CHAPTER 8.0
DEEP FOUNDATION DESIGN

Foundation design and construction involves assessment of factors related to engineering and economics. The selection of the most feasible foundation type requires consideration of both shallow and deep foundation types in relation to the characteristics and constraints of the project and site conditions. A cost evaluation is essential in the selection of the optimum foundation system.

Situations commonly exist where shallow foundations are inappropriate for support of structural elements. These situations may be related either to the presence of unsuitable soil layers in the subsurface profile, adverse hydraulic conditions, or tolerable movements of the structure. Deep foundations are designed to transfer load thru unsuitable subsurface layers to suitable bearing strata. Deep foundation types include several pile types (driven, non-driven, micropiles, etc.) and drilled shafts. The suitability of a deposit may depend on bearing capacity, settlement or scour considerations. Foundation engineers should interact with both structural and hydraulic engineers in the design of deep foundations. This manual will only present the basic concepts of deep foundation design that are similar for all deep foundation types. More advanced design and construction information on deep foundations can be found in other FHWA manuals and training courses related to specific deep foundation topics. This chapter will illustrate the use of those concepts for driven pile design and peripherally extend those concepts to drilled shafts.

8.1 DRIVEN PILE FOUNDATION DESIGN

For many years the use of a pile foundation has meant security to many designers. The temptation to use piles under every facility is great; detailing of plans is routine, quantity estimate is neat, and safe structural support apparently assured. Unfortunately, the rationale behind pile type selection, length, and allowable load, is usually based on peripheral factors such as local availability, outdated dynamic formulas or previous usage of certain pile types. Traditionally, the duty of determining if a pile is "good" has been passed on to the inspector in the field, who seldom has any training in foundation design or is given any pertinent information on which to base driven pile acceptability.

Foundation engineers agree that proper pile design is a most difficult task; requiring a combination of theoretical training, and experience in design and construction. Unlike other foundation types which may be installed to close tolerances in shallow excavations, piles are brutally forced far below ground surface with hammers that may generate stresses in excess of permitted static design levels. At present, a wide variety of piling is in use; each possessing inherent characteristics which affect determination of both capacity and driveability. Alternate pile types for a particular project can be evaluated by applying engineering judgment.

8.2 ALTERNATE PILE TYPE EVALUATION

There are many different methods of classifying piles such that several types may be suitable for a given situation. Some factors to be considered are as follows:

1. Pile Material

	Optimum Load Range	Optimum Length
Timber	30-80 Kips	20-40 ft.
Concrete	80-800 Kips	40-150 ft.
Steel H	80-400 Kips	40-160 ft.
Steel Pipe	70-1000 Kips	30-100 ft.

2. Pile Shape Effects

	Pile Types	Effects
Displacement	Steel Pipe (Closed end), 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 pile in granular soils
Tapered	Timber, Monotube	Increased densification of soils with less disturbance, high capacity for short length in granular soils

3. Subsurface Conditions

Typical Problem	Advice
Boulders overlying bearing stratum	Use heavy nondisplacement pile with a reinforced tip or manufactured point and include contingent predrilling item in contract.
Loose cohesionless soil	Use tapered pile to develop maximum skin friction.
Negative skin friction	Use smooth steel pile to minimize drag adhesion, and avoid battered piles. Provide bitumen coating in drag zone.
Deep soft clay	Use rough concrete pile to increase adhesion and rate of pore water dissipation.
Artesian Pressure	Do not use mandrel driven thin-wall shells as generated hydrostatic pressure may cause shell collapse; pile heave common to closed-end pipe.
Scour	Do not use tapered piles unless large part of taper extends well below scour depth. Design permanent pile capacity to mobilize soil resistance below scour depth.
Coarse Gravel Deposits	Use precast concrete piles where hard driving expected in coarse soils. DO NOT use H-piles as nondisplacement piles will penetrate at low blow count and cause unnecessary overruns.

4. Location and Topography

Problems to Consider:

- Driven piles may cause vibration damage.
- Access to remote area may restrict driving equipment size and, therefore, pile size.
- Local availability of certain materials may have decisive effects on pile selection.
- Waterborne operations may dictate use of shorter pile sections due to pile handling limitations.
- Steep terrain may make the use of certain pile equipment costly or impossible.

5. Structural Characteristics of the Proposed Superstructure

- Heavy structures may require stiff high capacity piles for lateral load resistance.
- Small, isolated structures may dictate small piles as mobilization costs for large driving equipment may be excessive.

Frequently consideration of pile type for a project will be influenced by the possible use of one pile type on several structures or at all footings on a particular structure. Designers should begin the selection process by choosing the two pile types which in their opinion will provide a cost-effective foundation for the project. The next step is evaluation between the selected alternates by determining required lengths and capacities at representative foundation locations.

8.3 DRIVEN PILE CAPACITY - STATIC ANALYSIS

The experienced foundation engineer has the ability to review boring data and classify zones in the subsoil with regard to relative pile support capability. However, without a rational method of design, this information will lead to pile selections that are at best wasteful, or at worst, dangerous.

Once the allowable structural load has been determined for prospective pile alternates, the pile length required to support that load must be determined. For many years this length determination was considered part of the "art of foundation engineering." In recent years more rational analytic procedures have been developed. Static analyses provide a useful design tool to select the most economical pile alternates. The methods which follow are established procedures which accurately account for the variables in pile length determination. The "art" remains in selecting appropriate soil strength values for the conditions and ascertaining the effects of pile installation on these values. For the typical project two static analyses will be required; the first to determine the length required for permanent support of the structures, and second to determine the soil resistance to be overcome during driving to achieve the estimated length. It must be stressed that each new site represents a new problem with unique boundary conditions. Experience with similar sites should not replace but refine the rational analysis methods presented herein.

8.4 COMPUTATION OF PILE CAPACITY

The ultimate capacity (Q_{ult}) of all driven piles may be expressed in terms of skin resistance (Q_s) and point resistance (Q_p);

$$Q_{ult} = Q_s + Q_p$$

The value of both Q_s and Q_p are determined in each layer based on either frictional or cohesive behavior of the soil. The strength of frictional soils is based on friction angle. Cohesive soil strength is based on undrained shear strength. The pile capacity of cohesive soil layers should not be computed with both friction angle and cohesion values.

8.4.1 Soils with Frictional Strength

A. Skin Resistance

1. Approximate Q_s for preliminary cost estimates (not recommended for design)
   Q_s (tons) = 0.02 N' D C_d (Reduce Q_s by 1/2 for H-piles) (8-1)

   Where: D = pile length below ground
   C_d = pile perimeter
   N′ = SPT value corrected for overburden pressure

2. Determining Q_s for design (Nordlund's Method)

This method is based on correlation with actual pile load test results. The pile shape and material are important factors included in this method.

$$Q_s = \sum_{0}^{D} K_\delta \, C_F \, P_d \, \frac{\sin(\omega + \delta)}{\cos\omega} \, C_d \, \Delta d \qquad (8\text{-}2)$$

Which simplifies for non-tapered piles (ω = 0) to the following:

$$Q_s = \sum_{0}^{D} K_\delta \, C_F \, P_d \sin\delta \, C_d \, \Delta d \qquad (8\text{-}3)$$

Where: Q_s = Total skin friction capacity
K_δ = Dimensionless factor relating normal stress and effective overburden pressure
P_d = Effective overburden pressure at the center of depth increment d
ω = Angle of pile taper measured from the vertical
δ = Friction angle on the surface of sliding
C_d = Pile perimeter
d = Depth increment below ground surface
C_F = Correction factor for K_δ when $\delta \neq \phi$ (soil friction angle)

To avoid numerical integration, computations may be performed for pile segments of constant diameter (ω = 0) within soil layers of the same effective unit weight and friction angle. Then equation 8-3 becomes:

$$q_s = K_\delta \, C_F \, P_d \sin\delta \, C_d \, D \qquad (8\text{-}4)$$

Where within the segment selected:

P_d = The average effective overburden pressure in segment D
C_d = The average pile perimeter
D = The segment length

q_s = The capacity of pile segment D (skin friction)

Equation 8-4 can be more easily understood if skin friction is related to the shear strength of granular soils, i.e., normal force times tangent of friction angle, N Tan ϕ. In equation 8-4 the terms K_δ C_F P_d represent the normal force against the pile, Sin δ represents the coefficient of friction between the pile and soil, and C_dD is the surface area in contact with the soil. In effect equation 8-4 is a summation of the N Tan ϕ shearing resistance against the sides of the pile.

3. Computational steps for non-tapered piles are as follows:

   a. Draw the existing effective overburden pressure (P_o) diagram.

   b. Choose a trial pile length.

   c. Subdivide the pile according to changes in the unit weight or soil friction angle (ϕ).

   d. Compute the average volume per foot of each segment (V).

   e. Enter Figure 8-1 with that volume and the pile type to determine δ/ϕ and compute δ.

   f. Enter the appropriate chart(s) in Figures 8-2 to 8-5 to determine K_δ for ϕ.

   g. If $\delta \neq \phi$, enter Figure 8-6 with ϕ and δ/ϕ to determine a correction factor C_F to be applied to K_δ.

   h. Determine the average values of effective overburden pressure and pile perimeter for each pile segment.

   i. Compute q_s from equation 8-4 for all pile segments and sum to find the ultimate frictional resistance developed by the pile.

For tapered piles Figures 8-2 to 8-5 must be entered with both φ and ω to determine K_δ. Also equation 8-2 should be used to compute the capacity of tapered piles. It is recommended that Nordlund's original paper in the May 1963 ASCE Journal (SMF) be referred to for numerical examples of tapered pile static analysis.

Selection of design friction angle should be done conservatively for piles embedded in coarse granular deposits. Pile load tests indicate that predicted skin friction is often overestimated; particularly in soil deposits containing either uniform sized or rounded particles. A conservative approach is to limit the shearing resistance by neglecting interlock forces. This results in maximum friction angle in predominately gravel deposits of 32^o for soft or rounded particles and 36^o for hard angular deposits. The Nordlund method also tends to overpredict capacity for piles larger than 24 inches in nominal width.

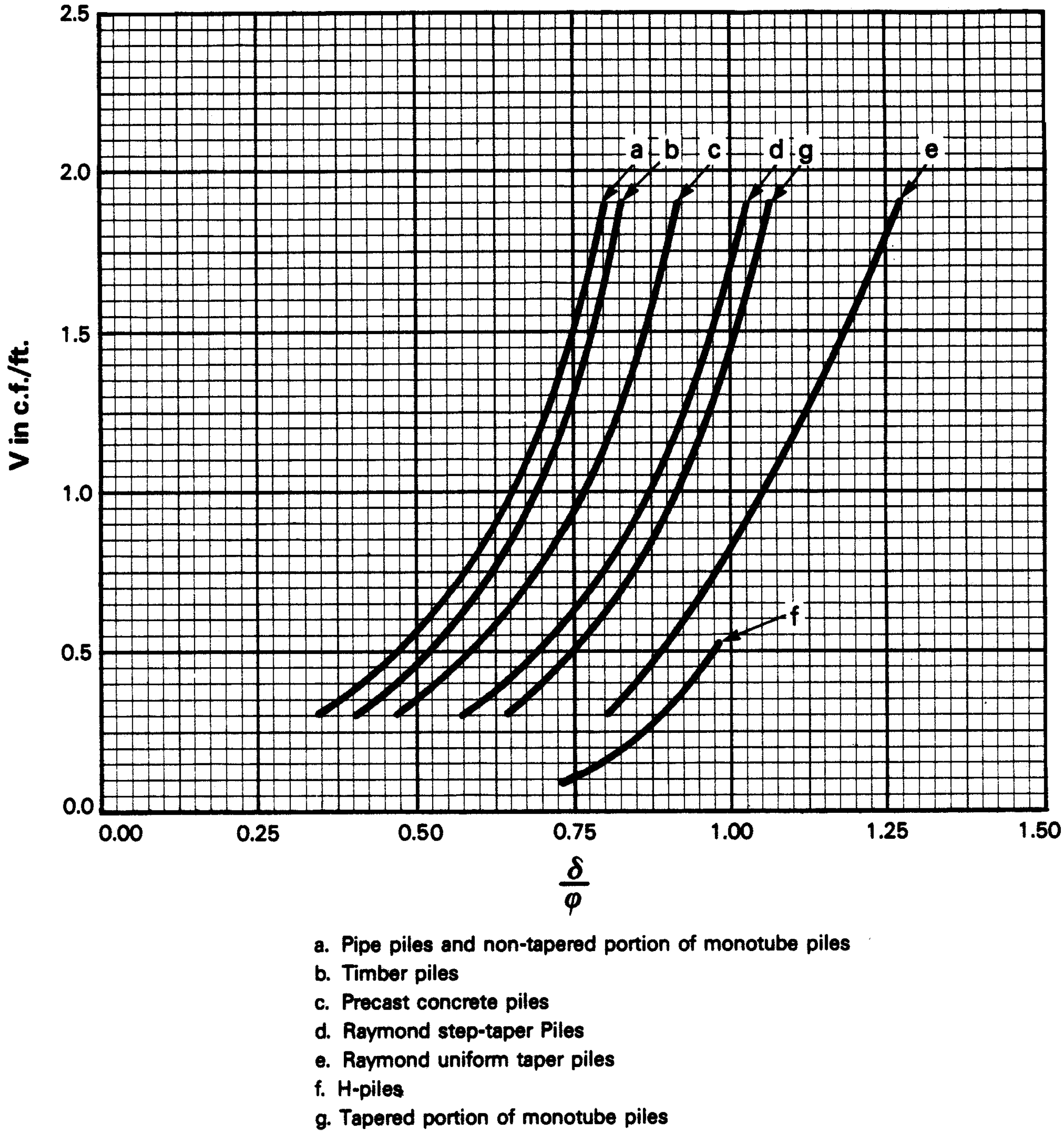


Figure 8-1: Relation of δ/ϕ and pile displacement, V, for various types of piles

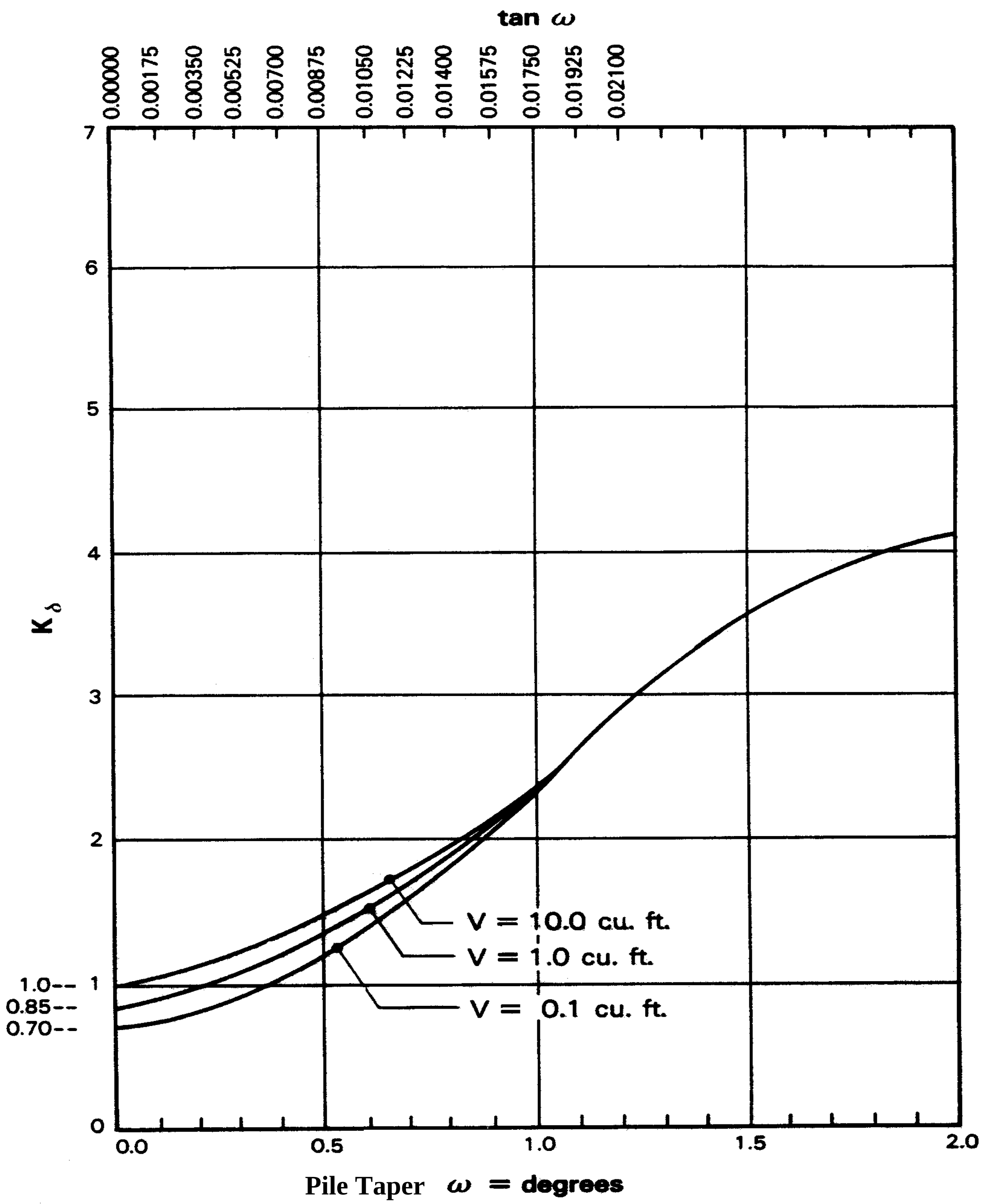


Figure 8-2: Design curves for evaluating K_δ for piles when $\phi = 25°$ (After Nordlund 1979)

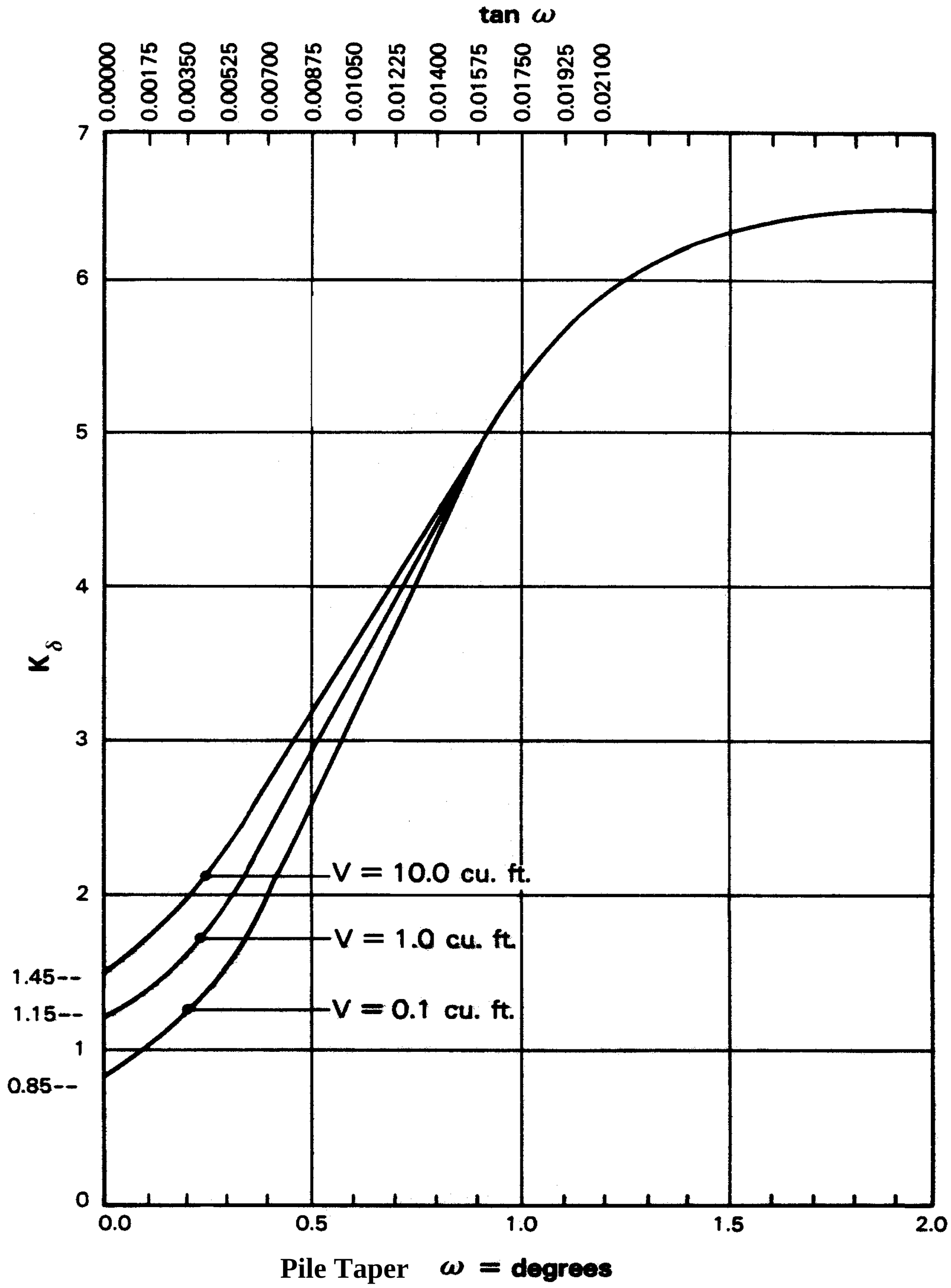


Figure 8-3: Design curves for evaluating K_δ for piles when $\phi = 30°$ (After Nordlund 1979)

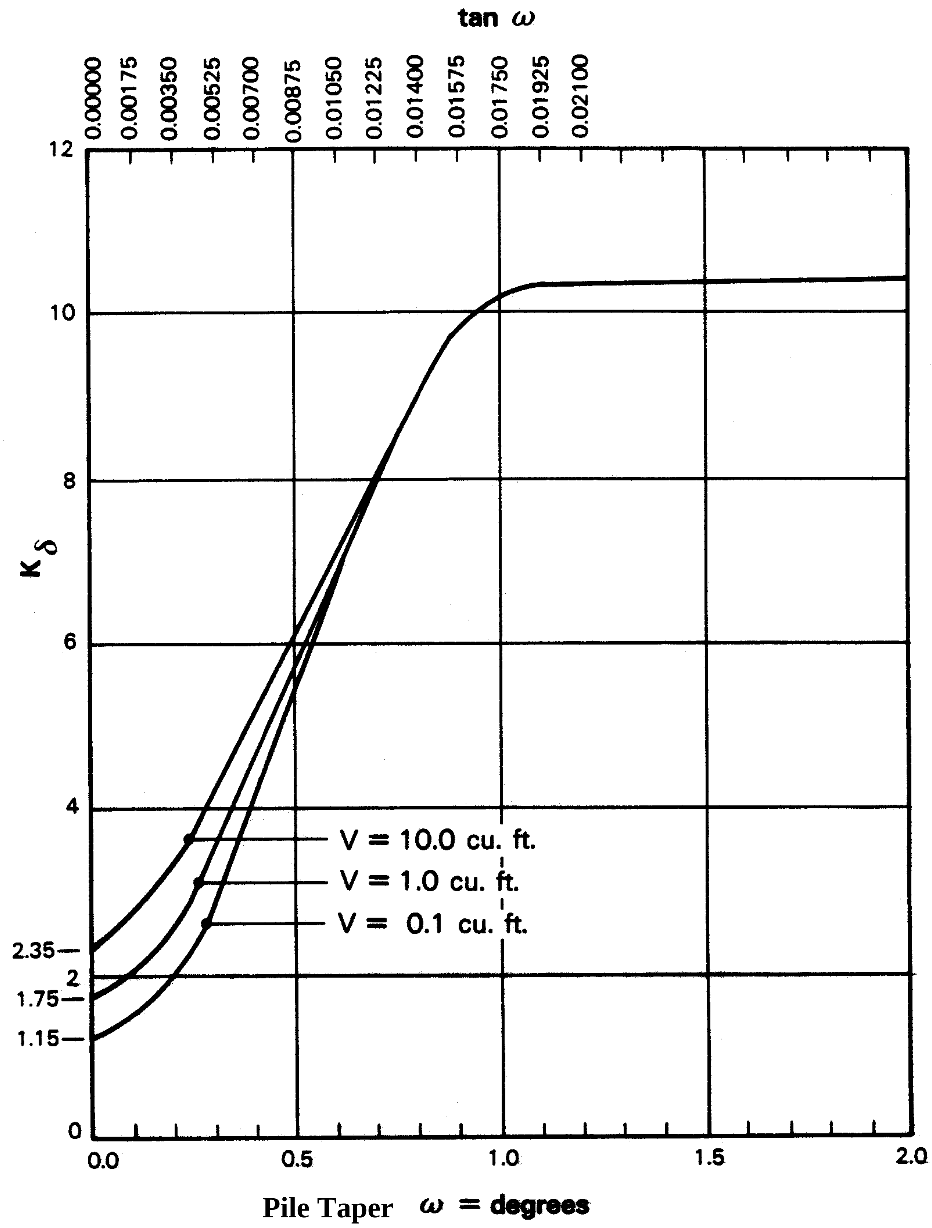


Figure 8-4: Design curves for evaluating K_δ for piles when $\phi = 35°$ (After Nordlund 1979)

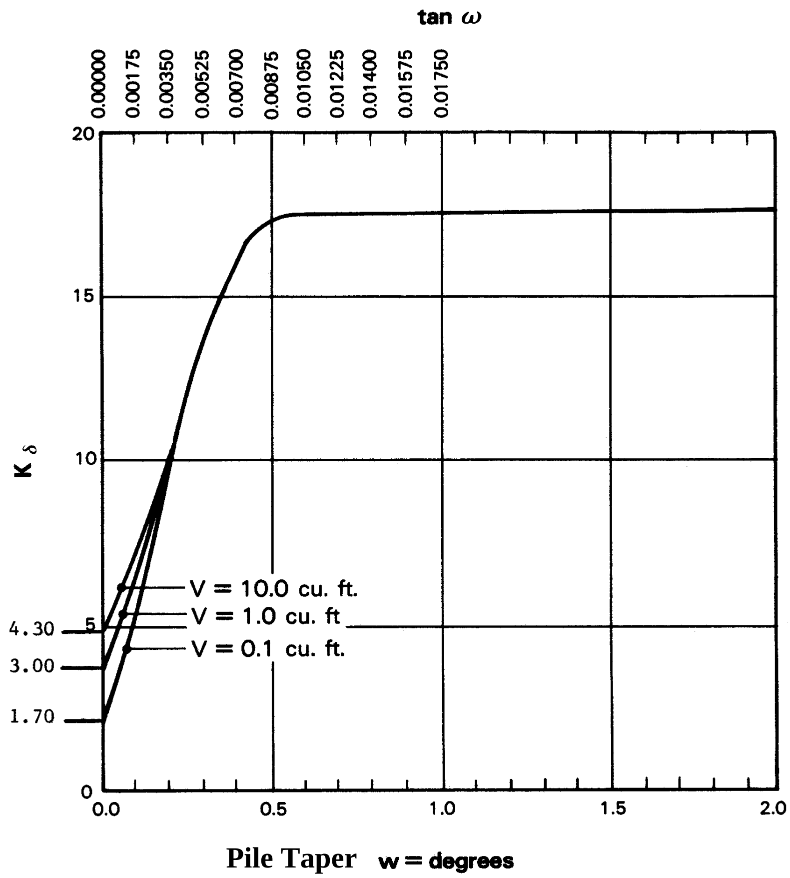


Figure 8-5: Design curves for evaluating K_{δ} for piles when $\phi = 40°$ (After Nordlund 1979)

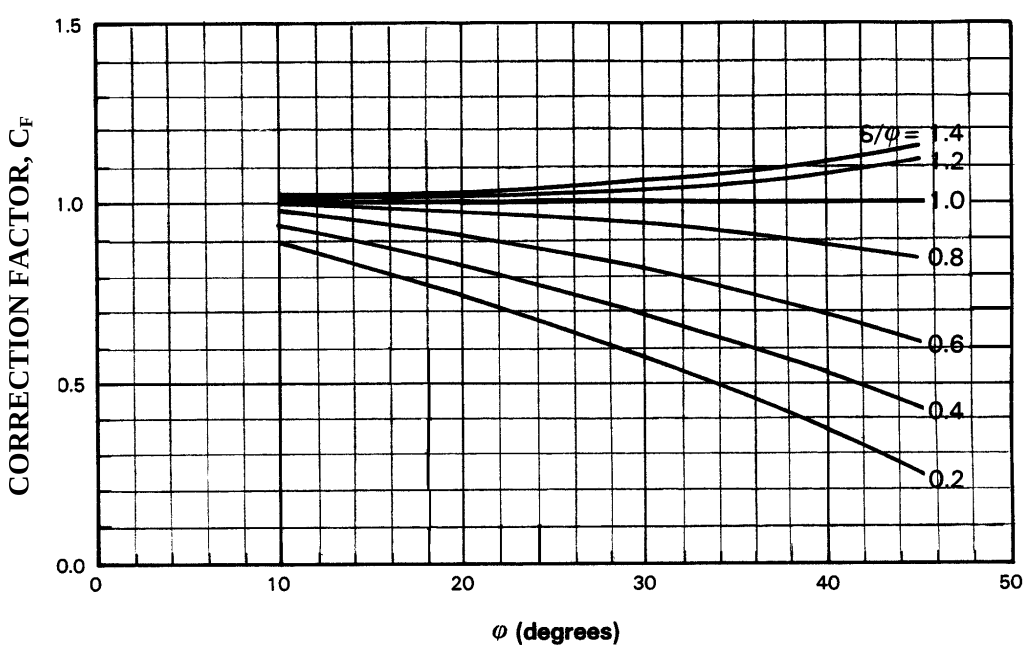


Figure 8-6: Correction factor, C_F for K_{δ} when $\delta \neq \phi$

B. Point Resistance

1. Approximate Q_p for preliminary cost estimate (not recommended for design).

$$Q_p \text{ (tons)} = 4\, N'\, A_p \tag{8-5}$$

Where: A_p = area of pile point
N' = SPT value corrected for overburden pressure

2. Determine Q_p for design (Thurman's Method)

$$Q_p = A_p\, \alpha\, P_D\, N'_q \tag{8-6}$$

Where: Q_p = end bearing capacity
A_p = pile end area
α = dimensionless factor dependent on depth-width relationship (see Figure 8-7A)
P_D = effective overburden pressure at the pile point
N'_q = bearing capacity factor from Figure 8-7A

The Q_p value is limited due to soil arching, which occurs around the pile point as the depth of tip embedment increases. For this reason, Nordlund has suggested limiting the overburden pressure at the pile point, P_D, to 3000 psf. More recently, other authors have suggested that further limitation must be placed on the end bearing so as not to compute unrealistic values. **Therefore, the Q_p value computed from the equation should be checked against the limiting value, Q_{LIM}, obtained from the product of the pile end area and the limiting point resistance in Figure 8-7B. The end bearing capacity should be taken as the lesser of Q_p or Q_{LIM}.**

The following criteria are suggested to determine the proper cross sectional end area to be used for H-piles.

a. Use the actual steel area if the point is founded on rock or in soil deposits having GRAVEL as the major component. Reinforced tips can be used to increase point area.

b. Use the enclosed area for all other soil deposits.

c. The ultimate pile capacity is the sum of all q_s values and Q_p which are below the deepest soil layer not considered suitable to permanently support the pile foundation. For scour piles, only sum Q_p and those q_s values below the anticipated scour depth.

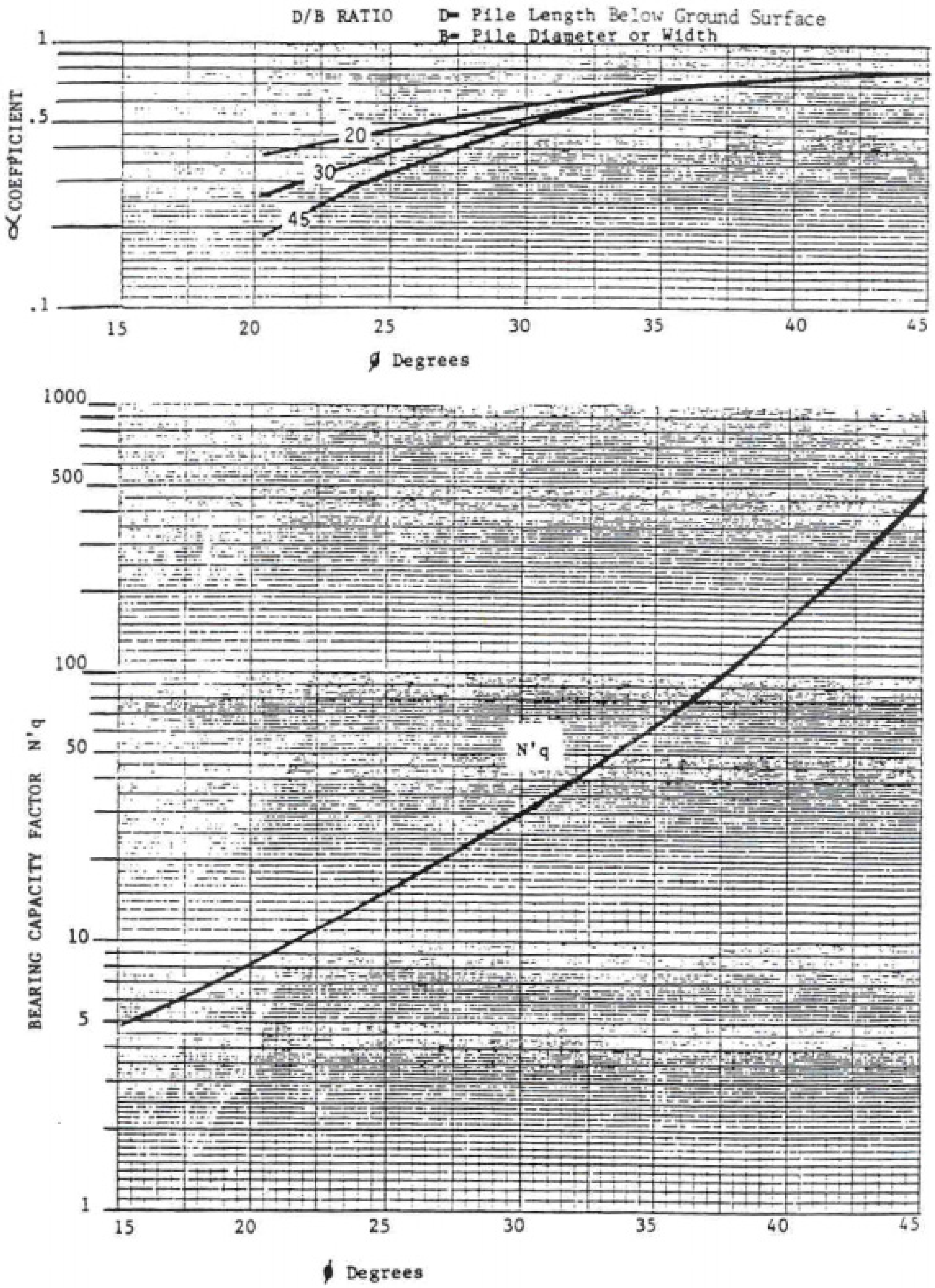


Figure 8-7A: Determination of ∀ coefficient and variation of bearing capacity factors with N

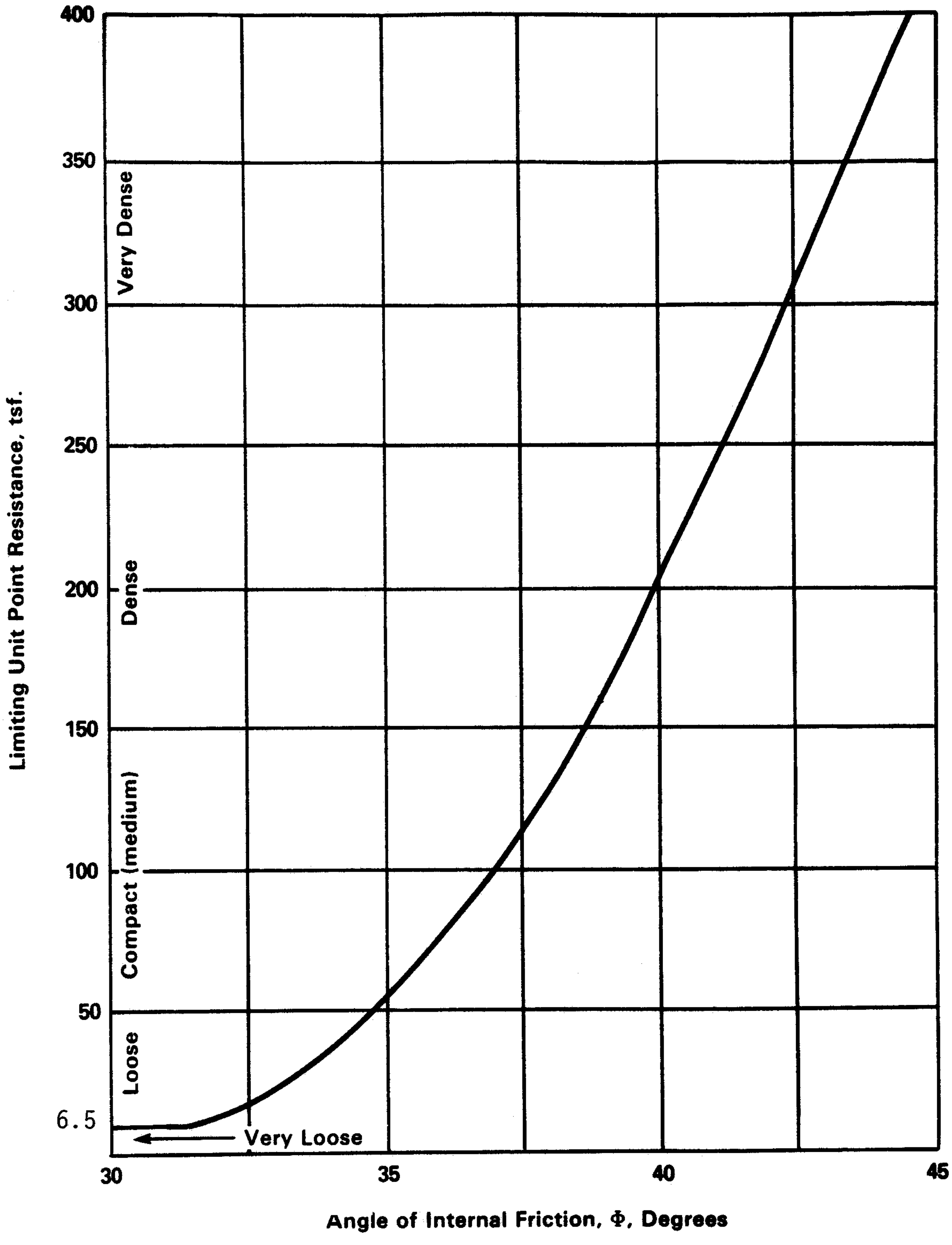
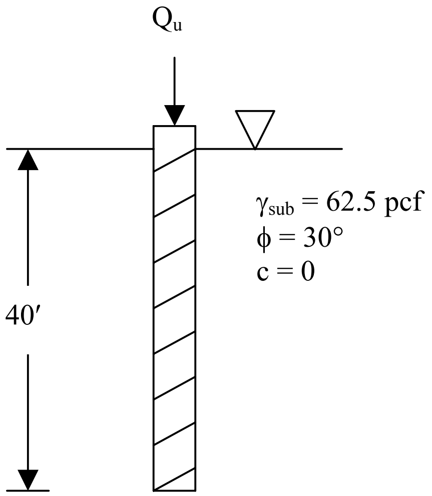


Figure 8-7B: Relationship between maximum unit pile point resistance and friction angle for cohesionless soils (After Meyerhof, 1976)

Example 8-1: Determine The Ultimate Capacity, Q_u, For The 1' Square Precast Concrete Pile



$$Q_u = A_p\, \alpha\, P_D\, N'_q + K_\delta\, C_F\, P_d\, \mathrm{Sin}\,(\delta + \omega)\, C_d\, D$$

Where the following terms are known from the problem

$A_p = 1$ sq.ft
$P_D = 40\ \gamma_{sub} = 2500$ psf
$P_d = 20\ \gamma_{sub} = 1250$ psf
$\omega = 0°$, $D = 40'$, $C_d = 4'$

Solution:

Find Point Resistance, Q_P:

Use Figure 8-7A to Find N'_q and α for $\phi = 30°$

$N'_q = 30$ $\qquad\qquad$ $\alpha = 0.5$ (for $\frac{D}{B} = 40$)

$Q_p = A_p \alpha P_D N'_q$

$= (1\text{ sq.ft})(0.5)(2500\text{ psf})\ 30 = 18.75$ tons

Check Limiting Point Resistance from Figure 8-7B

$Q_{Lim} = Q_{Lim}\ A_p = (6.5\text{ tsf})(1\text{ sq.ft}) = 6.5\text{ tons} \therefore Q_p = 6.5$ tons

Find Skin Resistance, Q_S: Use Figures 8-1, 8-3, and 8-6 with $\phi = 30°$

Figure 8-1 – For V = 1 cubic ft. per ft., and curve "C" for precast concrete piles;

$\frac{\delta}{\phi} = 0.76$, $\qquad$ Since $\phi = 30°$, $\quad \delta = 22.8°$

Fig. 8-3 – For $\omega = 0$, V = 1 cu.ft/ft ;

$K_\delta = 1.15$

Fig. 8-6 – For $\frac{\delta}{\phi} = 0.76$;

$C_F = 0.9$

$Q_s = K_\delta C_F P_d \sin\delta C_d D$

$Q_s = (1.15)(0.9)(1250 \text{ psf})(\sin 22.8)(4') 40'$
$Q_s = 40.1$ tons

$Q_{ult} = 6.5 + 40.1 = 46.6$ tons

8.4.2 Soils with Cohesive Strength

The method of installation has considerable effect on the capacity of a pile driven into cohesive soil. The following facts have been established:

A. Pile capacity immediately after driving is the lowest mobilized during its useful life. This capacity may be estimated from the remolded vane shear strength or from soil sensitivity.

B. Pile capacity begins increasing immediately after driving ceases. The rate of increase in capacity depends on:

1. Pile size and pile spacing.
2. Pile type.
3. Hammer size
4. Drainage characteristics of foundation soil.

Accurate estimates of rate of increase in capacity can only be found by installing piezometers within the group and monitoring the rate of pore pressure decrease. In general, piezometers should be installed within 3 pile diameters of the pile. The rate of pore pressure decrease can be estimated from procedures shown in geotechnical publications such as ASCE Proceedings, November 1979, Ismael & Klym, "Pore Pressures Induced by Pile Driving".

C. Except for low strength cohesive soils, the long-term strength of the soil supporting the piles will be less than the original soil strength due to remolding during pile installation. Soils with higher in situ strengths will exhibit a greater percentage reduction in strength available for long-term pile support. Therefore, a reduction factor should be applied to undrained shear strengths used in static analysis computations for pile side friction. Figure 8-8 shows the reduced pile adhesion values for corresponding undrained shear strength values. The ultimate bearing capacity of a pile in cohesive soil is:

$Q_{ult} = Q_s + Q_p$

$$Q_{ult} = C_a C_d D + 9 C_u A_p \tag{8-7}$$

Where: C_a = pile adhesion from Figure 8-8
C_d = perimeter of pile
D = pile segment length
C_u = undrained shear strength

A_p = pile end area

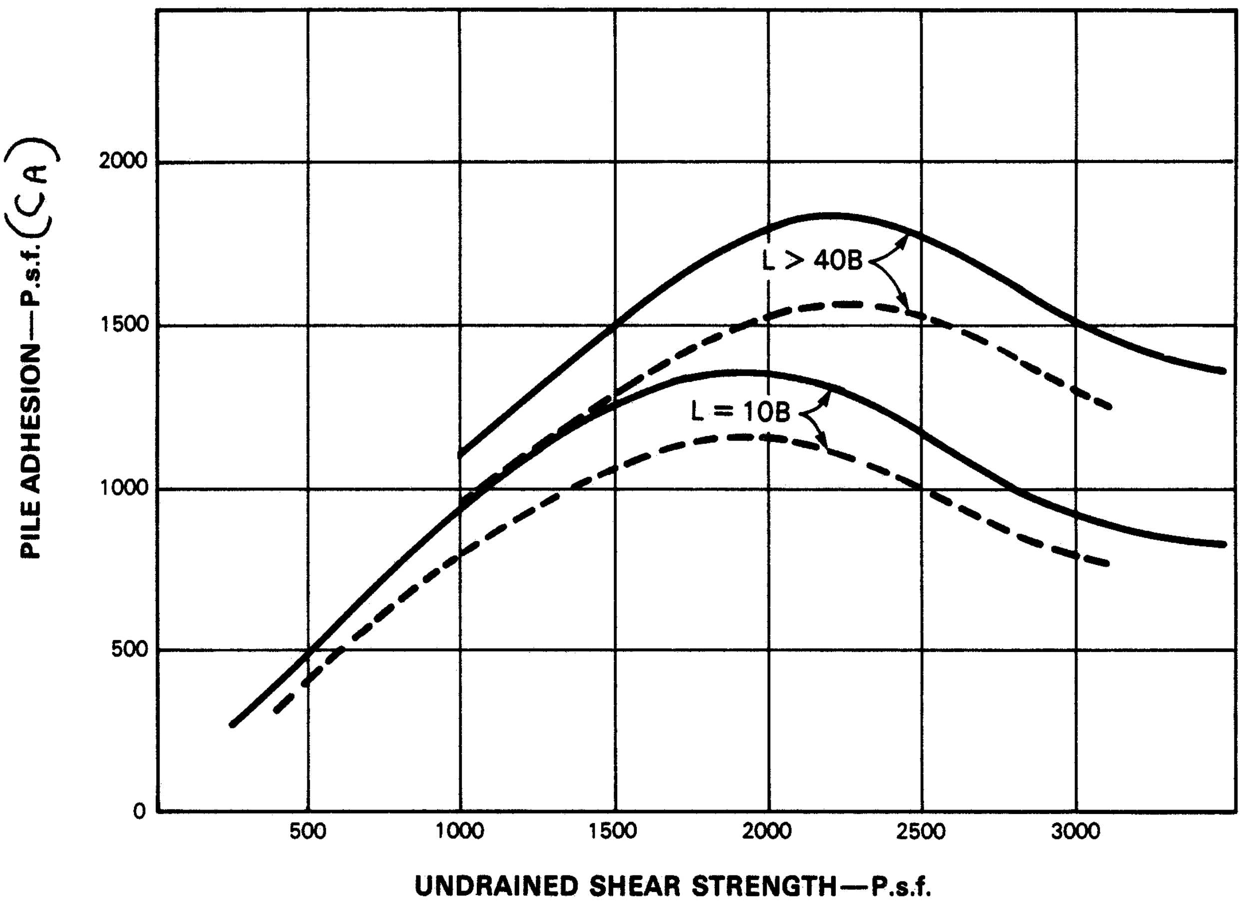
However, the Q_p value of 9 C_u A_p is usually taken as zero because substantial movement of the pile tip (~1/10 of the pile diameter) is needed to mobilize end bearing capacity.

In the case of H-piles, calculate:

1. The perimeter (C_d) as twice the sum of the widths of the flange and web.

2. The pile end area (A_p) as the product of the flange and web dimensions.

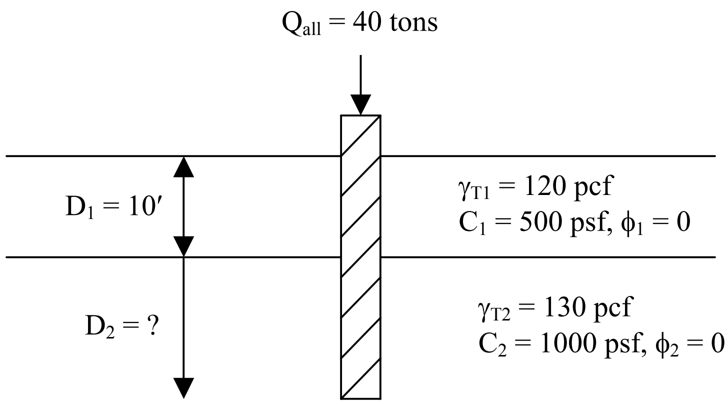
3. The adhesion (C_a) from the curves for corrugated piles in Figure 8-8 as the actual value involves two pile faces of smooth steel adhesion and two faces where pure soil shear occurs.



Legend: L = Distance from ground surface to bottom of clay layer or pile tip; whichever is less
B = Pile diameter
Concrete, timber, corrugated steel piles
Smooth steel piles

Figure 8-8: Adhesion values for piles in cohesive soils

Example 8-2: Determine the Required Pile Length To Resist A 40 Tons Load with A Safety Factor Of 2. Assume No Point Capacity For the 1′ Square Precast Concrete Pile.



Solution:

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

$C_{d1} = C_{d2} = 4 \times 1' = 4'$

From Figure 8-8

$C_{a1} = 500$ psf
$C_{a2} = 1100$ psf

$Q_u = 40 \text{ tons} \times 2 = 80 \text{ tons} = (500 \text{ psf})(4')(10') + (1100 \text{ psf})(4')D_2$

$$D_2 = \frac{80 - 10}{2.2} \approx 32'$$

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

8.5 PRACTICAL ASPECTS OF DRIVEN PILE DESIGN

Prediction of pile capacity is a powerful tool for the designer. Cost-effective pile types can be selected and pile lengths established in design with confidence, if common sense is used in applying the calculations.

First, the ultimate capacity, soil resistance contributing toward non-yielding support of the pile load, should be only considered below any unsuitable, compressible soil layers. The main reason piles are used is to transfer the structure load through poor soils to competent soils. Therefore, the resistance obtained in or above these layers should not contribute toward the required design load. For example, a pile to be driven through a dense sand layer overlying a soft clay layer and finally a deep gravel layer, should be designed to mobilize all necessary support capacity only in the gravel layer. Similarly, scour piles can only mobilize useful resistance below the expected scour depth.
Second, the driving capacity, soil resistance to be overcome to drive the pile to the required length, must be computed. This value is not the same as the design load which contains a safety factor and may

disregard soil resistance in overlying layers. This resistance may easily be obtained after the design pile length is established by adding up the static analysis values for all soil layers with consideration of soil strength loss due to driving in fine-grained soil layers. The driving capacity in fine-grained soil layers can be computed by dividing the static capacity of the soil layer by the soil sensitivity,ie., the ratio of undisturbed strength to the remolded strength. A special driving capacity case is the restrike capacity. Restrike capacity refers to the condition when all layers in the soil profile have had sufficient time to consolidate (setup) around the pile. No safety factor is applied to driving capacity because the actual soil resistance must be known by the designer to establish pile section or thickness, and for the contractor to estimate how big a hammer to use. In example 8-3, the static capacities computed in both the sand layer and gravel layer would be added to the capacity in the clay layer divided by the clay sensitivity.

Third, the designer should carefully choose the design safety factor used to establish the pile length required for a desired design load. The higher the safety factor in design, the greater the problem of pile installation in construction and the greater the need to perform wave equation analysis in design.

The safety factor selected should be based on both the quality of the subsurface information and the degree of construction control to be used for production pile driving. Assuming that procedures recommended in this manual are used for foundation investigation, the following safety factors on ultimate static capacity of piles supported by soil should be employed based on pile construction control method to be used. Piles supported on rock with RQD > 50 percent may use safety factor of 2.0 assuming driveability has been confirmed by wave equation analysis.

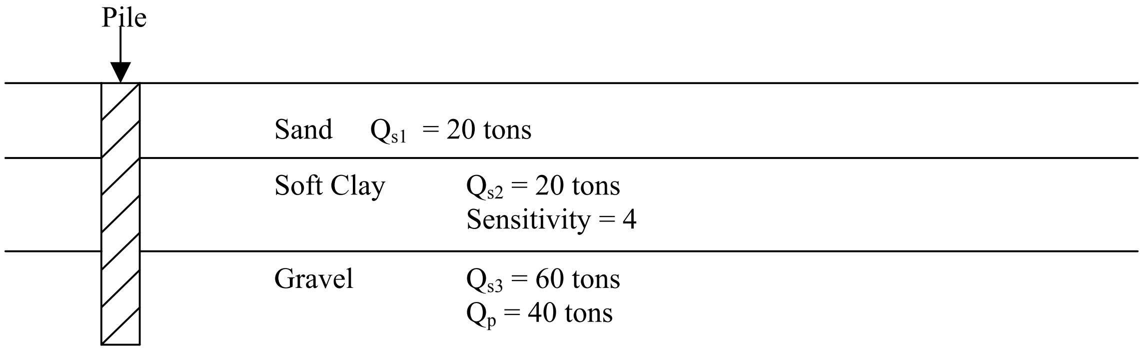
Control Method	**Safety Factor**
Static Load Test and Wave Equation	2.0
Dynamic Load Test and Wave Equation	2.25
Test Piles and Wave Equation	2.50
Wave Equation	2.75

Dynamic formula safety factors will be addressed in the construction control section.

8.5.1 Static Analysis Computer Programs

The user-friendly program SPILE was developed by FHWA in 1993 to permit rapid evaluation of the static capacity of alternate pile types. The user's manual is entitled "SPILE: Ultimate Static Capacity for Driven Piles" and is numbered FHWA TA-91-045. In 1998, a new Windows-based pile capacity program, DRIVEN, was developed to expand the capabilities of the SPILE program. The DRIVEN program program permits the user to enter the entire soil profile at a project. Based on this input, DRIVEN will calculate and plot pile capacities at predetermined intervals of depth for the profile depth. In addition the new analysis options featured in DRIVEN include multiple water tables, soft layer effects, scour effects, and open-end pipe pile design. Output options include ultimate capacity, driving capacity, and restrike capacity as well as an option to create a driveability file for subsequent wave equation analysis. The restrike capacity, which includes the setup capacity of all soil layers, is useful for evaluating the results of testing to verify post driving pile capacity.

Example 8-3: Find The Ultimate Capacity, The Driving Capacity And The Restrike Capacity For The Pile From The Static Capacity And Soil Values Listed In The Profile.



Solution:

Ultimate capacity = $Q_{s3} + Q_P = 60 + 40 = 100$ tons

Driving capacity = $Q_{s1} + (Q_{s2}/\text{Sensitivity}) + Q_{s3} + Q_P = 20 + \frac{20}{4} + 60 + 40 = 125$ tons

Restrike capacity = $Q_{s1} + Q_{s2} + Q_{s3} + Q_P = 20 + 20 + 60 + 40 = 140$ tons

8.6 DESIGN OF PILES FOR GROUP EFFECTS ON CAPACITY

The installation of a single pile results in disturbance in the surrounding soil that can extend outward a radial distance of six pile diameters depending on soil conditions and pile type. This disturbance can affect the capacity of piles within the group to carry load. The change in pile capacity due to group effects is commonly referred to as the efficiency of the pile group, E. The equation to determine the group capacity is:

$$P_{ult} = n\ E\ Q_{ult}$$

Where:

P_{ult} = group capacity
n = number of group piles
E = group efficiency, and
Q_{ult} = capacity of a single pile

Granular Soils

In the case of granular soil, the disturbance generally causes additional compaction that can result in increased capacity for piles in the group. However in difficult driving situations, contractors may use predrilling or jetting methods that reduce long-term capacity. Also battered piles are less effected by densification than non-battered piles.

Piles that are spaced at least 3 diameters center-to-center in granular soils general act as individual piles. Assuming no predrilling or jetting, the group capacity may be conservatively calculated as:

$$P_{ult} = n\ Q_{ult}$$

Cohesive Soils

Studies have shown fine-grained soils immediately adjacent to the driven pile are completely disturbed with lesser amount of disturbance extending beyond four pile diameters of the pile face. The magnitude of this disturbance is increased in pile groups as adjacent piles are driven. Driving solid cross section piles, such as closed end pipes, causes more disturbance than open-end pipe piles. Jetting or predrilling oversized holes with "mud" can also affect the soil adjacent to the pile and complicate load transfer. However, the time for consolidation of this disturbed zone is governed by the same general principle as previously used for spread footings except horizontal drainage is of primary importance.

$$t = \frac{TH^2}{C_h} \tag{6-4}$$

Where: C_h = The horizontal coefficient of consolidation.
T = The time factor

For single piles, H is usually one or two pile diameters. For closely spaced groups, H may be the distance to the group exterior or the maximum vertical drainage path. Consolidation is also aided by escape of water along the pile face, particularly if the pile material is timber or concrete. Practically, consolidation of this zone occurs on most projects before the contractor is ready to build the superstructure.

In clays, the long-term capacity of the piles in the group may also be affected depending on the spacing of the piles and whether the pile cap is in firm contact with the ground. In general the center-to-center spacing of piles in a group should not be less than 3 diameters. The issues of pile cap contact, pile batter, and special installation effects on capacity are beyond the scope of this manual. Conservative group capacity design can be achieved in cohesive soils as follows depending on pile center-to center spacing:

Spacing 3 diameters to 6 diameters; $P_{ult} = 0.7\ n\ Q_{ult}$
Spacing greater than 6 diameters; $P_{ult} = n\ Q_{ult}$

8.7 DESIGN OF PILES FOR LATERAL LOAD

The theory and design method for analyzing laterally loaded piles is beyond the scope of this basic manual. Guidance on lateral load analysis is provided in FHWA-IP-84-11 "Handbook on Design of Piles and Drilled Shafts under Lateral Load." FHWA also funded the development of a user-friendly computer program, COM624P, for lateral load analysis. The COM624P user's manual version 2.0 is numbered FHWA-TA-91-048.

8.8 SETTLEMENT OF PILE FOUNDATIONS

The analysis for settlement of pile foundations resembles that for spread footings in that both are based on those principles which govern soil consolidation. The major differences which must be considered in determining pile settlement magnitude and time for occurrence follow.

8.8.1 Transfer of load to soil

The load applied to a spread footing foundation is transmitted to the soil only from the bottom of the footing. The formula shown below easily can be used for spread footings as all terms in the equation are defined with respect to depth.

$$\Delta H = H \frac{C_c}{1+e_0} Log \frac{P_F}{P_0} \qquad (6\text{-}2a)$$

However, loads applied to pile foundations are transferred to the soil over the entire length of the pile as skin and point resistance. Referring to the formula for ΔH, neither the H nor P_F terms in the equation are well defined as varying proportions of the pile load are transferred to the soil at various depths depending on the consolidation properties of the soil. Fortunately, a simple method has been developed to approximate settlements due to subsoil consolidation for basic load transfer situations. This method, which is shown in Figure 8-9, is suitable for pile foundations installed in groups. Practically, pile groups present the greatest possibility of settlement as the total load applied to a group is transmitted deep below the pile tips whereas single piles transfer load in the immediate vicinity of the pile.

8.8.2 Effects of installation

Spread footings are placed in carefully prepared excavations where every effort is made not to disturb the foundation soil. Consolidation properties may be assumed to be as found from lab testing of undisturbed samples. Piles are most commonly installed by brutally forcing the pile below ground with a large hammer.

Long after installation, driven piles can retain large residual stresses which significantly influence load-settlement characteristics. Particularly noticeable in granular soils, these stresses tend to reduce observed settlements. Actual pile settlements in granular soils are generally negligible.

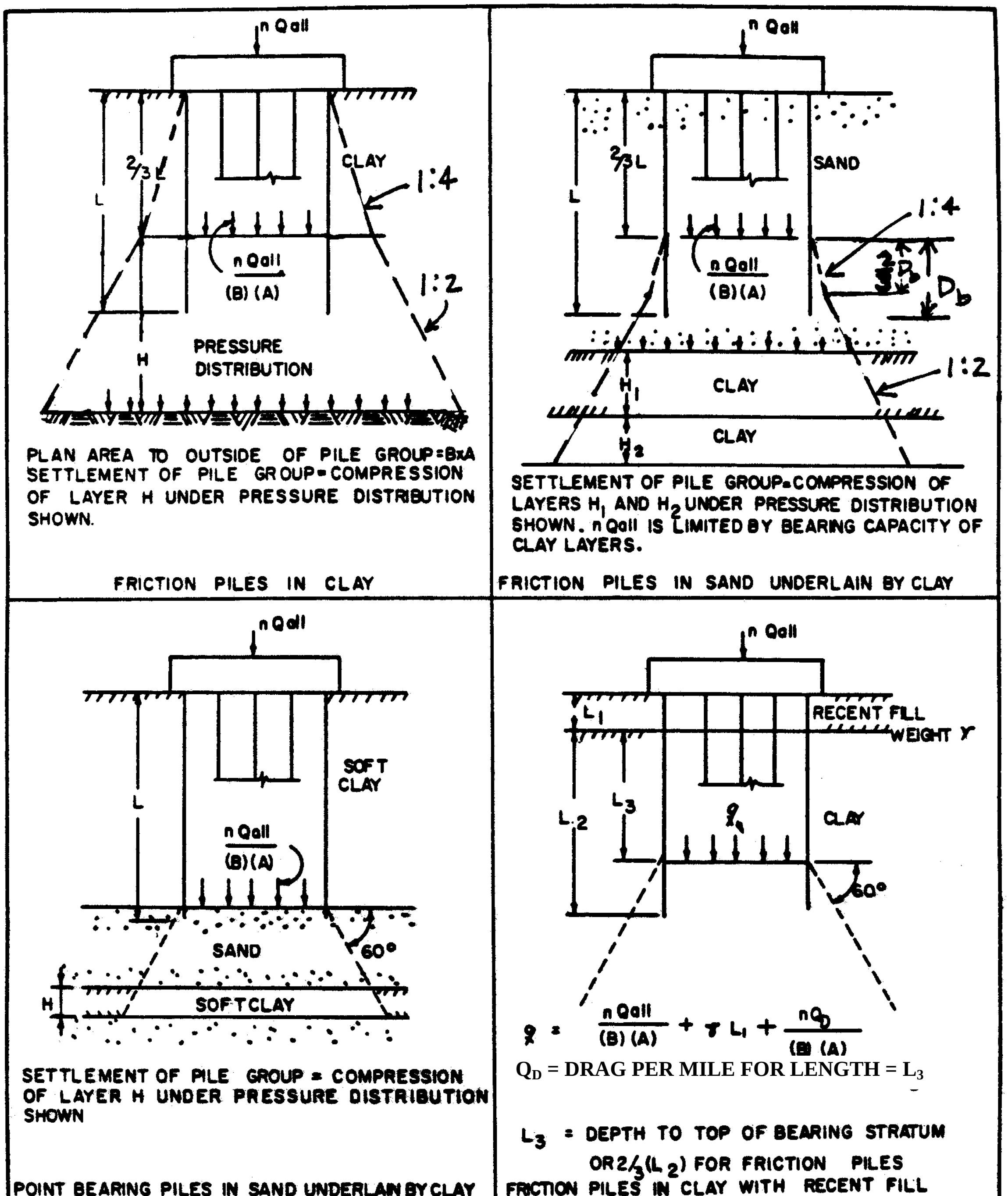
Practically, the foundation engineer should only be concerned about settlement of friction piles which terminate in cohesive soils and pile groups which terminate above compressible deposits. Settlement magnitude may be computed using Figure 8-9 and the methods shown for spread footings. Time rate of settlement can be estimated using the formula

$$t = \frac{TH_v^{\,2}}{C_v} \qquad (6\text{-}4)$$

Where: H_v is the maximum vertical drainage path in the clay layer(s) below the pile tips.
C_V is the coefficient of consolidation.

8.9 NEGATIVE SKIN FRICTION

Engineers typically think in terms of a pile transferring load to a soil which may consolidate and cause settlement. However, another mechanism called negative skin friction (or drag) exists, which may be the cause of large foundation movement. In this case compressible soil surrounding the pile consolidates under external loads and moves downward relative to the pile. This relative movement causes negative skin friction to develop and the soil load to be applied to the pile rather than vice versa. The most common instance of negative skin friction development is where fill is placed over a compressible deposit after piles have been driven. Such situations, if unanticipated, can cause large settlement and/or soil bearing capacity failure of friction piles or structural failure of end bearing piles. Negative skin friction can be particularly severe when end bearing, battered piles are affected because of additional torsional forces applied to the piles.



Notes:
1. Plan area to outside of pile group = B × A.
2. For relatively rigid pile cap, pressure distribution is assumed to vary with depth as shown.
3. For flexible slab or group of small separate caps, compute pressures by elastic solutions
4. Compute settlements by methods of Chapter 6

Figure 8-9: Settlement of pile groups

Computation of negative skin friction for cohesive soil involves use of the method previously shown in 8.4.2. Granular soils which are in contact with piles above the compressible deposit, can cause large forces. These granular drag forces can be estimated using the method shown previously for pile skin friction in granular soils. The amount of relative settlement between soil and pile that is necessary to mobilize drag is about 1/2 inch. At that movement the maximum value of drag is equal to the soil adhesion or friction resistance. The drag cannot exceed these values because slip of the soil along the pile occurs at this value. It is particularly important in the design of friction piles to determine the depth below which the pile will be unaffected by negative skin friction. Only below that depth can positive skin friction forces contribute to safely supporting the pile loading.

In past years, additional piles were added to structures to carry suspected drag loads. Such practice was very expensive as each pile contributed only a small portion of its structural capacity toward support of the structure. Recently, methods have been developed to protect the pile from negative drag loads. The most cost-effective treatment is the application of a slip-layer of bitumen to the pile portion which will be embedded in the zone of drag. Bituminous coatings can reduce drag by up to 90 percent. The major problem is protecting the coating during pile installation; especially through coarse surface soils. An inexpensive method of protecting the bitumen is to weld an oversized collar around the pile where the bitumen ends. The collar opens an adequate size hole to permit passage of the bitumen for moderate pile lengths. Additional information is included in NCHRP Project Report 24-5, Downdrag on Bitumen Coated Piles. Example of specification of bitumen coating of piles are shown in appendix G and H.

8.10 DRILLED SHAFTS

A drilled shaft is a machine (and/or hand) excavated shaft in soil or rock that is filled with concrete and reinforcing steel, with the primary purpose of structural support. A drilled shaft is usually circular in cross section and may be belled at the base to provide greater bearing area. A typical drilled shaft is shown in Figure 8-11. Other terminology commonly used to describe a drilled shaft includes: drilled pier, drilled caisson and bored pile. Rectangular drilled shafts are called barrettes.

Vertical load is resisted by the drilled shaft in base bearing and side friction. Horizontal load is resisted by the shaft in horizontal bearing against the surrounding soil or rock.

Characteristics:

The following special features distinguish drilled shafts from other types of foundations:

1. The drilled shaft is installed in a drilled hole, unlike the driven pile.

2. Wet concrete is cast and cures directly against the soil forming the walls of the borehole. Temporary steel casing may be necessary for stabilization of the open hole and may or may not be extracted.

3. The installation method for drilled shafts is adapted to suit the sub-surface conditions.

Advantages and Disadvantages of Drilled Shafts

1. Advantages

    a. Construction equipment is normally mobile and construction can proceed rapidly.
    b. The excavated material and the drilled hole can often be examined to ascertain whether or not

the soil conditions at the site agree with the projected soil profile. For end-bearing situations, the soil beneath the tip of the drilled shaft can be probed for cavities or for weak soil if desirable.

c. Changes in geometry of the drilled shaft may be made during the progress of the job if the subsurface conditions so dictate. These changes include adjustment in diameter and in penetration and the addition or exclusion of underreams.

d. The heave and settlement at the ground surface will normally be very small.

e. The personnel, equipment, and materials for construction are usually readily available.

f. The noise level from the equipment is less than for some other methods of construction.

g. The drilled shaft is applicable to a wide variety of soil conditions. For example, it is possible to drill through a layer of cobbles and for many feet into sound rock. It is also possible to drill through frozen ground.

h. A single drilled shaft can sustain very large loads so that a cap may not be needed.

i. Data bases which contain documented load transfer information are available that allow confident designs of drilled shafts to be made considering load transfer both in end bearing and in side resistance.

j. Use in constricted areas. The shaft occupies less area than the footing and thus can be built closer to railroads and existing structures.

k. Drilled shafts may be more economical than spread footing construction, especially when the foundation layer is deeper than 10' below the ground or at water crossings.

2. Disadvantages

a. Construction procedures are critical to the quality of the drilled shaft. Knowledgeable inspection is required.

b. Drilled shafts are not normally used in deep deposits of soft clay or in situations where artesian pressures exist.

c. Static load tests to verify ultimate capacity of large diameter shafts are very costly.

d. Lack of general knowledge of construction and design methods has restricted the use of drilled shafts and in some instances has led to improper design.

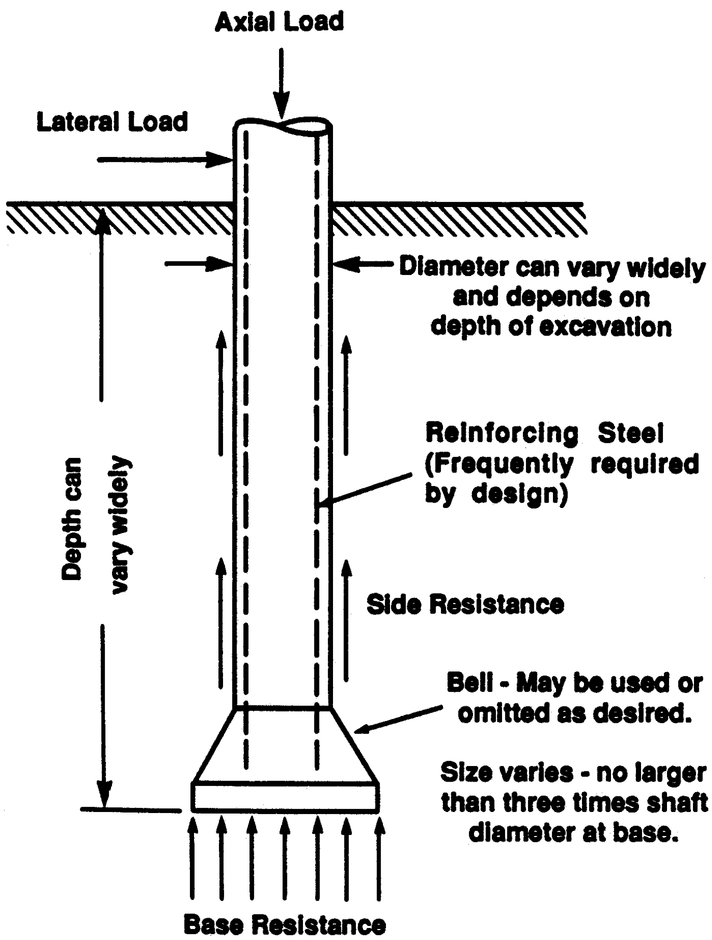


Figure 8-10: A typical drilled shaft

8.10.1 Design Procedure

Subsurface investigation for drilled shaft designs must include an assessment of the potential shaft construction methods as well as a determination of soil properties. The standard method for obtaining soil characteristics is similar to pile foundations and involves laboratory testing of undisturbed samples and the use of in situ techniques such as: the standard penetration test. Constructibility is difficult to assess from routine geotechnical investigations. Critical items such as hole caving, dewatering and obstructions can best be examined by drilling a full diameter test shaft hole during the exploration or design phase of the project. These test holes are usually done by local drilled shaft contractors under a short form contract from the highway agency. A detailed log should be made of the test hole including items such as type of drilling rig, rate of drilling, type of drill tools and augers used, etc. Such information should be made available for bidders. In addition, these test shaft holes may be cased with a "windowed" casing for inspection by designers and/or prospective bidders.

Subsurface Conditions Affecting Construction

a. The stability of the subsurface soils against caving or collapse when the excavation is made will determine whether a casing is necessary or not. The dry method of construction can be used only where the soils will not cave or collapse. The casing method must be used if there is danger of caving or collapse.

b. It must be determined if groundwater exists at the site and what rate of flow can be expected into a shaft excavation. The presence of groundwater will indicate if a tremie pour shaft will be needed or if a tremie seal must first be poured, the shaft dewatered, and then the remainder of the shaft poured in the dry. In either instance, the design must assure access to the top of the seal to allow the surface to be thoroughly cleaned prior to placing additional concrete. The shaft must be large enough to accommodate a worker or the top surface of a small diameter shaft seal must be located so that it is accessible.

c. Any artesian water conditions must be clearly identified in the contract documents. Artesian water flowing into a pour could spoil the concrete, or cause collapse or heaving of the soil at the excavation.

d. The presence of cobbles or boulders can cause difficulties in drilling. It is sometimes not easy to extract large pieces of rock, especially with the smaller diameter shafts.

e. The presence of existing foundations or structures.

f. Presence of landfill that could contain material that cannot be easily excavated, such as an old car body.

g. Presence of rock may require more sophisticated drilling methods or shooting with explosives.

h. Presence of a weak stratum just below the base of the drilled shaft. For this situation drilling may have to be extended below the weak stratum.

The total axial capacity of the drilled shaft is composed of two factors: the base capacity and the side capacity. The general formula is:

$$Q_T = Q_B + Q_S$$

Where: Q_T = Total axial capacity of the foundation
Q_B = Base capacity
Q_S = Side capacity

The procedures for estimation of drilled shaft capacity have improved significantly in the past decade. The major reason for this change is a data base has been developed on load transfer in skin friction and in end bearing. It is now well established that drilled shafts can carry a substantial portion of applied loads in skin friction. As with pile foundations, the ultimate skin friction is mobilized at a small downward movement of the shaft relative to the soil. End bearing resistance is developed in relation to the amount of deflection at the tip.

Separate analyses are required to determine skin friction and end bearing contributions in different soil types and rock. Details of these analyses can be found in FHWA publication IF 99-025 "Drilled Shafts: Construction Procedures and Design Methods." The general step by step outline of the design procedure which has been excerpted from that publication follows.

8.10.2 Step-by-Step Procedure for Drilled Shaft Design

1. Clay and Sand

   a. Develop soil profile, and obtain location of water table from available data.

   b. Obtain undrained shear strength of clay from laboratory testing of undisturbed specimens and/or from in-situ tests. Undrained shear strengths should be either those obtained from unconsolidated, undrained (UU) triaxial compression tests or should be converted from other test results to values that would have been obtained, approximately, had UU triaxial compression tests been conducted. For example, limit pressure values (P_L) from the pressuremeter divided by a theoretical cavity-expansion factor (of about 6) will normally lead to excessively high values of C_u and, ultimately, to unconservative design. Instead, (P_L) should be factored by a correlation factor that has been developed between C_u from UU triaxial tests and P_L for the soil formation under consideration.

   c. Obtain the N-values for sand from results of the Standard Penetration Test. These N-values are not to be corrected for fines or for overburden but should be the raw N-values obtained in the field.

   d. Review construction specifications and inspection procedures to ensure that high quality construction will be done.

   e. Obtain loadings for the drilled shafts, both axial and lateral. Take any possible downdrag into account.

   f. Select a factor of safety (or load and resistance factors), taking into account all of the pertinent information about the particular job. With good soil data the overall (global) factor of safety commonly ranges from 2 to 3.

   g. If clay exists at the ground surface:

      - Estimate the depth of the zone of seasonal moisture change and analyze for uplift.

      - If the depth of the zone of seasonal moisture change is more than 5 ft., consider eliminating skin friction to a depth greater than 5 feet.

      - If the lateral loads are significant, select a size and bending stiffness for the drilled shaft and compute the groundline deflection. If the computed deflection is more than 0.2 in., consider eliminating the clay skin friction to the first point of zero lateral deflection.

   h. Select the geometry of the drilled shaft and solve for the ultimate side and base resistances, employing appropriate equations for clays and sands.

      The ultimate base and side resistances are then divided by appropriate factors of safety and compared to the design load. For small-diameter shafts (base diameters less than 75 inches in clay or 50 inches in sand) or for shafts with base diameters that are large where reduced net ultimate base capacities (q_{br}) have been used, a global factor of safety can usually be applied. Otherwise, a partial factor of safety should be applied separately for side and end bearing values. Partial factors of safety should also be considered if there are significant differences in uncertainties of soil properties above the elevations of the bases of drilled shafts compared to those below the base elevation. Several design loading conditions must usually be considered, and the drilled shaft foundation should be sized for the most critical condition.

Where a global factor of safety is used, the design loads may be multiplied directly by the global factors of safety and compared with the sum of the ultimate side and base resistances to verify a trial geometry.

i. For the geometry selected and working load, compute short and long term settlements.

2. Rock

a. Perform subsurface explorations to obtain cores for laboratory strength testing, to obtain the RQD, to map the spacing and thickness of discontinuities, and to develop a profile of the subsurface conditions.

b. Obtain the compressive strength of the rock cores and the Young's modulus. If feasible, use the pressuremeter to obtain the Young's modulus of the rock mass.

c. Set up construction specifications to ensure a proper excavation, that excess loose material is removed when end bearing is part of design, and that the sides of the socket are roughened.

d. Obtain design loadings for the drilled shaft, both axial and lateral.

e. Select a global factor of safety, taking into account the fact that detailed information on discontinuities is very difficult to obtain and that the behavior of a drilled shaft is strongly influenced by the nature of the discontinuities.

f. Obtain values of ultimate loads by multiplying the design loads by the selected global factor of safety.

g. Select the trial geometry of the drilled shaft.

h. If the rock is weak (compressive strength of less than 750 psi), the design should depend on load transfer in side resistance. The design should follow procedures in FHWA IF 99-025 for load transfer in intermediate geomaterials. The settlement should be checked to see that it does not exceed 0.4 inches.

i. If the rock is strong, the design should be made on the basis of end bearing. The settlement under working load should not exceed the allowable settlement as dictated by the superstructure.

j. A load test program should be considered if there are serious questions about the quality of the rock.

8.10.3 Construction Methods

1. Dry Method

The dry method is applicable to soils above the water table that will not cave or slump when the hole is drilled to its full depth. A soil that meets this specification is a homogeneous stiff clay. The dry method can be employed in some instances with sands above the water table if the sands have some cohesion, or if they will stand for a period of time because of apparent cohesion.

The dry method can be used for soils below the water table if the soils are low in permeability so that only a small amount of water will seep into the hole during the time the excavation is open.

The dry method consists of drilling a hole using an auger or bucket drill, without casing, placing a rebar cage and then filling the hole with concrete.

2. Casing Method

The casing method is applicable to sites where soil conditions are such that caving or excessive deformation will occur when a hole is excavated. An example of such a site is a clean sand below the water table.

This method employs a cylindrical (usually steel) casing inside the hole to hold back the caving soil. The excavation is made by driving, vibrating, or pushing down a heavy casing to the proposed founding level and by removing the soil from the casing either continuously as driving proceeds or in one sequence after the casing has reached the founding level. The casing is sometimes left in the hole after the concrete is poured.

3. Slurry Displacement Method

This method is gaining in popularity. A bentonite slurry is introduced into the excavated hole to prevent caving or deformation of loose or permeable soils. Drilling continues through the slurry using an auger or clamshell mounted on a Kelly bar. When the desired depth is reached the rebar cage is lowered into the slurried hole. Concrete is then tremie-poured into the hole. Slurry is displaced by the heavier concrete and collected at the surface in a sump. The slurry may again be used in another hole.

8.10.4 Drilled Shaft Publications

FHWA publication "Drilled Shafts: Construction Procedures and Design Methods," FHWA-IF 99-025 contains information on vertical loading. FHWA publication, “Load Transfer for Drilled Shafts in Intermediate Geomaterials”, FHWA RD 95-172 contains information for vertical on weak rock. For lateral load information, consult either the "Handbook on Piles and Drilled Shafts Under Lateral Load'" FHWA-IP-84-11 or COM624P User's Manual Version 2.0, FHWA-TA-91-048. Consult Drilled Shafts: Construction Procedures and Design Methods, FHWA HI-88-042, for additional information on construction.

8.11 APPLE FREEWAY DESIGN EXAMPLE – PILE DESIGN

In this chapter the Apple Freeway is used to illustrate the pile design for support of the pier and abutment. Although drilled shafts may also be a feasible deep foundation design alternate for this structure, the details of the drilled shaft design are beyond the scope of this manual. The computation process for static analysis to determine pile capacity by Nordlund Method is presented along with the computation of pile driving resistance.

Site Exploration	Terrain Reconnaissance Site Inspection Subsurface Borings
Basic Soil Properties	Visual Description Classification Tests Soil Profile
Laboratory Testing	Po Diagram Test Request Consolidation Results Strength Results
Slope Stability	Design Soil Profile Circular Arc Analysis Sliding Block Analysis Lateral Squeeze
Embankment Settlement	Design Soil Profile Settlement Time – Rate Surcharge Vertical Drains
Spread Footing Design	Design Soil Profile Pier Bearing Capacity Pier Settlement Abutment Settlement Vertical Drains Surcharge
→	**Pile Design**
Construction Monitoring	Wave Equation Hammer Approval Embankment Instrumentation

Design Soil Profile
Static Analysis – Pier
- Pipe Pile
- H – Pile

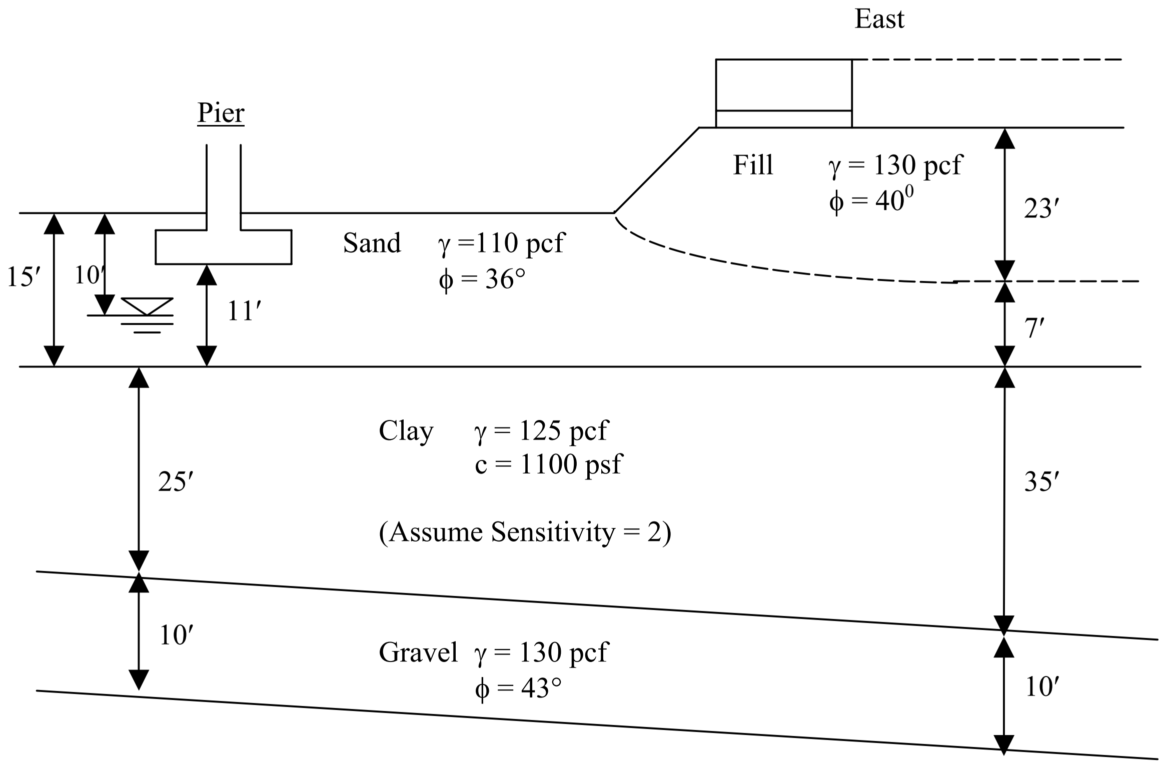
Static Analysis – abutment
- Pipe Pile
- H – Pile

Driving Resistance
Abutment Lateral Movement

Apple Freeway Design Example – Pile Design
Exhibit A

Given: The subsurface profile and soil properties shown below.

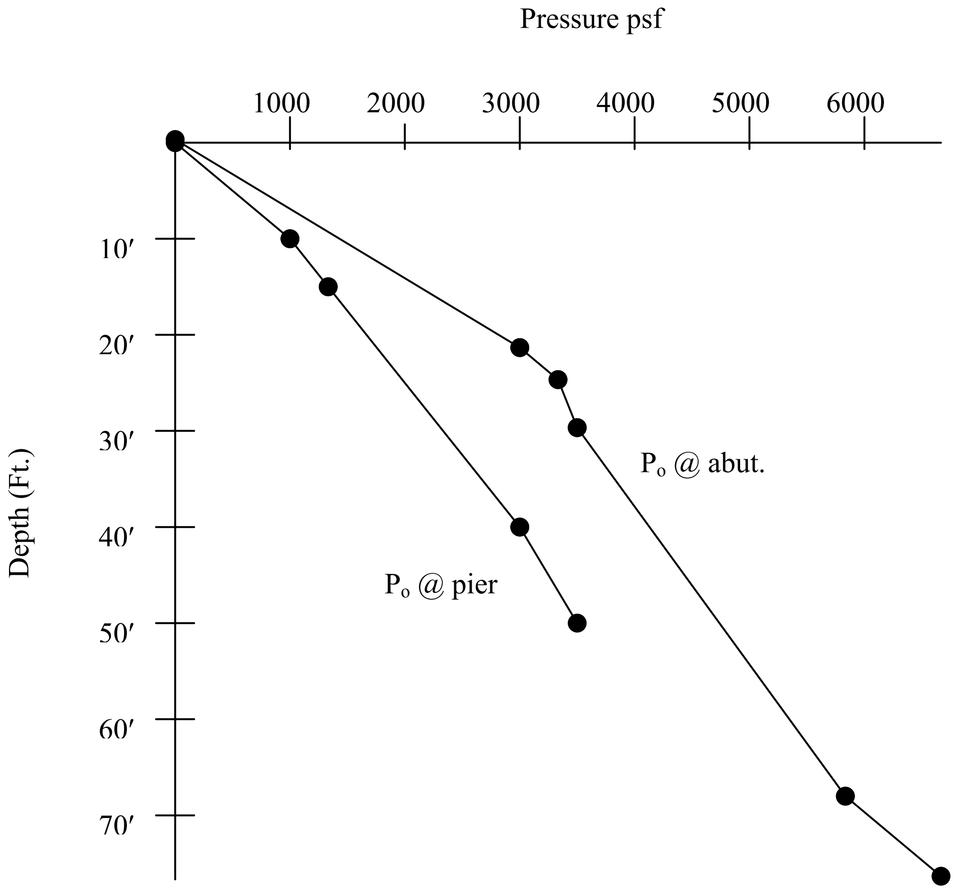
Required: Determine the allowable pile capacity using static analysis.



Consideration for Pile Type Selection:

- Spread footings would be feasible at both the pier and abutment except for settlements due to consolidation of the clay deposit. To eliminate these settlements any pile type selected must achieve capacity below the bottom of the clay deposit.
- End bearing will provide most of the ultimate resistance at either the pier or abutment location due to the minimal thickness of the gravel layer.
- The maximum estimated structural load of 2200 tons can be supported by either the gravel or the rock layer. However pile driveability appears to be an issue in design.
- Required loads, end bearing support, and difficult driving concerns would favor a straight-sided steel pile over either a timber or concrete or tapered pile.
- Static analyses will be used to determine if a displacement pile with an end plate or a non-displacement pile will provide the best choice for both bearing and driveability. The designer selected a 12” diameter closed end pipe pile and a 12” H-pile for alternate evaluation at this site. The structural engineer usually designs the pipe pile for a 70-ton design load and the H-pile for a 120-ton design load.

Step 1: Plot P_o diagram.



Static Pile Analysis – Pier

<u>A. For 12″ diameter pipe pile (Closed end, 70 ton design)</u>

Step 2A: Compute skin resistance.

- Sand Layer

4′ - 15′ Use Nordlund Method

D = 15 – 4 = 11′

$$V = \frac{\text{pile vol.}}{\text{foot}}$$

$$= \frac{\pi d^2}{4} = 0.785 \,{}^{\text{CF}}\!/_{\text{Ft.}}$$

$$\delta/\phi = 0.6$$

$\delta = (0.6)(36°) = 21.6°$

$K_\delta = 1.75$

Corr. Factor = 0.75

$C_d = \pi\, d = \pi\,(1) = 3.14'$

P_0 Avg. @ $\dfrac{4+15}{2} \sim 9.5' = 1050$ psf

$q_s = K_\delta\,(C_F)(P_0)(C_d)(\text{Sin}\ \delta)D$
$\quad = (1.75)(0.75)(1050)(3.14)(\text{Sin}\ 21.6)\ 11$
$q_s = 8.7$ tons

- Clay Layer

15′ - 40′

$q_s = C_a C_d D$

C_a (Adhesion) $\cong 1100$ psf

$q_s = 1100\ (3.14)\ 25 = 43.1$ tons

- *Gravel Layer (Try 4′ Embedment)

*Remember to reduce ϕ of 43° to maximum 36° value for hard, angular gravel skin friction.
40′ - 44′.

$q_s = (K_\delta)(C_F)(\bar{P}_o)(C_d)(\text{Sin}\ \delta)D$

$\delta/\phi = 0.6$

$\delta = (0.6)(36°) = 21.6°$

$K_\delta = 1.75$

$C_F = 0.7$

$P_0 = 3200$ psf

$q_s = (1.75)(0.7)(3200)(3.14)(\text{Sin}\ 21.6°)4$
$q_s = 9$ tons

Step 3A: Compute end bearing.

- Gravel Layer

P_o Max.

a. $Q_p = A_p \alpha P_D N'_q$
$= (0.785)(0.75)(3000)(300)$
$Q_p = 265$ tons

b. Q_{lim} = (Limiting Point Resist.) $\times A_p$
$= (320 \text{ tsf})(0.785)$
$Q_{lim} = 251$ tons

$\therefore Q_p = 251$ tons

It is obvious that any embedment in gravel layer will produce capacities > 200 tons. Therefore, estimate pile length to top of gravel.

Step 4A: Determine driving resistance (For 70 ton load with SF = 2).

$$Q_{drive} = q_{sand} + \frac{q_{clay}}{\text{sensitivity}} + (70^T \times 2)$$

$$= 8.7^T + \frac{43.1^T}{2} + 140^T$$

$Q_{drive} = 170^T$

<u>B. For *12″ H-Pile (120 ton Design Load)</u>
<u>*Assume 12x84 H-Section</u>

Step 2B: Compute Skin Resistance

- Sand Layer

4′ - 15′

$$V = \frac{24.6}{144} = 0.17 \, {}^{CF}\!/_{Ft.}$$

$$\delta/\phi = 0.80$$

$\delta = (0.80)(36°) = 28.8°$

$K_\delta = 1.30$

$C_F = 0.92$

$C_d = 4′$

$P_0 = 1050$

$q_s = (1.30)(0.92)(1050)(4)(\sin 28.8^o)\ 11$
$q_s = 13.0$ tons

- Clay Layer

15′ - 40′

$q_s = C_a C_d D$

$C_a = 1100$ psf
$q_s = 1100\ (4)\ 25 = 55$ tons

- *Gravel Layer (Try 4′ Embedment)

*Use $\phi_{Max} = 36°$

40′ - 44′

$q_s = (K_\delta)(C_F)(P_0)(C_d)(\sin \delta)D$

$$V = \frac{24.6}{144} = 0.17\, {}^{CF}\!/\!_{Ft.}$$

$${}^{\delta}\!/\!_{\phi} = 0.80$$

$\delta = (0.80)(36°) = 28.8°$

$K_\delta = 1.30$

$C_F = 0.92$

$C_d = 4'$

$P_0 = 3200$ psf

$q_s = (1.30)(0.92)(3200)(4)(\sin 28.8°)4$
$q_s = 14.7$ tons

Step 3B: Compute End Bearing (Use $\phi = 43°$) at 44′

a. $$Q_p = A_p \alpha P_D N'_q$$
$$= \frac{24.6}{144}(0.78)(3000)(300)$$
$$Q_p = 60 \text{ tons}$$

b. $Q_{lim} = q_{lim}A_P$

$= (320\text{ tsf})\ \dfrac{24.6}{144}$

$Q_{lim} = 54.7$ tons

$\therefore\ Q_p = 54.7$ tons

- Total useable soil capacity below clay is = $14.7^T + 54.7^T = 69.4$ Tons
- Total Required capacity is 240 Tons
- Extending pile to 50′ only increases Q_s to 37^{Tons}

Conclusion: Pile must bear on rock to develop 240 Tons capacity below clay layer. Therefore estimate pile length to rock.

Step 4B: Compute H – Pile Driving Resistance

$$Q_{driving} = Q_{lim} + \frac{q_{clay}}{sensitivity} + (120T \times 2)$$

$$= 13 + \frac{55}{2} + 240$$

$Q_{driving} = 280.5$ Tons

* Composed of 37 t skin friction in the gravel and 203 t in end bearing on rock.

STATIC PILE ANALYSIS - ABUTMENT @ STA 93 + 50

<u>A. For 12″ Diameter Pipe Pile</u>

Step 2A & 3A: Based on computation at the pier, the pipe pile will develop the 140 ton ultimate load at the top of the gravel layer, ie. an estimated length of 65′. However the driving resistance will increase.

Step 4A: Compute driving resistance.

- Fill (Use $\phi_{Max} = 36°$)

$q_s = (K_\delta)(C_F)(P_0)(C_d)(\sin\delta)\ C_d\ D$

$V = 0.785$ CF/Ft.

$\delta/\phi = 0.6$

$\delta = (0.6)(36°) = 21.6°$

$K_\delta = 1.75$

$C_F = 0.75$

$P_0 = 1650$ psf

$q_s = (1.75)(0.75)(1650)(3.14)(\text{Sin } 21.6°)23$
$q_s = 28.8$ tons

- Sand

$q_s = (1.75)(0.75)(3330)(3.14)(\text{Sin } 21.6°)7$
$q_s = 17.5$ tons

- Clay

$$q_s = \frac{C_a C_d d}{\text{sensitivity}}$$

$$q_s = \frac{(1100)(3.14)(1)35}{2} = 30.2 \text{ tons}$$

Driving resistance @ top of gravel 28.8 + 17.5 +30.2 + 140 = 216.5 tons

Step 5A: Check driving resistance in embankment.

Assume pile tip embedded 23′

$q_s = 28.8$ tons (from Step 4A)

$Q_p = A_p \alpha P_{0\,23'} N'q$
$= (0.785)(0.74)(2990)170$
$Q_p = 147.6^{\text{tons}} < q_{lim} = 200(0.785) = 157$ tons

$Q_{\text{Drive Emb}} = 28.8 + 147.6 = 176.4$ tons

* Pre augering may be required.

B. 12″ H – Pile (120^T design) -- 12×84 section

Steps 2B & 2C Estimate length to rock ie. 75′, as pier computation showed H-pile must bear on rock to achieve designed ultimate capacity.

Step 4B: Compute driving resistance.

- Fill

V = 0.17 CF/Ft.

$\delta/\phi = 0.80$

$\delta = (0.80)(\underset{\phi_{max}}{36°}) = 28.8°$

$K_\delta = 1.30$

$C_F = 0.92$

$q_s = (1.30)(0.92)(1495)(4)(\text{Sin } 28.8°)23$
$q_s = 39.6$ tons

- Sand

$q_s = (1.30)(0.92)(3303)(4)(\text{Sin } 28.8°)7$
$q_s = 26.6$ tons

- Clay

$$q_s = \frac{C_a C_d d}{\text{sensitivity}}$$

$$q_s = \frac{(1100)(4)35}{2} = 38.5 \text{ tons}$$

- Gravel

$q_s = (1.30)(0.92)(5600)(4)(\text{Sin } 28.8°)10$
$q_s = 64.5$ tons

$Q_{Drive} = 39.6 + 26.6 + 38.5 + [\underbrace{64.5 + 175.5}_{240^T}]$

$Q_{Drive} = 344.7$ tons

Step 5B: Check H – Pile driving resistance in embankment

Assume pile tip embedded 23′

$q_s = 39.6$ tons (From Step 4B)

$q_p = A_p \alpha P_{0\ 23'} N'q$

$$q_p = \frac{24.6}{144}(0.74)(2990)170$$

$q_p = 32.1 \text{ tons} < q_{lim} = (200)(0.17) = 34 \text{ tons}$

$Q_{Drive\ Emb.} = 39.6^T + 32.1^T = 71.7$ tons

* Preaugering may not be required.

<u>Summary of the Pile Design Phase for the Apple Freeway Design Problem</u>

- Design Soil Profile

  Strength value selected for all layers.

- Static Analysis - Pier

  12" - 70 T Pipe Pile - 36' length required
  12" - 120 T H-Pile - 46' length required.

- Static Analysis Abutment

  12" - 70 T Pipe Pile - 65' length required
  12" - 120 T H-Pile - 75' length required.

- Driving Resistance

  Driving Resistances computed for both pipe and H-piles to permit design check of pile section overstress.

  Pipe pile will require pre-augering through embankment.

- Abutment Lateral Movement

  3" possible horizontal movement even with a pile foundation unless recommended waiting period observed prior to pile driving.