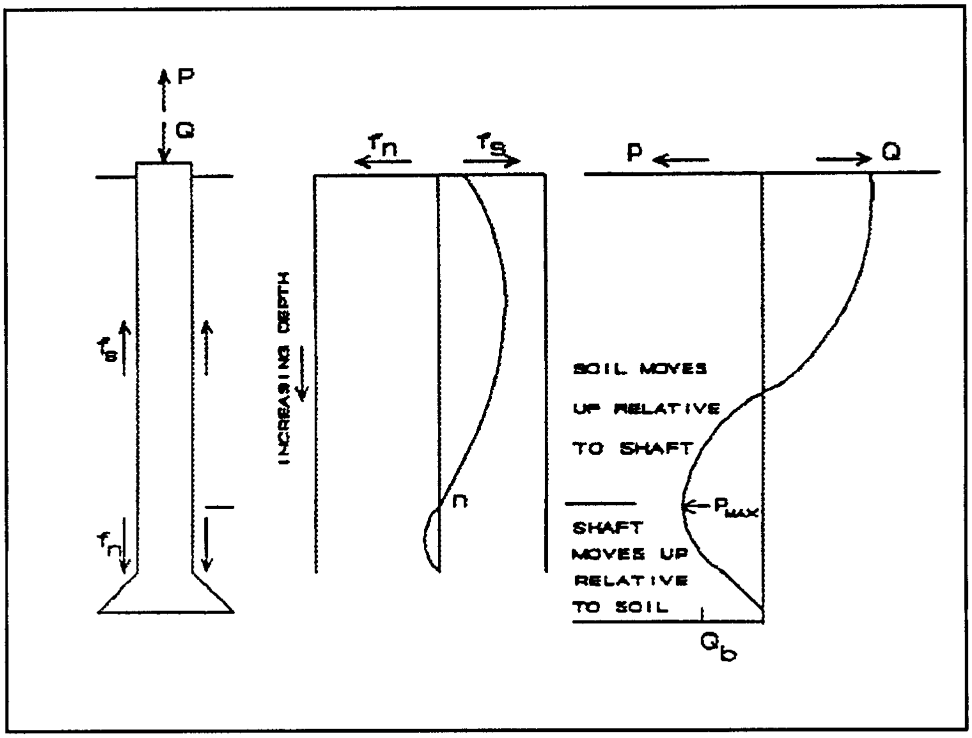


Figure 3-20. Deep foundation resisting uplift thrust

where

ρ_{bu} = ultimate base displacement, inches

B_b = base diameter, inches

ϵ_{50} = strain at 1/2 of maximum deviator stress from consolidated undrained or unconsolidated undrained triaxial test conducted at a confining pressure equal to the soil overburden pressure, fraction

Typical values for ϵ_{50} are 0.007, 0.005, and 0.004 for stiff clays with cohesion C_u of 1 to 2, 2 to 4, and 4 to 8 ksf, respectively (FHWA-RD-85-106). The ultimate base displacement ρ_{bu} for the Vijayvergiya model is 4 percent of the base diameter, where ρ_{bu} occurs at loads equal to the bearing resisting force of the soil Q_{bu}. Plunging failure occurs if an attempt is made to apply greater loads. Base displacement from consolidation theory is calculated relative to the initial effective stress on the soil beneath the base of the shaft prior to placing the structural loads. AXILTR may calculate large settlements for small applied loads on the shaft if the preconsolidation stress (maximum past pressure) is less than the initial effective stress (i.e., an underconsolidated soil). Effective stresses in the soil below the shaft base caused by shaft loads are calculated using the Boussinesq stress theory.

(d) Skin resistance. The shaft skin friction load-transfer functions applied by AXILTR as shown in Figure 3-22b are the Seed and Reese (1957) model, and of Kraft, Ray, and Kagawa (1981) models. The Kraft, Ray, and Kagawa model requires an estimate of a curve fitting constant R that can be obtained from

$$G = G_i \left[1 - \frac{\tau R}{\tau_{max}} \right] \tag{3-35}$$

where

G = soil shear modulus at an applied shear stress τ, ksf

G_i = initial shear modulus, ksf

τ = shear stress, ksf

τ_{max} = shear stress at failure, ksf

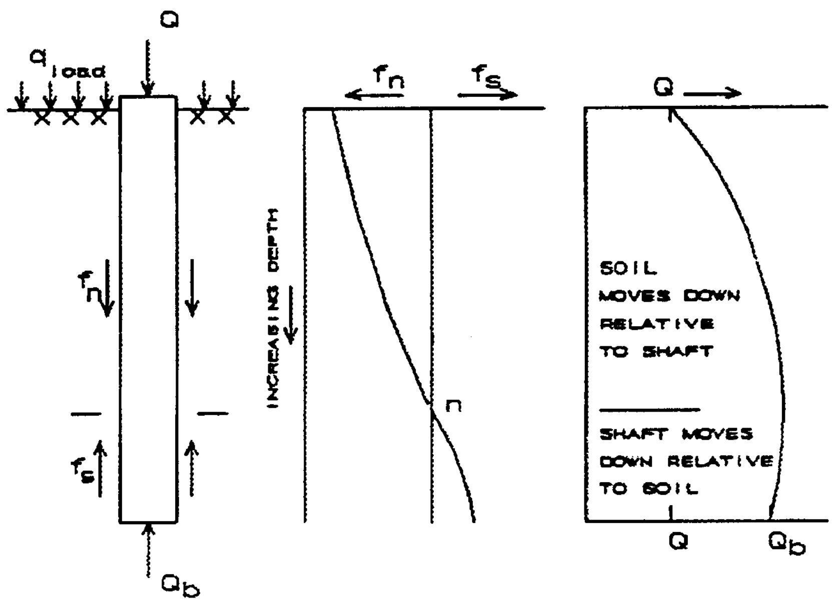


Figure 3-21. Deep foundation resisting downdrag

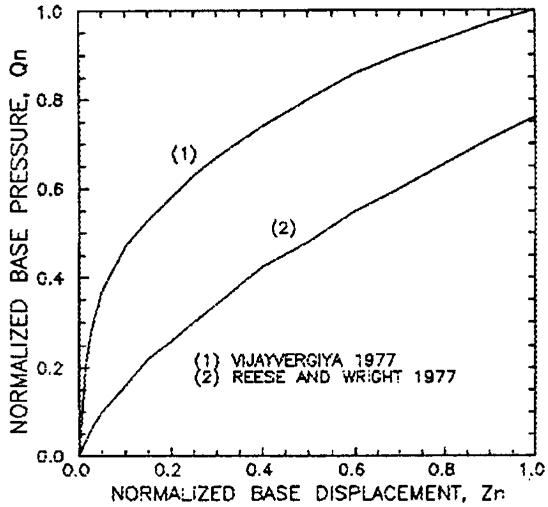
R = curve fitting constant, usually near 1.0

Curve fitting constant R is the slope of 1 - G/G_i versus τ/τ_{max} and should be assumed unity it not known.

(e) Other load-transfer functions. Other functions may be input into AXILTR for each soil layer up to 11. Each function consists of 11 data points that are the ratio of the mobilized skin friction/maximum mobilized skin friction f_s/f_{su} correlated with displacement such as in Figure 3-22b. The value f_{su} is taken as the soil shear strength if not known. The 11 displacement points in meters (inches) are input only once and become applicable to all of the load-transfer functions; therefore, f_s/f_{su} of each load-transfer function must be correlated with displacement.

(f) Influence of soil movement. Soil movement, whether heave or settlement, alters shaft performance. The magnitude of soil heave or settlement is calculated in AXILTR using swell or recompression indexes, compression indexes, swell pressure of each soil layer, maximum past pressure, water table depth, and depth of the soil that is subject to soil movement. The swell index is the slope of the rebound log pressure/void ratio curve of consolidation test results as described in ASTM D 4546. The recompression index is the slope of the log pressure/void ratio curve for pressures less than the maximum past pressure. AXILTR assumes that the swell and recompression indexes are the same. The compression index is the slope of the linear portion of the log pressure-void ratio for pressures exceeding the maximum past pressure. The maximum past pressure is the greatest effective pressure applied to a soil. Swell pressure is defined as the pressure when it prevents soil swell described in Method C of ASTM D 4546.

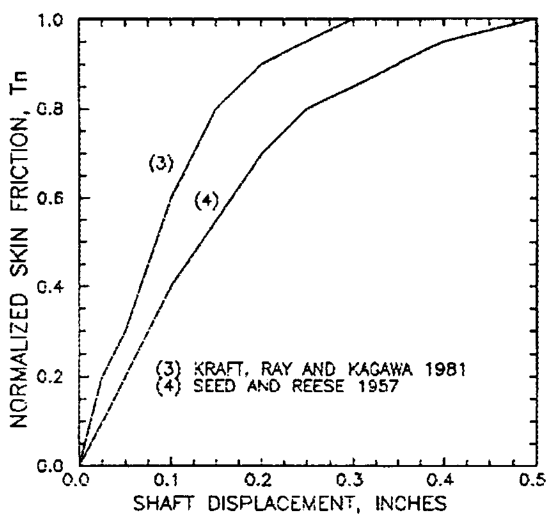
e. Load-dispalcement relationship. Settlement for given loads should be estimated to check that the expected settlement will be within acceptable limits. Load-displacement relationships are estimated by theory of elasticity and empirical load-transfer relationships. Settlement analysis using computer programs based on nonlinear load-transfer functions applicable to actual soil conditions are also reasonably reliable and cost effective. The skin friction and base load transfer curves should be used together to estimate



$$Q_n = \frac{\text{BASE PRESSURE, } q_b}{\text{ULTIMATE BASE PRESSURE, } q_{bu}}$$

$$Z_n = \frac{\text{BASE DISPLACEMENT, } \rho_b}{\text{ULTIMATE BASE DISPLACEMENT, } \rho_{bu}}$$

a. BASE TRANSFER (q-z) FUNCTIONS



$$T_n = \frac{\text{MOBILIZED SKIN FRICTION, } f_s}{\text{MAXIMUM MOBILIZED SKIN FRICTION, } f_{su}}$$

ρ_s = SHAFT DISPLACEMENT, INCHES

b. SHAFT TRANSFER (t-z) FUNCTIONS

Figure 3-22. Load-transfer curves used in AXILTR

settlement for a wide variety of load conditions and to provide a complete analysis of load-displacement behavior. Settlement due to consolidation and creep are site specific and will be considered depending on the types of soils in which the foundation is to be constructed.

(1) Elastic method. Linear elastic analysis is used to determine short-term settlement, but may underestimate long-term settlement. Loads at the pile or shaft base applied to underlying soil should be checked for consolidation settlement using methods in TM 5-818-1 of AXILTR if a highly compressible soil layer exists beneath the tip. The Randolph and Wroth method (1978) is recommended to quickly estimate settlement for piles or straight shafts:

$$\rho = \frac{Q \xi \mu}{2 \pi G_L' \tanh(\mu L)} \tag{3-36a}$$

A similar equation for underreamed shafts can be deduced as follows:

$$\rho = \frac{Q \xi \mu \eta (1 - \nu_s)}{2[\pi \eta (1 - \nu_s) \tanh(\mu L) + \xi B_b \mu] G_L'} \tag{3-36b}$$

where

$$\xi = \ln \left[\frac{5 L G_L' \eta (1 - \nu_s)}{B_s G_L} \right]$$

$$\mu = \left[\frac{8 G_L}{\xi E_p B_s^2} \right]^{1/2}$$

L = embedded length of pile of shaft, feet

Q = applied load, kips

ρ = settlement for load Q, feet

ν_s = Poisson's ratio

η = interaction factor of upper with lower soil layer, $0.85 B_s / B_b$

E_p = shaft elastic modulus, ksf

G_L = soil shear modulus at depth L, ksf

G_L' = average soil shear modulus, ksf

B_b = base diameter, feet

B_s = shaft diameter, feet

This method accounts for local softening or a weak stratum near the shaft.

(2) Semiempirical method. Total settlement for piles or drilled shafts ρ (Vesic 1977) is

$$\rho = \rho_p + \rho_b + \rho_s \tag{3-37}$$

where

ρ = total settlement at the pile or shaft top, feet

ρ_p = settlement from axial pile or shaft deformation, feet

ρ_b = tip (base) settlement from load transferred through the shaft to the tip, feet

ρ_s = tip settlement from load transmitted to the soil from skin friction along the shaft length, feet

(a) Axial compression (Vesic 1977) is

$$\rho_p = (Q_b + \alpha_s Q_s) \frac{L}{A E_p} \tag{3-38a}$$

where

Q_b = load at the pile tip, kips

α_s = load distribution factor along pile length, 0.5 to 0.7; usually assume 0.5

Q_s = load taken by skin friction, kips

L = pile or shaft length, feet

A = cross section area of pile, feet^2

E_p = pile or shaft modulus of elasticity, ksf

Axial compression should usually be calculated by assuming that $Q_s = Q_{su}$, the ultimate skin resistance in equation 3-1 or 3-3, because most skin friction will be mobilized before end bearing is significant, unless the pile is bearing on a hard stratum. The value of Q_b is then calculated by subtracting Q_s from the design load Q_d. Otherwise, loads Q_b and Q_s supporting the pile load Q_d should be estimated using load-transfer curves as follows:

(b) Settlement at the pile or shaft tip (Vesic 1977) is

$$\rho_b = \frac{C_b Q_b}{B_s q_{bu}} \qquad (3\text{-}38b)$$

$$\rho_s = \frac{C_s Q_s}{L q_{bu}} \qquad (3\text{-}38c)$$

where

C_b = empirical tip coefficient, Table 3-10

C_s = empirical shaft coefficient, $[\,0.93 + 0.16\,(L/B_s)^{0.5}\,]\ C_b$

Table 3-10
Empirical Tip Coefficient C_b

Soil	Driven Piles	Drilled Shafts
Sand (dense to loose)	0.02 to 0.04	0.09 to 0.18
Clay (stiff to soft)	0.02 to 0.03	0.03 to 0.06
Silt (dense to loose)	0.03 to 0.05	0.09 to 0.12

The bearing stratum extends a minimum $10B_b$ beneath the pile or shaft tip, and stiffness in this stratum is equal to or greater than stiffness at the tip elevation. C_b will be less if rock is closer to the pile tip than $10B_b$. Settlement is $0.88\rho_b$ if rock exists at $5B_b$ and 0.5ρ if rock is B below the pile or shaft tip. Consolidation settlement should not be significant and should not exceed 15 percent of the total settlement.

(3) Load-transfer functions. Skin friction *t-z* curves and base resistance *q-z* curves may be used to transfer vertical loads to the soil. Curves in Figure 3-23 for clays and Figure 3-24 for sands were determined from drilled shafts with internal instruments for separating skin friction and base resistance. These curves include elastic compression and may be used to estimate settlements ρ_s and ρ_b which include ρ_p for shafts < 20 feet long. The value ρ_p from equation 3-38a should be added for long shafts.

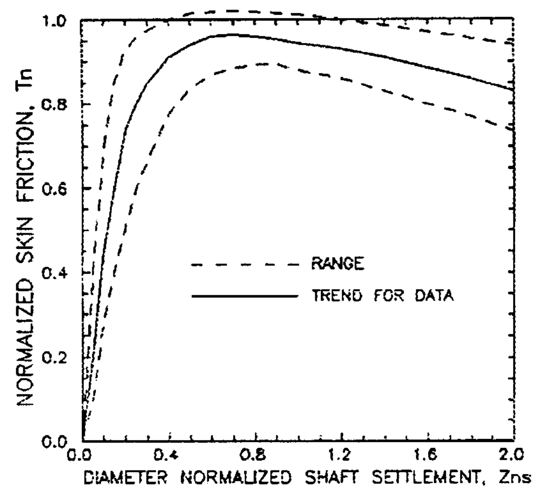
(4) Computer programs. Programs available at WES for estimating settlement from axial loads using base and shaft load-transfer functions are CAXPILE and AXILTR. These programs may be applied to either piles or shafts and consider multilayer soils. Some load-transfer functions are included and others may be input. Noncircular piles or shafts should be converted to circular cross sections by assuming equivalent area for square or rectangular cross sections. The cross-sectional area of H-piles calculated as the flange width b_f times section depth d, shown in Table 1-3, should be converted to an equivalent circular cross section.

(a) CAXPILE. This program considers downward vertical loads on shaft with variable diameter (WES Instruction Report-K-84-4).

(b) AXILTR. This program, Appendix C (available from the Soil and Rock Mechanics Division, Geotechnical Laboratory, U.S. Army Engineer Waterways Experiment Station), considers straight shafts with uniform cross sections are/or underreamed drilled shafts. AXILTR calculates settlement or uplift of piles caused by pullout loads and by soil heave or settlement.

f. Application. A drilled shaft is to be constructed in expansive soil characterized as two layers as shown in the tabulation on the following page. Soil Poisson's ratio $\nu_s = 0.4$. The shaft elastic modulus E_p = 432,000 ksf. A cone penetration test indicated q_c > 24 ksf. The shaft must support a design load Q_d = 300 kips with displacement less than 1 inch. The FS = 3. A schematic diagram of this shaft divided into 50 increments NEL = 50 and placed 10 feet into layer 2 is given in Figure C-1. Solution for the design according to Table 3-8 is given in Table 3-11. The shaft should also be checked for structural integrity as described in Chapter 2.

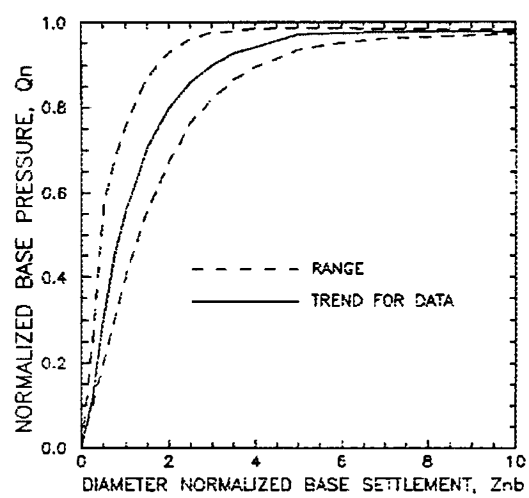
Parameter	**Layer 1** **0 - 40 ft**	**Layer 2** **40 - 50 ft**
Specific gravity, G_s	2.68	2.65
Initial void ratio, e_o	0.80	0.37
Water content, percent	30.00	13.10
Swell pressure, σ_s , ksf	4.80	6.00
Swell index, C_s	0.10	0.10
Compression index, C_c	0.20	0.20
Cohesion, C_u , ksf	2.00	4.00
Friction angle, ϕ, deg	0.00	0.00
Coefficient of earth pressure at rest, K_o	0.70	2.00
Maximum past pressure, σ_p , ksf	7.00	10.00
Plasticity index, PI, percent	38.00	32.00
Liquid limit, LL, percent	70.00	60.00
Elastic soil modulus, E_s , ksf	400.00	1,000.00
Shear soil modulus, G, ksf	143.00	357.00



$$T_n = \frac{\textit{MOBILIZED SKIN FRICTION, } f_s}{\textit{MAXIMUM MOBILIZED SKIN FRICTION, } f_{su}}$$

$$Z_{ns} = \frac{\textit{SETTLEMENT, } \rho_s}{\textit{SHAFT DIAMETER, } B_s}, \textit{ PERCENT}$$

b. SHAFT TRANSFER (t-z) FUNCTION

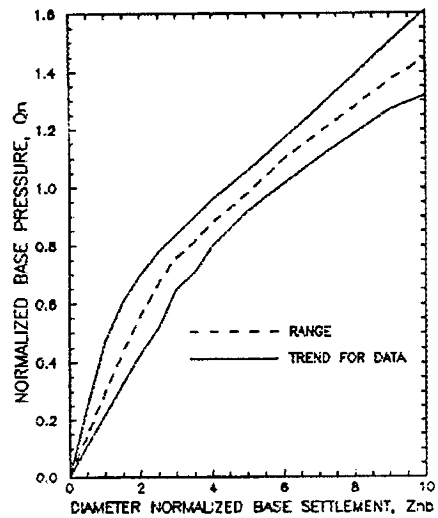


$$Q_n = \frac{\textit{END BEARING PRESSURE, } q_b}{\textit{ULTIMATE END BEARING, } q_{bu}}$$

$$Z_{nb} = \frac{\textit{BASE SETTLEMENT, } \rho_b}{\textit{BASE DIAMETER, } B_b}, \textit{ PERCENT}$$

a. BASE TRANSFER (q-z) FUNCTION

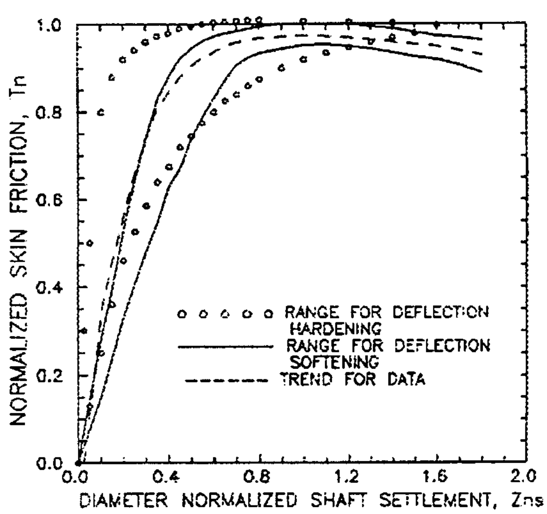
Figure 3-23. General load-transfer curves for clay



$$Q_n = \frac{\text{END BEARING PRESSURE, } q_b}{\text{ULTIMATE END BEARING, } q_{bu}}$$

$$Z_{nb} = \frac{\text{BASE SETTLEMENT, } \rho_b}{\text{BASE DIAMETER, } B_b}, \text{ PERCENT}$$

a. BASE TRANSFER (q-z) FUNCTION



$$T_n = \frac{\text{MOBILIZED SKIN FRICTION, } f_s}{\text{MAXIMUM MOBILIZED SKIN FRICTION, } f_{su}}$$

$$Z_{ns} = \frac{\text{SETTLEMENT, } \rho_s}{\text{SHAFT DIAMETER, } B_s}, \text{ PERCENT}$$

b. SHAFT TRANSFER (t-z) FUNCTION

Figure 3-24. General load-transfer curves for sand

Table 3-11
Application of Drilled Shaft Design

Step	Procedure	Description
1	Select shaft length	The shaft is selected to penetrate 10 ft into layer 2, a firm stratum, with L = 50 ft; additional analyses can be performed with L < 50 ft to determine an optimum length
2	Evaluate q_{bu}	From equation 3-22, $q_{bu} = F_r N_c C_c \le 80$ ksf $F_r = 1,\ C_u = 4$ ksf, $N_c = 6\,[(1 + 0.2\,(L/B_b)] \le 9$ $= 6\,[1 + 0.2\,(50/5)] = 18;$ so $N_c = 9$ $q_{bu} = 1 \times 9 \times 4 = 36$ ksf
3	Evaluate f_{sui}	From equation 3-16, $f_{sui} = \alpha_a \times C_u$ Layer 1, equation 3-26b: 0-40 ft $\alpha_a = 0.9 - 0.01PI$ $= 0.9 - 0.01 \times 38$ $= 0.52$ $f_{su1} = 0.52 \times 2 = 1.04$ ksf Layer 2, equation 3-26a: 50 - 60 ft $\alpha_a = 0.7 - 0.01PI$ $= 0.7 - 0.1 \times 32$ $= 0.38$ $f_{su2} = 0.38 \times 4 = 1.52$ ksf From Table 3-8, $\alpha_a = 0.55$ Layer 1: $f_{su1} = 0.55 \times 2$ $= 1.1$ ksf ≤ 5.5 ksf Layer 2: $f_{su2} = 0.55 \times 4$ $= 2.2$ ksf ≤ 5.5 ksf

(Sheet 1 of 5)

Table 3-11 (Continued)

Step	Procedure	Description
4	Evaluate Q_{bu} and Q_{su} for the shaft and base diameters	From equation 3-1b, $Q_{bu} = q_{bu} \times A_b = 36 \times \pi \times 2.5^2 = 706.9$ kips From equation 3-3, $Q_{sui} = A_{si} f_{sui} = \pi B_s \Delta L f_{sui}$ $Q_{su1} = \pi \times 2 \times 35 \times f_{su1} = 219.9 f_{su1}$ $Q_{su2} = \pi \times 2 \times 5 \times f_{su2} = 31.4 f_{su2}$ From equations 3-26, $Q_{su1} = 219.9 \times 1.04 = 228.7$ kips $Q_{su2} = 31.4 \times 1.52 = 47.7$ kips $Q_{su} = 228.7 + 47.7 = 276.4$ kips From Table 3-8, $\alpha_a = 0.55$ $Q_{su1} = 219.9 \times 1.1 = 241.9$ kips $Q_{su2} = 31.4 \times 2.2 = 69.1$ kips $Q_d = 300 < 327.8 = Q_a$; okay $Q_{su} = 241.9 + 69.1 = 311.0$ kips Select the least Q_{su} = 276.4 kips
5	Check $Q_d \le Q_a$	$Q_u = Q_{bu} + Q_{su}$ $= 706.9 + 276.4 = 983.3$ kips; $Q_a = 983.3/3 = 327.8$ kips
6	Evaluate shaft for other loads	Figure C-2c, Appendix C, for this shaft in expansive soil indicates heave < 1 inch even when subject to 300-kip pullout force
7	Evaluate maximum settlement ρ for given Q_d	From equation 3-36b, $\rho = \dfrac{12 Q \xi \mu \eta (1 - \nu_s)}{2[\pi \eta (1 - \nu_s) \tanh(\mu L) + \xi B_b \mu] G_L'}$ $= \dfrac{12 \times 300 \times 2.323 \times 0.34 \times 0.6 \times 0.267}{2[\pi \times 0.34 \times 0.6 \times 0.87 + 2.323 \times 5 \times 0.0267] \times 143}$ $= 0.18$ inch

(Sheet 2 of 5)

Table 3-11 (Continued)

Step	Procedure	Description
		where
		$$\xi = \ln\left[\frac{5LG_L'\eta(1 - \nu_s)}{B_sG_L}\right]$$
		$$= \ln\left[\frac{5 \times 50 \times 143 \times 0.34(1 - 0.4)}{2 \times 357}\right]$$
		$$= 2.323$$
		$$\mu = \left[\frac{8G_L}{\xi EB_s^2}\right]^{1/2}$$
		$$= \left[\frac{8 \times 357}{2.323 \times 432{,}000 \times 2^2}\right]^{1/2}$$
		$$= 0.0267$$
		$$\tanh \mu L = \tanh 1.335 = 0.87$$
		$$\eta = 0.85 \times (B_s/B_b) = 0.85 \times (2/5) = 0.34$$
		$$G_L = 357 \text{ ksf}$$
		$$G_L' = 143 \text{ ksf}$$
		From equation 3-37,
		$$\rho = \rho_p + \rho_b + \rho_s$$
		From equation 3-38a,
		$$\rho_p = (Q_b + \alpha_s Q_s)\frac{L}{AE}$$
		$$= (23.6 + 0.5 \times 276.4)\frac{50}{\pi 1^2 \times 432{,}000}$$
		$$= 0.07 \text{ inch}$$
		where
		$$Q_s = Q_{su} = 276.4 \text{ kips}$$
		$$Q_b = Q_d - Q_s = 300 - 276.4 = 23.6 \text{ kips}$$

(Sheet 3 of 5)

Table 3-11 (Continued)

Step	Procedure	Description

From equation 3-38b,

$$\rho_b = \frac{12\,C_b Q_b}{B_s q_{bu}}$$

$$= \frac{12 \times 0.06 \times 23.6}{2 \times 36}$$

$$= 0.24 \text{ inch}$$

From equation 3-38c,

$$\rho_s = \frac{12\,C_s Q_s}{L q_{bu}}$$

$$= \frac{12 \times 0.1 \times 276.4}{50 \times 36}$$

$$= 0.18 \text{ inch}$$

where

$$C_s = [0.93 + 0.16(L/B_s)^{0.5}]\,C_b$$

$$= [0.93 + 0.16(50/2)^{0.5}] \times 0.06$$

$$= 0.1$$

Therefore,

$$\rho = 0.07 + 0.24 + 0.18 = 0.49 \text{ inch}$$

Settlement should be < 0.49 inch because resistance from the 5-ft underream is disregarded

From Figure 3-23, base load-transfer functions (assume 90-percent skin friction is mobilized:

$$Q_b = Q_d - 0.9Q_{su}$$

$$= 300 - 248.8 = 51.2 \text{ kips}$$

$$Q_b/Q_{bu} = 51.2/706.9 = 0.07; \text{ therefore,}$$

$$Z_{nb} = 0.2 \text{ percent Figure 3-23a}$$

$$\rho = 12 \times B_b Z_{nb}/100$$

$$= 12 \times 5 \times 0.2/100 = 0.12 \text{ inch}$$

Shaft: assume $f_s/f_{su} = 0.9$; therefore,

$$Z_{ns} = 0.4 \text{ percent from Figure 3-23b}$$

(Sheet 4 of 5)

Table 3-11 (Concluded)

Step	Procedure	Description
		$\rho = 12 \times B_s Z_{ns}/100$ $= 12 \times 2 \times 0.4/100 = 0.10$ inch
		The shaft is longer than 20 ft, $\rho_p = 0.07$ inch must be added to determine total settlement ρ
		$\rho = 0.07 + 0.12 + 0.10 = 0.29$ inch
		Program AXILTR , Figure C-2a, Appendix C, indicates 0.2 inch for a 300-kip load using $\alpha_e = 0.9$
		All of the above analyses indicate total settlement < 0.5 inch
8	Check computed ≤ specified settlement	Specified settlement is 1.0 inch; this exceeds the calculated settlement; okay

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