University of Anbar
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CHAPTER ONE

PILE FOUNDATIONS

LECTURE
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1.1. Introduction

Piles are **structural members made of steel, concrete, or timber**. They are **deep foundations** where the **depth (or length)** is significantly **larger than the width**. Deep foundations require **special equipment and skill** and hence **cost more**. Therefore, **piles** are considered **only** in **situations** where **shallow foundations prove to be inadequate** (e.g., **large loads or poor soil conditions)**. **Piles** are **recommended** in the following situations:

1. In weak ground conditions

When the **soil conditions near the surface are poo**or, **shallow foundations will not be able to carry the building loads, and deep foundations are required. If bedrock is present at reasonable depth, it is possible to drive the pile into the bedrock and transfer the entire load to the bedrock (Figure 1.1a).** Generally, when the pile is driven into a thick deposit of soil, the **load applied** on **the pile head is transferred to the soil through the pile tip (or point) and the pile shaft**, as shown in Figure 1.1b.

2. For carrying lateral loads

Tall buildings, earth-retaining structures, transmission towers, and chimneys can be subjected to large lateral loading due to wind loads, earth *pressures, or seismic loads.* Unlike shallow foundations, the **piles can resist lateral loads very effectively** (Figure 1.1c). Sometimes these **piles** are **installed at an angle to the vertical to resist the lateral load** and are known as **batter piles**.

3. In expansive or collapsible soil

Expansive soils pose a significant threat to **low-rise buildings**, **roads**, and other infrastructure in many parts of the world. **Shallow foundations placed** in expansive soil can undergo repeated **swelling and shrinkage due to seasonal variations**, which cause considerable **damage** to the **superstructure**. Piles can be **driven well beyond the depths** where the expansive soil are present, so this problem can be avoided (Figure 1.1d). **Collapsible soil** such as loess become weaker when **saturated and undergo large settlements**. Here, too, piles can be driven beyond the depths where such problematic soils are present.

4. For resisting uplift

In **transmission towers, offshore platforms**, and situations where the **basement or pump house lies below the water table**, the **foundations** must be able to **resist uplift**. **Piles** can be **very effective** for such situations (Figure 1.1e). Sometimes, the **bottoms of the piles** are **enlarged to provide anchorage against uplift**. Such piles are known as "**belled**" or "**underreamed**" piles.

5. For bridge abutments

Bridge abutments and **piers** are usually **constructed over pile foundations** to avoid the **loss of bearing capacity** that a **shallow foundation may suffer because of soil erosion at the ground level** (Figure 1.1f).

6. As compaction piles

Under certain circumstances, **piles are driven into granular soil to achieve proper compaction of soil close to the ground surface**. These piles are called **compaction piles**. The **lengths** of compaction piles depend on the **relative density of the soil before and after the compaction** and the **required depth of compaction**. These **piles** are **generally short**; however, some **field tests** are necessary to determine **a reasonable length**.

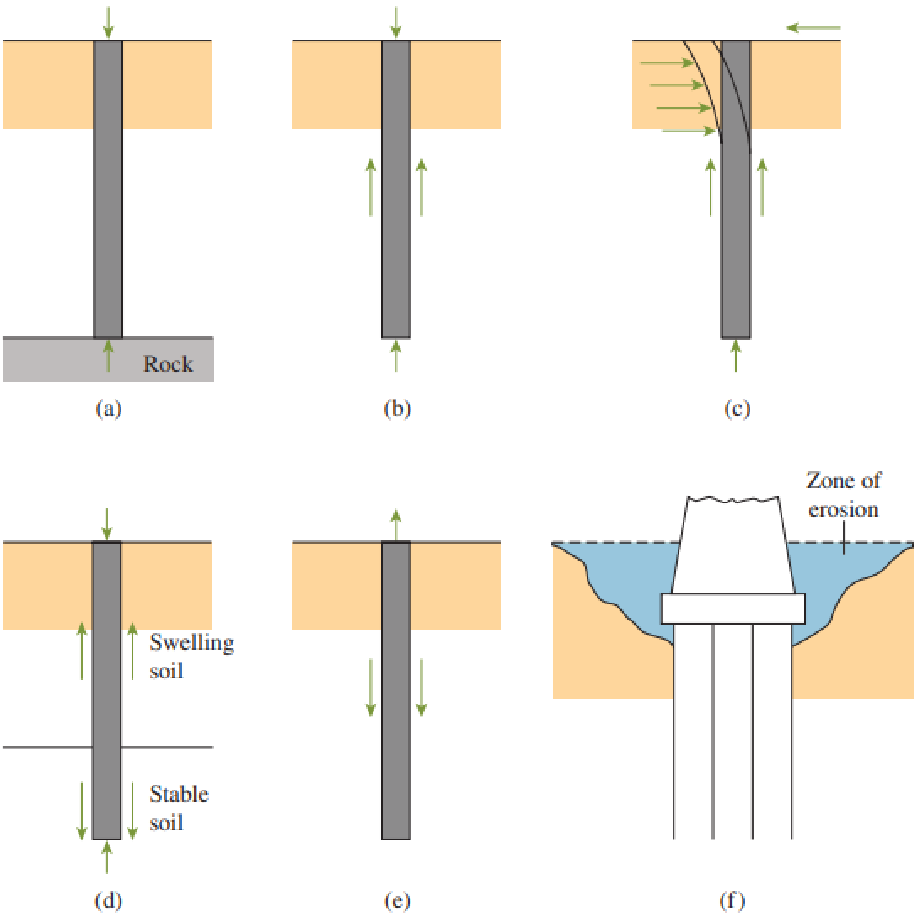


Fig. 1.1 Some of the conditions that require pile foundations

1.2 Types of Piles and Their Structural Characteristics

- **Different types of piles** are used in practice, **depending** on the **type of load to be carried, soil conditions, location of the water table, and the installation technique that is required**.
- **Piles** can be divided into the following categories with the general descriptions for conventional **steel, concrete, timber**, and **composite piles**.

1.2.1 Steel Piles

- ***Steel piles*** generally are either ***pipe piles*** or ***rolled steel* H-*section piles***. ***Pipe piles*** can be **driven** into the **ground** with their **ends open or closed**. ***Wide-flange and I-section steel beams*** can also be **used** as **piles**. However, **H-section piles** are **usually preferred** because their **web and flange thicknesses are equal**. (In wide-flange and I-section beams, the web thicknesses are smaller than the thicknesses of the flange.). **Table 1.1** gives the dimensions of some standard H-section steel piles used in the United States. **Table 1.2** shows selected pipe sections frequency used for piling purposes.
- **In many cases**, the **pipe piles** are **filled with concrete after they have been driven and they become composite piles**.
- The allowable structural capacity for **steel piles** is

$$Q_{all} = A_s f_s \qquad \textbf{(1.1)}$$

where

A_s = cross-sectional area of the steel

f_s = allowable stress of steel (≈ 0.33–$0.5f_y$)

- When necessary, steel piles are **spliced by welding or by riveting**. Figure 1.2a shows a typical splice by welding for an H-pile. A typical splice by welding for a pipe pile is shown in Figure 1.2b. Figure 1.2c is a diagram of a splice of an H-pile by rivets or bolts.
- When hard **driving conditions** are expected, such as driving through **dense gravel, shale, or soft rock, steel piles** can be **fitted with driving points or shoes**. Figures 1.2d and 1.2e are diagrams of two types of shoes used for pipe piles.

- **Steel piles** may be **subject to corrosion**. For example, **swamps, peats, and other organic soil** are **corrosive**. **Soils** that have a **pH greater than 7 are not so corrosive**.

Table 1.1a Common H-Pile Sections Used in the United States (SI Units).

Designation, size (mm) × weight (kg/m)	Depth d_1 (mm)	Section area ($m^2 \times 10^{-3}$)	Flange and web thickness w (mm)	Flange width d_2 (mm)	Moment of inertia ($m^4 \times 10^{-6}$) I_{xx}	I_{yy}
HP 200 × 53	204	6.84	11.3	207	49.4	16.8
HP 250 × 85	254	10.8	14.4	260	123	42
× 62	246	8.0	10.6	256	87.5	24
HP 310 × 125	312	15.9	17.5	312	271	89
× 110	308	14.1	15.49	310	237	77.5
× 93	303	11.9	13.1	308	197	63.7
× 79	299	10.0	11.05	306	164	62.9
HP 330 × 149	334	19.0	19.45	335	370	123
× 129	329	16.5	16.9	333	314	104
× 109	324	13.9	14.5	330	263	86
× 89	319	11.3	11.7	328	210	69
HP 360 × 174	361	22.2	20.45	378	508	184
× 152	356	19.4	17.91	376	437	158
× 132	351	16.8	15.62	373	374	136
× 108	346	13.8	12.82	371	303	109

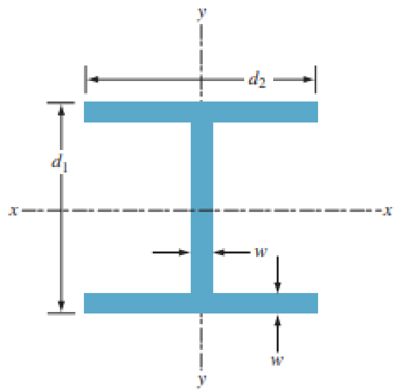


Table 1.1b Common H-Pile Sections Used in the United States (English Units)

Designation size (in.) × weight (lb/ft)	Depth d_1 (in.)	Section area (in^2)	Flange and web thickness w (in.)	Flange width d_2 (in.)	Moment of inertia (in^4)	
					I_{xx}	I_{yy}
HP 8 × 36	8.02	10.6	0.445	8.155	119	40.3
HP 10 × 57	9.99	16.8	0.565	10.225	294	101
× 42	9.70	12.4	0.420	10.075	210	71.7
HP 12 × 84	12.28	24.6	0.685	12.295	650	213
× 74	12.13	21.8	0.610	12.215	570	186
× 63	11.94	18.4	0.515	12.125	472	153
× 53	11.78	15.5	0.435	12.045	394	127
HP 13 × 100	13.15	29.4	0.766	13.21	886	294
× 87	12.95	25.5	0.665	13.11	755	250
× 73	12.74	21.6	0.565	13.01	630	207
× 60	12.54	17.5	0.460	12.90	503	165
HP 14 × 117	14.21	34.4	0.805	14.89	1220	443
× 102	14.01	30.0	0.705	14.78	1050	380
× 89	13.84	26.1	0.615	14.70	904	326
× 73	13.61	21.4	0.505	14.59	729	262

Table 1.2 Selected Pipe Pile Sections (SI Units and English Units)

Outside diameter (mm)	Wall thickness (mm)	Area of steel (cm^2)
219	3.17	21.5
	4.78	32.1
	5.56	37.3
	7.92	52.7
254	4.78	37.5
	5.56	43.6
	6.35	49.4
305	4.78	44.9
	5.56	52.3
	6.35	59.7
406	4.78	60.3
	5.56	70.1
	6.35	79.8
457	5.56	80
	6.35	90
	7.92	112
508	5.56	88
	6.35	100
	7.92	125
610	6.35	121
	7.92	150
	9.53	179
	12.70	238

Outside diameter (in.)	Wall thickness (in.)	Area of steel (in^2)
$8\frac{5}{8}$	0.125	3.34
	0.188	4.98
	0.219	5.78
	0.312	8.17
10	0.188	5.81
	0.219	6.75
	0.250	7.66
12	0.188	6.96
	0.219	8.11
	0.250	9.25
16	0.188	9.34
	0.219	10.86
	0.250	12.37
18	0.219	12.23
	0.250	13.94
	0.312	17.34
20	0.219	13.62
	0.250	15.51
	0.312	19.30
24	0.250	18.7
	0.312	23.2
	0.375	27.8
	0.500	36.9

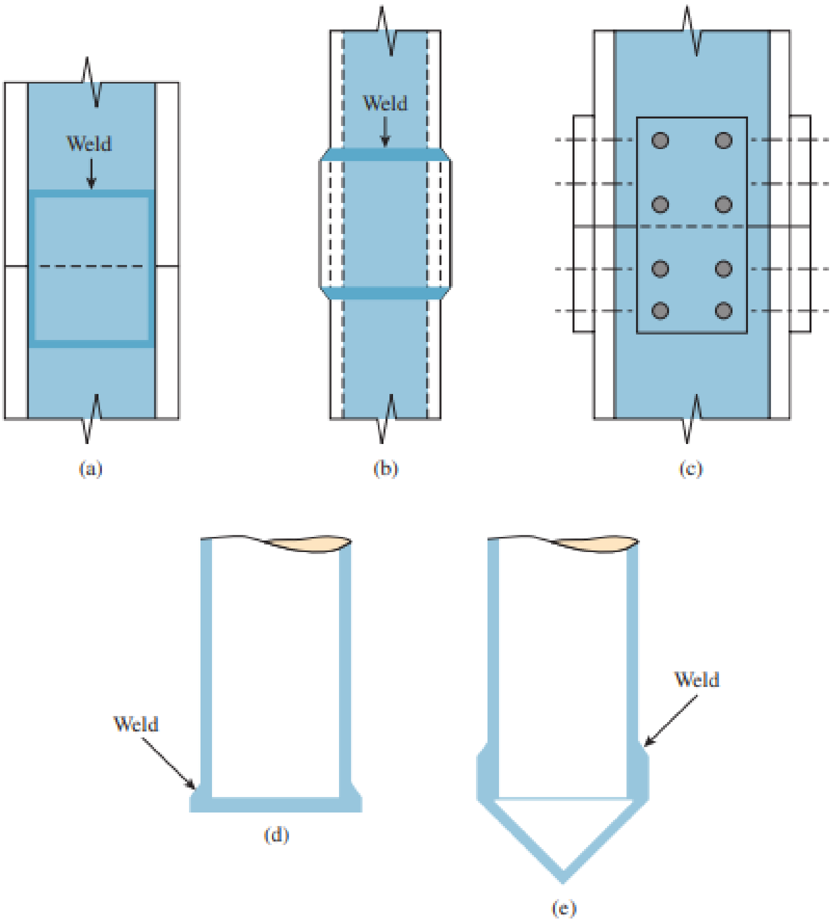


Figure 1.2 Steel piles: (a) splicing of H-pile by welding; (b) splicing of pipe pile by welding; (c) splicing of H-pile by rivets and bolts; (d) flat driving point of pipe pile; (e) conical driving point of pipe pile

- To **offset the effect of corrosion**, an **additional thickness of steel** (**over the actual designed cross-sectional area**) is generally recommended. In many circumstances, **factory-applied epoxy coatings on piles work satisfactorily against corrosion**. These coatings are **not easily damaged by pile driving**. **Concrete encasement of steel piles** in most corrosive zones also **protects** against **corrosion**.

1.2.2 Concrete Piles

- **Concrete piles** may be divided into **two basic** categories:**(a) precast piles and (b) cast-in-situ piles. Precast piles** can be prepared by using **ordinary reinforcement**, and they can be **square or octagonal in cross section** (see Figure 1.4).
- **Reinforcement** is provided to enable the **pile to resist the bending moment developed during pickup and transportation**, **the vertical load, and the bending moment caused by a lateral load**.
- The **piles** are **cast** to **desired lengths** and **cured before being transported to the work sites**.

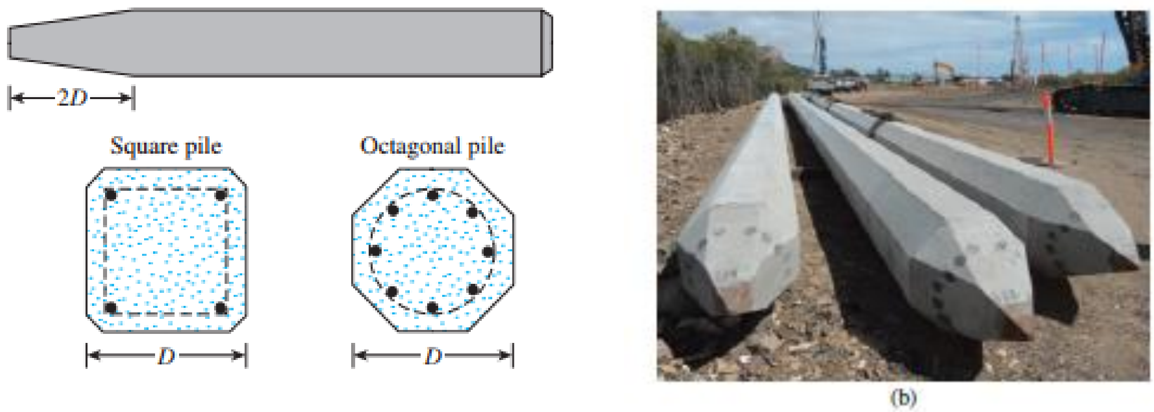


Figure 1.4 Precast piles with ordinary reinforcement: (a) schematic diagram; (b) photograph of 24 m long octagonal piles ready for driving

Precast piles can also be **prestressed** by the use of **high-strength steel prestressing cables.** The **ultimate strength** of these cables is about **1800 MN/m^2**. During casting of the piles, the cables are pretensioned to about 900 to 1300 MN/m^2, and concrete is poured around them. After curing, the cables are cut, producing a compressive force on the pile section. Table 1.3 gives additional information about prestressed concrete piles with square and octagonal cross sections.

Cast-in-situ, or cast-in-place, *piles* are **built by making a hole** in the **ground and then filling it with concrete**. Various types of cast-in-place concrete piles are currently used in construction, and most of them have been patented by their manufacturers. These piles may be divided into two broad categories: (a) **cased and** (b) **uncased**. Both types may have a **pedestal at the bottom**.

Cased piles are made by **driving a steel casing into the ground with the help of a mandrel placed inside the casing**. When the pile **reaches the proper depth the mandrel is withdrawn and the casing is filled with concrete.** Figures 1.5a, 1.5b, 1.5c, and 1.5d show some examples of cased piles without a pedestal. Figure 1.5e shows a cased pile with a pedestal. The pedestal is an expanded concrete bulb that is formed by dropping a hammer on fresh concrete.

Cased Pile

$$Q_{all} = A_s f_s + A_c f_c \qquad (1.2)$$

where

A_s = area of cross section of steel
A_c = area of cross section of concrete
f_s = allowable stress of steel
f_c = allowable stress of concrete

Uncased piles

$$Q_{all} = A_c f_c \qquad (1.3)$$

- Figures 1.5f and 1.5g are **two types of uncased pile**, one with a pedestal and the other without. The **uncased piles** are made **by first driving the casing to the desired depth and then filling it with fresh concrete. The casing is then gradually withdrawn.**

Table 1.3 Typical Prestressed Concrete Pile in Use (SI Units)

Pile shape[a]	D (mm)	Area of cross section (cm^2)	Perimeter (mm)	Number of strands: 12.7-mm diameter	Number of strands: 11.1-mm diameter	Minimum effective prestress force (kN)	Section modulus ($m^3 \times 10^{-3}$)	Design bearing capacity (kN), Strength of concrete (MN/m^2): 34.5	Design bearing capacity (kN), Strength of concrete (MN/m^2): 41.4
S	254	645	1016	4	4	312	2.737	556	778
O	254	536	838	4	4	258	1.786	462	555
S	305	929	1219	5	6	449	4.719	801	962
O	305	768	1016	4	5	369	3.097	662	795
S	356	1265	1422	6	8	610	7.489	1091	1310
O	356	1045	1168	5	7	503	4.916	901	1082
S	406	1652	1626	8	11	796	11.192	1425	1710
O	406	1368	1346	7	9	658	7.341	1180	1416
S	457	2090	1829	10	13	1010	15.928	1803	2163
O	457	1729	1524	8	11	836	10.455	1491	1790
S	508	2581	2032	12	16	1245	21.844	2226	2672
O	508	2136	1677	10	14	1032	14.355	1842	2239
S	559	3123	2235	15	20	1508	29.087	2694	3232
O	559	2587	1854	12	16	1250	19.107	2231	2678
S	610	3658	2438	18	23	1793	37.756	3155	3786
O	610	3078	2032	15	19	1486	34.794	2655	3186

[a]S = square section; O = octagonal section

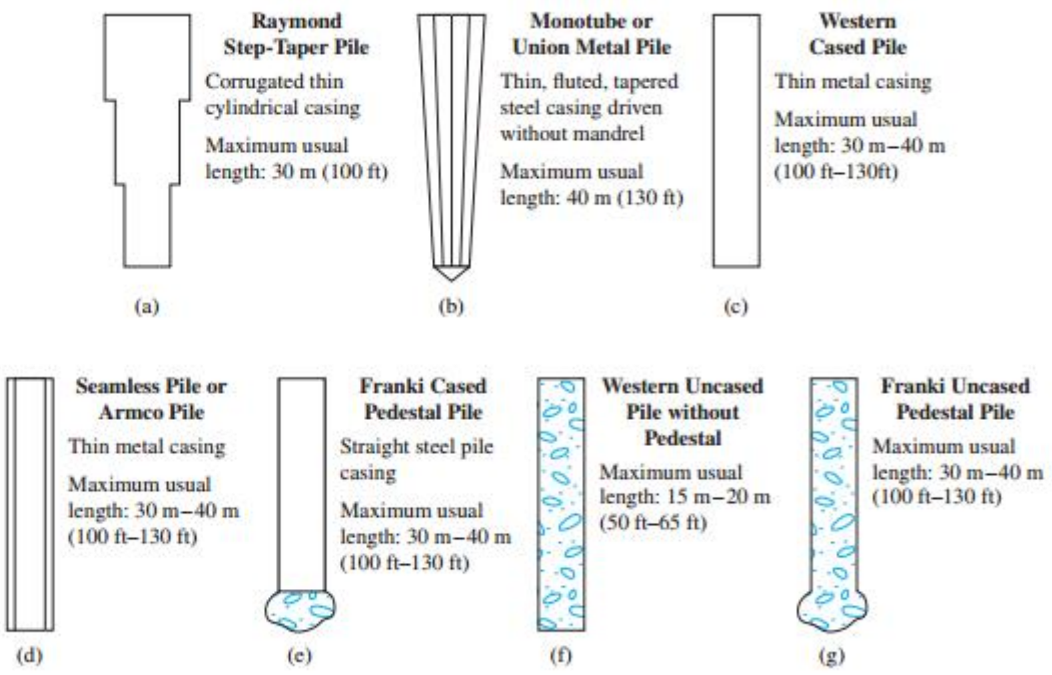


Figure 1.5 Cast-in-place concrete piles

1.2.3 Timber Piles

Timber piles are **tree trunks that have had their branches and bark carefully trimmed off.** The **maximum length** of most timber piles is **10 to 20 m**. To qualify for use as a pile, the **timber** should be **straight, sound, and without any defects**. The American Society of Civil Engineers' *Manual of Practice*, No. 17 (1959), divided timber piles into three classes:

1. ***Class A*** *piles* carry **heavy loads**. The minimum diameter of the butt should be 356 mm.
2. ***Class B*** *piles* are used to **carry medium loads**. The minimum butt diameter should be 305 to 330 mm.
3. ***Class C*** *piles* are used in **temporary construction work**. They can be **used permanently** for **structures** when the entire **pile is below the water table.** The minimum butt diameter should be 305 mm.

- In any case, a **pile tip** should not have a **diameter less** than 150 mm.
- Timber piles **cannot withstand hard driving stress**; therefore, the **pile capacity** is generally **limited**.

1.2.4 Composite Piles

- The upper and lower portions of *composite piles* are made of **different materials**. For example, composite piles may be **made of steel and concrete or timber and concrete**.
- **Steel-and-concrete piles** consist of a **lower portion of steel and an upper portion of cast-in-place concrete.** This type of pile is used when the length of the pile required for adequate **bearing exceeds the capacity of simple cast-in-place concrete piles.**
- **Timber-and-concrete piles** usually consist of a **lower portion of timber pile** below the **permanent water table and an upper portion of concrete**. In any case, forming proper joints between two

dissimilar materials is difficult, and for that reason, composite piles are not widely used. Nowadays, **fiber reinforced polymer (FRP) composite piles are widely used for waterfront structures.**

Comparison of Pile Types

Table 1.4 gives a common of the advantage and disadvantages of the various types of pile based on the pile material.

Table 1.4 Comparisons of Pile made of Different Materials

Pile type	Usual length of piles (m)	Maximum length of pile (m)	Usual load (kN)	Approximate maximum load (kN)	Comments
Steel	15–60	Practically unlimited	300–1200	Eq. (8.1)	*Advantages* a. Easy to handle with respect to cutoff and extend to the desired length b. Can stand high driving-stresses c. Can penetrate hard layers such as dense gravel, soft rock d. High load-carrying capacity *Disadvantages* a. Relatively costly material b. High level of noise during pile driving c. Subject to corrosion d. H-piles may be damaged or deflected from the vertical during driving through hard layers or past major obstructions
Precast concrete	*Precast*: 10–15 *Prestressed*: 10–35	*Precast*: 30 *Prestressed*: 60	300–3000	*Precast*: 800–900 *Prestressed*: 7500–8500	*Advantages* a. Can be subjected to hard driving b. Corrosion resistant c. Can be easily combined with concrete superstructure *Disadvantages* a. Difficult to achieve proper cutoff b. Difficult to transport
Cased cast-in-place concrete	5–15	15–40	200–500	800	*Advantages* a. Relatively cheap b. Possibility of inspection before pouring concrete c. Easy to extend *Disadvantages* a. Difficult to splice after concreting b. Thin casings may be damaged during driving
Uncased cast-in-place concrete	5–15	30–40	300–500	700	*Advantages* a. Initially economical b. Can be finished at any elevation *Disadvantages* a. Voids may be created if concrete is placed rapidly b. Difficult to splice after concreting c. In soft soils, the sides of the hole may cave in, thus squeezing the concrete
Wood	10–15	30	100–200	270	*Advantages* a. Economical b. Easy to handle c. Permanently submerged piles are fairly resistant to decay *Disadvantages* a. Decay above water table b. Can be damaged in hard driving c. Low load-bearing capacity d. Low resistance to tensile load when spliced

(Continued)

1.3 Estimating Pile Length

Selecting the type of pile to be used and estimating its necessary length are fairly difficult tasks that require good judgment. In addition to being broken down into the classification given in Section 1.2, **piles can be divided into three major categories, depending on their lengths and the mechanisms of load transfer to the soil:**

(a) point bearing piles,

(b) friction piles, and

(c) compaction piles.

1.4 Point Bearing and Friction Piles

Piles generally carry the applied column load through skin friction along the pile shaft and the bearing capacity at the **pile point (or tip).** When the pile carries the ultimate load Q_u, the ultimate shaft resistance and ultimate point resistance are denoted by Q_s and Q_p, respectively. From equilibrium considerations,

$$\mathbf{Q_u = Q_P + Q_s} \qquad (6.5)$$

- In **point bearing piles**, it is assumed that the entire load is transferred to the soil as through the point, so $\mathbf{Q_s \approx 0}$.
- In **friction piles**, it is assumed that the entire load is transferred through the pile shaft in the form of friction or adhesion, with $\mathbf{Q_p \approx 0}$.

Point Bearing Piles

In situations where the soil near the **ground surface is weak and cannot support shallow foundations**, **pile foundations can be used**. Especially when there is **bedrock or a stiff stratum (e.g., stiff clay or dense sand)** located at **relatively shallow depths**, it is possible to drive the piles through the weak soil and transfer the load to the underlying stiff stratum, as shown in Figure 1.7a. The pile can socket into the **stiff stratum by extending a few meters**. In point bearing piles,

$$\mathbf{Q_u = Q_p} \qquad (1.6)$$

Friction Piles

When there is **no stiff stratum within reasonable depth, point bearing piles can become expensive**. Here it is **necessary to rely on the shaft resistance,** which comes **from skin friction or adhesion**. The point resistance becomes insignificant; hence $Q_p \approx 0$ (Figure 1.7b). Therefore, in friction piles,

$$Q_u \approx Q_s \qquad (1.7)$$

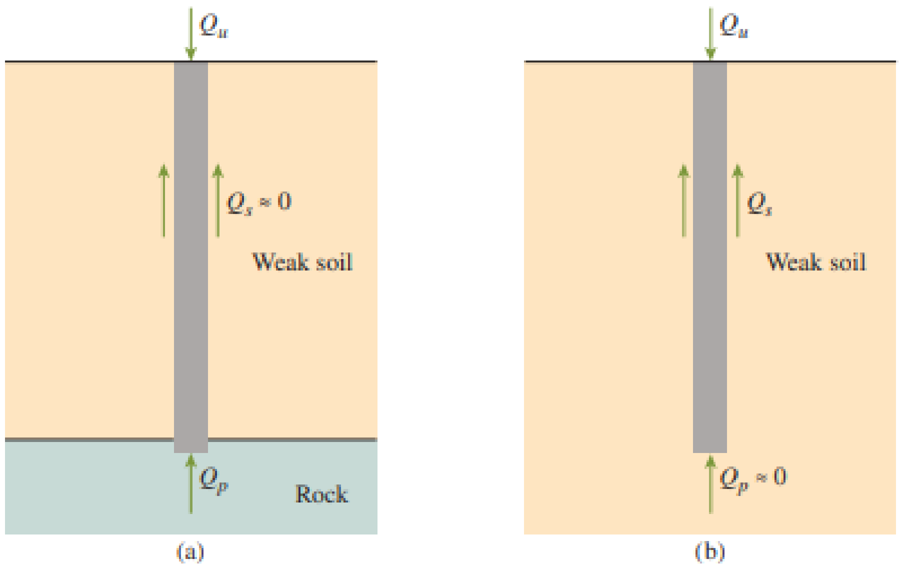


Figure 1.7 (a) Point bearing piles; (b) friction piles

1.5 Installation of Piles

- There are **different ways of installing piles**, depending on **the type of pile and the ground conditions**. **Steel, timber, and precast concrete piles are generally driven into the ground using an impact hammer or a vibratory hammer.**
- A traditional impact pile driver allows a heavy hammer to slide up and down between guide rails, hitting the pile head and making the tip penetrate the ground by a few millimeters for every blow. The weight is raised by **power from compressed air, steam, hydraulics, diesel,** or simply by manual labor, as in ancient civilizations (Figures 1.9a and 1.9b).
- The vibratory hammer was developed in the Soviet Union during World War II. **Vibratory pile drivers produce less noise and are preferred near residential or office buildings**. **They are effective in medium-dense granular soil and with steel piles**. Figure 1.9 shows the schematic diagrams of two impact hammers and a vibratory hammer.
- In Figures 1.9a and 1.9b, the pile is driven into the **ground by raising and dropping a heavy weight**, known as the ram. In the vibratory hammer, two counterrotating weights produce a vertical sinusoidal load that drives the pile into the ground (Figure 1.9c). The horizontal components of the centrifugal forces induced by the two rotating weights cancel each other out.
- **Cast-in-place (or bored) concrete piles** are **installed** by placing a **reinforcement cage into a cylindrical hole in the ground and pouring fresh concrete.**

- Installation of piles generally causes **lateral displacement of the surrounding soil.** The extent of displacement depends on the method of installation and other factors. **Driven piles generally cause high displacements:** the **larger the diameter, the larger the displacements**. **Cast-in-place piles (or bored piles)** literally cause no displacement and are known as **nondisplacement piles**.
- **H-piles and open-ended pipe piles cause little displacement** and are known as **low-displacement piles**.
- **Precast concrete, timber, or closed-end pipe piles** that are driven into the ground are often **medium-to high-displacement piles, depending on the diameter**.

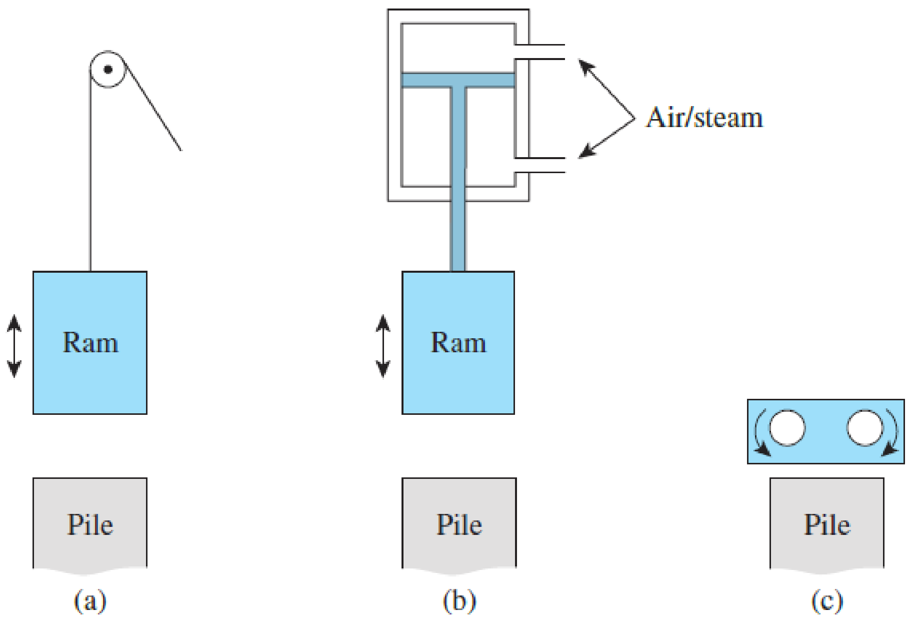


Figure 1.9 Pile-driving hammers: (a) drop hammer; (b) pneumatic hammer; (c) Vibratory hammer

1.6 Load Transfer Mechanism

The load transfer mechanism from a pile to the soil is complicated. To understand it, consider a pile of length *L*, as shown in Figure 1.10a. The load on the pile is gradually increased from zero to $Q_{(z=0)}$ at the ground surface. Part of this load will be resisted by the side friction developed along the shaft, Q_1 , and part by the soil below the tip of the pile, Q_2 . Now, how are Q_1 and Q_2 related to the total load? If measurements are made to obtain the load carried by the pile shaft, $Q_{(z)}$, at any depth *z*, the nature of the variation found will be like that shown in curve 1 of Figure 1.10b. The *frictional resistance per unit area* at any depth *z* may be determined as

$$f_{(z)} = \frac{\Delta Q_{(z)}}{(p)(\Delta z)} \qquad (1.8)$$

Where

p = perimeter of the cross section of the pile. Figure 1.10c shows the variation of $f_{(z)}$ with depth.

If the load *Q* at the ground surface is gradually increased, maximum frictional resistance along the pile shaft will be fully mobilized when the relative displacement between the soil and the pile is about 5 to 10 mm, irrespective of the pile size and length *L*. However, the maximum point resistance $Q_2 = Q_p$ will not be mobilized until the tip of the pile has moved about 10 to 25% of the pile width (or diameter). (The lower limit applies to driven piles and the upper limit to bored piles). At ultimate load (Figure 1.10d and curve 2 in Figure 1.10b), $\boldsymbol{Q_{(z=0)} = Q_u}$. Thus,

$$\boldsymbol{Q_1 = Q_s}$$

and

$$\boldsymbol{Q_2 = Q_p}$$

The preceding explanation indicates that Q_s (or the unit skin friction, *f*, along the pile shaft) is developed at a *much smaller pile displacement compared with the point resistance*, Q_p.

At ultimate load, the failure surface in the soil at the pile tip (a bearing capacity failure caused by Q_p) is like that shown in Figure 6.9e. Note that pile foundations are deep foundations and that the soil fails mostly in a *punching mode*. That is, a *triangular zone*, I, is developed at the pile tip, which is pushed downward without producing any other visible slip surface. In dense sands and stiff clayey soils, a *radial shear zone*, II, may partially develop.

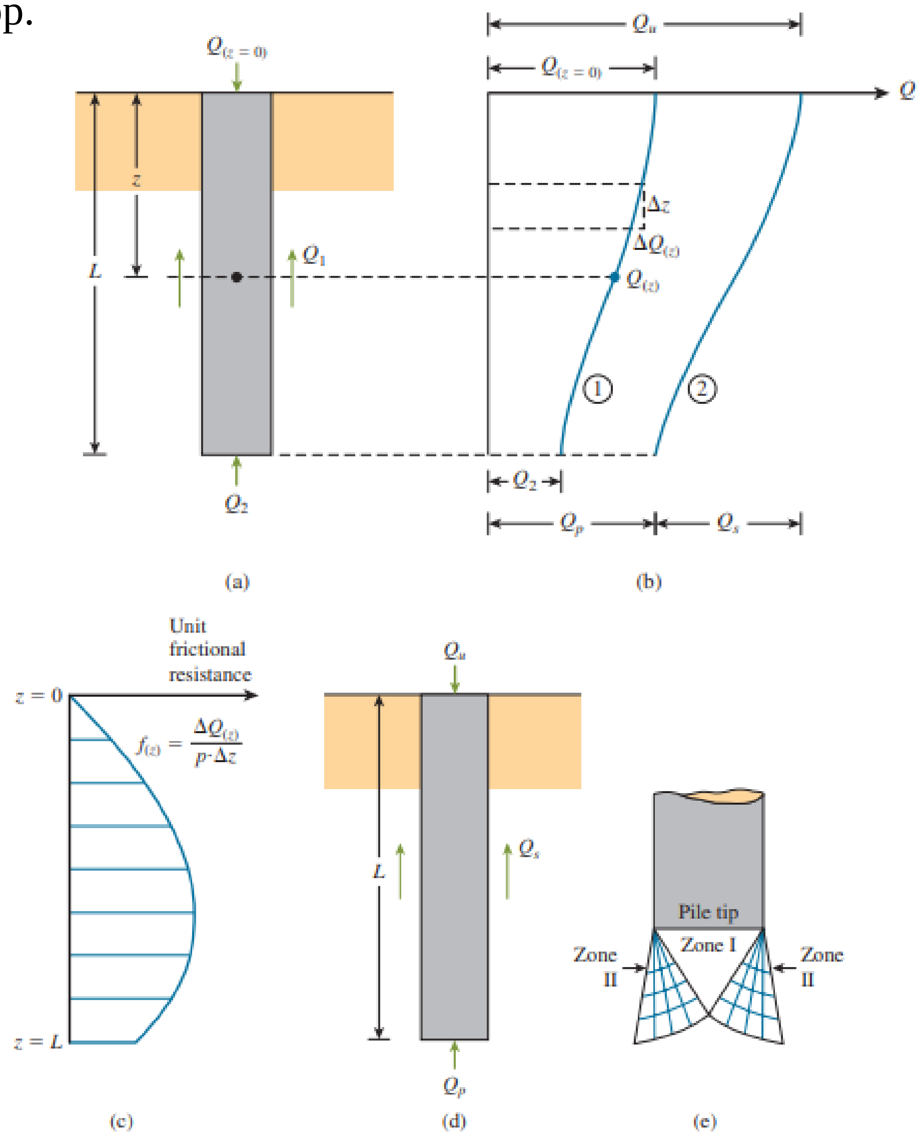


Figure 1.10 Load transfer mechanism for piles

1.7 Equations for Estimating Pile Capacity

The ultimate load-carrying capacity Q_u of a pile is given by the equation

$$Q_u = Q_p + Q_s \qquad (1.9)$$

Q_p = load-carrying capacity of the pile point

Q_s = frictional resistance (skin friction) derived from the soil–pile interface (see Figure 1.11)

Numerous published studies cover the determination of the values of Q_p and Q_s . Excellent reviews of many of these investigations have been provided by Vesic (1977), Meyerhof (1976), and Coyle and Castello (1981). These studies afford an insight into the problem of determining the ultimate pile capacity.

Point Bearing Capacity, Q_p

The ultimate bearing capacity of shallow foundations was discussed in Chapter 3. According to Terzaghi's equations,

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma \qquad \text{(for shallow square foundations)}$$

and

$$q_u = 1.3c'N_c + qN_q + 0.3\gamma BN_\gamma \qquad \text{(for shallow circular foundations)}$$

Similarly, the general bearing capacity equation for shallow foundations was given in Chapter 4 (for vertical loading) as

$$q_u = c'N_cF_{cs}F_{cd} + qN_qF_{qs}F_{qd} + \tfrac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}$$

Hence, in general, the ultimate load-bearing capacity may be expressed as

$$q_u = c'N_c^* + qN_q^* + \gamma BN_\gamma^* \qquad (1.10)$$

where N_c^*, N_q^*, and N_γ^* are the bearing capacity factors that include the necessary shape and depth factors.

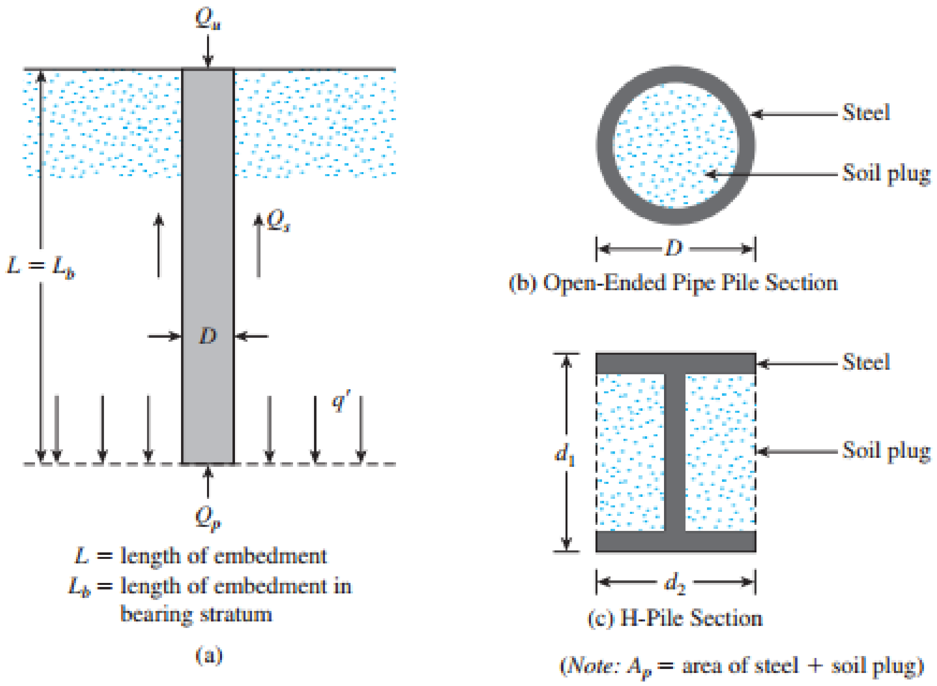


Figure 1.11 Ultimate load-carrying capacity of pile

Pile foundations are deep. However, the ultimate resistance per unit area developed at the pile tip, q_p , may be expressed by an equation similar in form to Eq. (1.10), although the values of N^*_c , N^*_q , and N^*_γ will change. The notation used in this chapter for the width of a pile is *D*. Hence, substituting *D* for *B* in Eq. (1.10) gives

$$q_u = q_p = c'N_c^* + qN_q^* + \gamma DN_\gamma^* \quad (1.11)$$

Because the width D of a pile is relatively small, the term γDN_γ^* may be dropped from the right side of the preceding equation without introducing a serious error; thus, we have

$$q_p = c'N_c^* + q'N_q^* \quad (1.12)$$

- **Note that the term *q* has been replaced by *q'* in Eq. (1.12), to signify effective vertical stress. Thus, the point bearing of piles is**

$$Q_p = A_p q_p = A_p(c'N_c^* + q'N_q^*) \quad (1.13)$$

where

A_p = area of pile tip
c' = cohesion of the soil supporting the pile tip
q_p = unit point resistance
q' = effective vertical stress at the level of the pile tip
N_c^*, N_q^* = the bearing capacity factors

Frictional Resistance, Q_s

The frictional, or skin, resistance of a pile may be written as

$$Q_s = \Sigma p \, \Delta L f \quad (1.14)$$

where

p = perimeter of the pile section
ΔL = incremental pile length over which p and f are taken to be constant
f = unit friction resistance at any depth z

The various methods for estimating Q_p and Q_s are discussed in the next several sections. It needs to be reemphasized that, in the field, for full mobilization of the point resistance (Q_p), **the pile tip must go through a displacement of 10 to 25% of the pile width (or diameter)**.

Allowable Load, Q_{all}

After the total ultimate load-carrying capacity of a pile has been determined by summing the point bearing capacity and the frictional (or skin) resistance, a reasonable factor of safety should be used to obtain the total allowable load for each pile, or

$$Q_{all} = \frac{Q_u}{FS}$$

where

Q_{all} = allowable load-carrying capacity for each pile
FS = factor of safety

- The factor of safety generally used ranges **from 2 to 3**, depending on the uncertainties surrounding the calculation of ultimate load.

1.8 Meyerhof's Method for Estimating Q_p Sand

The point bearing capacity, q_p , of a pile in sand generally increases with the depth of embedment in the bearing stratum and reaches a maximum value at an embedment ratio of $L_b/D = (L_b/D)_{cr}$. Note that in a homogeneous soil L_b is equal to the actual embedment length of the pile, L. However, where a pile has penetrated into a bearing stratum, $L_b < L$. Beyond the critical embedment ratio, $(L_b/D)_{cr}$, the value of q_p remains constant $(q_p = q_l)$. That is, as shown in Figure 1.12 for the case of a homogeneous soil, $L = L_b$.

For piles in sand, $c' = 0$, and Eq. (1.13) simplifies to

$$Q_p = A_p q_p = A_p q' N_q^* \qquad (1.15)$$

The variation of N^*q with soil friction angle ϕ' is shown in **Figure 1.13**. The interpolated values of N^*_q for various friction angles are also given in **Table 1.6**. However, Q_p should not exceed the limiting value $A_p q_l$; that is,

$$Q_p = A_p q' N_q^* \leq A_p q_l \qquad (1.16)$$

The limiting point resistance is

$$q_l = 0.5\, p_a N_q^* \tan \phi' \quad (1.17)$$

where

p_a = atmospheric pressure (=100 kN/m^2 or 2000 lb/ft^2)
ϕ' = effective soil friction angle of the bearing stratum

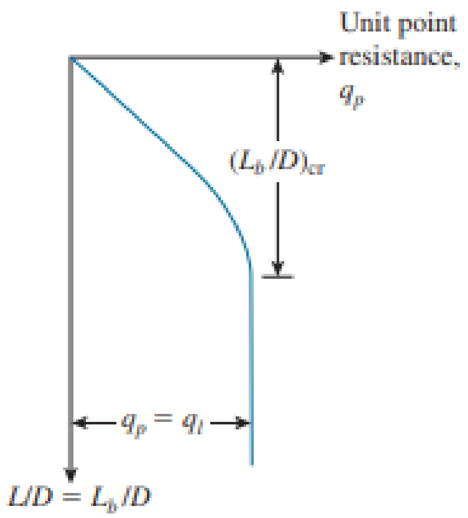


Figure 1.12 Nature of variation of unit point resistance in a homogeneous sand

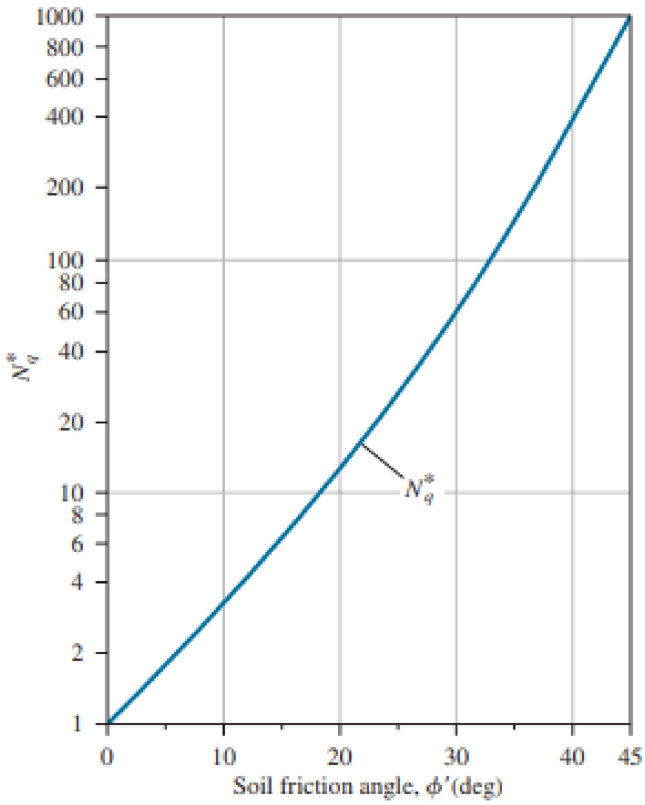


Figure 1.13 Variation of the maximum values of N^*_q with soil friction angle ϕ.

Table 1.6 Interpolated Values of N^*_q Based on Meyerhof's Theory

Soil friction angle, ϕ' (deg)	N_q^*
20	12.4
21	13.8
22	15.5
23	17.9
24	21.4
25	26.0
26	29.5
27	34.0
28	39.7
29	46.5
30	56.7
31	68.2
32	81.0
33	96.0
34	115.0
35	143.0
36	168.0
37	194.0
38	231.0
39	276.0
40	346.0
41	420.0
42	525.0
43	650.0
44	780.0
45	930.0

Clay ($\phi = 0$)

For piles in *saturated clays* under undrained conditions ($\phi = 0$), the net ultimate load can be given as

$$Q_p \approx N_c^* c_u A_p = 9 c_u A_p \tag{1.18}$$

where c_u = undrained cohesion of the soil below the tip of the pile.

Example 1.1

Consider a 20 m long concrete pile with a cross section of 0.407 m × 0.407 m fully embedded in sand. For the sand, given: unit weight, $\gamma = 18$ kN/m^3; and soil friction angle, $\phi' = 35°$. Estimate the ultimate point Q_p with each of the following:

a. Meyerhof's method

SOLUTION

Part a

From Eqs. (12.18) and (12.19),

$$Q_p = A_p q' N_q^* \le A_p(0.5 p_a N_q^* \tan \phi')$$

For $\phi' = 35°$, the value of $N_q^* \approx 143$ (Table 12.6). Also, $q' = \gamma L = (18)(20) = 360$ kN/m^2. Thus,

$$A_p q' N_q^* = (0.407 \times 0.407)(360)(143) \approx 8528 \text{ kN}$$

Again,

$$A_p(0.5 p_a N_q^* \tan \phi') = (0.407 \times 0.407)[(0.5)(100)(143)(\tan 35)] \approx 829 \text{ kN}$$

Hence, Q_p = **829 kN**.

Example 1.2

Consider a pipe pile (flat driving point—see Figure 12.2d) having an outside diameter of 457 mm. The embedded length of the pile in layered saturated clay is 20 m. The following are the details of the subsoil:

Depth from ground surface (m)	Saturated unit weight, γ (kN/m^3)	c_u (kN/m^2)
0–3	16	25
3–10	17	40
10–30	18	90

The groundwater table is located at a depth of 3 m from the ground surface. Estimate Q_p by using

a. Meyerhof's method

SOLUTION

Part a

From Eq. (12.20),

$$Q_p = 9c_u A_p$$

The tip of the pile is resting on a clay with $c_u = 90\ \text{kN/m}^2$. So,

$$Q_p = (9)(90)\left[\left(\frac{\pi}{4}\right)\left(\frac{457}{1000}\right)^2\right] = \mathbf{132.9\ kN}$$

1.10 Correlations for Calculating Q_p with SPT Results

- On the basis of field observations, Meyerhof (1976) also suggested that the ultimate point resistance q_p in a homogeneous granular soil ($L = L_b$) may be obtained from standard penetration numbers as

$$q_p = 0.4p_a N_{60}\frac{L}{D} \leq 4p_a N_{60} \qquad (1.20)$$

where

N_{60} = the average value of the standard penetration number near the pile point (about $10D$ above and $4D$ below the pile point)

p_a = atmospheric pressure ($\approx 100\ \text{kN/m}^2$ or $2000\ \text{lb/ft}^2$)

Briaud et al. (1985) suggested the following correlation for q_p in granular soil with the standard penetration resistance N_{60}.

$$q_p = 19.7p_a(N_{60})^{0.36} \qquad (1.21)$$

Meyerhof (1956) also suggested that

$$q_p \approx q_c \qquad (1.22)$$

where q_c = cone penetration resistance.

Example 1.3

Consider a concrete pile that is 0.305 m × 0.305 m in cross section in sand. The pile is 12 m long. The following are the variations of N_{60} with depth.

Depth below ground surface (m)	N_{60}
1.5	8
3.0	10
4.5	9
6.0	12
7.5	14
9.0	18
10.5	11
12.0	17
13.5	20
15.0	28
16.5	29
18.0	32
19.5	30
21.0	27

a. Estimate Q_p using Eq. (9.37).
b. Estimate Q_p using Eq. (9.38).

Solution

Part a

The tip of the pile is 12 m below the ground surface. For the pile, $D = 0.305$ m. The average of N_{60} 10D above and about 5D below the pile tip is

$$N_{60} = \frac{18 + 11 + 17 + 20}{4} = 16.5 \approx 17$$

From Eq. (9.37)

$$Q_p = A_p(q_p) = A_p\left[0.4p_a N_{60}\left(\frac{L}{D}\right)\right] \leq A_p(4p_a N_{60})$$

$$A_p\left[0.4p_a N_{60}\left(\frac{L}{D}\right)\right] = (0.305 \times 0.305)\left[(0.4)(100)(17)\left(\frac{12}{0.305}\right)\right] = 2488.8 \text{ kN}$$

$$A_p(4p_a N_{60}) = (0.305 \times 0.305)[(4)(100)(17)] = 632.6 \text{ kN} \approx 633 \text{ kN}$$

Thus, $Q_p =$ **633 kN**

Part b

From Eq. (9.38),

$$Q_p = A_p q_p = A_p[19.7p_a(N_{60})^{0.36}] = (0.305 \times 0.305)[(19.7)(100)(17)^{0.36}]$$

$$= \mathbf{508.2 \text{ kN}}$$

1.11 Frictional Resistance (Qs) in Sand

According to Eq. (1.14), the frictional resistance

$Qs = \sum p \, \Delta L \, f$

The unit frictional resistance, f, is hard to estimate. In making an estimation of f, several important factors must be kept in mind:

1. The nature of the pile installation. For driven piles in sand, the vibration caused during pile driving helps densify the soil around the pile. The zone of sand densification may be as much as 2.5 times the pile diameter, in the sand surrounding the pile.

2. It has been observed that the nature of variation of f in the field is approximately as shown in Figure 1.16. The unit skin friction increases with depth more or less linearly to a depth of L' and remains constant thereafter. The magnitude of the critical depth L' may be 15 to 20 pile diameters. A conservative estimate would be

$$L' < 15D \qquad (1.23)$$

3. At similar depths, the unit skin friction in loose sand is higher for a high displacement pile, compared with a low-displacement pile.

4. At similar depths, bored, or jetted, piles will have a lower unit skin friction compared with driven piles.

Taking into account the preceding factors, we can give the following approximate relationship for f (see Figure 1.16):

For $z = 0$ to L',

$$f = K\sigma'_o \tan \delta' \qquad (1.24)$$

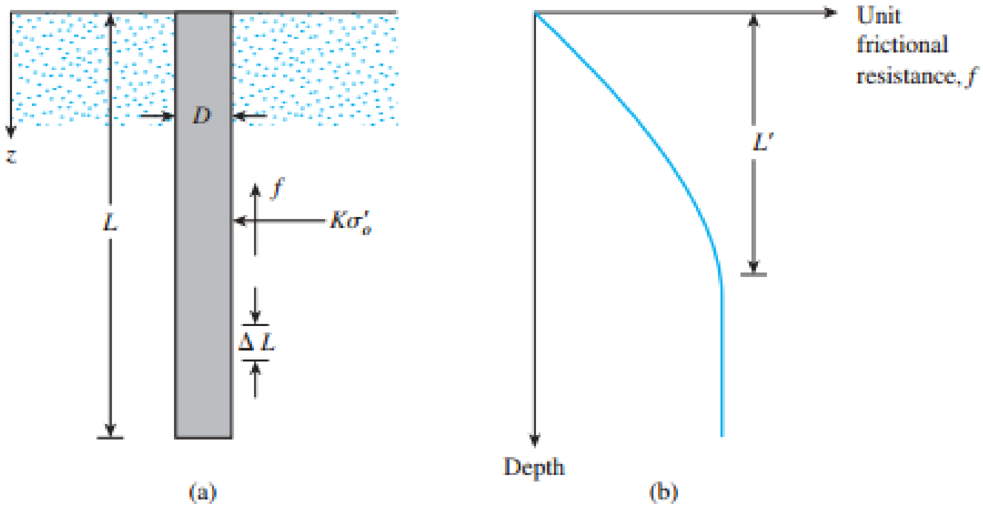


Fig.(6.16) Unit frictional resistance for piles in sand

and for $z = L'$ to L,

$$f = f_{z=L'} \tag{1.25}$$

In these equations,

K = effective earth pressure coefficient
σ'_o = effective vertical stress at the depth under consideration
δ' = soil-pile friction angle

In reality, the magnitude of K varies with depth; it is approximately equal to the Rankine passive earth pressure coefficient, K_p, at the top of the pile and may be less than the at-rest pressure coefficient, K_o, at a greater depth.

Based on presently available results, the following average values of K are recommended for use in Eq. (1.25):

Pile type	K
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$

The values of δ' from various investigations appear to be in the range from $0.5\phi'$ to $0.8\phi'$.

Based on load test results in the field, Mansur and Hunter (1970) reported the following average values of K.

H-piles. . . . . . $K = 1.65$

Steel pipe piles. . . . . . $K = 1.26$

Precast concrete piles. . . . . . $K = 1.5$

Correlation with Standard Penetration Test Results

- Meyerhof (1976) indicated that the average unit frictional resistance, f_{av} , for high-displacement driven piles may be obtained from average standard penetration resistance values as

$$f_{av} = 0.02p_a(\overline{N}_{60}) \tag{1.28}$$

where

$(\overline{N}_{60})$ = average value of standard penetration resistance

p_a = atmospheric pressure ($\approx$100 kN/m^2 or 2000 lb/ft^2)

For low-displacement driven piles

$$f_{av} = 0.01p_a(\overline{N}_{60}) \tag{1.29}$$

Briaud et al. (1985) suggested that

$$f_{av} \approx 0.224p_a(\overline{N}_{60})^{0.29} \tag{1.30}$$

Thus,

$$Q_s = pLf_{av} \tag{1.31}$$

Example 1.4

Refer to the pile described in Example 6.3. Estimate the magnitude of Qs for the pile. a. Use Eq. (1.28). b. Use Eq. (1.30). c. Considering the results in Example 6.3, determine the allowable load-carrying capacity of the pile based on Meyerhof's method and Briaud's method. Use a factor of safety, FS = 3.

Solution

The average N_{60} value for the sand for the top 12 m is

$$\overline{N}_{60} = \frac{8 + 10 + 9 + 12 + 14 + 18 + 11 + 17}{8} = 10.25 \approx 10$$

Part a

From Eq. (9.45),

$$f_{av} = 0.02p_a(\overline{N}_{60}) = (0.02)(100)(10) = 20 \text{ kN/m}^2$$

$$Q_s = pLf_{av} = (4 \times 0.305)(12)(20) = \mathbf{292.8 \text{ kN}}$$

Part b

From Eq. (9.47),

$$f_{av} = 0.224p_a(\overline{N}_{60})^{0.29} = (0.224)(100)(10)^{0.29} = 43.68 \text{ kN/m}^2$$

$$Q_s = pLf_{av} = (4 \times 0.305)(12)(43.68) = \mathbf{639.5 \text{ kN}}$$

Part c

$$\text{Meyerhof's method: } Q_{all} = \frac{Q_p + Q_s}{FS} = \frac{633 + 292.8}{3} = \mathbf{308.6 \text{ kN}}$$

$$\text{Briaud's method: } Q_{all} = \frac{Q_p + Q_s}{FS} = \frac{508.2 + 639.5}{3} = \mathbf{382.6 \text{ kN}}$$

So the allowable pile capacity may be taken to be about **345 kN**. ■

Example 1.5

Refer to Example 1.1. For the pile, estimate the frictional resistance Q_s. a. Based on Eqs. (1.24) and (1.25). Use K = 1.3 and $\delta=0.8\phi$.

SOLUTION

Part a

From Eq. (12.42), $L' = 15D = (15)(0.407) \approx 6.1$ m. Refer to Eq. (12.43):

At $z = 0$: $\sigma'_o = 0$, $f = 0$

At $z = 6.1$ m: $\sigma'_o = (6.1)(18) = 109.8 \text{ kN/m}^2$

So

$$f = K\sigma'_o \tan\delta' = (1.3)(109.8)[\tan(0.8 \times 35)] \approx 75.9 \text{ kN/m}^2$$

Thus,

$$Q_s = \frac{(f_{z=0} + f_{z=6.1\text{m}})}{2} pL' + f_{z=6.1\text{m}}\, p(L - L')$$

$$= \left(\frac{0 + 75.9}{2}\right)(4 \times 0.407)(6.1) + (75.9)(4 \times 0.407)(20 - 6.1)$$

$$= 376.87 + 1717.56 = 2094.43 \text{ kN} \approx \mathbf{2094\ kN}$$

1.12 Frictional (Skin) Resistance in Clay

- Estimating the frictional (or skin) resistance of piles in clay is almost as difficult a task as estimating that in sand, due to the presence of several variables that cannot easily be quantified. Several methods for obtaining the unit frictional resistance of piles are described in the literature. We examine some of them next.

1. λ Method

This method, proposed by Vijayvergiya and Focht (1972), is based on the assumption that the displacement of soil caused by pile driving results in a passive lateral pressure at any depth and that the average unit skin resistance is

$$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u) \qquad (1.51)$$

where

$\bar{\sigma}'_o$ = mean effective vertical stress for the entire embedment length
c_u = mean undrained shear strength ($\phi = 0$)

- The value of λ changes with the depth of penetration of the pile. (See Table 6.9.) Thus, the total frictional resistance may be calculated as

$$Q_s = pLf_{av}$$

Care should be taken in obtaining the values of $\bar{\sigma}'_o$ and c_u in layered soil. Figure 9.20 helps explain the reason. Figure 9.20a shows a pile penetrating three layers of clay. According to Figure 9.20b, the mean value of c_u is $(c_{u(1)}L_1 + c_{u(2)}L_2 + \cdots)/L$. Similarly, Figure 9.20c shows the plot of the variation of effective stress with depth. The mean effective stress is

$$\bar{\sigma}'_o = \frac{A_1 + A_2 + A_3 + \cdots}{L} \qquad (1.52)$$

where $A_1, A_2, A_3, \ldots$ = areas of the vertical effective stress diagrams.

Table 6.9 Variation of λ with Pile Embedment Length, L

Embedment length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

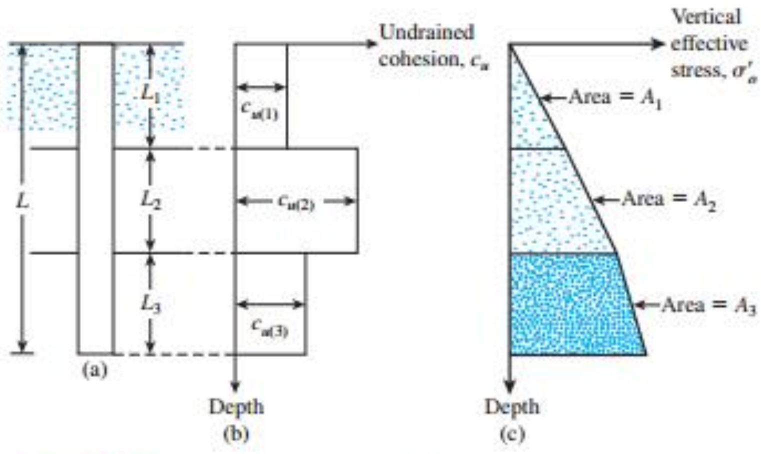


Figure 9.20 Application of λ method in layered soil

2. α Method

According to the α method, the unit skin resistance in clayey soils can be represented by the equation

$$f = \alpha c_u \qquad (1.53)$$

where α = empirical adhesion factor.

The approximate variation of the value of *a* is shown in Table 6.10.

The ultimate side resistance can thus be given as

$$Q_s = \Sigma f p \,\Delta L = \Sigma \alpha c_u p \,\Delta L \qquad (1.59)$$

Table 6.10 Variation of α (Interpolated Values Based on Terzaghi, Peck and Mesri, 1996)

$\frac{c_u}{p_a}$	α
≤ 0.1	1.00
0.2	0.92
0.3	0.82
0.4	0.74
0.6	0.62
0.8	0.54
1.0	0.48
1.2	0.42
1.4	0.40
1.6	0.38
1.8	0.36
2.0	0.35
2.4	0.34
2.8	0.34

Note: p_a = atmospheric pressure ≈ 100 kN/m^2 or 2000 lb/ft^2

Example 1.7

Refer to the pipe pile in saturated clay shown in Figure 9.24. For the pile,

a. Calculate the skin resistance (Q_s) by (1) the α method, (2) the λ method, and (3) the β method. For the β method, use $\phi'_R = 30°$ for all clay layers. The top 10 m of clay is normally consolidated. The bottom clay layer has an OCR = 2. (Note: diameter of pile = 457 mm)

b. Using the results of Example 9.2, estimate the allowable pile capacity (Q_{all}). Use FS = 4.

Solution

Part a

(1) From Eq. (9.59),

$$Q_s = \Sigma \alpha c_u p \Delta L$$

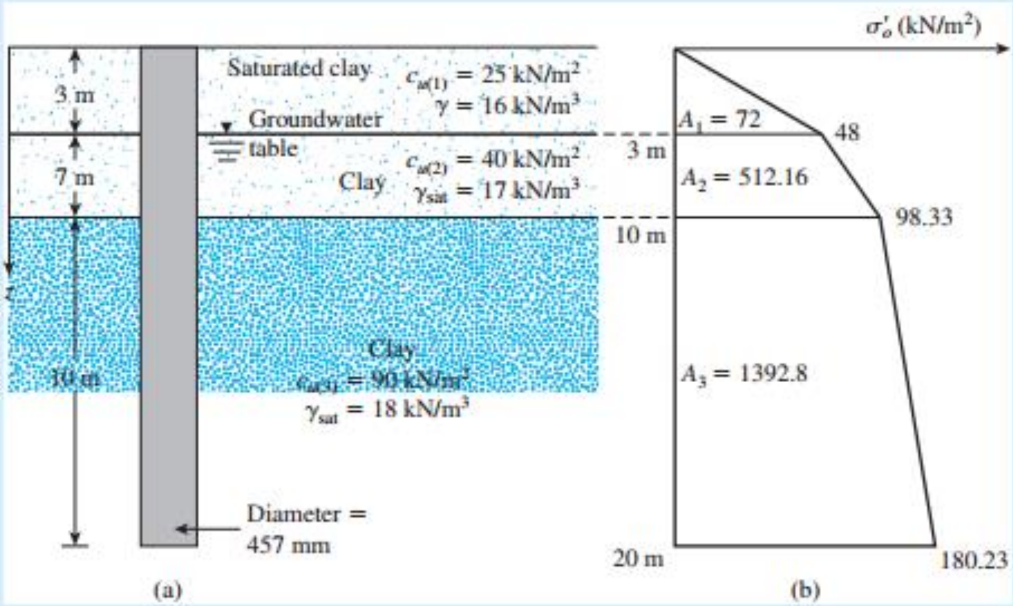


Figure 9.24 Estimation of the load bearing capacity of a driven-pipe pile

Hence,

$$Q_s = pLf_{av} = \pi(0.457)(20)(38.81) = \mathbf{1114.4\ kN}$$

(3) The top layer of clay (10 m) is normally consolidated, and $\phi'_R = 30°$. For $z = 0$–3 m, from Eq. (9.64), we have

$$f_{av(1)} = (1 - \sin \phi'_R) \tan \phi'_R \, \overline{\sigma}'_o$$

$$= (1 - \sin 30°)(\tan 30°)\left(\frac{0 + 48}{2}\right) = 6.93 \text{ kN/m}^2$$

Similarly, for $z = 3$–10 m.

$$f_{av(2)} = (1 - \sin 30°)(\tan 30°)\left(\frac{48 + 98.33}{2}\right) = 21.12 \text{ kN/m}^2$$

For $z = 10$–20 m from Eq. (9.65),

$$f_{av} = (1 - \sin \phi'_R)\tan \phi'_R \sqrt{\text{OCR}}\, \overline{\sigma}'_o$$

For OCR = 2,

$$f_{av(3)} = (1 - \sin 30°)(\tan 30°)\sqrt{2}\left(\frac{98.33 + 180.23}{2}\right) = 56.86 \text{ kN/m}^2$$

So,

$$Q_s = p[f_{av(1)}(3) + f_{av(2)}(7) + f_{av(3)}(10)]$$
$$= (\pi)(0.457)[(6.93)(3) + (21.12)(7) + (56.86)(10)] = \mathbf{1058.45\ kN}$$

Part b

$$Q_u = Q_p + Q_s$$

From Example 9.2,

$$Q_p \approx 151\ \text{kN}$$

Again, the values of Q_s from the α method, λ method, and β method are close. So,

$$Q_s = \frac{1050 + 1114.4 + 1058.45}{3} \approx 1074\ \text{kN}$$

$$Q_{all} = \frac{Q_u}{FS} = \frac{151 + 1074}{4} = 306.25\ \text{kN} \approx \mathbf{306\ kN}$$

1.13 Elastic Settlement of Piles

The total settlement of a pile under a vertical working load Q_w is given by:

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)} \qquad (1.80)$$

where

$s_{e(1)}$ = elastic settlement of pile
$s_{e(2)}$ = settlement of pile caused by the load at the pile tip
$s_{e(3)}$ = settlement of pile caused by the load transmitted along the pile shaft

If the pile material is assumed to be elastic, the deformation of the pile shaft can be evaluated, in accordance with the fundamental principles of mechanics of materials, as

$$s_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p} \qquad (1.81)$$

where

Q_{wp} = load carried at the pile point under working load condition
Q_{ws} = load carried by frictional (skin) resistance under working load condition
A_p = area of cross section of pile
L = length of pile
E_p = modulus of elasticity of the pile material

The magnitude of ξ varies between 0.5 and 0.67 and will depend on the nature of the distribution of the unit friction (skin) resistance f along the pile shaft.

The settlement of a pile caused by the load carried at the pile point may be expressed in the form:

$$s_{e(2)} = \frac{q_{wp}D}{E_s}(1 - \mu_s^2)I_{wp} \tag{1.82}$$

where

D = width or diameter of pile
q_{wp} = point load per unit area at the pile point = Q_{wp}/A_p
E_s = modulus of elasticity of soil at or below the pile point
μ_s = Poisson's ratio of soil
I_{wp} = influence factor ≈ 0.85

The settlement of a pile caused by the load carried by the pile shaft is given by a relation similar to Eq. (1.82), namely,

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL}\right)\frac{D}{E_s}(1 - \mu_s^2)I_{ws} \tag{1.84}$$

where

p = perimeter of the pile
L = embedded length of pile
I_{ws} = influence factor

Note that the term Q_{ws}/pL in Eq. (9.84) is the average value of f along the pile shaft. The influence factor, I_{ws}, has a simple empirical relation (Vesic, 1977):

$$I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}} \tag{1.85}$$

Example 1.8

The allowable working load on a prestressed concrete pile 21-m long that has been driven into sand is 502 kN. The pile is octagonal in shape with $D = 356$ mm (see Table 9.3a). Skin resistance carries 350 kN of the allowable load, and point bearing carries the rest. Use $E_p = 21 \times 10^6$ kN/m^2, $E_s = 25 \times 10^3$ kN/m^2, μ_s 0.35, and $\xi = 0.62$. Determine the settlement of the pile.

Solution

From Eq. (9.81),

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$$

From Table 9.3a for $D = 356$ mm, the area of pile cross section. $A_p = 1045$ cm^2, Also, perimeter $p = 1.168$ m. Given: $Q_{ws} = 350$ kN, so

$$Q_{wp} = 502 - 350 = 152 \text{ kN}$$

$$s_{e(1)} = \frac{[152 + 0.62(350)](21)}{(0.1045 \text{ m}^2)(21 \times 10^6)} = 0.00353 \text{ m} = 3.35 \text{ mm}$$

From Eq. (9.82),

$$s_{e(2)} = \frac{q_{wp}D}{E_s}(1 - \mu_s^2)I_{wp} = \left(\frac{152}{0.1045}\right)\left(\frac{0.356}{25 \times 10^3}\right)(1 - 0.35^2)(0.85)$$

$$= 0.0155 \text{ m} = 15.5 \text{ mm}$$

Again, from Eq. (9.84),

$$s_{e(3)} = \left(\frac{Q_{ws}}{pL}\right)\left(\frac{D}{E_s}\right)(1 - \mu_s^2)I_{ws}$$

$$I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}} = 2 + 0.35\sqrt{\frac{21}{0.356}} = 4.69$$

$$s_{e(3)} = \left[\frac{350}{(1.168)(21)}\right]\left(\frac{0.356}{25 \times 10^3}\right)(1 - 0.35^2)(4.69)$$

$$= 0.00084 \text{ m} = 0.84 \text{ mm}$$

Hence, total settlement is

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)} = 3.35 + 15.5 + 0.84 = \mathbf{19.69\ mm}$$

1.14 Pile Load Tests

- High-rise buildings often require several piles to support the building loads. Varying soil conditions, unreliable soil parameters, and the assumptions and simplifications in the theoretical model used in the prediction contribute to the variability in the ultimate load Q_u. The pile load test is a good way to verify the load-carrying capacity of a pile .
- Figure 1.21 shows schematic diagrams of two different pile load test arrangements for testing **axial compression** in the field. The main difference between the two is the way the horizontal reaction beam is held in place. In Figure 1.21a, a kent-ledge, consisting of heavy weights, is required to hold the reaction beam in place , and the hydraulic jack is used to jack against the beam and hence apply the pile load .
- In Figure 1.21b, two reaction piles, located far away from the test pile, anchor the horizontal reaction beam to the ground .
- The loads are applied in increments as specified by the relevant standards ,with sufficient time between the load increments.

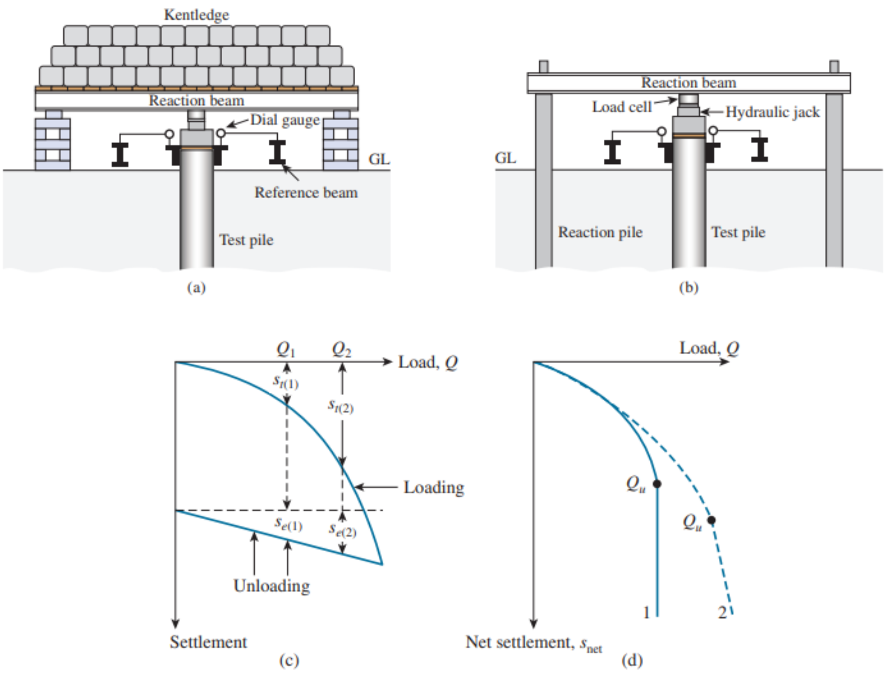


Figure 1.21 Pile load test: (a) using kentledge, (b) using reaction pile, (c) load vs. total settlement plots, and (d) load vs. net settlement

- Generally, the piles are loaded well beyond their working loads (e.g., 2 times). On reaching the maximum load for the test, the pile is unloaded in steps. The dial gauges measure the settlement of the pile head where they are mounted.
- The load test procedure just described requires the application of step loads on the piles and the measurement of settlement and is called a load-controlled test .
- Another technique used for a pile load test is the constant- rate-of-penetration test ,wherein the load on the pile is continuously increased to maintain a constant rate of penetration, which can vary from 0.25 to 2.5 mm/min. This test gives a loadsettlement plot similar to that obtained from the load-controlled test. Another type of pile load test is cyclic loading, in which an incremental load is repeatedly applied and removed.

1.15 Negative Skin Friction

- Negative skin friction is a downward drag force exerted on a pile by the soil surrounding it. Such a force can exist under the following conditions, among others:

1. If a fill of clay soil is placed over a granular soil layer into which a pile is driven, the fill will gradually consolidate. The consolidation process will exert a downward drag force on the pile (see Figure 1.24a) during the period of consolidation.
2. If a fill of granular soil is placed over a layer of soft clay, as shown in Figure 2.24b, it will induce the process of consolidation in the clay layer and thus exert a downward drag on the pile.

3. Lowering of the water table will increase the vertical effective stress on the soil at any depth, which will induce consolidation settlement in clay. If a pile is located in the clay layer, it will be subjected to a downward drag force.

- In some cases, the downward drag force may be excessive and cause foundation failure. This section outlines two tentative methods for the calculation of negative skin friction.

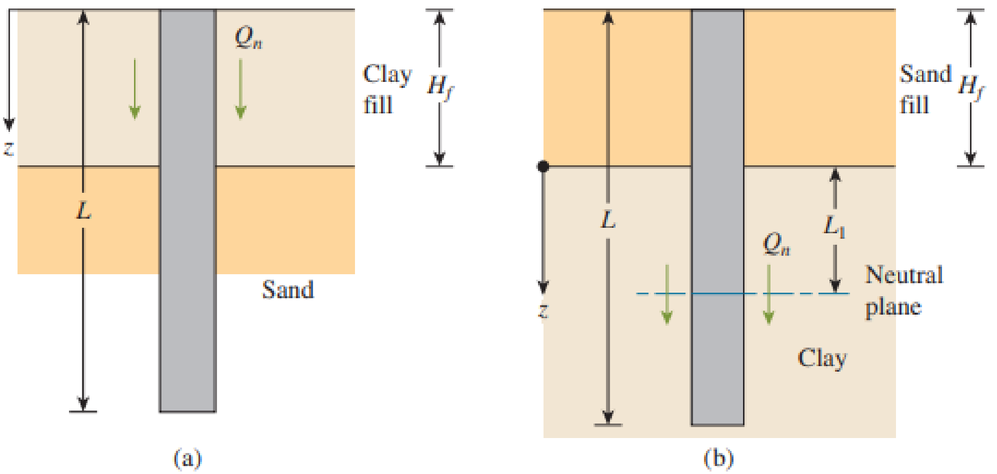


Figure 1.24 Negative skin friction

1.16 Tension Piles

- Tension piles may be used beneath buildings to resist uplift from hydrostatic pressure. They also may be used to support structures over expansive soils. Overturning caused by wind, ice loads, and broken wires may produce large tension forces for power transmission towers. In this type of situation the piles or piers supporting the tower legs must be designed for both compressive and tension forces.
-

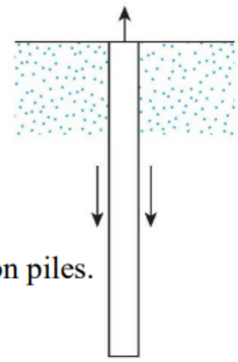
Uplift Piles:

If the net force on individual pile is a tension force then the pile must resist the tension which is done by the pile weight and the shaft resistance of the pile hence.

$$(Q_{uplift})_{ult} = (Q_S)_{ult} + W_{Pile}$$

$$(Q_{uplift})_{all} = \frac{(Q_{uplift})_{ult}}{FS} \; \ldots\ldots FS \geq 3$$

Qs is calculated as stated previously for compression piles.



Group Piles

1.17 Group Efficiency

In most cases, piles are used in groups, as shown in Figure 1.44, to transmit the structural load to the soil. A *pile cap* is constructed over *group piles*. The cap can be in contact with the ground, as in most cases (see Figure 1.44a), or well above the ground, as in the case of offshore platforms (see Figure 1.44b).

Determining the load-bearing capacity of group piles is extremely complicated and has not yet been fully resolved. When the piles are placed close to each other, a reasonable assumption is that the stresses transmitted by the piles to the soil will overlap (see Figure 1.44c), reducing the load-bearing capacity of the piles. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles. In practice, the minimum center-to-center pile spacing, *d*, is 2.5*D* and, in ordinary situations, is actually about 3 to 3.5*D*.

The efficiency of the load-bearing capacity of a group pile may be defined as

$$\eta = \frac{Q_{g(u)}}{\sum Q_u} \quad (1.127)$$

where

η = group efficiency
$Q_{g(u)}$ = ultimate load-bearing capacity of the group pile
Q_u = ultimate load-bearing capacity of each pile without the group effect

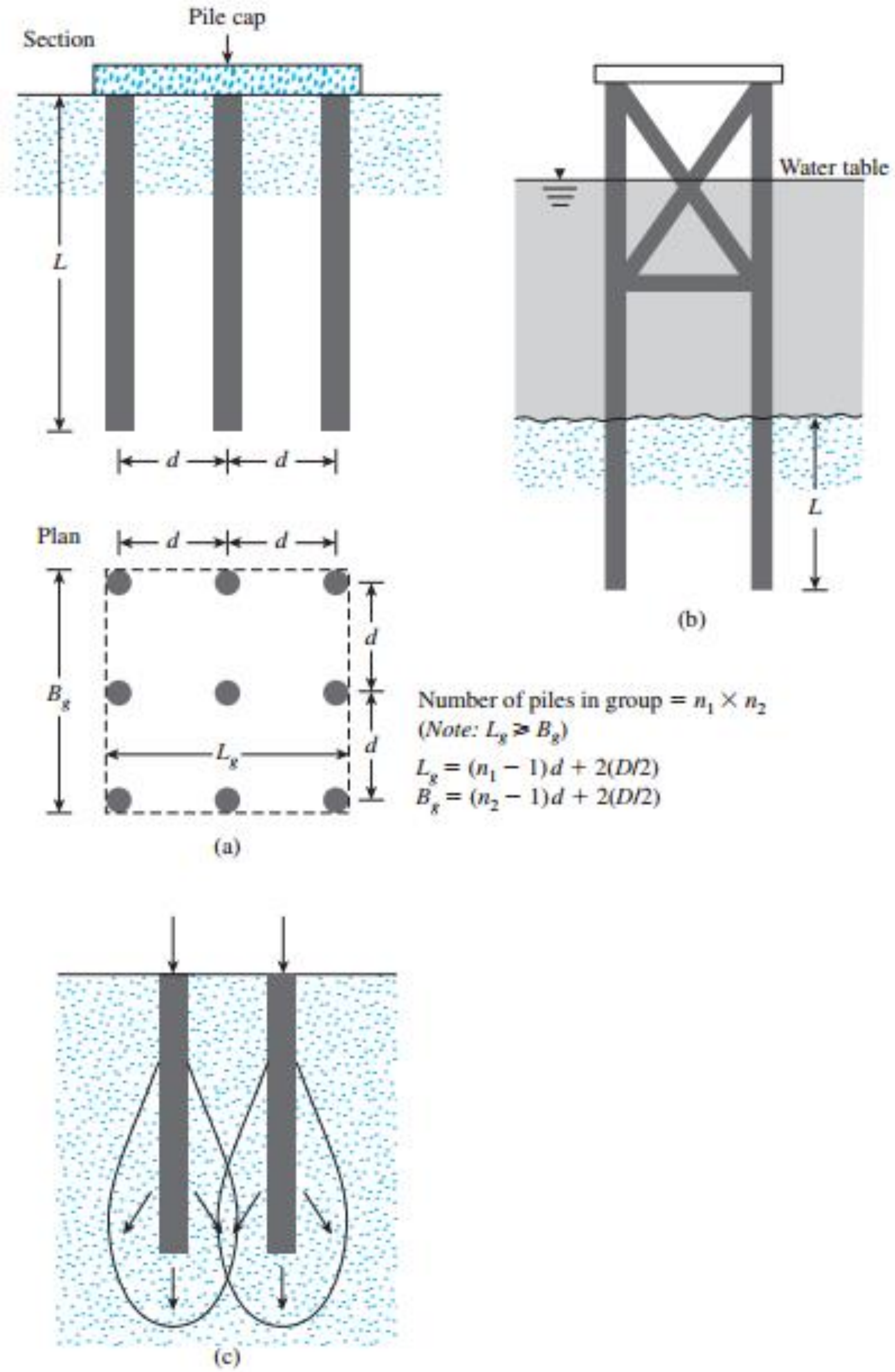


Fig. 1.44 Group piles

Piles in Sand

Many structural engineers use a simplified analysis to obtain the group efficiency for *friction piles*, particularly in sand. This type of analysis can be explained with the aid of Figure 1.44a. Depending on their spacing within the group, the piles may act in one of two ways:

(1) as a *block*, with dimensions $L_g \times B_g \times L$, or (2) as *individual piles*.

(2) as *individual piles*. If the piles act as a block, the frictional capacity is $\hat{f}_{av} p_g \hat{L} \approx Q_{g(u)}$.

p_q = perimeter of the cross section of block=
$2(n_1 + n_2 - 2)d + 4D$, and f_{av} = average unit frictional resistance.]

Similarly, for each pile acting individually, $Q_u < pLf_{av}$. (Note: p = perimeter of the cross section of each pile.) Thus,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{f_{av}[2(n_1 + n_2 - 2)d + 4D]L}{n_1 n_2 p L f_{av}} = \frac{2(n_1 + n_2 - 2)d + 4D}{p n_1 n_2} \quad (1.128)$$

Hence,

$$Q_{g(u)} = \left[\frac{2(n_1 + n_2 - 2)d + 4D}{p n_1 n_2}\right] \Sigma Q_u \quad (1.129)$$

From Eq. (9.129), if the center-to-center spacing d is large enough, $\eta > 1$. In that case, the piles will behave as individual piles. Thus, in practice, if $\eta < 1$, then

$$Q_{g(u)} = \eta \Sigma Q_u$$

and if $\eta \geq 1$, then

$$Q_{g(u)} = \Sigma Q_u$$

- There are several other equations like Eq. (1.129) for calculating the group efficiency of friction piles.

Ultimate Capacity of Group Piles in Saturated Clay

Figure 1.47 shows a group pile in saturated clay. Using the figure, one can estimate the ultimate load-bearing capacity of group piles in the following manner:

Step 1. Determine $\Sigma Q_u = n_1 n_2 (Q_p + Q_s)$. From Eq. (1.18)

$$Q_p = A_p[9c_{u(p)}]$$

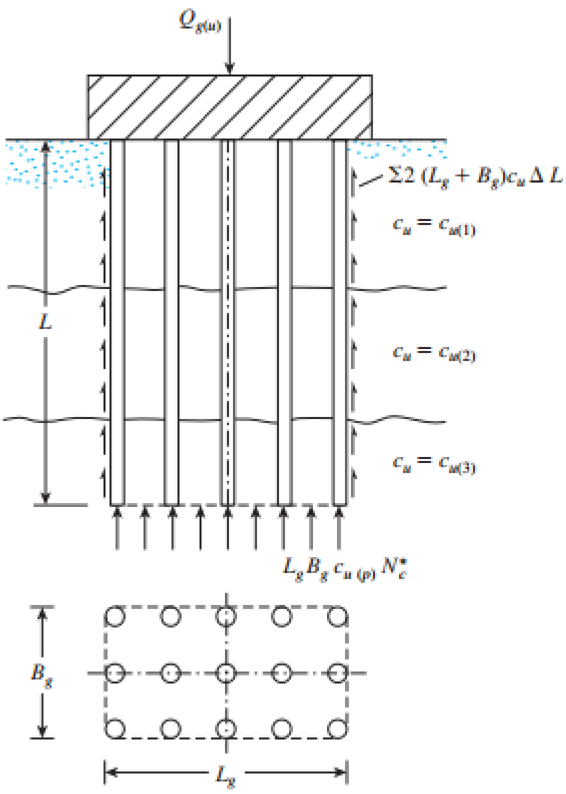


Fig.1.47 Ultimate capacity of group piles in clay

where $c_{u(p)}$ = undrained cohesion of the clay at the pile tip.
Also, from Eq. (9.59),

$$Q_s = \Sigma\, \alpha p c_u \Delta L$$

So,

$$\Sigma\, Q_u = n_1 n_2 [9 A_p c_{u(p)} + \Sigma\, \alpha p c_u \Delta L] \tag{1.130}$$

Step 2. Determine the ultimate capacity by assuming that the piles in the group act as a block with dimensions $L_g \times B_g \times L$. The skin resistance of the block is

$$\Sigma\, p_g c_u \Delta L = \Sigma\, 2(L_g + B_g) c_u \Delta L$$

Calculate the point bearing capacity:

$$A_p q_p = A_p c_{u(p)} N_c^* = (L_g B_g) c_{u(p)} N_c^*$$

Obtain the value of the bearing capacity factor N_c^* from **Fig. 1.48**). Thus, the ultimate load is

$$\Sigma\, Q_u = L_g B_g c_{u(p)} N_c^* + \Sigma\, 2(L_g + B_g) c_u \Delta L \tag{1.131}$$

Step 3. Compare the values obtained from Eqs. (9.130) and (9.131). The *lower* of the two values is $Q_{g(u)}$.

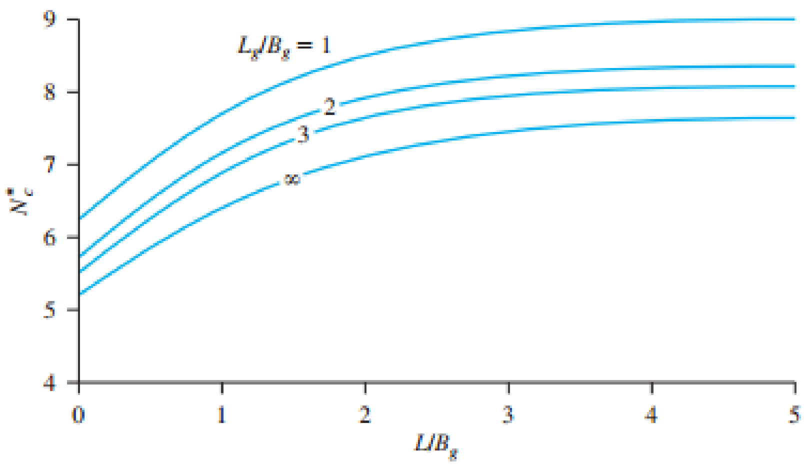


Fig. 1.48 Variation of N_c^* with L_g/B_g and L/B_g

Example 1.14

The section of a 3×4 group pile in a layered saturated clay is shown in Figure 12.56. The piles are square in cross section (356 mm $\times$ 356 mm). The center-to-center spacing, d, of the piles is 889 mm. Determine the allowable load-bearing capacity of the pile group. Use FS = 4. Note that the groundwater table coincides with the ground surface.

SOLUTION

From Eq. (12.138),

$$\Sigma Q_u = n_1 n_2 [9 A_p c_{u(p)} + \alpha_1 p c_{u(1)} L_1 + \alpha_2 p c_{u(2)} L_2]$$

From Figure 12.56, $c_{u(1)} = 50.3 \text{ kN/m}^2$ and $c_{u(2)} = 85.1 \text{ kN/m}^2$.

For the top layer with $c_{u(1)} = 50.3 \text{ kN/m}^2$,

$$\frac{c_{u(1)}}{p_a} = \frac{50.3}{100} = 0.525$$

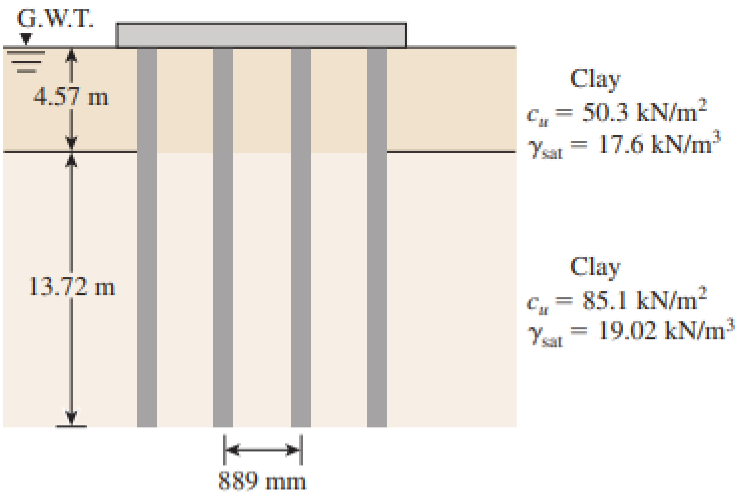


FIGURE 12.56 Group pile of layered saturated clay

From Table 12.11, $\alpha_1 \approx 0.68$. Similarly,

$$\frac{c_{u(2)}}{p_a} = \frac{85.1}{100} \approx 0.85$$
$$\alpha_2 = 0.51$$

$$\Sigma Q_u = (3)(4)\begin{bmatrix}(9)(0.356)^2(85.1) + (0.68)(4 \times 0.356)(50.3)(4.57) \\ + (0.51)(4 \times 0.356)(85.1)(13.72)\end{bmatrix} = 14011 \text{ kN}$$

For piles acting as a group.

$$L_g = (3)(0.889) + 0.356 = 3.023 \text{ m}$$
$$B_g = (2)(0.889) + 0.356 = 2.134 \text{ m}$$
$$\frac{L_g}{B_g} = \frac{3.023}{2.134} = 1.42$$
$$\frac{L}{B_g} = \frac{18.29}{2.134} = 8.57$$

From Figure 12.55, $N_c^* = 8.75$. From Eq. (12.139),

$$\begin{aligned}\Sigma Q_u &= L_g B_g c_{u(p)} N_c^* + \Sigma 2(L_g + B_g) c_u \, \Delta L \\ &= (3.023)(2.134)(85.1)(8.75) + (2)(3.023 + 2.134)[(50.3)(4.57) + (85.1)(13.72)] \\ &= 19217 \text{ kN}\end{aligned}$$

Hence, $\Sigma Q_u = 14{,}011$ kN.

$$\Sigma Q_{\text{all}} = \frac{14{,}011}{\text{FS}} = \frac{14{,}011}{4} \approx \mathbf{3503 \text{ kN}}$$

1.17 Elastic Settlement of Group Piles

In general, the settlement of a group pile under a similar working load per pile increases with the width of the group (B_g) and the center-to-center spacing of the piles (d). Several investigations relating to the settlement of group piles have been reported in the literature, with widely varying results. The simplest relation for the settlement of group piles was given by Vesic (1969), namely,

$$s_{g(e)} = \sqrt{\frac{B_g}{D}} s_e \qquad (1.139)$$

where

$s_{g(e)}$ = elastic settlement of group piles

B_g = width of group pile section

D = width or diameter of each pile in the group

s_e = elastic settlement of each pile at comparable working load (see Section 12.18)

For group piles in sand and gravel, for elastic settlement, Meyerhof (1976) suggested the empirical relation

$$s_{g(e)}(\text{mm}) = \frac{0.96q\sqrt{B_g}I}{N_{60}} \quad (1.140)$$

where

$$q = Q_g/(L_g B_g) \text{ (in kN/m}^2) \quad (1.141)$$

and

L_g and B_g = length and width of the group pile section, respectively (m)

N_{60} = average standard penetration number within seat of settlement ($\approx B_g$ deep below the tip of the piles)

$$I = \text{influence factor} = 1 - L/8B_g \geq 0.5 \quad (1.142)$$

L = length of embedment of piles (m)

Similarly, the group pile settlement is related to the cone penetration resistance by the formula

$$S_{g(e)} = \frac{qB_g I}{2q_c} \quad (1.143)$$

where q_c = average cone penetration resistance within the seat of settlement. [Note that in Eq. (12.144), all quantities are expressed in consistent units.]

Example 1.17

Consider a 3 × 4 group of prestressed concrete piles, each 21 m long, in a sand layer. The details of each pile and the sand are similar to that described in Example 12.14. The working load for the pile group is 6024 kN (3 × 4 × Q_{all}—where Q_{all} = 502 kN as in Example 12.14), and d/D = 3. Estimate the elastic settlement of the pile group. Use Eq. (12.139).

SOLUTION

$$s_{e(g)} = \sqrt{\frac{B_g}{D}} s_e$$

$$B_g = (3-1)d + \frac{2D}{2} = (2)(3D) + D = 7D = (7)(0.356 \text{ m}) = 2.492 \text{ m}$$

From Example 12.14, s_e = 19.69 mm. Hence,

$$s_{e(g)} = \sqrt{\frac{2.492}{0.356}}(19.69) = \mathbf{52.09 \text{ mm}}$$

1.18 Pile groups Subjected to moment

To calculate the ultimate bearing capacity for each pile in a group subjected to moment. The following formula can be used:.

$$P_{min}^{max} = \frac{\sum V}{n} \pm \frac{M_x * y}{\sum y^2} \pm \frac{M_y * x}{\sum x^2} = \sum V \left(\frac{1}{n} \pm \frac{e_y * y}{\sum y^2} \pm \frac{e_x * x}{\sum x^2}\right)$$

Where:

P = Total pile reaction resulting from moment and load.

Σv= Sum of vertical loads acting on the foundation.

M_x, M_y= Moments about x and y axes, respectively.

n = No. of piles.

y& x =Maximum Distance of pile from x and y axes.

Σy^2 &Σx^2 = Sum of the squares of the distance to each pile.

<u>For Example:</u>

$\Sigma y^2 = 6 * (0.5\ S)^2 = 1.5S^2$

$\Sigma x^2 = 4 * (S)^2 = 4\ S^2$

$\Sigma x^2 = 8\ x_1^2 + 8\ x_2^2$

$\Sigma y^2 = 8\ y_1^2 + 8\ y_2^2$