University of Anbar
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CHAPTER ONE

DRILLED-SHAFT AND CAISSON FOUNDATIONS

LECTURE
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2017

1.1 Introduction

The terms *caisson, pier, drilled shaft,* and *drilled pier* are often used interchangeably in foundation engineering; all refer to a *cast-in-place pile generally having a diameter of about 750 mm* or more, with or without steel reinforcement and with or without an enlarged bottom. Sometimes the diameter can be as small as 305 mm.

The use of drilled-shaft foundations has several advantages:

1. A single drilled shaft may be used instead of a group of piles and the pile cap.
2. Constructing drilled shafts in deposits of dense sand and gravel is easier than Driving piles.
3. Drilled shafts may be constructed before grading operations are completed.
4. When piles are driven by a hammer, the ground vibration may cause damage to nearby structures.
5. Piles driven into clay soils may produce ground heaving and cause previously driven piles to move laterally.
6. There is no hammer noise during the construction of drilled shafts**.**
7. Because the base of a drilled shaft can be enlarged, it provides great resistance to the uplifting load.
8. The surface over which the base of the drilled shaft is constructed can be visually inspected.
9. The construction of drilled shafts generally utilizes mobile equipment.
10. Drilled shafts have high resistance to lateral loads.

1.2 Types of Drilled Piers

Drilled piers may be described under four types. All four types are similar in construction technique, but differ in their design assumptions and in the mechanism of load transfer to the surrounding earth mass. These types are illustrated in **Fig. 1.1** and as following:

1. *Straight –shaft end-bearing piers* develop their support from end-bearing on strong soil, " hardpan" or rock. The overlying soil is assumed to contribute nothing to the support of the load imposed on the pier**[Fig.1.1(a)]**.
2. *Straight-shaft side wall friction piers* pass through overburden soils that are assumed to carry none of the load, and penetrate far enough into an assign bearing stratum to develop design load capacity by side wall friction between the pier and bearing stratum to develop design load capacity by side wall friction between the pier and bearing stratum**[Fig. 1.1(b)]**.
3. *Combination of straight shaft side wall friction and end bearing piers* are of the same construction as the two mention above, but with both side wall friction and end bearing assigned a role in carrying the design load. When carried into rock, this pier may be referred to as a socketed pier or a " drilled pier with rock socket"**[Fig. 1.1(c)]**.

4. *Belled or underreamed piers* with a bottom bell or underream[Fig.1.1(d)]. A greater percentage of the imposed load on the pier top is assumed to be carried by the base.

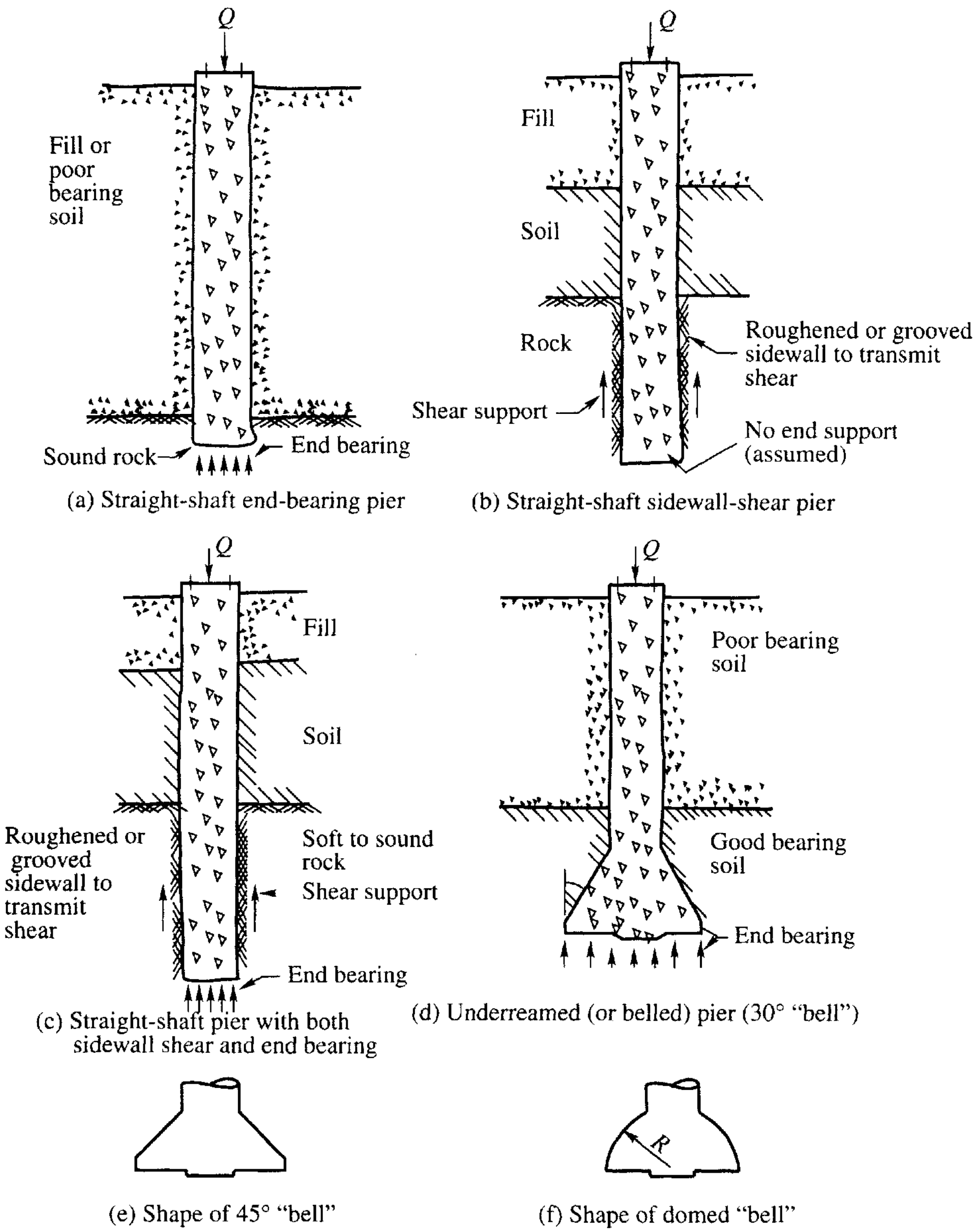


Fig 1.1 Types of drilled piers and underream shapes (Woodward et al., 1972)

1.3 Advantages and Disadvantages of Drilled Pier Foundations

Advantages

1. Pier of any length and size can be constructed at the site
2. Construction equipment is normally mobile and construction can proceed rapidly
3. Inspection of drilled holes is possible because of the larger diameter of the shafts
4. Very large loads can be carried by a single drilled pier foundation thus eliminating the necessity of a pile cap
5. The drilled pier is applicable to a wide variety of soil conditions
6. Changes can be made in the design criteria during the progress of a job
7. Ground vibration that is normally associated with driven piles is absent in drilled pier construction
8. Bearing capacity can be increased by underreaming the bottom (in non-caving materials)

Disadvantages

1. Installation of drilled piers needs a careful supervision and quality control of all the materials used in the construction
2. The method is cumbersome. It needs sufficient storage space for all the materials used in the construction
3. The advantage of increased bearing capacity due to compaction in granular soil that could be obtained in driven piles is not there in drilled pier construction
4. Construction of drilled piers at places where there is a heavy current of ground water flow due to artesian pressure is very difficult

1.4 Construction Procedures

There are three major types of construction methods: the dry method, the casing method,
and the wet method.

Dry Method of Construction

This method is employed in soils and rocks that are above the water table and that will not cave in when the hole is drilled to its full depth. The sequence of construction, shown in **Figure 1.2**, is as follows:

Step 1. The excavation is completed (and belled if desired), using proper drilling tools, and the spoils from the hole are deposited nearby. **(See Fig. 1.2a.)**

Step 2. Concrete is then poured into the cylindrical hole. **(See Fig. 1.2b.)**

Step 3. If desired, a rebar cage is placed in the upper portion of the shaft. **(See Fig. 1.2c.)**

Step 4. Concreting is then completed, and the drilled shaft will be as shown in **Fig. 1.2d**.

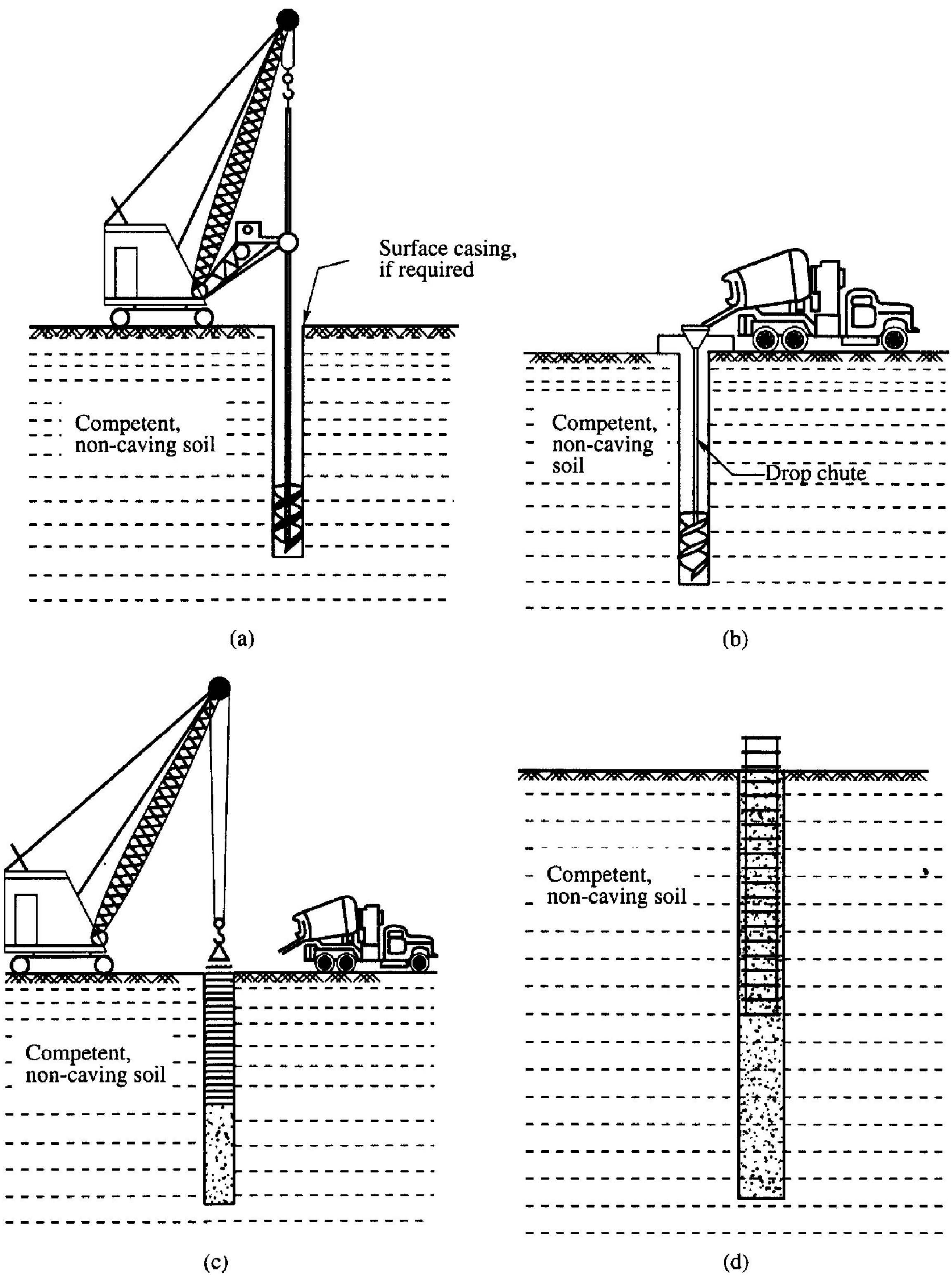


Fig. 1.2 Dry method of construction: (a) initiating drilling; (b) starting concrete pour; (c) placing rebar cage; (d) completed shaft (Based on O'Neill and Reese, 1999)

Casing Method of Construction

This method is used in soils or rocks in which caving or excessive deformation is likely to occur when the borehole is excavated. The sequence of construction is shown in Fig. 1.3 and may be explained as follows:

Step 1. The excavation procedure is initiated as in the case of the dry method of construction. (See Fig. 1.3a.)

Step 2. When the caving soil is encountered, bentonite slurry is introduced into the borehole. (See Fig. 10.3b.) Drilling is continued until the excavation goes past the caving soil and a layer of impermeable soil or rock is encountered.

Step 3. A casing is then introduced into the hole. (See Fig. 1.3c.)

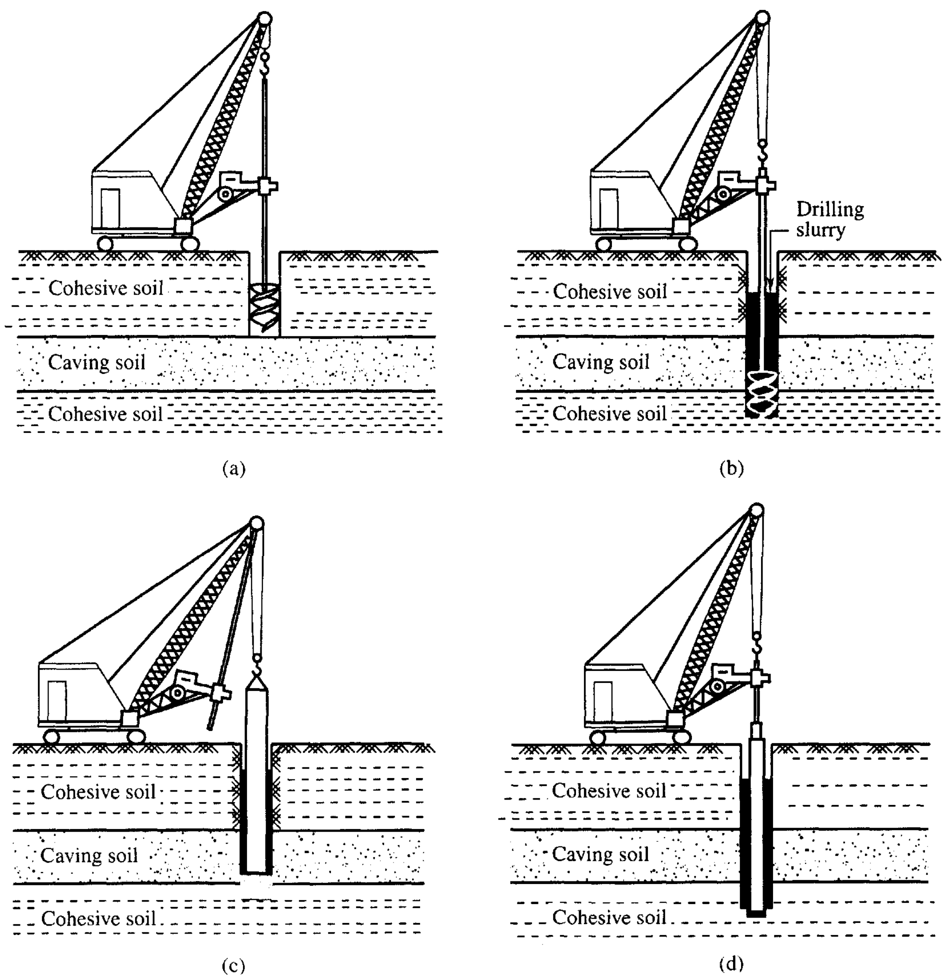
Step 4. The slurry is bailed out of the casing with a submersible pump. (See Fig. 1.3d.)

Step 5. A smaller drill that can pass through the casing is introduced into the hole, and excavation continues. (See Fig. 1.3e.)

Step 6. If needed, the base of the excavated hole can then be enlarged, using an underreamer. (See Fig. 1.3f.)

Step 7. If reinforcing steel is needed, the rebar cage needs to extend the full length of the excavation. Concrete is then poured into the excavation and the casing is gradually pulled out. (See Fig. 1.3g.)

Step 8. Fig. 1.3h shows the completed drilled shaft.



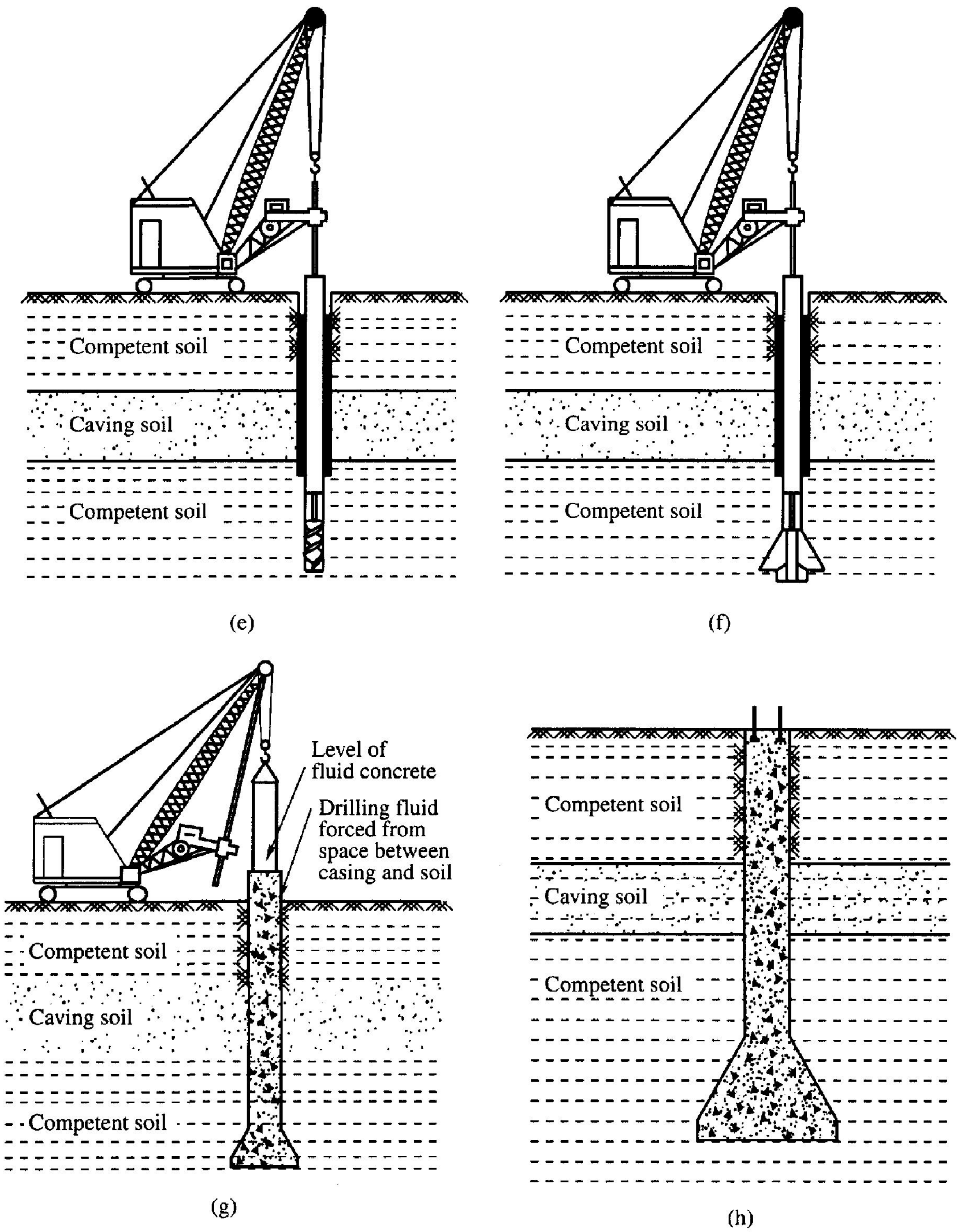


Fig. 1.3 Casing method of construction: (a) initiating drilling; (b) drilling with slurry; (c) introducing casing; (d) casing is sealed and slurry is being removed from interior of casing; (e) drilling below casing; (f) underreaming; (g) removing casing; (h) completed shaft (Based on O'Neill and Reese, 1999)

Wet Method of Construction

This method is sometimes referred to as the *slurry displacement method.* Slurry is used to
keep the borehole open during the entire depth of excavation. (See Figure 1.4) Following
are the steps involved in the wet method of construction:

Step 1. Excavation continues to full depth with slurry. (See Figure 1.4a.)

Step 2. If reinforcement is required, the rebar cage is placed in the slurry. (See Figure 1.4b.)

Step 3. Concrete that will displace the volume of slurry is then placed in the drill hole. (See Figure 1.4c.)

Step 4. Figure 1.4d shows the completed drilled shaft.

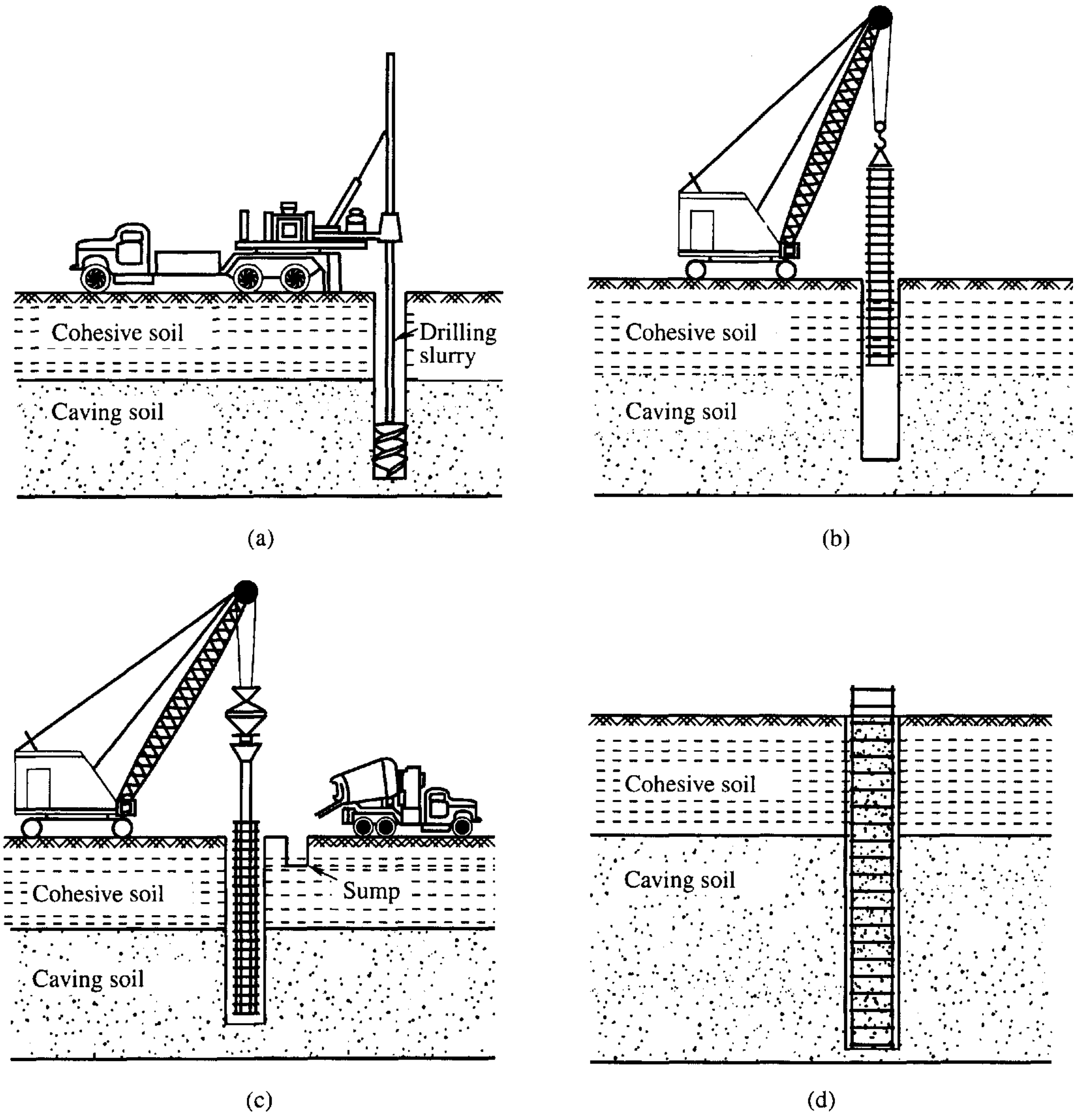


Figure 10.4 Slurry method of construction: (a) drilling to full depth with slurry; (b) placing rebar cage; (c) placing concrete; (d) completed shaft (After O’Neill and Reese, 1999)

1.5 DESIGN CONSIDERATIONS

The process of the design of a drilled pier generally involves the following:
1 . The objectives of selecting drilled pier foundations for the project.
2. Analysis of loads coming on each pier foundation element.
3. A detailed soil investigation and determining the soil parameters for the design.
4. Preparation of plans and specifications which include the methods of design, tolerable settlement, methods of construction of piers, etc.
5.The method of execution of the project.

In general the design of a drilled pier may be studied under the following headings:
1. Allowable loads on the piers based on ultimate bearing capacity theories.
2. Allowable loads based on vertical movement of the piers.
3. Allowable loads based on lateral bearing capacity of the piers.

In addition to the above, the uplift capacity of piers with or without underreams has to be evaluated. The following types of strata are considered.
1 . Piers embedded in homogeneous soils, sand or clay.
2. Piers in a layered system of soil.
3. Piers socketed in rocks.
It is better that the designer select shaft diameters that are multiples of 150 mm (6 in) since these are the commonly available drilling tool diameters.

For the design of ordinary drilled shafts without casings, a minimum amount of vertical steel reinforcement is always desirable. Minimum reinforcement is 1% of the gross cross-sectional area of the shaft. For drilled shafts with nominal reinforcement, most building codes suggest using a design concrete strength, fc , on the order of $f'c/4$. Thus, the minimum shaft diameter becomes

$$D_s = \sqrt{\frac{Q_w}{\left(\frac{\pi}{4}\right)(0.25)f_c'}} = 2.257\sqrt{\frac{Q_w}{f_c'}} \qquad (1\text{-}1)$$

where
D_s = diameter of the shaft
$f'c$ = 28-day concrete strength
Q_w = working load of the drilled shaft
A_{gs} = gross cross-sectional area of the shaft

If drilled shafts are likely to be subjected to tensile loads, reinforcement should be continued
for the entire length of the shaft.

Concrete Mix Design

The concrete mix design for drilled shafts is not much different from that for any other concrete structure. When a reinforcing cage is used, consideration should be given to the ability of the concrete to flow through the reinforcement. In most cases, a concrete slump of about 15.0 mm (6 in.) is considered satisfactory. Also, the maximum size of the aggregate should be limited to about 20 mm (0.75 in.).

1.6 Estimation of Load-Bearing Capacity - General

The load-transfer mechanism from drilled shafts to soil is similar to that of piles as last described chapter. The ultimate load-bearing capacity of a drilled shaft **(Fig. 1.5)** is

$$Q_u = Q_p + Q_s \qquad (1\text{-}2)$$

where

Q_u= ultimate load

Q_p = ultimate load-carrying capacity at the base

Q_s = frictional (skin) resistance

The equation for the ultimate base load is similar to that for shallow foundations:

$$Q_p = A_p(c'N_c^* + q'N_q^* + 0.3\gamma D_b N_\gamma^*) \qquad (1\text{-}3)$$

Where

N_c^*, N_q^*, N_γ^* = the bearing capacity factors

q'= vertical effective stress at the level of the bottom of the pier

D_b= diameter of the base (see Fig. 1.5a and b)

A_p= area of the base= $\pi/4 D_b^2$

In most cases, the last term (containing N_γ^*) is neglected except for relatively short drilled shafts, so

$$Q_p = A_p(c'N_c^* + q'N_q^*) \qquad (1\text{-}4)$$

The net load-carrying capacity at the base (that is, the gross load minus the weight of the drilled shaft) may be approximated as

$$Q_{p(net)} = A_p(c'N_c^* + q'N_q^* - q' = A_p[c'N_c^* + q'(N_q^* - 1)] \qquad (1\text{-}5)$$

The expression for the frictional, or skin, resistance, Q_s, is similar to that for piles

$$Q_s = \int_0^{L_1} pf\,dz \tag{1-6}$$

Where

p=shaft perimeter=πD_s

f=unit frictional (or skin)resistance

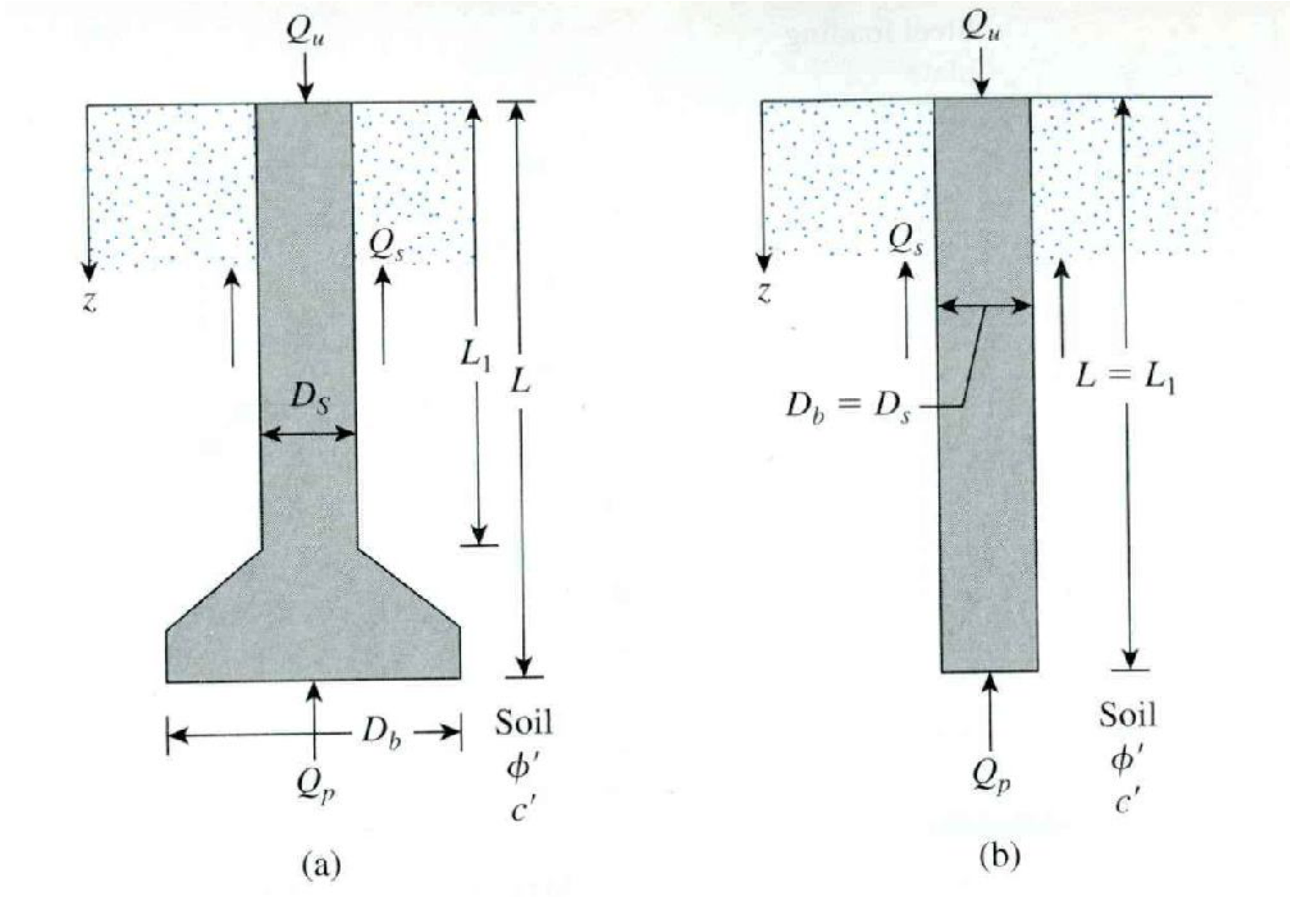


Figure 1.5 Ultimate bearing capacity of drilled shafts: (a) with bell and (b) straight shaft

The following two sections describe the procedures for obtaining the ultimate and allowable load-bearing capacities of drilled shafts in sand and clay.

1.6 Drilled Shafts in Sand: Load-Bearing Capacity

Estimation of Q_p

For drilled shafts in sand, c'=0 and , hence Eq. (1-5) simplifies to

$$Q_{p(net)} = A_p q'(N_q^* - 1) \quad (1\text{-}7)$$

Determination of N_q^* is always a problem for deep foundation, as in the case of piles. Note, however, that all shafts are drilled, unlike the majority of piles, which are driven. The values of N_q^* given by Vesic(1963) are approximately the lower bound, and hence are used in this chapter **(Fig. 1-6)**

The frictional resistance at ultimate load, Q_s, developed in a drilled shaft may be calculated from the relation given in Eq.(1-6), in which

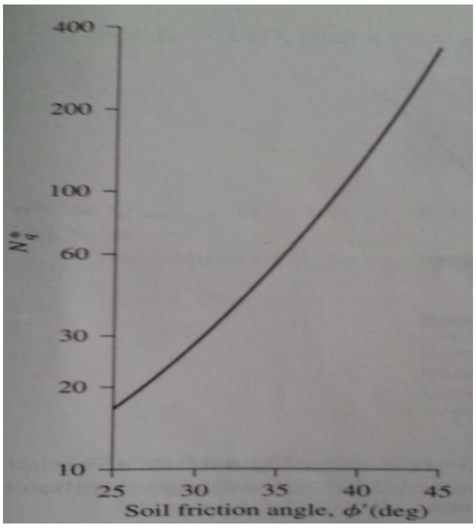


Fig. 1.6 Vesic's bearing capacity factors, N_q^*, for deep foundations

The magnitude of $Q_{p(\text{net})}$ also can be reasonably estimated from a relationship based on the analysis of Berezantzev et al. (1961) that can be expressed as

$$Q_{p(net)} = A_p q'(\omega N_q^* - 1) \quad (1.8)$$

where

N_q^*= bearing capacity factor = $0.21e^{0.17\phi'}$ (See Table 1.1) (1.9)

ω = correction factor = $f\,(L/D_b)$

In Eq. (1.9), ϕ' is in degrees. The variation of v (interpolated values) with L/D_b is given in Fig. (1.7).

Table 1.1 Variation of N_q^* with ϕ' [Eq. (1.9)]

ϕ' (deg)	N_q^*
25	14.72
26	17.45
27	20.68
28	24.52
29	29.06
30	34.44
31	40.83
32	48.39
33	57.36
34	67.99
35	80.59
36	95.52
37	113.22
38	134.20
39	159.07
40	188.55
41	223.49
42	264.90
43	313.99
44	372.17
45	441.14

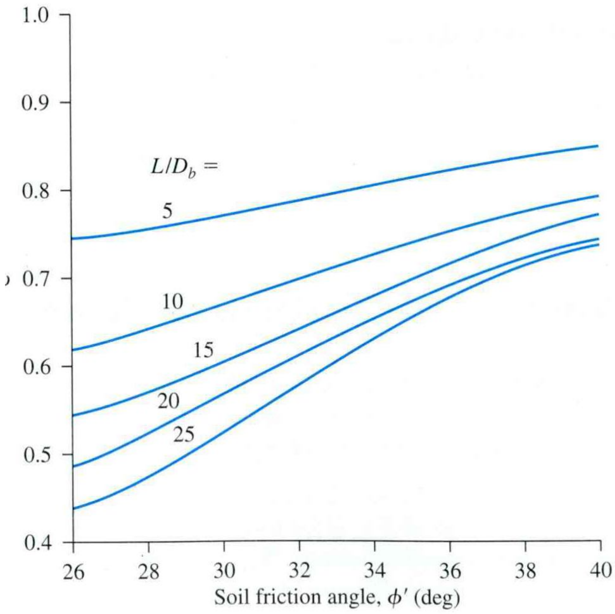


Fig. 1.7 Variation of ω with ϕ' and L/D_b

Estimation of Q_s

The frictional resistance at ultimate load, Q_s , developed in a drilled shaft may be calculated as

$$Q_s = \int_0^{L_1} pf\,dz \qquad (1\text{-}6)$$

p= shaft perimeter= πD_s
f= unit frictional (or skin) resistance= $K\sigma_o' \tan\delta'$ (1-10)
K= earth – pressure coefficient ≈ K_o= 1-sinϕ'
σ_o'= effective vertical stress at any depth z
Thus,

$$Q_s = \int_0^{L_1} pf\,dz = \pi D_s(1 - sin\emptyset') \int_0^{L_1} \sigma_o' \tan\delta'\,dz \qquad (1\text{-}11)$$

The value of σ_o' will increase to a depth of about $15D_s$ and will remain constant thereafter, as shown in Figure 1.8.
For cast-in-pile concrete and good construction techniques, a rough interface develops and, hence, δ'/ϕ' may be taken to be one. With poor slurry construction,

$\delta'/\phi' < 0.7$ to 0.8.

Allowable Net Load, $Q_{all\,(net)}$

An appropriate factor of safety should be applied to the ultimate load to obtain the net allowable load, or

$$Q_{net(all)} = \frac{Q_{p(net)} + Q_s}{FS} \tag{1-12}$$

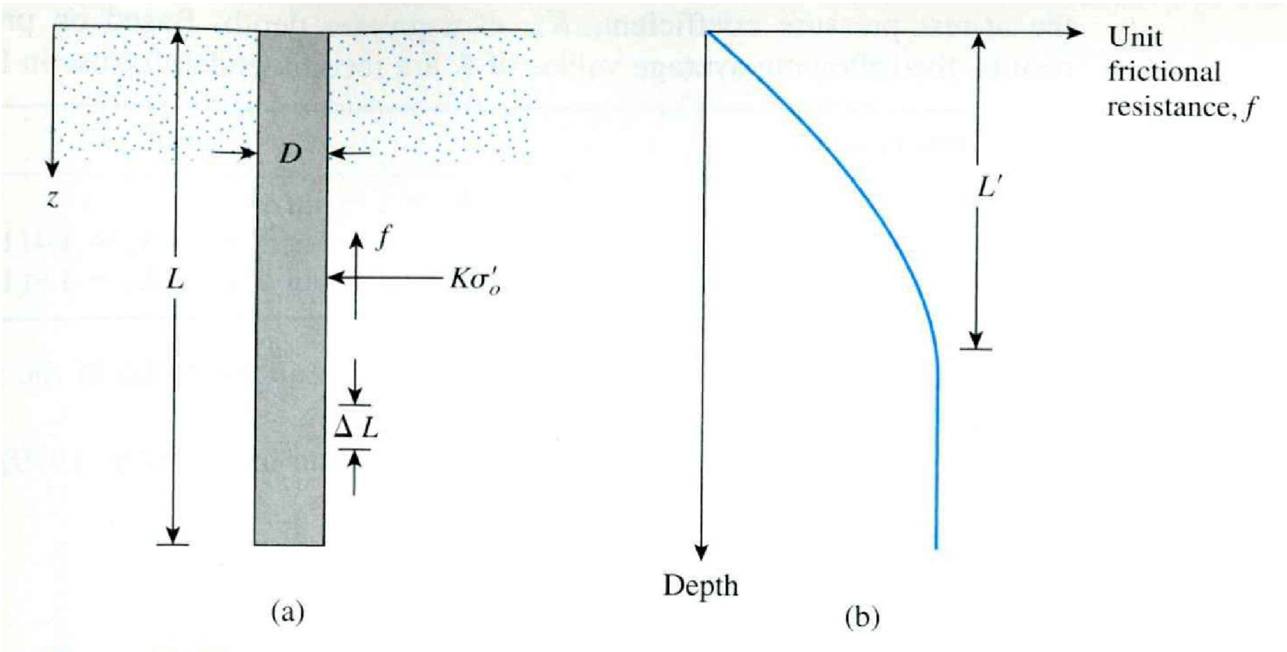


Fig. 1.8 Unit frictional resistance for piles in sand

1.7 Load Bearing Capacity Based on Settlement

On the basis of a database of 41 loading tests, Reese and O'Neill (1989) proposed a method for calculating the load-bearing capacity of drilled shafts that is based on settlement. The method is applicable to the following ranges:

1. Shaft diameter: D_s = 0.52 to 1.2 m (1.7 to 3.93 ft)
2. Bell depth: L = 4.7 to 30.5 m (15.4 to 100 ft)
3. Field standard penetration resistance: $N60$ = 5 to 60
4. Concrete slump = 100 to 225 mm (4 to 9 in.)

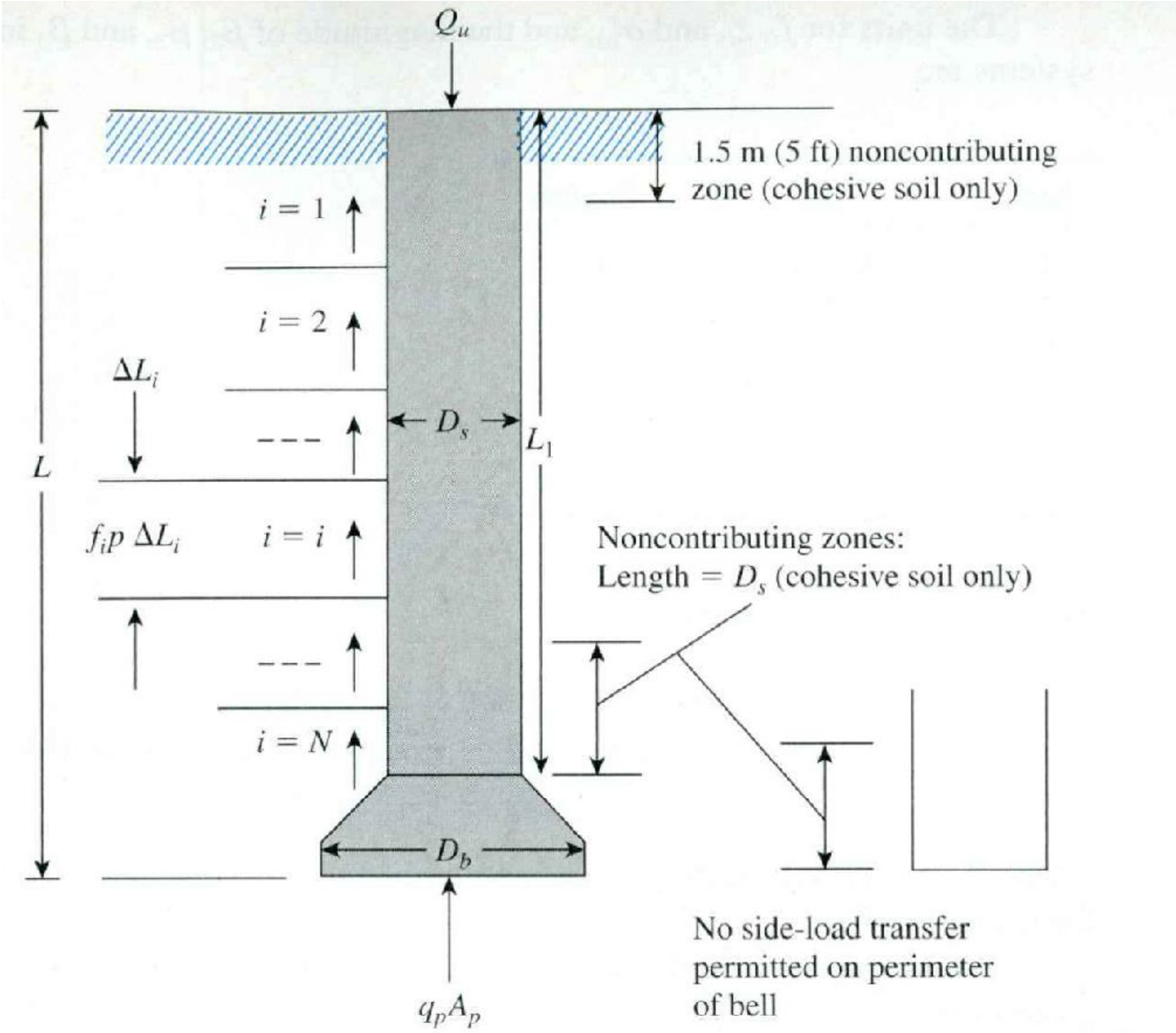


Fig. 1.9 Development of Eq.(1-13)

Reese and O'Neill's procedure (see Figure 10.10) gives

$$Q_{u(net)} = \sum_{i=1}^{N} f_i p \Delta L_i + q_p A_p \qquad (1\text{-}13)$$

where

f_i = ultimate unit shearing resistance in layer i

p = perimeter of the shaft = pD_s

qp = unit point resistance

Ap = area of the base = $(\pi/4)D^2_b$

Following are the relationships for determining $Q_{u(net)}$ in granular soils. Based on **Eq. (1-13)**

$$f_i = \beta \sigma'_{ozi} \leq 192\ kN/m^2 \qquad (1\text{-}14)$$
$$\beta = 1.5 - 0.244 z_i^{0.5} \qquad (0.25 \leq \beta \leq 1.2) \qquad (1\text{-}15)$$

(where z_i is in m)

$$q_p\left(\frac{kN}{m^2}\right) = 57.5\ N_{60} \leq 4310\frac{kN}{m^2} \qquad (for\ D_b < 1.27m) \qquad (1\text{-}16)$$

If $D_b \geq 1.27$m, excessive settlement may occur. In that case, q_p may be replaced by

$$q_{pr} = \frac{1.27}{D_b(m)} q_p \qquad (1\text{-}17)$$

Figs. 1.10 and 1.11 may now be used to calculate the allowable load $Q_{all(net)}$ based on the desired level of settlement. Example 1.2 shows the method of calculating the net allowable load.

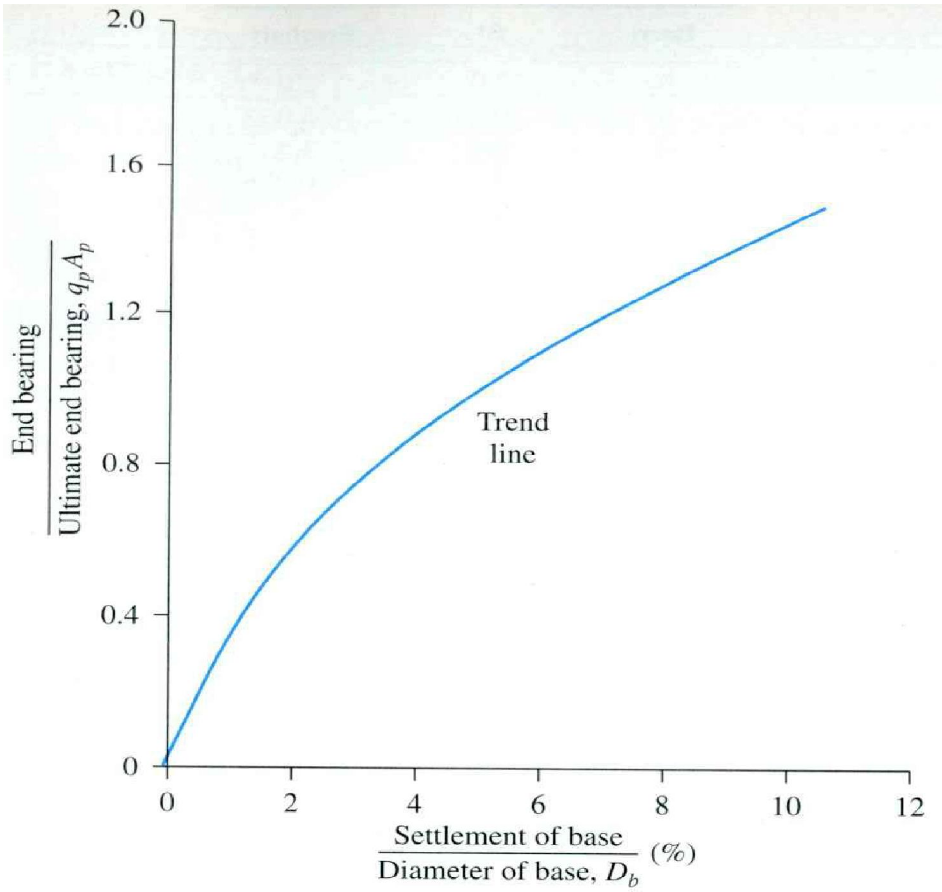


Fig. 1-10 Normalized based-load transfer versus settlement of sand

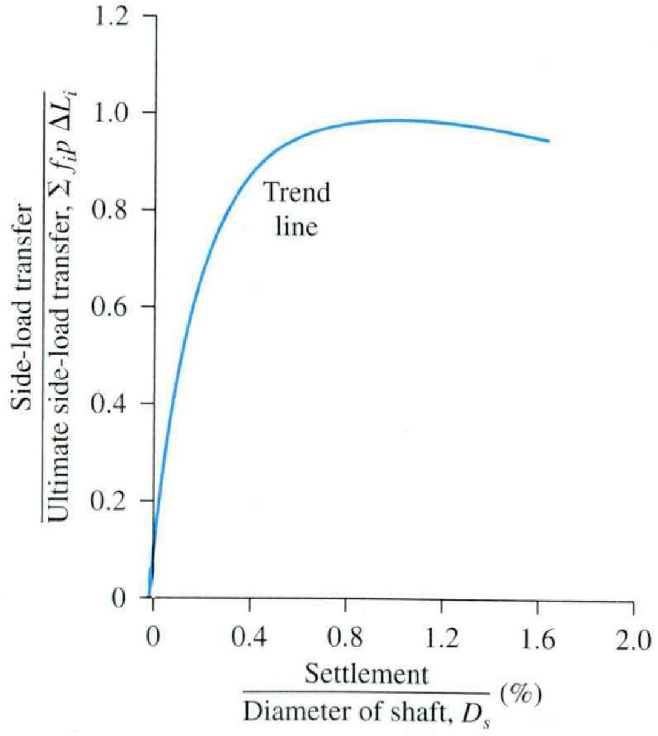


Fig. 1.11 Normalized side-load transfer versus settlement in sand

Example 1.1
Example 1.2

1.8 Drilled Shafts in Clay: Load-Bearing Capacity

From Eq.(1-5), For saturated clays with ϕ =0, N_q^* =1; hence the net base resistance becomes

$$Q_{p(net)} = A_p c_u N_c^* \quad (1\text{-}18)$$

Where c_u= undrained cohesion

The bearing capacity factor N_c^* is usually taken to be 9. When the L/D_b≥ 4, N_c^*=9, which is the condition for most drilled shafts.

Experiments by **Whitaker and Cooke (1966)** showed that, for belled shafts, the full value of N_c^* =9 is realized with a base movement of about 10 to 15% of D_b. Similarly, for straight shafts (D_b=D_s), the full value of N_c^*=9 is obtained with a base movement of about 20% of D_b.

The expression for the skin resistance of drilled shafts in clay is

$$Q_s = \sum_{L=0}^{L=L_1} \alpha^* \, c_u p \Delta L \quad (1\text{-}19)$$

Where p= perimeter of the shaft cross section.
the value of α^* that can be used in Eq. (1-19) has not yet been fully established however, the field test results available at this time indicate that α^* may vary between 1.0 to 0.3. **Kulhawy and Jackson (1989)** reported the field-test result of 106 straight drilled shafts—65 in uplift and 41 in compression. The best correlation obtained from the results is

$$\alpha^* = 0.21 + 0.25\left(\frac{p_a}{c_u}\right) \leq 1 \quad (1\text{-}20)$$

Where p_a= atmospheric pressure=100 kN/m^2.

So, conservatively, we may assume that

$$\alpha^* = 0.4 \quad (1\text{-}21)$$

Load-Bearing Capacity Based on Settlement

Reese and O'Neill (1989) suggested a procedure for estimating the ultimate and allowable (based on settlement) bearing capacities for drilled shafts in clay. According to this procedure, we can use Eq. (1-13) for the net ultimate load, or

$Q_{u(net)} = \sum_{i=1}^{N} f_i p \Delta L_i + q_p A_p$ (1-13)

The unit skin friction resistance can be given as

$f_i = \alpha_i^* c_{u(i)}$ (1-22)

The following values are recommended for α_i^*:

α_i^* =0 for the top 1.5m (5 ft) and bottom 1 diameter, D_s, of the drilled shaft. (*Note:* If $D_b > D_s$, then $a^* = 0$ for 1 diameter above the top of the bell and for the peripheral area of the bell itself.)

α_i^* =0.55 elsewhere.

The expression for q_p (point load per unit area) can be given as

$q_p = 6c_{ub}\left(1 + 0.2\frac{L}{D_b}\right) \leq 9c_{ub} \leq 40p_a$ (1-23)

where
c_{ub} = average undrained cohesion within a vertical distance of $2D_b$ below the base
p_a = atmospheric pressure

If D_b is large, excessive settlement will occur at the ultimate load per unit area, q_p, as given by Eq. (1.23). Thus, for D_b >1.91 m (75 in.), q_p may be replaced by

$q_{pr} = F_r q_p$ (1-24)

Where

$F_r = \frac{2.5}{\varphi_1 D_b(mm) + \varphi_2} \leq 1$ (1-25)

$\varphi_1 = 2.78 \times 10^{-4} + 8.26 \times 10^{-5}(\frac{L}{D_b}) \leq 5.9 \times 10^{-4}$ (1-26)

and

$\varphi_2 = 0.065[c_{ub}(\frac{kN}{m^2})]^{0.5}$ (1-27)

Figures (1-12) and (1-13) may now be used to evaluate the allowable load-bearing capacity, based on settlement. (Note that the ultimate bearing capacity in **Figure (1-13)** is q_b, not q_{br}). To do so

Step 1. Select a value of settlement, *s*.

Step 2. Calculate $\sum_{i=1}^{N} f_i p \Delta L_i$ *and* $q_b A_p$

Step 3. Using **Figures 1.12 and 1.13** and the calculated values in Step 2, determine the *side load* and the *end bearing load.*

Step 4. The sum of the side load and the end bearing load gives the total allowable load.

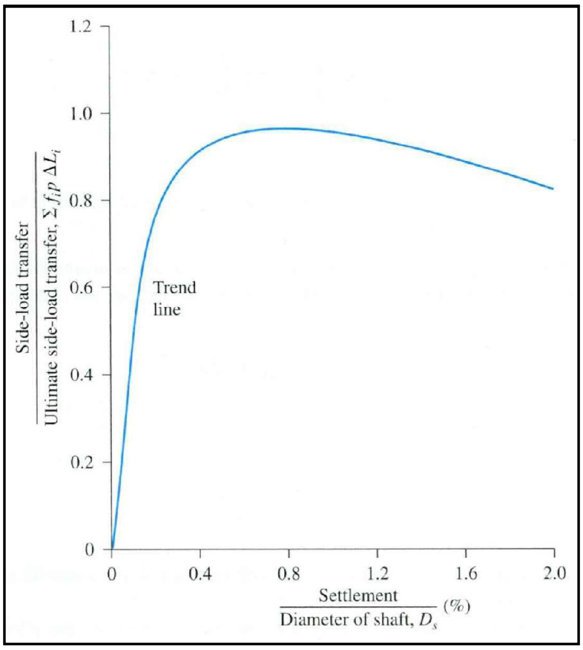


Fig 1.12 Normalized side-load transfer versus settlement in cohesive soil

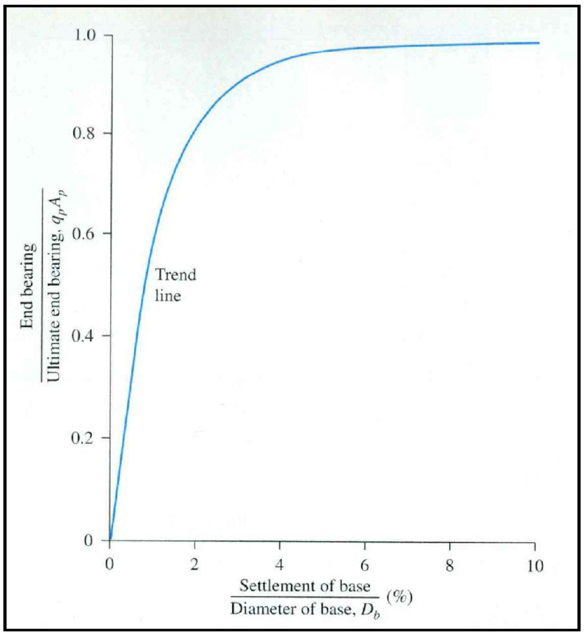


Fig. 1.13 Normalized base-load transfer versus settlement in cohesive soil

Example 1.3
Example 1.4

1.9 Lateral Load- Carrying Capacity

The lateral load-carrying capacity of piers can be analyzed in a manner similar to that presented in last section for piles. Therefore, it will not be repeated here.

1.10 Caissons

1.10.1 Types of Caissons

Caissons are divided into three major types:
(1) open caissons,
(2) box caissons (or closed caissons), and
(3) pneumatic caissons.

Open caissons (**Figure 1.14**) are concrete shafts that remain open at the top and bottom during construction. The bottom of the caisson of the caisson has a cutting edge. The caisson is sunk into place, and soil from the inside of the shaft is removed by grab buckets until the bearing stratum is reached. The shafts may be circular, square, rectangular, or oval. Once the bearing stratum is reached, concrete is poured into the shaft (under water) to form a seal at its bottom. When the concrete seal hardens, the water inside the caisson shaft is pumped out. Concrete is then poured into the shaft to fill it. Open caissons can be extended to great depths, and the cost of construction is relatively low. However, one of their major of disadvantages is the lack of quality control over the concrete poured into the shaft for the seal. Also, the bottom of the caisson cannot be thoroughly cleaned out. An alternative method of open-caisson construction is to drive some sheet piles to form an enclosed area, which is filled with sand and is generally referred to as a *sand island.* The caisson is then sunk through the sand to the desired bearing stratum. This procedure is somewhat analogous to sinking a caisson when the ground surface is above the water table.

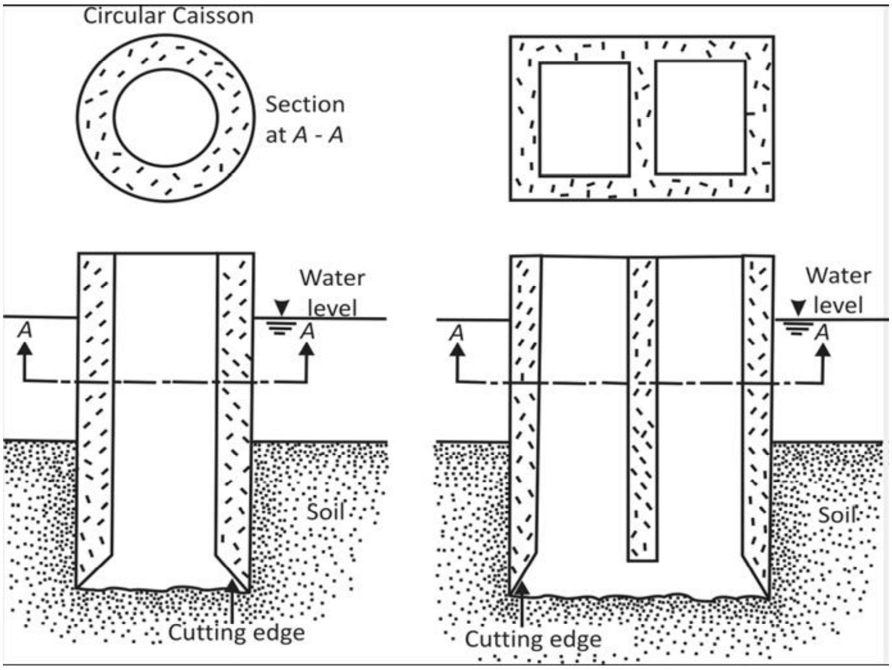


Fig. 1.14 Open caisson

Box caissons **(Figure 1. 15)** are caissons with closed bottoms. They are constructed on land and then transported to the construction site. They are gradually sunk at the site by filling the inside with sand, ballast, water, or concrete. The cost for this type of construction is low. The bearing surface must be level, and if it is not, it must be leveled by excavation.

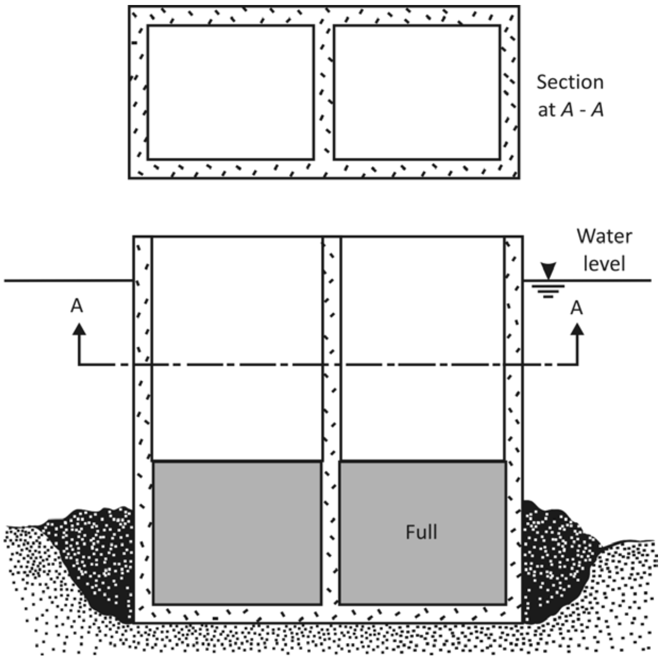


Fig. 1.15 Box caisson

***Pneumatic caissons* (Figure 1.16)** are generally used for depths of about (15-40 m). This type of caisson is required when an excavation cannot be kept open because the soil flows into the excavated area faster than it can be removed. A pneumatic caisson has a work chamber at the bottom that is at least (*3* m) high. In this chamber, the workers excavate the soil and place the concrete. The air pressure in the chamber is kept high enough to prevent water and soil from entering. Workers usually do not counter severe discomfort when the chamber pressure is raised to about *15* lb/in2(.100 kN/m^2) above atmospheric pressure. Beyond this pressure, decompression periods are required when the workers leave the chamber. When chamber pressures of about (300 kN/m^2) above atmospheric pressure are required, workers should not be kept inside the chamber for more than *1122* hours at a time. Workers enter and leave the chamber through a steel shaft by means of a ladder. This shaft is also used for the removal of excavated soil and the placement of concrete. For large caisson construction, more than one shaft may be necessary, an airlock is provided for each one. Pneumatic caissons gradually sink as excavation proceeds. When the bearing stratum is reached, the work chamber is filled with concrete. Calculation of the load-bearing capacity of caissons is similar to that for drilled shafts. Therefore, it will not be further discussed in this section.

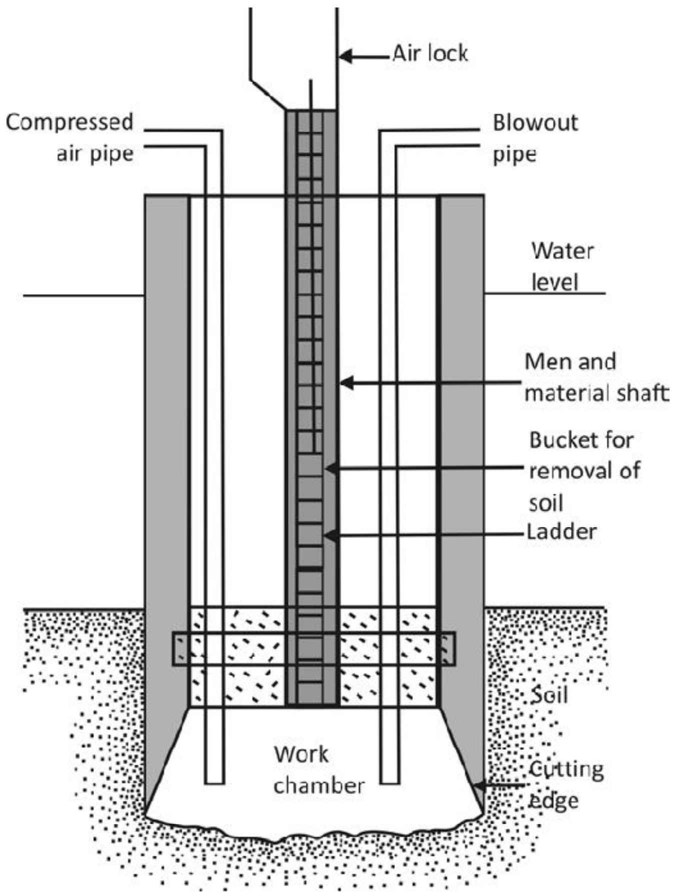


Fig 1.16 Pneumatic caisson

1.11 Thickness of Concrete Seal in Open Caissons

we mentioned that, before dewatering the caisson, a concrete seal is placed at the bottom of the shaft (**Figure 1.17**) and allowed to cure for some time. The concrete seal should be thick enough to withstand an upward hydrostatic force from it bottom after dewatering is complete and before concrete fills the shaft. Based on the theory of elasticity the thickness, *t,* according to Teng (1962) is

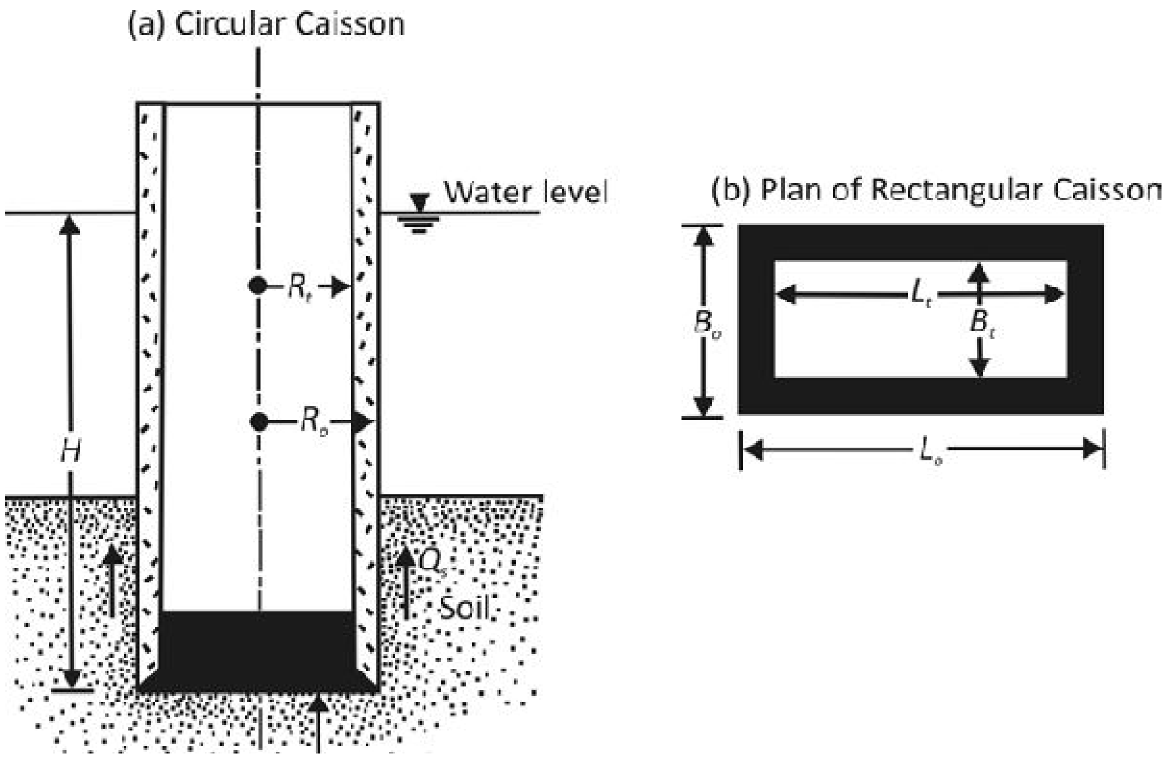


Fig. 1.17 Calculation of the thickness of seal for an open caisson

$$t = 1.18R_i\sqrt{\frac{q}{fc}} \qquad \text{(circular caisson)} \qquad (1\text{-}28)$$

and

$$t = 0.866B_i\sqrt{\frac{q}{fc[1+1.61\left(\frac{L_i}{B_i}\right)]}} \qquad \text{(rectangular caisson)} \qquad (1\text{-}29)$$

Where

R_i =inside radius of a circular caisson

q =unit bearing pressure at the base of the caisson

f_c=allowable concrete flexural stress (≈*.0.1-0.2* of f_c' where f_c' is than 28day compressive strength of concrete)

B_i, L_i = inside with and length, respectively, of rectangular caisson

According to **Figure 1.17**, the value of *q* in **Equations (1-28 and 1-29)** can be approximated as

$q \approx H\gamma_w - t\gamma_c$ (1-30)

Where
γ_c=unit weight of concrete

The thickness of the seal calculated by **Equations (1-28 and 1-29)** will be sufficient to protect it from cracking immediately after dewatering. However, two other conditions should also be checked for safety.

1. Check for Perimeter Shear an Contact Face of Seal and Shaft

According to **Figure 1-17**, the net upward hydrostatic force from the bottom of the seal is $A_i H\gamma_w - A_i t\gamma_c$(where$A_i = \pi R_i^2$ for circular caissons and $A_i = B_i L_i$ for rectangular caissons). So the perimeter shear developed is

$$\upsilon = \frac{A_i H\gamma_w - A_i t\gamma_c}{p_i t} \qquad (1\text{-}31)$$

Where
p_i= inside perimeter of the caisson

Note that
$p_i = 2\pi R_i$ (for circular caissons) (1-32)

And that
$p_i = 2(B_i + L_i)$ (for rectangular caisson) (1-33)

The perimeter shear given by **Eq. (1-31)** should be less than the permissible shear stress, υ_u, or

$$\nu\left(\frac{MN}{m^2}\right) \le \nu_u\left(\frac{MN}{m^2}\right) = 0.17\phi\sqrt{f_c'\left(\frac{MN}{m^2}\right)} \qquad (1\text{-}34)$$

Where
ϕ=0.85

2. Check for Buoyancy
If the shaft is completely dewatered, the buoyant upward force,F_u , is

$$F_u = (\pi R_o^2)H\gamma_w \quad \text{(for circular caissons)} \qquad (1\text{-}35)$$

$F_u = (B_o L_o) H \gamma_w$ (for rectangular caissons) (1-36)

The downward force, F_d , is caused by the weight of the caisson and the seal and by the skin friction at the caisson-soil interface, or

$$F_d = W_c + W_s + Q_s \tag{1-37}$$

Where
W_c=weight of caisson
W_s=weight of seal
Q_s=skin friction

If $F_d > F_u$, the caisson is safe from buoyancy. However, if $F_d < F_u$ dewatering the shaft completely will be unsafe. For that reason, the thickness of the seal should be increased by Δt [over the thickness calculated by using **Equation (1-28) or (1-29)]** or

$$\Delta t = \frac{F_u - F_d}{A_t \gamma_c} \tag{1-38}$$

Example 1-5