
CHAPTER 19

DRILLED PIERS OR CAISSONS

19-1 INTRODUCTION

The *drilled pier* is constructed by drilling a cylindrical hole of the required depth and subsequently filling it with concrete. The shaft may be straight or the base may be enlarged by underreaming. This structural member is also termed as follows:

- a. Drilled shaft
- b. Drilled caisson (or sometimes, simply, a caisson)
- c. Bored pile (but usually restricted to $D < 760$ mm)

If the base is enlarged the member takes one of these names:

- d. Belled pier (or belled caisson)
- e. Underreamed foundation

These several configurations are shown in Fig. 19-1.

The term *caisson* is also used to describe large prefabricated box-type structures that can be sunk through soft ground or water at a site to provide a dry work space.

This chapter will focus primarily on the analysis and design of drilled piers.

19-2 CURRENT CONSTRUCTION METHODS

Early drilled piers were constructed by digging the shaft and/or bell by hand although drilling methods using horse power were in use in the early 1900s. Early methods include the Chicago and Gow methods shown in Fig. 19-2. In the Chicago method, workers excavated a circular pit to a convenient depth and placed a cylindrical shell of vertical boards or staves held in place by an inside compression ring. Excavation depth then continued to the next board length and a second tier of staves was set, etc., to the required shaft depth. The tiers could be set at a constant diameter or stepped in about 50 mm.

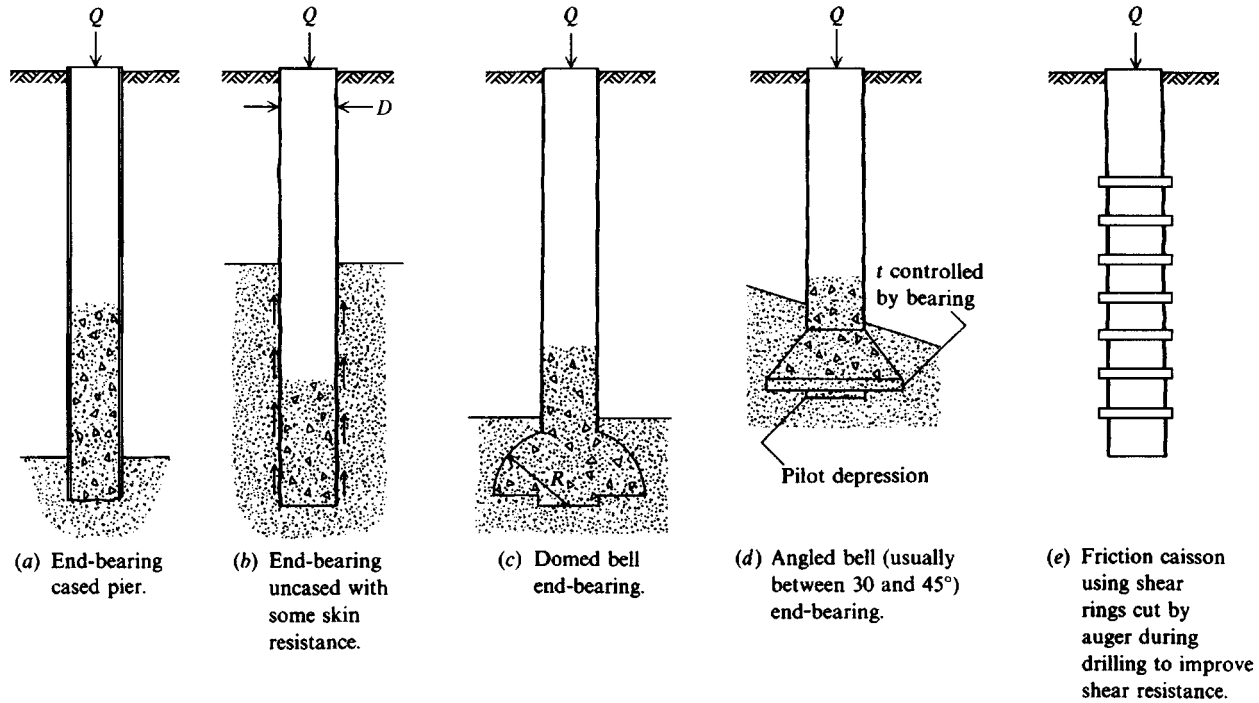


Figure 19-1 Common drilled pier configurations. Such a structure is considered a pile if shaft diameter $D < 760$ mm; a pier if $D > 760$ mm.

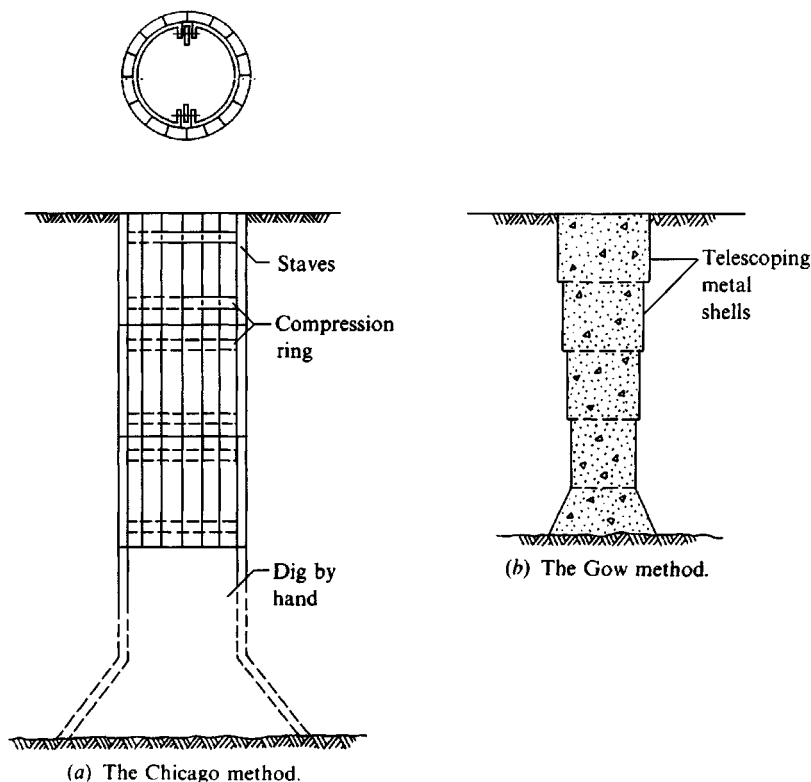


Figure 19-2 Early methods of caisson construction.

The Gow method, which used a series of telescoping metal shells, is about the same as the current method of using casing except for the telescoping sections reducing the diameter on successive tiers.

The shaft base can be enlarged for additional bearing if the base soil does not cave (i.e., if founded in a fairly stiff nonfissured clay). Many of the early piers were founded on rock.

Drilled piers—particularly large-diameter ones—are not often used in groups. Most often a drilled pier interfaces with a single column carrying a very large superstructure load. Reinforcing bars may be required either for the full pier depth L_p or only in the upper moment-active zone (about $L_p/2$). The rebars—if used—are to carry any tensile Mc/I stress from shaft moment. Reinforcing bars may not be required in those cases where the pier requires steel casing that is filled with concrete to form a metal-encased shaft.

The shaft moment may result from using a fixed-base column, from accidental misalignment of the load-carrying column with the pier shaft (a $P-\Delta$ type effect not known at the time the pier is designed), or from lateral loads from the superstructure (which are usually known). Since the pier shaft is embedded in the soil, where its temperature is a relatively constant value, T&S steel is used only as a designer prerogative or if the local building official requires its use.

The reinforcing bars are usually prewired—including vertically spaced tie bars—into a designed pattern called a *rebar cage*, which can be set as a unit into the pier shaft cavity into about 1 m of previously poured concrete (so that the bars are not in contact with earth) and

the remaining space filled with concrete to form a vertically reinforced structural member. Where the rebars are not required for the full depth, some concrete is placed, the rebar cage is set, and then the shaft pour is continued.

The shaft supports for the Chicago and Gow methods were usually left in place since the pier did not rely on shaft friction. Furthermore, they were not very easy to remove after the concrete had been poured.

Currently, labor and insurance costs for hazardous conditions preclude hand digging shafts, so machine digging is universally used. There are three basic methods (site variables may require a mix of methods, however).

1. DRY METHOD. Here the production sequence is as in Fig. 19-3. First the shaft is drilled (and belled if required). Next the shaft is partly filled with concrete as in Fig. 19-3*b* with the rebar cage then set and the shaft completed. Note that the rebar cage should never go all the way to the bottom, for a minimum concrete cover is required, but it may extend nearly the full shaft depth rather than approximately one-half as shown here.

This method requires site soils be noncaving (cohesive) and the water table be below the base or the permeability so low the shaft can be drilled (pumped possibly) and concreted before it fills with sufficient water to affect the concrete strength.

2. CASING METHOD. This method is outlined in Fig. 19-4. Casing is used at sites where caving or excessive lateral deformation toward the shaft cavity can occur. It is also used where it is desired to seal the hole against groundwater entry but to do this requires an impermeable stratum below the caving zone into which the casing can be socketed. Note that until the casing is inserted, a slurry is used to maintain the hole. After the casing is seated the slurry is bailed out and the shaft extended to the required depth in the dry stratum. Depending on the site and project requirements the shaft below the casing will be decreased to at least the ID of the casing—sometimes 25 to 50 mm less for better auger clearance.

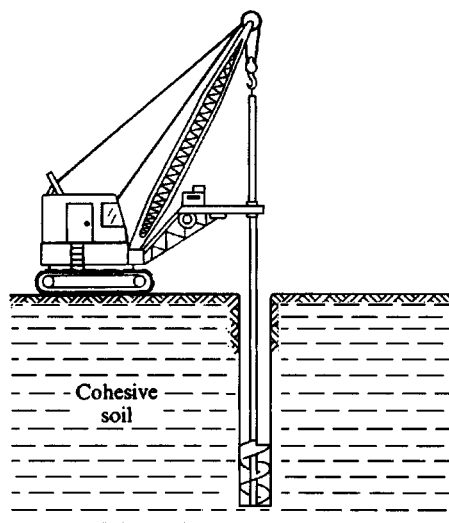
The casing may be left in place or pulled. If it is left in place the annular space between casing OD and soil (currently filled with slurry or drilling fluid) is displaced with pressure-injected grout (a cement + water + additives) mixture. By inserting a tube to the base of the slurry and pumping grout the slurry is displaced over the top so the void is filled with grout.

Alternatively, the casing can be pulled but with great care to ensure the following:

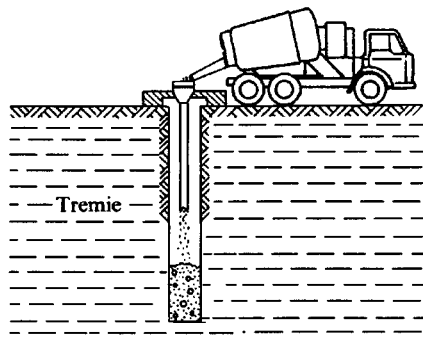
- a. Concrete inside casing is still in a fluid state.
- b. Concrete “head” is always sufficiently greater than the slurry head that concrete displaces slurry and not vice versa.

Pulling the casing may result in a substantially oversize top shaft zone—depending on how close the casing OD and initial shaft ID match. The oversize is seldom of consequence but may need to be known so that the total shaft volume can be compared to concrete volume used to ensure the shaft does not contain any accidental voids. The change in shaft diameters will produce an increase in capacity from the ledge-bearing Q_L .

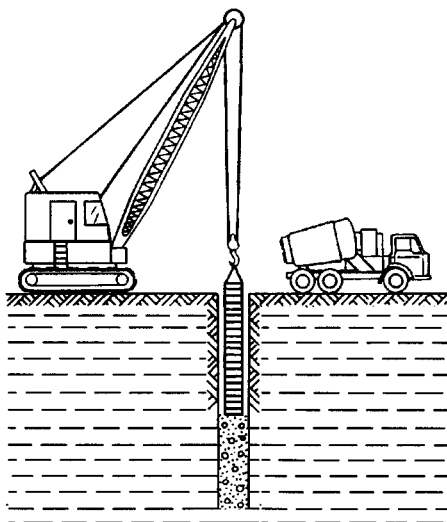
3. SLURRY METHOD. This method is applicable for any situation requiring casing. It is required if it is not possible to get an adequate water seal with the casing to keep groundwater out of the shaft cavity. The several steps are outlined in Fig. 19-5. Note that it is essential in this method that a sufficient slurry head is available (or that the slurry density can be increased



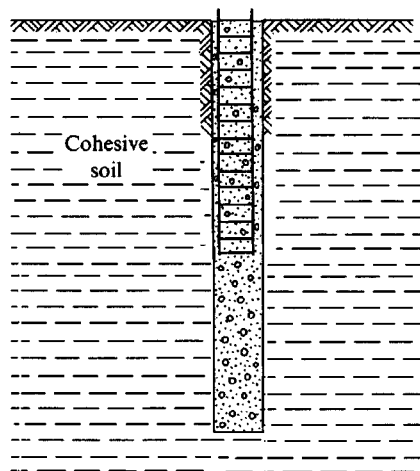
(a) Drill shaft to required depth.



(b) Place concrete through tremie (and use limited free fall).



(c) Pull-out tremie and set rebar cage to depth required.



(d) Completed shaft.

Figure 19-3 Dry method of drilled pier construction.

as needed) so the inside pressure is greater than that from the GWT or from the tendency of the soil to cave. Many of the considerations of slurry trench construction discussed in Sec. 14-9 are equally applicable here.

Bentonite is most commonly used with water to produce the slurry ("bentonite slurry") but other materials (admixtures) may be added. Some experimentation may be required to obtain optimum percentage for a site, but amounts in the range of 4 to 6 percent by weight of admixture are usually adequate.

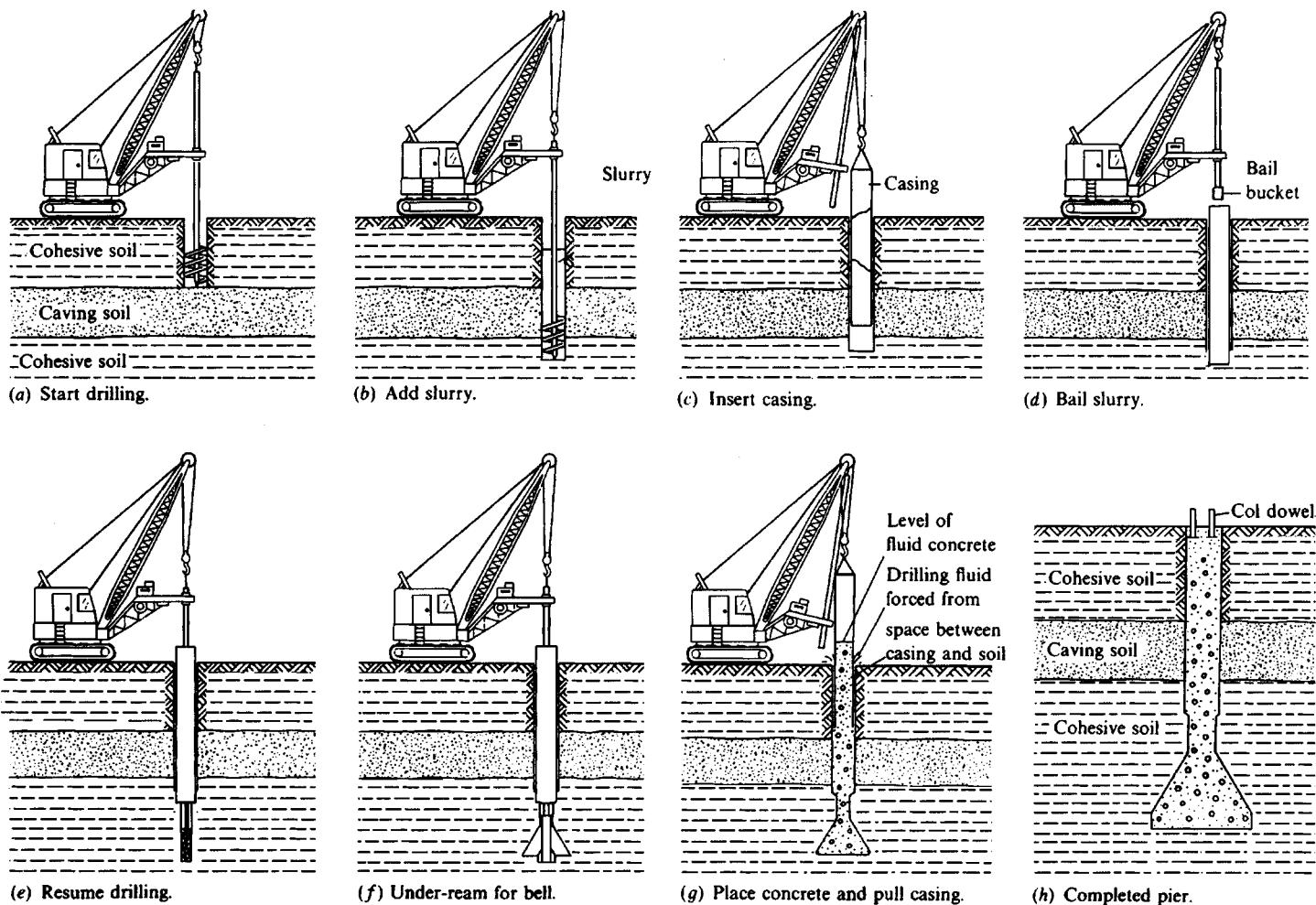
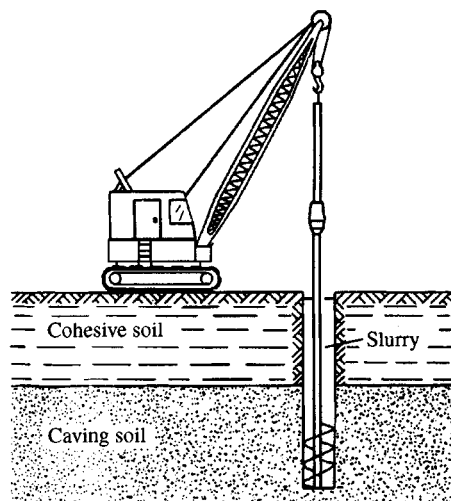
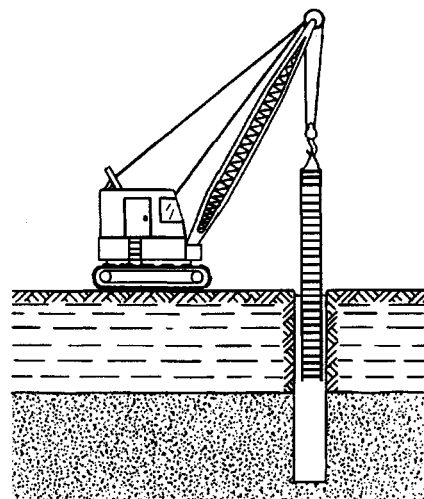


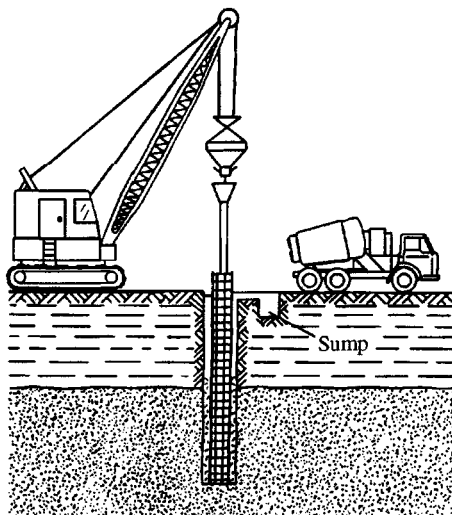
Figure 19-4 Casing method of drilled pier construction.



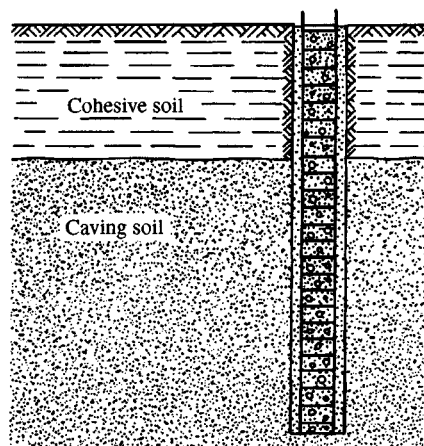
(a) Drill into caving soil and add slurry as necessary for adequate head and to required depth.



(b) Pull drill and insert rebar cage.



(c) Add tremie and pump cement.
Catch displaced slurry in sump pit.



(d) Completed shaft.

Figure 19-5 Slurry method of drilled pier construction.

The bentonite should be well mixed with water so that the mixture is not lumpy. The slurry should be capable of forming a filter cake on the shaft wall and of carrying the smaller (say, under 6 mm) excavated particles in suspension. Sometimes if the local soil is very clayey it may be used to obtain an adequate slurry. The shaft is generally not underreamed for a bell since this procedure leaves unconsolidated cuttings on the base and creates a possibility of trapping slurry between the concrete base and bell roof.

With the slurry method the following are generally desirable:

- a. Not have slurry in the shaft for such a long time that an excessively thick filter cake forms on the shaft wall; a thick cake is difficult to displace with concrete during shaft filling.
- b. Have the slurry pumped and the larger particles in suspension screened out with the “conditioned” slurry returned to the shaft just prior to concreting.
- c. Exercise care in excavating clay through the slurry so that pulling a large fragment does not cause sufficient negative pore pressure or suction to develop and collapse a part of the shaft.

When the shaft is complete the rebar cage is set in place and a tremie installed (this sequence is usually necessary so that the tremie does not have to be pulled to set the cage and then reinserted—almost certain to produce a slurry-film discontinuity in the shaft). Concrete made using small aggregate is pumped and great care is taken that the tremie is always well submerged in the concrete so a minimum surface area is exposed and contaminated with slurry. It appears that the concrete will adequately displace slurry particles from the rebar cage so a good bond can be obtained, and as previously noted, if the shaft is not open too long the filter cake is reasonably displaced as well.

19-3 WHEN TO USE DRILLED PIERS

Drilled piers can be used in most cases requiring pile foundations. Where the site soil requires use of deep foundations one should make a comparative analysis to determine whether piles or drilled piers are more economical.

Drilled piers have the following direct advantages:

- a. They eliminate the need for pile caps, for dowels can be placed in the wet concrete at the required plan location (even if pier center is slightly misaligned) for direct column attachment.
- b. They use fewer but with larger diameter shafts.
- c. Their use eliminates much of the vibration and noise associated with pile driving.
- d. They can go through a boulder soil where driven piles might be deflected. Boulders of size less than about one-third the shaft diameter can be directly removed. Others may be broken with special tools, or a temporary casing can be installed to give access for hand drilling and blasting larger rocks.
- e. It is easy to enlarge the top portion of the pier shaft to allow for larger bending moments.
- f. Almost any diameter shaft in the range of 0.460 to 3.5 m can be produced.
- g. Larger-diameter shafts (if cased) allow direct inspection of bearing capacity and soil at shaft base.

There are a few disadvantages:

- a. They cannot be used if a suitable bearing stratum is not close enough to the ground surface (and assuming that the soil to the competent stratum is unreliable for skin resistance).
- b. Bad weather conditions may make drilling and/or concreting difficult.

- c. There may be ground loss if adequate precautions are not taken.
- d. One must dispose of soil from drilling ("spoil") and any slurry that is used.

19-4 OTHER PRACTICAL CONSIDERATIONS FOR DRILLED PIERS

Several practical considerations of importance in drilled pier construction include shaft alignment, disposal of slurry, concrete quality control, underreaming, and ground loss.

Shaft Alignment

It is often difficult to get a drilled pier perfectly aligned either in plan or elevation. If the plan location is within about 150 mm this is usually satisfactory. Much larger misalignment may require an adjustment in design for the additional moment resulting from eccentricity of the design load.

Maximum vertical misalignment as suggested by ACI Committee 336 (1988) is as follows:

Category A. Unreinforced shafts extending through materials offering minimal lateral restraint—not more than $0.125 \times$ diameter

Category B. Same, but soil is competent for lateral restraint—not more than $0.015 \times$ shaft length

Category C. Reinforced concrete shaft—to be determined on a site basis by the project engineer

Slurry Disposal

Slurry disposal is always a problem. One might use a (or several) large storage tank(s) on-site as temporary storage so the slurry can be reconditioned and reused to keep the total required volume to a minimum. One may construct a storage pit for the same purpose. Ultimately, however, the remaining residue must be hauled to a suitable disposal site.

Concrete Quality Control

Concrete is often specified in the 28 to 35 MPa range to reduce the shaft diameter. The slump should be in the range of 125 to 150 mm. Some persons suggest slumps in the range of 125 to 250 mm but one should check whether adequate (and reliable) strength can be obtained at slumps over 150 mm. Higher slumps are more necessary in slurry construction than for cased or uncased piers. Proprietary plasticizers are available to improve flowability (reason for large slumps) and eliminate arching. These might be appropriate for the dry method or with casing. Use of a plasticizer in the slurry method might be a viable solution, but there should be reasonable certainty that there will be no adverse chemical reactions with the slurry constituents.

To ensure reasonable shaft continuity, one should compare the shaft and concrete volumes for each pier. Several highly specialized nondestructive test procedures are available to measure shaft continuity (and quality, e.g., for voids) where a defective shaft is suspected [see Olson and Thompson (1985)] and the concrete has hardened. Sometimes a small-diameter core is taken from a suspect shaft.

Test cylinders are routinely taken to have a record of the concrete strength used. This aspect is usually set up by the project engineer using ACI guidelines. The top 1.5 m of the shaft should be vibrated to ensure adequate density.

Underreaming

Underreaming or bellling can be done in noncaving soils to enlarge the base to increase the bearing capacity where the base is founded on soil. For bases on rock the bearing capacity of the rock is often at least as large as that of the shaft based on f'_c of the concrete.

Belling produces unconsolidated cuttings on the base soil. Some of these may be isolated into the reamer seat (pilot depression of Fig. 19-1*d*). Alternatively, a temporary casing can be installed and an inspector lowered to the base to remove the cuttings by hand and to check the soil strength with a pocket penetrometer.

Bells may enlarge the base up to about four times the shaft diameter. As there would be great difficulty in placing rebars, the enlarged base is seldom reinforced. By using a maximum slope on the underream of 45° , two-way action shear is usually adequate so that the shaft does not "punch" through the bell. Bending should not be of concern for the short moment arm of about $1.5D$ maximum. Also note the concrete is placed in a fluid state so that it flows to a substantial contact pressure against the soil from the hydrostatic head. After hardening the soil provides substantial "confinement" to the bell to aid in resisting bending and punching failure.

Ground Loss

When the shaft is drilled the loss of lateral support will allow the surrounding soil to squeeze into the hole, decreasing its diameter. The squeeze can result in surface subsidence in the vicinity of the hole. The amount, of course, is directly related to the reduction in hole volume. Lukas and Baker (1978) suggest that a convenient method of determining whether hole squeezing will be a problem depends on the squeeze ratio R_s , which is the inverse of the s_u/p'_o ratio of Sec. 2-11.9

$$R_s = p'_o/s_u \quad (19-1)$$

where p'_o = effective overburden pressure

s_u = undrained shear strength

If $R_s < 6$ squeezing may take place but usually it is slow enough that it is of no consequence.

If $R_s > 6$ squeezing is almost certain to take place, and if R_s is on the order of 8 to 9 it will occur so rapidly it will be taking place as the hole is being excavated.

The foregoing is based on experiences in Chicago clay, and the ratio may be somewhat different at other locations.

The ground loss can be controlled in the following ways:

1. Rapid shaft excavation and replacement with concrete
2. Use of a shaft liner
3. Use of the slurry method

Either of the two latter options increases project costs, and many contractors do not like to use the slurry method because of the resulting mess and cleanup.

19-5 CAPACITY ANALYSIS OF DRILLED PIERS

Drilled piers are widely used to carry compressive loads. They are also used to carry tension loads—particularly under power line and antenna tower legs. They may carry lateral loads or a combination of vertical and lateral loads. The tension load case as given for piles in Sec. 16-14 can be written (here using Q instead of P) as

$$Q_{ult,t} = \sum Q_{si} + Q_b + W \quad (19-2)$$

where $\sum Q_{si}$ = sum of perimeter $\times f_s \times \Delta L$ of the several (or single) shaft elements making up total length L —ultimate value

Q_b = bell pullout resistance and/or any point suction. Similarly as for piles the point suction contribution is transient so is seldom used.

W = total pier weight including shaft and bell

Safety factors in the range of 2 to 4 are common, giving an allowable tension load of either

$$Q_{a,t} = \frac{Q_{ult,t}}{SF}$$

or, preferably, but not much used,

$$Q_{a,t} = \frac{\sum Q_{si}}{SF_s} + \frac{Q_b}{SF_b} + \frac{W}{SF_w} \quad (19-2a)$$

The use of partial safety factors as in Eq. (19-2a) is preferable since we might use $SF_s = 3$ or 4 for the skin resistance component because of uncertainties, an $SF_b = 2$ to 5 on the bell if Q_b is included, and an SF_w of about 1.4 since the volume of concrete and resulting weight of the pier are reasonably well known. The structural design would require that the allowable concrete stress in tension plus rebar allowable tension stress be sufficient to carry the tension design load $Q_{d,t} \leq Q_{a,t}$.

19-5.1 Pier Capacity in Compression

The ultimate capacity of a drilled pier (see Fig. 19-6) in compression is the smaller of

$$Q_{ult} = \sum Q'_{si} + \sum Q_L + Q_p \quad (19-3a)$$

or

$$Q_{ult} = \sum Q_{si} + \sum Q_L + Q'_p \quad (19-3b)$$

where $\sum Q_{si}$ = ultimate skin resistance as defined in Eq. (19-2)

$\sum Q'_{si}$ = limiting skin resistance, generally $< Q_{si}$

Q_p = ultimate point bearing

Q'_p = point bearing just at transition from ultimate to limiting skin resistance, and is generally $< Q_p$

$\sum Q_L$ = bearing resistance from any ledges produced by changes in shaft diameter or shear rings

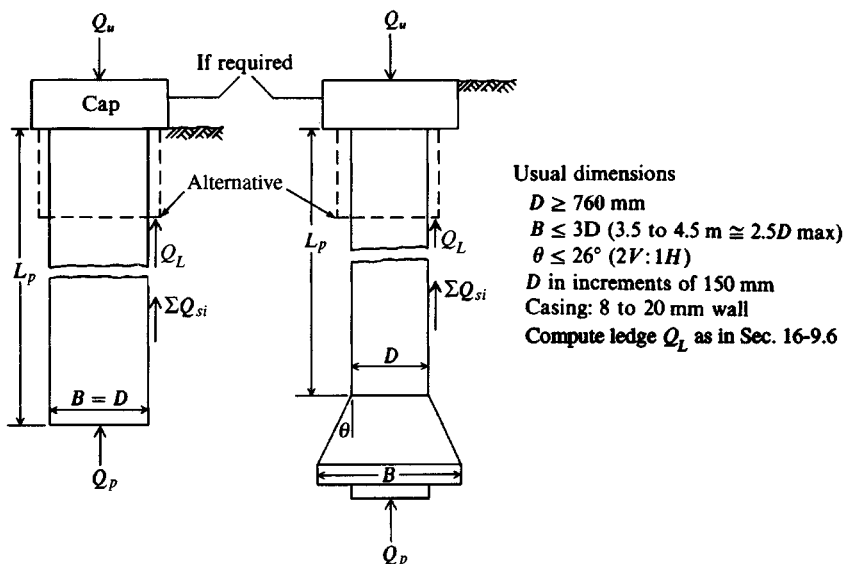


Figure 19-6 Capacity of straight- or stepped-shaft and belled piers. Commonly used dimensions are shown, but other dimensions can also be used.

The rationale for Eqs. (19-3) is based on load tests for both piles and drilled piers where the maximum skin resistance is developed at very small shaft movements on the order of about 3 to 10 mm. As a ratio the movements are on the order of $0.002D$ to $0.01D$. The movement necessary to develop ultimate bearing resistance is on the order of $0.005B$ to $0.05B$ where B = base diameter = D for straight shafts. The base displacement to develop maximum point resistance is much smaller for dense sand than for clay, which is often near 0.03 to $0.05B$.

The load test in Fig. 19-7 illustrates load resistance development as a combination of two separate effects. The pier is 762-mm diameter \times 7.01-m long and was selected because of the particular clarity and the nearly ideal load-transfer curves that are developed. Most load tests produce similar results but less clearly. Here we have the following:

1. At application of the first load increment of approximately 110 kN, skin resistance develops along nearly the full shaft length. The skin resistance contribution Q_{si} for any segment length ΔL can be obtained as the difference in shaft load at the top and base of the element. The sum of all these Q_{si} contributions for this load increment is simply the load $Q = 110$ kN.
2. With the second load increment to approximately 285 kN the load-transfer curve shifts to the right, but we see again that the tip load of about 45 kN is negligible.
3. The third load increment (to 735 kN), however, appears to produce a "limiting" shaft skin resistance with a small increase in point load (from 45 to about 80 kN). Also note:
 - a. The limiting skin resistance is analogous to the "residual" soil strength in a direct shear test.
 - b. The limiting skin resistance is not constant. In the upper 1.5 m and the bottom 1.0 m there is almost no skin resistance (in these two zones the curve is nearly vertical).
 - c. The point load is now the Q'_p of Eq. (19-3).
4. Next, the fourth load stage of 1250 kN is applied to develop what one could define as Q_{ult} for the pier. The point load has increased nearly the amount of the load increase (1250–

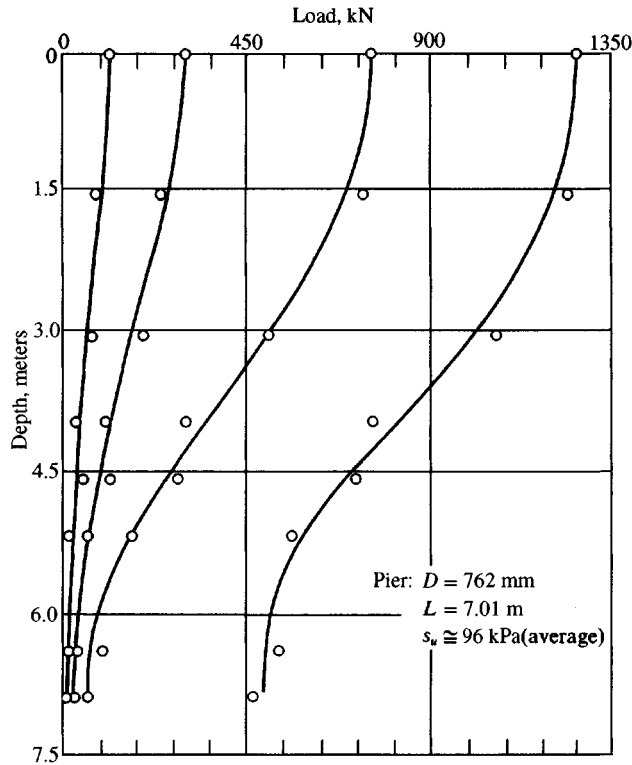


Figure 19-7 Load distribution for drilled pier. [From Reese and O'Neill (1969); converted to SI by author.]

735 = 515 vs. 500 – 80 = 420). An inspection of the load-transfer curve for load stage 4 shows that it is nearly identical in shape to that from the 735-kN load stage 3 load. In comparing the last two load transfer curve shapes it is clear that the skin resistance only increases a small amount from load stage 3 to 4, with the major part of the load increment being carried directly by the point. The load transfer curve for load stage 4 approximates this by its lateral displacement to the right, so the shaft curve profile is similar to curve 3, but the bottom is shifted by very nearly the load increment.

Considering these load stages and again referring to Fig. 19-7, we see that we can define the following:

$$Q_{ult} = 1250 \text{ kN}$$

$$Q_p = 490 \text{ kN} \quad (\text{read directly from the load-transfer curve at the tip level})$$

from which the skin resistance component is computed as

$$\sum Q'_{si} = 1250 - 490 = 760 \text{ kN}$$

Since s_u in the 7.01 pier depth was about 96 kPa we can compute a full-depth α coefficient as

$$\begin{aligned} \alpha &= \frac{\sum Q'_{si}}{L \times p' \times s_u} \\ &= \frac{760}{7.01 \times \pi \times 0.762 \times 96} = 0.47 \end{aligned}$$

however, we probably should have used a length $L = 7.01 - 1.5 - 1.0 = 4.5$ m (and $\alpha \cong 0.73$) since the upper 1.5 m and lower 1 m of the shaft has negligible skin resistance at ultimate load.

If the pier load were increased to, say, 1560 kN or more, we may speculate that the load-transfer curve would become nearly vertical to a greater depth; and the point load would increase, with the settlement greatly increasing.

From this description of events in a load test, together with Eqs. (19-3a) and (19-3b), we see that estimating the capacity of a drilled pier—particularly without the guidance of a load test—is not a simple task in spite of the relatively simple format of the equations. Obviously, if ultimate values of skin resistance and point bearing occurred at about the same amounts of displacement the problem would be much simpler.

Because the shaft and point maximum load capacities are not developed simultaneously, many practitioners use either point bearing or skin resistance rather than a combination. This practice is common in the United States (and is not unduly conservative when the point is founded on rock or very dense bearing soil). Others, primarily in Europe, often try to use some kind of interaction to obtain the pier capacity as a combination of skin resistance and point bearing. This approach is also given by Reese et al. (1976) and later by Reese (1978) based on his extensive research. As given by Reese et al. (1976) the pier capacity *in clay* is

$$Q_{ult} = \sum Q_{si} + Q_p \quad (19-4)$$

where $\sum Q_{si} = \sum \alpha s_{us} \times p' \times \Delta L$

$$Q_p = N_c c A_p = 9 s_{u,p} A_p$$

α = reduction coefficient from Table 19-1 based on installation process

$s_{u,s}$ = average undrained shear strength *along shaft length* ΔL ; use $s_{u,s}$ = cohesion in range of $0 \leq \phi \leq 10^\circ$

p' = average pier perimeter in shaft length ΔL

ΔL = element length over which $s_{u,s}$ can be taken as a constant value

$s_{u,p}$ = average undrained shear strength from about $0.5B$ above base to about $3B$ below the base

TABLE 19-1

Average α values to estimate shaft skin resistance of drilled piers in clay

Method of pier construction	Limiting f_s^*	
	α^\dagger	kPa
Dry or using lightweight drilling slurry	0.5	90
Using drilling mud where filter cake removal is uncertain	0.3	40
Belled piers on about same soil as on shaft sides		
By dry method	0.3	40
Using drilling mud where filter cake removal is uncertain	0.15	25
Straight or belled piers resting on much firmer soil than around shaft	0	0

* $f_s = \alpha s_u \cong f_{s(\text{limiting})}$.

† For soil-to-concrete; use values of 0.25 to 0.1 for cased piers where adhesion is to the steel shell. Use higher values for driven casing. After Reese et al. (1976)

$$A_p = \text{area of base} = 0.7854B^2$$

$$B = \text{base width}$$

For the immediate settlement to be tolerable *in clay* it was recommended that the allowable design load be

$$Q_a = \frac{Q_{ult}}{SF} \geq Q_d \quad (19-5)$$

with the SF in the range of 1.5 to 4. Alternatively, or where the base is on clay with $OCR > 1$,

$$Q_a = \sum Q_{si} + \frac{Q_p}{3} \geq Q_d \quad (19-6)$$

The premise of Eq. (19-6) is that by reducing the base load by a factor of 3 the small slip necessary to mobilize Q_{si} is well within settlement tolerances. *Use the smaller Q_a from either Eq. (19-5) or Eq. (19-6) above.*

For piers in *sand* Reese et al. (1976) suggest using Eq. (19-4) with the terms separated as

$$\left. \begin{aligned} \sum Q_{si} &= \sum K \bar{p}_o \tan \delta (p' \times \Delta L) \\ \text{and} \quad Q_p &= \frac{q_p}{\alpha_p} A_p \end{aligned} \right\} \quad (19-6a)$$

where the new variables are as follows:

K = shaft lateral pressure factor, conservatively taken as follows:

Depth to base, m	K
≤ 7.5	0.7
$7.5 < L \leq 12$ m	0.6
> 12	0.5

\bar{p}_o = average effective overburden pressure to midheight of ΔL

$\delta = \phi$ for pier shaft in sand because of the rough concrete interface

q_p = maximum point pressure for an assumed 5 percent point displacement which, based on load tests, is suggested as follows:

Sand state	q_p	
	kPa	ksf
Loose (not likely used)	0	0
Medium (possibly used)	1600	32
Dense (very likely used)	4000	80

α_p = base reduction factor to limit base settlement to 25 mm (1 in.) and given as

$$\begin{aligned} \text{SI:} \quad & 2.0B \text{ (base width } B \text{ in meters)} \\ \text{Fps:} \quad & 0.6B \text{ (} B \text{ in feet)} \end{aligned}$$

SPT or CPT correlations may be used to estimate the angle of internal friction ϕ in Eq. (19-6a) unless better data is available, since the lateral pressure coefficient K as given above is considered to be conservative. One would never found a drilled pier base on loose sand and probably would not place the point on a medium dense sand unless a more competent stratum is at a substantially greater depth.

If Eq. (19-6a) is used, the immediate settlement should not be a problem, since it is based on a 25-mm maximum settlement through use of the α_p factor. The allowable pier design load Q_a is

$$Q_a = \frac{Q_{ult}}{SF}$$

It is recommended to use $SF = 1.0$ when $Q_{ult} =$ point value from Eq. (16-6a) with $\Delta H_p \approx 25$ mm; use $SF = 1.5$ to 4 when skin resistance is included in Q_{ult} and with point settlement ΔH_p now somewhat less than 25 mm.

We should note that Eqs. (19-3) and (19-4) are theoretically correct and that Eqs. (19-6) are empirical. Any difference between the theoretical equations and load-test values are from using incorrect design parameters to estimate the skin resistance and point capacity, or an oversimplification of using L rather than ΔL in a summation process. The parameters suggested by Reese et al. (1976) are from a fairly limited data base + use of some reported load-test data of others, and the correlation is generally very good. As with any of the correlation-type data, however, the reader should expect some scatter as more test data are accumulated—either from errors or from natural variability in soils from different geographic regions. Further, locally obtained parameters in these equations may provide better designs than the use of global (of universal application) parameters.

The computation for the α coefficient for skin resistance illustrates how wide variations can be reported in the literature (ranging from about 0.15 to 1). Here with the simple load test discussed earlier we could obtain 0.47 or 0.60 depending on what is used for shaft length. It is common to use a single factor for the full shaft length. In a load test where data can easily be back-computed it might be better to use shaft segments of ΔL . Practice tends to simplify the computations by using the effective shaft length and average shear strength values. Practice also tends to use the effective shaft length and average soil parameters for piles in cohesionless soils as well. According to Reese et al. (1976) the effective shaft length for skin resistance should exclude the top 1.5 m (5 ft) and the bell perimeter or, for straight shafts, the bottom 1.5 m (or 5 ft).

19-5.2 Other Methods for Point Bearing Capacity

Besides using Eq. (19-6a), one can compute the pier base capacity using the Terzaghi bearing-capacity equations from Table 4-1 as

$$Q_a = \frac{Q_{ult}}{SF} = \frac{A_p}{SF} (1.3cN_c + L'\gamma N_q + 0.4\gamma B_p N_\gamma) \quad (19-7)$$

For the case of the base on either clay ($\phi = 0$) or sand ($c = 0$),

$$Q_a = \frac{A_p \times 9c}{SF} \quad (\text{clay})$$

$$Q_a = \frac{A_p}{SF} (L'\gamma N_q + 0.4\gamma B_p N_\gamma) \quad (\text{sand})$$

We can also use the Hansen equations, where

$$Q_a = \frac{A_p}{SF} (cN_{cs}d_c + L'\gamma N_{qs}d_q + 0.5B_pN_{\gamma}s_{\gamma}) \quad (19-7a)$$

or for $\phi = 0$

$$Q_a = \frac{A_p}{SF} 5.14 \times s_u (1 + s'_c + d'_c) + L'\gamma$$

where A_p = pier point area (bell area if one is used)

B_p = width of pier point [shaft or bell (if used)]

L' = about $15 \times$ shaft diameter for Terzaghi equations, and effective length L_p for the Hansen equations

Meyerhof (1956) suggested equations using the SPT and CPT for the allowable bearing capacity for spread footings for a 25-mm settlement, and with the statement they should be *doubled* for pier bases. After doubling by the author these equations become

$$\text{SPT:} \quad Q_a = A_p \frac{N_{55}}{0.052} \quad (\text{kN}) \quad (19-8)$$

$$\text{CPT:} \quad Q_a = A_p \frac{q_c}{40} \quad (\text{kN}) \quad (19-9)$$

where q_c is given in kPa.

For drilled piers socketed into rock the allowable bearing capacity q_a can be computed as in Example 4-14 of Sec. 4-16 so that the allowable point

$$Q_a = A_p q_a$$

Drilled piers socketed into rock some depth D_r will have a substantial skin resistance capacity as well as point bearing. This may allow using a reduced shaft diameter in this region.

The socket skin resistance capacity [see Benmokrane et al. (1994)] can be expressed as

$$Q_s = \pi B_r D_r \lambda \sqrt{q_u} \quad (\text{MN})$$

where B_r = shaft diameter in rock socket at depth D_r

q_u = unconfined compression strength of the smaller of the rock or the pier shaft concrete, MPa

λ = adjustment factor, usually ranges between 0.2 for smooth-sided and 0.3 for rough-sided shafts. Others have suggested values of 0.45 for fairly smooth sides and 0.6 for rough sides.

19-5.3 General Capacity Analysis for Drilled Piers

For the usual case of a drilled pier in soil the analysis is essentially identical to that for a pile, and the computer program PILCAPAC can be used. The two basic differences are that the shaft is usually round (and larger than a pile) and some adjustment in the α factor must be made if the pier is constructed by the slurry method.

19-6 SETTLEMENTS OF DRILLED PIERS

The settlement of a pier is the axial shortening of the shaft + the point settlement, written as

$$\Delta H = \sum \Delta H_{si} + \Delta H_p$$

where $\sum H_{si}$ = accumulation of shaft axial compression, $\frac{P_i \Delta L}{AE}$

ΔH_p = point settlement due both to the point bearing pressure and to settlement caused by skin resistance

The computer program PILCAPAC in Example 16-7 and Example 19-1 (following) also computed pier settlement by this method.

If we do not have a computer program we can estimate that the settlement should not be more than 25 mm if the recommendations for Q_{pu} made by Reese (1978) are followed. The resulting design ΔH should be 25/SF since Q_{pu} = ultimate value and is always divided by an SF.

We may use Meyerhof's equations [Eqs. (19-8) and (19-9)] as alternatives, which are suggested not to give more than $\Delta H = 25$ mm for the allowable design pressure q_a .

We may also use the stress coefficients from Table 18-1 and our best estimate as to which of the three table cases (1, 2, or 3) applies. From the stress influence coefficients, compute a stress profile for a depth of influence $L_i \approx 4$ to $5B$ below the base and compute the average stress increase Δq_{av} . Next make some kind of estimate for the stress-strain modulus E_s in this depth and solve the following:

$$\Delta H_p = \epsilon \times D_i = \frac{\Delta q_{av} L_i}{E_s}$$

for the point settlement term.

The methodology of program PILCAPAC will be used to illustrate both capacity and settlement analysis in the following example.

Example 19-1. Use program PILCAPAC and compute the estimated ultimate pier capacity for the "slurry" pier [one of the four "piles" tested and reported in ASCE SP No. 23 (see Finno (1989))]. See Fig. E16-7a for the soil profile. This pier had a nominal 24-in. diameter shaft in the upper 9 ft and 18 in. below. Thus, there is one ledge [the program will allow any number—you have to specify the number of layers to the ledge and the upper and lower diameter in millimeters (or inches)]. The concrete $f'_c = 6000$ psi and the pier length is 50 ft. *Fps units are used in this example since the original source uses those units and it would be difficult to check results if converted to SI.*

Solution. A data file was created and named ASCEPL2.DTA as shown on the output sheets (Fig. E19-1). Most of the soil data are contained in the table labeled "Soil Data for Each Layer." Although only layers 2 through 8 provide skin resistance, nine layers are shown. The ninth (bottom) layer is for computing point capacity. Shown are both the assumed ϕ and δ angles of the soil. The K factor is computed as described in Example 16-7.

Note that for friction in the sand the friction angle $\delta = \phi$ since the concrete is poured against the soil—or at least flowed against the soil as the casing in the top depth was pulled.

The α factors are all 1.25 in the bottom three clay layers and are substantially larger than the Reese recommendations given earlier. The value of $\alpha = 1.25$ was selected for two reasons: (1) The soil is below the GWT; the contractor had some drilling problems, so this part of the shaft may have been enlarged somewhat (it was stated that the concrete volume was about 10 percent larger than the theoretical shaft volume). (2) The concrete had a slump between 9 and 10 in. (a very high value), so it would tend to give a large lateral pressure, which would in turn give a larger undrained cohesion

than that used. Rather than do a numbers shuffle (increase the shaft diameter, increase cohesion) it was easier just to increase α .

I elected to use the Terzaghi equation for point capacity since the Hansen equation had been used in Example 16-7. I had to stay with the computer during execution, for the program asks how many diameter changes occur for a drilled pier (ITYPE = 5) and the number of soil layers from the top down to the change (here 1 change and 2 layers down from the top).

Figure E19-1

```

++++++DATA FILE NAME FOR THIS EXECUTION:  ASCEPL2.DTA

ASCE DRILLED "SLURRY" PIER TEST IN GT SP-23, FIG. 4, P 9--ALPHA METHOD

NO OF SOIL LAYERS =      9              IMET (SI > 0) =      0

PILE LENGTH FROM GROUND SURFACE TO POINT, PLEN =  50.000 FT
                                           FILE TYPE:  DRILL PIER

      PILE DIAMETER =  1.500 FT
      DRIVE POINT DIAM =  .000 FT

      POINT X-AREA = 1.767      SQ FT

SOIL DATA FOR EACH LAYER:
LAYER NO  EFF WT  PHI  DELTA  COHES  ALPHA  K-FACT  THICK  PERIMETR
          K/FT*3  deg  deg    KSF      FT      FT
1       .110   25.00   .0   1.000   .907   1.000   2.00   6.283
2       .115   36.00  36.0   .000   .000   1.600   7.00   6.283
3       .115   32.00  32.0   .000   .000   1.400   4.00   4.712
4       .115   32.00  32.0   .000   .000   1.400   2.00   4.712
5       .060   36.00  36.0   .000   .000   1.700   8.00   4.712
6       .060    .00    .0   .964   1.250   1.000   9.00   4.712
7       .060    .00    .0   .964   1.250   1.000   9.00   4.712
8       .060    .00    .0   .964   1.250   1.000   9.00   4.712
9       .060    .00    .0   .964   1.000   1.000  10.00   4.712

THERE ARE  1 STEP CHANGES IN X-SECTION AND ALSO SHAFT MAY BE TAPERED

FOR ABRUPT X-SECT CHANGE =  1
      DIAM D1, D2 =  2.000  1.500
      NET AREA =  1.374      QULT USES D1 =  2.00
EXTRA DATA FOR CHECKING TERZAGHI STEP LOAD
NC, NQ, NG =  44.034  28.515  27.490
SC, SG, QBAR =  1.300  .600  1.025
COMPUTE QULT =  31.125      STEP LOAD PBASET =  42.7792 KIPS

+++++TERZAGHI BEARING CAPACITY METHOD USED--IBRG =  2

      FILE POINT IS ROUND W/AREA =  1.7672 SQ FT
      BASED ON DIAM =  1.500 FT

PILE POINT AND OTHER DATA
PILE LENGTH, PLEN =  50.00 FT      UNIT WT OF SOIL =  .060 K/FT*3
      PHI-ANGLE =  .000 DEG      SOIL COHES =  .96 KSF
      EFFEC OVERBURDEN PRESSURE AT PILE POINT QBAR =  3.81 KSF

EXTRA DATA FOR HAND CHECKING TERZAGHI POINT LOAD
NC, NQ, NG =  5.700  1.000  .000
SC, SG, QBAR =  1.300  .600  3.815
COMPUTE QULT =  10.958      POINT LOAD PBASET =  19.3654 KIPS

+++++ IN ROUTINE USING ALPHA-METHOD FOR SKIN RESISTANCE--IPILE =  5
    
```

I,QBAR = 2 .623	DELTA ANG DELTA(I) = 36.00						
KFACT(I) = 1.6000	FRIC FORCE SFRIC = 31.827						
I,QBAR = 2 .623	DEL ANG D1,D2 = 36.00 .00						
KFACT(I) = 1.6000	FRIC FORCE SFRIC = 31.827						
I,QBAR = 3 1.255	DELTA ANG DELTA(I) = 32.00						
KFACT(I) = 1.4000	FRIC FORCE SFRIC = 20.694						
I,QBAR = 3 1.255	DEL ANG D1,D2 = 32.00 .00						
KFACT(I) = 1.4000	FRIC FORCE SFRIC = 20.694						
I,QBAR = 4 1.600	DELTA ANG DELTA(I) = 32.00						
KFACT(I) = 1.4000	FRIC FORCE SFRIC = 13.192						
I,QBAR = 4 1.600	DEL ANG D1,D2 = 32.00 .00						
KFACT(I) = 1.4000	FRIC FORCE SFRIC = 13.192						
I,QBAR = 5 1.955	DELTA ANG DELTA(I) = 36.00						
KFACT(I) = 1.7000	FRIC FORCE SFRIC = 91.029						
I,QBAR = 5 1.955	DEL ANG D1,D2 = 36.00 .00						
KFACT(I) = 1.7000	FRIC FORCE SFRIC = 91.029						
IN ROUTINE ALPHAM FOR I = 6 H1 = 9.00							
ALPHA(I) = 1.250							
SHAFT PERIMETER PER(I) = 4.712	ADHES = 51.106						
IN ROUTINE ALPHAM FOR I = 7 H1 = 9.00							
ALPHA(I) = 1.250							
SHAFT PERIMETER PER(I) = 4.712	ADHES = 51.106						
IN ROUTINE ALPHAM FOR I = 8 H1 = 9.00							
ALPHA(I) = 1.250							
SHAFT PERIMETER PER(I) = 4.712	ADHES = 51.106						
TOTAL ACCUMULATED SKIN RESISTANCE = 310.0595							
USING THE ALPHA METHOD GIVES TOTAL RESISTANCE, PSIDE = 310.060 KIPS							
WITH TOP 2.00 FT OMITTED							
TOTAL PILE CAPACITY USING TERZAGHI POINT LOAD = 372.20 KIPS							
SETTLEMENTS COMPUTED FOR AXIAL DESIGN LOAD = 372.2 KIPS							
USING SHAFT MODULUS OF ELAST ES = .6358E+06 KSF							
LAYER NO	THICK FT	X-AREA SQ FT	PTOP KIPS	SKIN R KIPS	PBOT KIPS	ELEM DH	SUM DH IN
1	2.00	3.142	372.2	.0	372.2	.0045	.0045
2	7.00	3.142	372.2	31.8	340.4	.0150	.0195
3	4.00	1.767	340.4	20.7	319.7	.0141	.0336
4	2.00	1.767	319.7	13.2	306.5	.0067	.0402
5	8.00	1.767	306.5	91.0	215.5	.0223	.0625
6	9.00	1.767	215.5	51.1	164.4	.0183	.0808
7	9.00	1.767	164.4	51.1	113.2	.0133	.0941
8	9.00	1.767	113.2	51.1	62.1	.0084	.1026
SETTLEMENT DATA:			DQ, BMAX = 210.62	1.50			
			SOIL THICKNESS HTOT = 50.00				
			HTOT/BMAX & FOX FAC = 33.33	.500			
			FOR MU = 0.35 AND SOIL Es = 450.0 KSF				
			COMPUTED POINT SETTLEMENT, DP = 1.8482 IN				
			TOTAL PILE/PIER SETTLEMENT (BUTT MOVEMENT) = DP + DH = 1.9507 IN				

Figure E19-1 (continued)

The resulting output is shown on Fig. E19-1, and we can make the following comparison:

Computed	Load test
$Q_u = 372$ kips	340 (after 4 weeks) kips 413 (after 43 weeks)
$\Delta H = 1.95$ in.	Between 2 and 2.5 in.

This comparison indicates that the estimated soil properties were fairly good (with aging not considered, both ϕ and α are too low); that aging is a factor; and that pile/pier loads are not easy to predict. The use of the computer program clearly indicates that the best predictions for capacity and settlement are made by considering the several soil layers making up a site profile rather than trying to obtain a single site parameter such as α or β . It is usually easier to back-compute from known values; however, note that the ϕ angles were not readjusted to obtain a better fit and the α factor was selected with some justification.

As a final comment, there were 24 predictors for these tests and not one got a quality value. One was about 30 percent over—the others ranged from about 50 to 60 percent of the load test. Most did not include a ledge contribution Q_L , which is larger (since it bears on the sand) than the point capacity Q_p , which is in clay.

///

19-7 STRUCTURAL DESIGN OF DRILLED PIERS

Since the pier shaft is supported by the surrounding soil, column slenderness effects do not have to be considered. Thus, the design is considerably simplified. Design requirements are usually met if the shaft diameter is large enough to carry the design load without exceeding the allowable concrete and steel (if used) stresses.

The bell dimensions should be adequate to resist a punching failure and have adequate bending resistance as a plain concrete member, because reinforcement would be difficult to place.

For unreinforced pier shafts the allowable concrete stress in a number of building codes is

$$f_c = 0.25 f'_c \quad (19-10)$$

For ordinary reinforced drilled piers we can design conservatively as

$$P = A_c f_c + A_s f_s \geq P_d \quad (19-11)$$

where A_i = cross-sectional areas of concrete and steel, respectively

f_i = allowable concrete and steel stresses, respectively

$$f_c = 0.25 f'_c$$

$$f_s = 0.40 f_y$$

In many cases the pier shaft must be designed for both bending and an axial load. This issue is not directly addressed in most building codes nor in the ACI 318- or by ACI Committee 336. If we use the ACI 318- as a guide, a reinforced pier shaft for axial load can be designed using the factored axial load P_u for tied rebars (usual case) as

$$P_u = 0.80 \phi (0.85 f'_c A_c + f_y A_s) \quad (19-12)$$

For bending with axial load one should consult a textbook on reinforced concrete design of short columns with bending since strain compatibility between concrete and steel is necessary unless $P/A + Mc/I$ gives compressive stress everywhere on the cross section. A round column computer program is most useful for this analysis since it is a computationally intensive iterative process.

When the drilled pier casing is left in place it may be used to increase the shaft capacity either by using a transformed section ($A_t = A_g + nA_s$) or as

$$P = A_c f_c + A_s f_a$$

where A_s = effective area of casing steel (after reduction for corrosion has been made). Alternatively, the casing can be used to increase the allowable concrete stress f_c as follows:

$$f_c = 0.30 f'_c + \frac{1.5 t f_y}{D} \leq 0.40 f'_c \quad (19-13)$$

where t = casing thickness after deduction for corrosion, mm or in.

D = ID of casing, mm or in.

f_y = yield stress of casing steel, MPa or ksi

This recommendation is given by the Chicago Building Code (Sec. 13-132-400).

19-8 DRILLED PIER DESIGN EXAMPLES

We will illustrate some of the preceding design discussion with the following two design examples.

Example 19-2. For the soil profile given in Fig. E19-2 we must make a trial pier design in order to develop an economic comparison with piles. For the pier use $f'_c = 28$ MPa with a 150-mm slump. By inspection of the GWT elevation we see it will be necessary to use the slurry method since we could not seal the water out of the hole with a casing socketed into the sand. The upper part of the pier shaft will use an arbitrary 1 percent of rebars (a designer decision since only axial load is present).

The design axial load $P_d = 3000$ kN.

Required. Make a preliminary design recommendation.

Solution.

Step 1. Find the approximate shaft diameter based on the allowable concrete stress of $f_c = 0.25 f'_c = 0.25 \times 28 = 7$ MPa. Let us write

$$0.7854 D^2 f_c = P_d$$

Substituting and solving, we find

$$D = \sqrt{\frac{3}{0.7854 \times 7}} = 0.74 \text{ m}$$

Step 2. Estimate the pier length $L = 11$ m (into dense sand), and find the estimated point capacity neglecting any skin resistance as a first trial. Use the Reese (1978) recommendations:

$$q_p = 4000 \text{ kPa (dense sand)} \quad A_p = 0.7854 B^2 \quad \alpha_p = 2.0$$

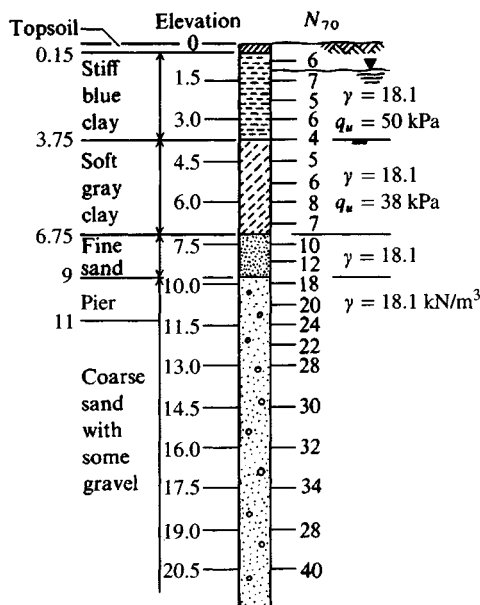


Figure E19-2

Substituting into Eq. (16-6a), we obtain

$$Q_p = \frac{q_p A_p}{\alpha_p} = \frac{4000 \times 0.7854 B^2}{2B} = 1571 B$$

Since this result is for a 25-mm settlement, we can use an SF = 1 and directly solve for pier diameter B , giving

$$B = \frac{Q_d}{Q_p} = \frac{3000}{1571} = 1.91 \text{ m (rather large)}$$

At this point it would appear that we must use either a large-diameter shaft or a bell. We cannot bell in sand, so let us look at alternatives. First, the Meyerhof equation [Eq. (19-8)] may help. Averaging N_{70} for the four values in the approximate influence depth below the base, we have 24 and $N_{55} = 24 \times 70/55 = 31$. Directly substituting into Eq. (19-8), we obtain

$$q_a = N_{55}/0.052 = 31/0.052 = 596 \rightarrow 600 \text{ kPa}$$

The required point diameter is

$$0.7854 D^2 \times 600 = 3000 \rightarrow D = \sqrt{\frac{3000}{0.7854 \times 600}} = 2.52 \text{ m} \gg 1.91$$

We might be able to obtain some skin resistance from the clay and sand layers to reduce the point load. The L for layer 1 is $L \approx 3.75 - 0.15 = 3.60$ m; for layer 2, $L \approx 6.75 - 3.75 = 3.0$ m. Use $\alpha = 1$ for both layers (clay is both below GWT and soft). Also arbitrarily estimate the required pier shaft = 1.372 m.

$$\text{For layer 1: } \pi \times 1.372 \times 50 \times 3.60 = 775 \text{ kN}$$

$$\text{For layer 2: } \pi \times 1.372 \times 38 \times 3.0 = 490 \text{ kN}$$

$$\text{Total} = Q_{sc} = 1265 \text{ kN}$$

For the sand, we estimate $\phi = 32^\circ = \delta$; $\gamma' = 18.1 - 9.81 = 8.3 \text{ kN/m}^3$; $\Delta L = 11.0 - 0.15 - 3.6 - 3.0 = 4.25 \text{ m}$; $z_o = 11.0 - 4.25/2 = 8.8 \text{ m}$; $K = 0.60$ (Reese value for $L < 12 \text{ m}$). Then

$$\bar{q}_o = \gamma' z_o = 8.3 \times 8.8 = 73 \text{ kPa}$$

$$Q_{ss} = K \bar{q}_o \tan \delta (\pi \times D) \Delta L = 0.6 \times 73 \times \tan 32^\circ (\pi \times 1.372 \times 4.25) = 501 \text{ kN}$$

$$\text{Total side resistance } \sum Q_s = Q_{sc} + Q_{ss} = 1265 + 501 = 1766 \text{ kN}$$

$$\text{Net point load } Q_p = Q_d - \sum Q_s = 3000 - 1766 = 1234 \text{ kN}$$

$$\begin{aligned} \text{Shaft load (concrete } \gamma_c = 23.6 \text{ kN/m}^3) &= 0.7854 \times 1.372^2 \times 23.6 \times 11 \\ &= 384 \text{ kN} \end{aligned}$$

$$\text{Total point load} = 1234 + 384 = 1618 \text{ kN}$$

Using Eq. (19-6a) for a point settlement of 25 mm, we can write

$$Q_p = \frac{q_p A_p}{\alpha_p} = \frac{4000 \times 0.7854 \times 1.372^2}{2 \times 1.372} = 2155 > 1618 \quad (\text{O.K.})$$

We may be able to use a pier with dimensions as follows:

$$\text{Shaft diameter } D = 1.372 \text{ m}$$

$$L = 11 \text{ m}$$

The major question is whether an $\alpha = 1.0$ is valid. Note that the overall SF is rather small.

Comments.

1. This is a fairly large-diameter shaft—so is the load.
2. It would not be practical to use a bell in the clay—even if the base were on the sand, for that sand is somewhat loose and settlement would be a problem.
3. Piles may be a more viable option since they can be driven into the dense sand and their lengths would also be on the order of 11 m.
4. A lower f'_c could be used but may not be allowed by the local code.
5. One may consider a point-bearing pier on rock if the depth is not over 30 to 35 m down and the stratum is reasonably competent. The greater length is offset by a smaller-diameter shaft.

////

Example 19-3. Make a preliminary design for a drilled pier to be founded on the firm clay at depth -27 m of Fig. E19-3a. The top 3.5 m of depth is in a water-bearing sand-gravel stratum. The pier is to carry 10 500 kN, and we will use $f'_c = 35 \text{ MPa}$. Use an SF = 2 on the skin resistance, and use a belled base if necessary.

Solution. From Fig. E19-3a estimate the base $s_u = 145 \text{ kPa}$. Take the average shaft $s_u = 120 \text{ kPa}$. We should actually divide the 27-m thick stratum into several layers and obtain $s_{u,av}$ for each.

The dry method (Fig. 19-3) of pier installation will be used. First, a casing will be socketed into the clay about 1 m below the sand-gravel, material for a water seal and then the shaft excavation will proceed.

Step 1. For $f'_c = 35 \text{ MPa}$ the allowable $f_c = 0.25 \times 35 = 8.750 \text{ MPa}$. Also we have

$$0.7454 D^2 f_c = 10\,500 \text{ kN}$$

Rearranging and solving for a trial shaft diameter, we find

$$D = \frac{10\,500}{0.7854 \times 8.750 \times 1000} = 1.23 \text{ m}$$

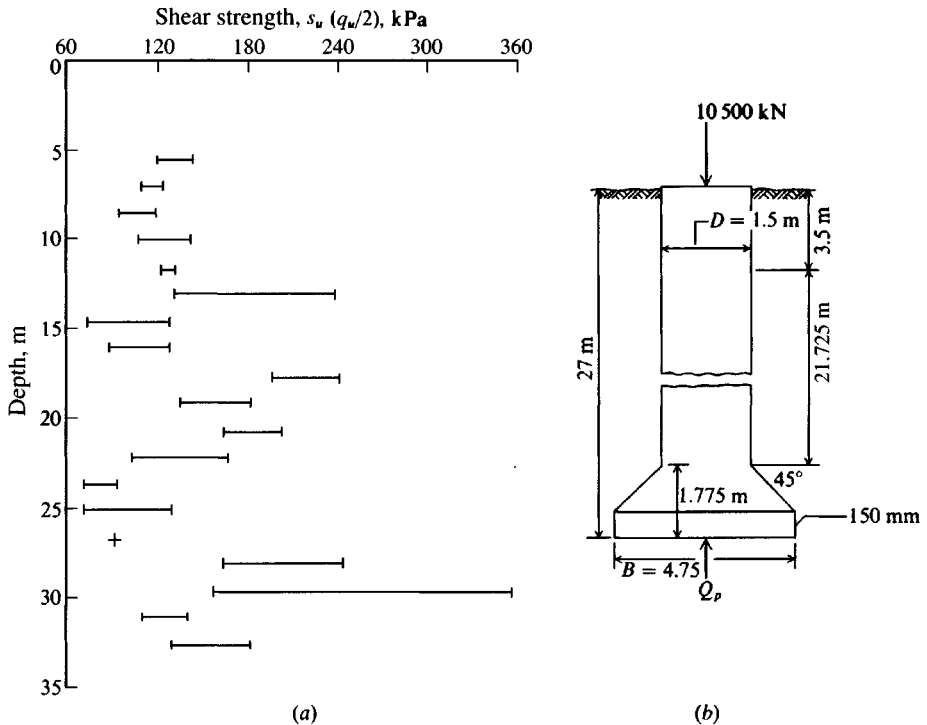


Figure E19-3

Step 2. Estimate the shaft friction resistance. We will try $D = 1.5$ m, giving a shaft perimeter $p' = \pi D = 4.71$ m. The effective shaft length for cohesive skin resistance is

$$L' = L - 3.5 \text{ m of sand-gravel} = 27 - 3.5 = 23.5 \text{ m}$$

From Table 19-1 obtain the Reese value of $\alpha = 0.5$, which is very conservative. From Fig. 16-14 we can obtain $\alpha = 0.7$ to 0.8 . We should in a real case divide the 27-m shaft into several layers, with the top layer being about 1.5 m, the second layer 2.0 m (the sand-gravel), then layers based on the s_u profile; obtain an average s_u for each layer and an α for each layer using either Fig. 16-14 or Eq. (16-12a).

We could also use PILCAPAC for the analysis but obtain printouts whereby we analyze the skin resistance and point capacity and apply a suitable SF to see if the system is adequate. That program also allows a belled base. We would make the point layer thick enough that we could add any needed intermediate layers with minor adjustments to the data file.

To get the general idea of pier design/analysis we will *incorrectly* use a single $\alpha = 0.5$ for the full shaft length.

Check that $0.5 \times 120 = 60 \text{ kPa} < 86$, the limiting value in Table 16-1. Then

$$\sum Q_{si} = \alpha \times s_u \times p' \times L' = 0.5 \times 120 \times 4.71 \times 23.5 = 6641 \text{ kN} \ll 10\,500$$

It is immediately evident that either we have to use a larger shaft, a larger α , or a bell. We will use a bell, which reduces the shaft length for friction resistance but creates a substantial gain in point bearing Q_p . Estimate a bell height of 1.75 m, giving $L' = 23.5 - 1.75 = 21.75$ m and a revised

$$\sum Q_{si} = 60 \times 4.71 \times 21.75 = 6150 \text{ kN}$$

Step 3. Compute bell dimensions. We will use an SF = 2 on the skin resistance. Noting that Reese suggests using $Q_p/3$ to provide a bearing value so the settlement $\Delta H \leq 25$ mm, we find

$$Q_{p,a} = \frac{s_u \times 9 \times A_p}{3} = \frac{145 \times 9 \times 0.7854 D_b^2}{3} = 341.65 D_b^2$$

The bell must carry $P_b = 10500 - 6150/2 = 7425$ kN. Equating these expressions, we find

$$341.65 D_b^2 = 7425 \rightarrow D_b = \sqrt{\frac{7425}{341.65}} = 4.66 \text{ m}$$

Use $D_b = 4.75$ m to find $D_b/D_s = 4.75/1.5 = 3.17$, which is close to the maximum allowed. The revised bell depth (see Fig. E19-3b for geometry) is

$$H_b = 0.15 + (4.75 - 1.50)/2 = 1.775 \text{ m} \approx 1.75 \text{ used} \quad (\text{O.K.})$$

Step 4. Check potential ground loss from possible “squeezing.”

For this we will estimate $\gamma_{\text{wet}} = 19.8 \text{ kN/m}^3$ and $\gamma' = 10 \text{ kN/m}^3$ for full shaft length. Thus,

$$\begin{aligned} \text{At 10 m depth: } p'_o &= 10(\gamma') = 10(10) = 100 \text{ kPa} \\ s_u &= 120, \text{ giving } \frac{p'_o}{s_u} = \frac{100}{120} = 0.83 \ll 6 \text{ to } 8 \end{aligned}$$

$$\begin{aligned} \text{At 20 m depth: } p'_o &= 25(10) = 250 \text{ kPa} \\ s_u &= 120, \text{ giving } \frac{p'_o}{s_u} = \frac{250}{120} = 2.5 < 6 \text{ to } 8 \end{aligned}$$

It appears that ground loss from squeezing will not be a problem here.

Step 5. Check axial shortening—use the effective shaft length = $27 - 1.775 = 25.2$ m even though a part is the “bell.” Assume the average shaft load $P = \sum Q_{si} = 6150$: Then

$$\begin{aligned} A_s &= 0.7854 \times 1.5^2 = 1.767 \text{ m}^2 \\ E_c &= 4700 \sqrt{f'_c} \quad (\text{Table 8-3}) \\ &= 4700(35)^{0.5} = 27\,800 \text{ MPa} \end{aligned}$$

The axial shortening is

$$\Delta H_s = \frac{PL}{A_s E} = \frac{6150(25.2)}{1.767(27\,800)} = 3.2 \text{ mm}$$

Since the point should displace not more than 25 mm the total immediate ΔH of the pier should not exceed 30 mm; any consolidation settlement would be additional.

Summary.

Use the dry method with a casing to about 5 m depth.

Use $D = 1.50$ m (Fig. E19-3b).

Use $B = 4.75$ m.

Total settlement under 30 mm.

Squeezing or ground loss does not seem a problem.

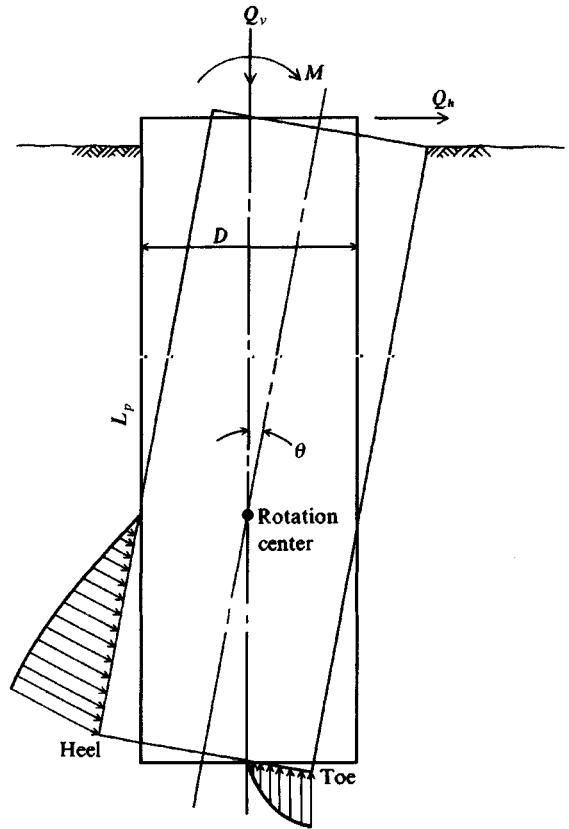


Figure 19-8 Idealization of rigid pier rotation with rotation angle θ greatly exaggerated. The toe and heel pressures will be highly indeterminate. Whereas toe pressure is nearly vertical, the heel pressure has both horizontal and vertical components, giving the slope shown.

19-9 LATERALLY LOADED DRILLED PIER ANALYSIS

Laterally loaded drilled piers can be analyzed using program FADBEMLP (B-5). There is some opinion that a short rigid pier is so stiff that the shaft will rigidly rotate about a point designated the center of rotation (see Fig. 19-8) and that a resisting moment will develop on the base from the toe and heel pressure profiles qualitatively shown. This moment is not accounted for in the usual FEM lateral pile program (unless we inspect the output from a trial run and arbitrarily select a possible base moment, which is input as an additional base node load on a subsequent trial).

It is immediately evident that if Fig. 19-8 is a correct representation of rigid pier-soil interaction, modeling it would be nearly impossible in any FEM/FD computer program unless one has a load test for a guide. In the author's opinion this model is not likely to develop unless the pier L_p/D ratio is less than about 2 except at lateral loads far in excess of the design load, e.g., lateral load tests are commonly taken to the limiting resistance of the pile or pier where the design load may only be one-fourth to one-half the ultimate load. Very short stub piers with L_p/D less than about 2 can probably be analyzed as footings with a passive pressure on the shaft about as accurately as trying to treat the stub pier as a rigid laterally loaded pier.

For larger L_p/D ratios the pier shaft, being substantially stiffer than the soil, will carry the lateral force similar to a laterally loaded pile. In any case, one can make a lateral pile-type analysis and inspect the output displacement of the bottom node. If there is a horizontal

displacement in the load direction much over 1 or 2 mm the analytical model *may* be inadequate *or* the lateral load is too large for the pier-soil system.

Lateral load tests on drilled piers of small L_p/D ratios tend to confirm that the base rotation of Fig. 19-8 is seldom of consequence. For example, Bhushan et al. (1978) report test results of a series of short drilled piers in the range of $L_p/D = 15/4 = 3.75$ to $22/4 = 5.5$. Some of the 1.22-m diameter shafts had 1.677-m diameter bells installed. They reported no discernible difference in capacity for shafts with bells versus no bells. Davisson and Salley (1968) reported the results of four laterally loaded test piers. For lateral loads up to about 450 kN the differences between the displacements of belled and straight-shaft piers were negligible. At near ultimate loads, however, the displacement differences were noticeable, with the bell tending to reduce the lateral displacement. Referring to Fig. 19-8 we see that in a rigid shaft rotation any bell should decrease rotation and increase the lateral load capacity of the pier.

To illustrate that the lateral pile FEM provides a reasonable solution, we will analyze a laterally loaded short drilled pier reported by Bhushan and Askari (1984). By citing a reference I do not use an excessive amount of text space for test details, and the reader can gain experience in trying to follow the work of others in developing his or her own experience base.

Bhushan et al. (1978) and Bhushan and Askari (1984) suggested that predicted displacements (that is, values computed in some manner) are in the range of two to six times measured values for laterally loaded piers. It should be noted in passing that a number of methods have been suggested in the *ASCE Geotechnical Journal*. Obviously if some of these give predictions in error by a factor of six [and most suggestions have been made since about 1960] they were worthless to begin with and should not have been published. The author readily concedes, however, that it is common at a site with similar piers (or piles) for lateral load test measurements to differ by ± 20 percent—sometimes more. The cause is the natural heterogeneity of the soil, which prompted the author to comment in Sec. 16-14 that one should not spend great effort in exactly matching a load test for site parameters. Any parameters obtained in this manner are strictly applicable for that test, and if they happen to match values for an adjacent test it is more a happy coincidence than computational rigor.

What one should try to do with load-test data is obtain average site parameters that are, one hopes, in an easy-to-use format so that changes can be made using commonly used soil parameters such as ϕ and s_u .

If you have a pier located on a slope refer to Sec. 16-15 for the necessary methodology to estimate the lateral modulus of subgrade reaction k_s .

Example 19-4. Use your computer program FADBEMLP and analyze pier No. 1 of Table 1 of Bhushan and Askari (1984). Figure E19-4a illustrates the general test setup as interpreted by the author. Figure E19-4b is the FEM used. The second node at 0.2 m from top was included since the lateral displacement of this node was given in Table 3 of the reference, which summarized the test results.

Solution. Obtain soil parameters as needed. The reference gave $\phi = 36^\circ$ and an average $\gamma = 99$ pcf, which the author rounds to $\gamma = 16 \text{ kN/m}^3$ since we will use all SI units. The load cases were given as follows:

LC	$P(2)$, kN	$P(1) = P(2) \times 4.88\text{m}$, kN·m
1	5.36	$-5.36(4.88) = -26.16$
2	9.00	-43.92
3	18.37	-86.64

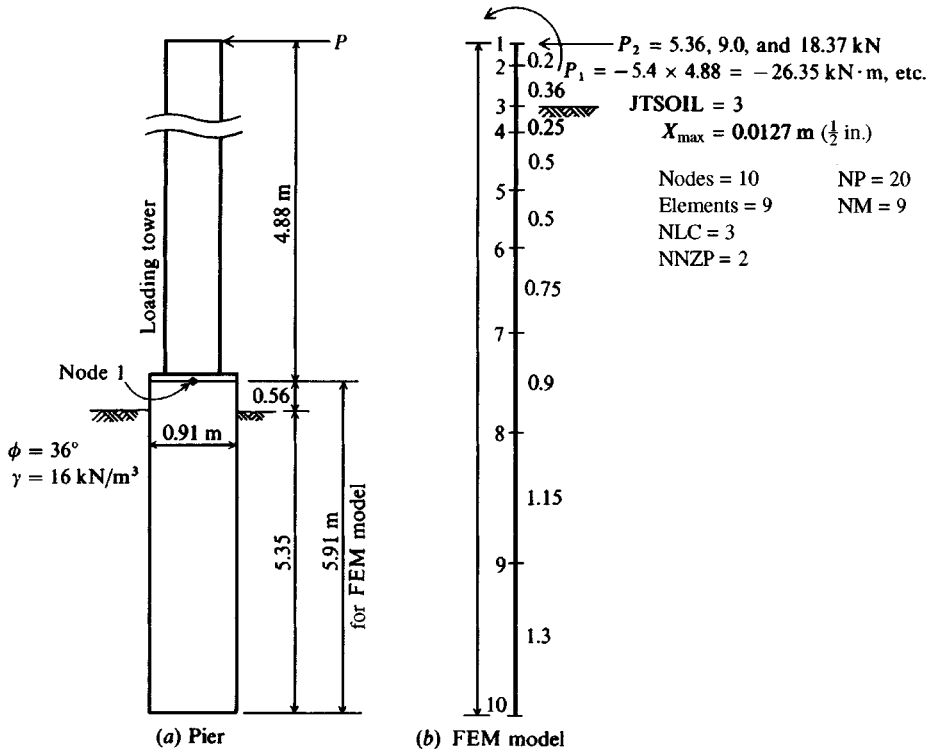


Figure E19-4a, b

Note that these are very small loads for piers of this size. We will use nine elements with lengths taken as shown in Fig. E19-4b. Use short elements in the upper region, grading into larger values. The ground line starts at node 3, giving JTSOIL = 3. Other data are as follows:

$$f'_c = 40 \text{ MPa} \quad (\text{given})$$

$$\text{Compute } E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{40} = 29\,700 \text{ MPa}$$

$$\text{Estimate maximum } \delta_h = 1/4 \text{ in.} = 0.0254/4 \text{ m}$$

$$C = 1/(0.0254/4) = 160 \text{ m}^{-1} \quad (\text{rounded})$$

$$C_m = 1 + (460/910)^{0.75} = 1.6 \quad [\text{see Eq. (16-26)}]$$

$$\text{Use shape factors } F_{w1} = 1.5 \text{ and } F_{w2} = 3 \quad [\text{see Eq. (16-26a)}]$$

$$\text{For } \phi = 36^\circ \text{ obtain } N_q = 38; \quad N_\gamma = 40 \quad (\text{Table 4-4})$$

$$\text{AS} = F_{w1} \times C \times C_m (0.5 \gamma B N_\gamma)$$

$$= 1.5 \times 160 \times 1.6 (0.5 \times 16 \times 0.91 \times 40) = 111\,820 \text{ kN/m}^3$$

$$\text{BS} = F_{w2} \times C \times C_m (\gamma Z^n N_q) = 3.0 \times 160 \times 1.6 \times 16 \times 38 Z^n = 466\,994 Z^n$$

We will arbitrarily use $n = 0.5$.

$$\text{The input } k_s = 112\,000 + 500\,000 Z^{0.5} \text{ (some rounding)}$$

$$\text{The moment of inertia } I = \frac{\pi D^4}{64} = \frac{\pi \times 0.91^4}{64} = 0.033\,66 \text{ m}^4$$

$$\text{With drilled shafts take } \text{FAC1} = \text{FAC2} = 1$$

DRILLED PIER FROM BHUSHAN & ASKARI IN ASTM STP 835--PP 140-156

***** THIS OUTPUT FOR DATA FILE: EXAM194.DTA

SOLUTION FOR Laterally Loaded Pile--ITYPE = 1 *****

NO OF NP = 20 NO OF ELEMENTS, NM = 9 NO OF NON-ZERO P, NNZP = 2
 NO OF LOAD CASES, NLC = 3 NO OF CYCLES NCYC = 1
 NODE SOIL STARTS JTISOIL = 3
 NONLINEAR (IF > 0) = 0 NO OF BOUNDARY CONDIT NZX = 0
 MODULUS KCODE = 2 LIST BAND IF > 0 = 0
 IMET (SI > 0) = 1

MEMNO	NP1	NP2	NP3	NP4	LENGTH	WIDTH	INERTIA, M**4
1	1	2	3	4	.200	.910	.33660E-01
2	3	4	5	6	.360	.910	.33660E-01
3	5	6	7	8	.250	.910	.33660E-01
4	7	8	9	10	.500	.910	.33660E-01
5	9	10	11	12	.500	.910	.33660E-01
6	11	12	13	14	.750	.910	.33660E-01
7	13	14	15	16	.900	.910	.33660E-01
8	15	16	17	18	1.150	.910	.33660E-01
9	17	18	19	20	1.300	.910	.33660E-01

THE INITIAL INPUT P-MATRIX ENTRIES

NP	LC	P(NP,LC)
1	1	-26.160
2	1	5.360
1	2	-43.920
2	2	9.000
1	3	-86.640
2	3	18.370

MOD OF ELASTICITY E = 29700. MPA

GROUND NODE REDUCTION FACTORS FOR PILES, FAC1,FAC2 = 1.00 .50

EQUATION FOR KS = 112000.0 + 500000.0*Z** .50

THE NODE SOIL MODULUS, SPRINGS AND MAX DEFL:

NODE	SOIL MODULUS	SPRING,KN/M	MAX DEFL, M
1	.0	.0	.0250
2	.0	.0	.0250
3	112000.0	15356.3	.0250
4	181000.0	86754.3	.0250
5	545012.7	229931.8	.0250
6	671017.0	388930.8	.0250
7	819106.8	617804.7	.0250
8	963469.3	905963.4	.0250
9	1118231.0	1249183.0	.0250
10	1268503.0	720690.9	.0250

BASE SUM OF NODE SPRINGS = 4214616.0 KN/M NO ADJUSTMENTS

* = NODE SPRINGS HAND COMPUTED AND INPUT

Figure E19-4c

MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC = 1

MEMNO	MOMENTS--NEAR	END 1ST, KN-M	NODE	SPG FORCE, KN	ROT, RADS	DEFL, M	SOIL Q, KPA	P-, KN-M	P-, KN
1	-26.160	27.232	1	.00	-.00008	.00011	.00	-26.16	5.36
2	-27.232	29.159	2	.00	-.00007	.00009	.00	.00	.00
3	-29.160	30.229	3	1.07	-.00006	.00007	7.84	.00	.00
4	-30.229	29.990	4	4.75	-.00006	.00005	9.92	.00	.00
5	-29.990	26.289	5	6.93	-.00004	.00003	16.42	.00	.00
6	-26.289	17.004	6	4.98	-.00003	.00001	8.58	.00	.00
7	-17.004	6.704	7	-.94	-.00001	.00000	1.24	.00	.00
8	-6.704	.211	8	-5.80	.00000	-.00001	6.17	.00	.00
9	-.211	.000	9	-5.48	.00000	.00000	4.91	.00	.00
			10	-.16	.00000	.00000	.29	.00	.00
SUM SPRING FORCES =		5.35 VS SUM APPLIED FORCES =		5.36		KN			

(*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++
 NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++

MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC = 2

MEMNO	MOMENTS--NEAR	END 1ST, KN-M	NODE	SPG FORCE, KN	ROT, RADS	DEFL, M	SOIL Q, KPA	P-, KN-M	P-, KN
1	-43.920	45.720	1	.00	-.00013	.00019	.00	-43.92	9.00
2	-45.720	48.957	2	.00	-.00013	.00016	.00	.00	.00
3	-48.957	50.753	3	1.80	-.00011	.00012	13.16	.00	.00
4	-50.753	50.352	4	7.98	-.00010	.00009	16.66	.00	.00
5	-50.352	44.137	5	11.63	-.00007	.00005	27.56	.00	.00
6	-44.137	28.549	6	8.35	-.00005	.00002	14.41	.00	.00
7	-28.549	11.257	7	-1.57	-.00002	.00000	2.08	.00	.00
8	-11.257	.354	8	-9.73	.00000	-.00001	10.35	.00	.00
9	-.354	.000	9	-9.21	.00001	-.00001	8.24	.00	.00
			10	-.27	.00001	.00000	.48	.00	.00
SUM SPRING FORCES =		8.99 VS SUM APPLIED FORCES =		9.00		KN			

(*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++
 NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++

MEMBER MOMENTS, NODE REACTIONS, DEFLECTIONS, SOIL PRESSURE, AND LAST USED P-MATRIX FOR LC = 3

MEMNO	MOMENTS--NEAR	END 1ST, KN-M	NODE	SPG FORCE, KN	ROT, RADS	DEFL, M	SOIL Q, KPA	P-, KN-M	P-, KN
1	-86.636	90.313	1	.00	-.00027	.00037	.00	-86.64	18.37
2	-90.313	96.920	2	.00	-.00025	.00032	.00	.00	.00
3	-96.919	100.606	3	3.60	-.00021	.00023	26.24	.00	.00
4	-100.608	100.014	4	15.93	-.00019	.00018	33.24	.00	.00
5	-100.014	87.796	5	23.25	-.00014	.00010	55.11	.00	.00
6	-87.796	56.890	6	16.77	-.00009	.00004	28.93	.00	.00
7	-56.890	22.496	7	-2.99	-.00004	.00000	3.97	.00	.00
8	-22.496	.745	8	-19.30	.00000	-.00002	20.53	.00	.00
9	-.745	.000	9	-18.34	.00001	-.00001	16.42	.00	.00
			10	-.57	.00001	.00000	1.01	.00	.00
SUM SPRING FORCES =		18.34 VS SUM APPLIED FORCES =		18.37		KN			

(*) = SOIL DISPLACEMENT > XMAX SO SPRING FORCE AND Q = XMAX*VALUE ++++++
 NOTE THAT P-MATRIX ABOVE INCLUDES ANY EFFECTS FROM X > XMAX ON LAST CYCLE ++++++

Figure E19-4c (continued)

With these data for input (see data set EXAM194.DTA on your diskette), we obtain the computer output shown on Fig. E19-4c. The displacements are summarized as follows:

<i>LC</i>	Measured δ_h , mm	Computed δ_h , mm	$R = \frac{\text{Computed}}{\text{Measured}}$
1	0.074	0.09	1.22
2	0.163	0.16	1.00
3	0.0351	0.32	0.91

Discussion of output

1. The computed output compares quite well with the load test values. The foregoing data represent some revisions to the execution given in the fourth edition; that is, k_s is adjusted for factor C_m , an improved (smaller) F_{wi} is used, and we have taken into account that the k_s should be representative of the small displacements (under $\frac{1}{4}$ in.) of this system.
2. With such a large shaft and such small lateral loads, the computed and measured δ_h are almost meaningless. What one generally hopes to avoid is a measured $\delta_h = 50$ mm when the computed value is only 20 or 25 mm.
3. The equation for k_s is not an “after the fact” development, so it can be used with reasonable confidence for other cases.
4. One might question if a shaft diameter this large should be considered a “deep” beam.
5. The program makes several self-checks, so it would seem it is making correct computations—or at least correct for this set of input.
6. The displacements at the bottom three nodes are either zero or so near zero that we can say they are. That is, the shaft—at least in this load range—is behaving similarly to any laterally loaded pile.
7. The ground line moment (node 3) is readily checked for all three cases as simply the input moment $+0.56P_h$. For $LC = 3$ we obtain

$$M_{gl} = 86.64 + 0.56 \times 18.37 = \mathbf{96.92 \text{ kN} \cdot \text{m}}$$

as on the output sheet for node 3.

////

19-10 DRILLED PIER INSPECTION AND LOAD TESTING

The drilled pier (or caisson) usually carries a very large load, so structural integrity is an absolute necessity. This is partially achieved by an inspection of the shaft cavity. When the shaft is cased, a person may enter to check the base for loose material. If the base is in rock, it can be checked for cracks or voids and loose material; however, present technology is at a state where equipment is available to precondition the shaft sides and to clean the base of loose material. When the base is on soil, it is often desirable to check the bearing capacity manually (and visually), using a pocket penetrometer to obtain the unconfined compression strength q_u at a number of points similar to the testing illustrated in Fig. 3-9a. A visual comparison of the actual shaft soil with the original boring logs is of much value. Usually at this point it is not too late to make a rapid redesign if the shaft soil is found to be different from the original borings. When the shaft is not cased, the diameter is too small for an inspector to enter, or hazardous gas is being emitted, it may be possible to lower a video camera to obtain an indirect visual

check of shaft conditions. If a video camera is not available, it may be possible to get some indication of shaft condition and vertical alignment by lowering a light into the cavity. If the light disappears, the shaft is not vertical; soil crumbs may be visible on the pier base soil (if the shaft is vertical and not too deep); the condition of the shaft sides may be visible at least in the upper part.

It is usually specified that the inspector do at least the following:

1. Perform a specified number of slump tests on the wet concrete.
2. Take a specified number of concrete cylinders for later strength testing.
3. Observe and compare the volume of concrete placed in the pier shaft (and bell if used) to the shaft volume. It is self-evident that if less than the shaft volume of concrete is placed, there is some kind of discontinuity in the shaft. This is usually the first verification of pier integrity.

There are electronic test devices [see, for example, Lin et al. (1991)] that can measure a seismic wave down the shaft after the concrete has hardened (nondestructive testing, NDT) to ascertain whether any voids or discontinuities are present. A core sample is considered to be more reliable, but it is usually too costly (and permanently damages the pier some amount); it may be done if the concrete strength f'_c is suspect or if litigation is pending.

The ACI committee 336 has two current specifications, titled *Standard Specification for the Construction of Drilled Piers* and *Design and Construction of Drilled Piers*, which can be obtained from the ACI; they give a number of suggested inspection procedures to ensure the quality of the drilled pier.

Pier load testing. Load-testing a drilled pier for its capacity is a difficult task, since large piers carry substantial load and conventional testing, similar to that for piles, requires a large load frame (see Fig. 17-7c).

A recent development is to put a high-capacity hydraulic jack, termed an *O-cell*, onto a plate 1 on the base soil of the pier (shaft or bell), and an upper plate 2 against which the bell/shaft is poured. Hydraulic and electronic pickup lines are routed to the ground surface for later use. When the pier concrete hardens, the jack is activated to attempt to separate plates 1 and 2; the resistance can be related to point bearing. If the lower plate 1 has been referenced to a known elevation (a surface reference frame), the change in elevation caused by the jack load is related to point settlement and to side skin resistance. This pier load test is termed an *O-test* (also an *upside-down load test*, because the load is applied at the base and pier movement is upward) and has been in use since about 1985 [Goodwin (1993), Meyer and Schade (1995)].

PROBLEMS

In any economic analysis assume f'_c costs (\$100/7 MPa per m^3) over the base strength of 21 MPa—that is, 28 MPa costs \$100/ m^3 more than 21 MPa strength concrete; 35 MPa is \$200/ m^3 more, etc.

19-1. Compute α for the three 9-m ΔL increments of clay in Example 19-1.

19-2. In Example 19-1 what ϕ angle for the sand layers together with $\alpha = 0.5$ for the 27 m of clay and the computed point value would give the load test value $P_u \approx 410$ kN? Is this angle realistic (you should try to obtain a copy of the original source)?

- 19-3.** Using the given ϕ angles and $\alpha = 0.5$, what s_u would you have to use to give the load test value of $P_u \approx 410$ kN for Example 19-1? Remember the point value also changes, so that Q_p must also be recomputed.
- 19-4.** Verify the skin resistance computations shown on Fig. E19-1.
- 19-5.** Compare the quantity of concrete required in Example 19-2 to that required if we extended the shaft to bedrock at 33 m below the ground surface and the rock $q_a = 28$ MPa.
- 19-6.** What shaft diameter would be required for the drilled pier of Example 19-2 with the point at -21 m elevation?
- 19-7.** For Example 19-3, what shaft length is required to eliminate the need for a bell? Would it be more economical to increase the shaft diameter D_s ? Use a single α as in the example.
- 19-8.** Redo Example 19-3 using at least four clay layers instead of one and compute α for each layer using Eq. (16-12a). Use $Q_p = Q_{ult}/3$ for the point contribution and skin resistance $SF = 2$ as in the example.
- 19-9.** Would the drilled pier of Example 19-3 be more economical using $f'_c = 28$ MPa (example uses 35 MPa)?
- 19-10.** Design a drilled pier for a column load of 4500 kN using the soil profile shown in Fig. P19-10. Soil data is from “undrained” tests.

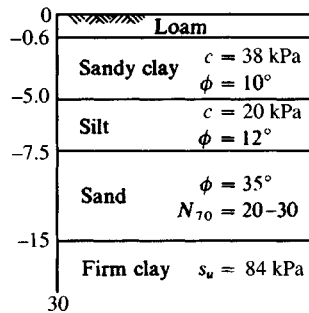


Figure P19-10

- 19-11.** Design a drilled pier for the soil profile of Fig. P19-11 for a 5000-kN axial load. Use a bell if it will be more economical.

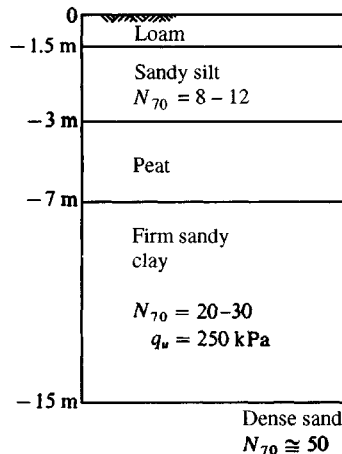


Figure P19-11

All of the following problems require use of your lateral pile/pier program FADBEMLP.

- 19-12. Verify the output of Fig. E19-4c using data set EXAM194.DTA on your program diskette.
- 19-13. Verify the c_m side resistance factor of 1.6 for Example 19-4. Do you think 1.6 or 2.0 is a better value for these piers?
- 19-14. Redo Example 19-4 using $I = 0.0370 \text{ m}^4$ (a 10 percent increase from the example) to allow some increase in stiffness for the rebar cage. If we assume the pier contained 15 No. 20 rebars on a radius of 0.70 m, what is the computed moment of inertia I ? How does this compare to the moment of inertia of the gross section actually used of 0.03366 m^4 ?
- 19-15. Redo Example 19-4 using the exponent $n = 0.4, 0.75$, and 1.0 . Compare your results with the output given (which used $n = 0.5$). Plot P_h versus δ_h for each n value onto the same curve together with the measured values for a visual comparison.
- 19-16. Make a literature search for a laterally loaded drilled pier in a cohesive soil and see if you can back-compute the ground line displacements using your program FADBEMLP.

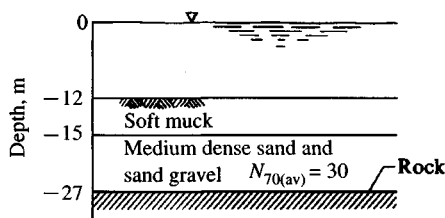


Figure P19-17

- 19-17. Outline considerations you think necessary to design a large-diameter pile or caisson/pier (whatever you want to call it) for a bridge pier for the water-soil-rock profile of Fig. P19-17. The pier top is 6 m above water and carries an axial load of 36 500 kN and a lateral load of 500 kN.