SECTION 4C
PILE FOUNDATIONS

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4C.1 INTRODUCTION

Piles are vertical or slightly inclined members used to transmit the loads of the superstructure to lower layers in the soil mass. The load transfer mechanism relies either on the skin resistance occurring along the surface contact of the pile with the soil or on the end bearing on a dense or firm layer. The design of some piles can also be based on the utilization of both the skin resistance and the end bearing to carry the applied load jointly. In general, pile foundations are relied upon to transfer the load acting on the superstructures in situations where the use of shallow foundations becomes inadequate or unreliable. Such situations include (1) the top soil layers have a weak bearing capacity, with the soil layer at greater depth possessing a high bearing capacity; (2) large values of concentrated loads are to be transmitted from the superstructure to the foundation; and (3) the structure to be designed is very sensitive to unequal settlements.

Materials usually used to make piles are concrete, steel, and timber. The upper part of the pile connected to the superstructure is referred to as the pile head. The middle part is called the shaft, the lower is the pile tip. The pile cross section can either be maintained throughout the length of the pile or it can be tapered to a rather pointed pile. The cross section can be circular, octagonal, hexagonal, square, triangular, or H-shaped. Figure 4C.1 illustrates typical pile shapes and various cross sections.

Piles can be classified into two types—displacement piles and nondisplacement piles. Displacement piles are those which displace the soil to allow for the pile penetration. These piles can be of solid cross section, driven into the ground, and left in position. Timber, steel, prestressed concrete piles, and precast concrete piles are of this type. Displacement piles are also obtained by driving shell (hollow) piles by means of an internal steel mandrel onto which the shell is threaded. After the mandrel is pulled out, the shell pile is filled with concrete internally. The Raymond pile is a mandrel-driven steel-shell pile; the Western pile is a mandrel-driven concrete-shell pile. Another method for obtaining displacement piles is driving a pilelike body into the ground and withdrawing it while filling the void with concrete (Franki pile). Nondisplacement piles are those in which the soil is removed to accommodate the pile. Typically a borehole is formed in the ground, then concrete is cast in place in the hole.

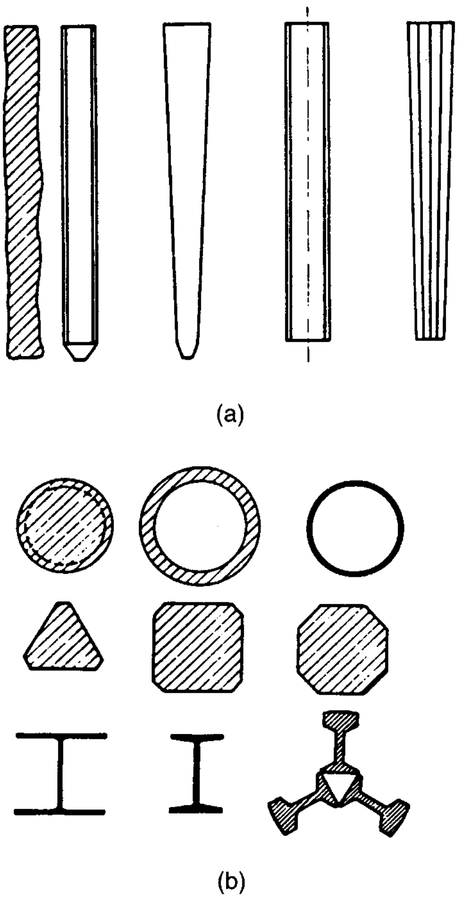


FIGURE 4C.1 Typical pile shapes and cross sections. (a) Piles shapes. (b) Various pile cross sections.

4C.2 ALLOWABLE STRESSES IN PILES

In pile foundation design it is necessary to determine the required number of piles, their cross section, and their length. This will require knowledge about the pile loading capacity as well as their allowable stresses. This section will present the allowable design stresses for service loads as adopted in the design guides of the U.S. Army Corps of Engineers and published by the American Society of Civil Engineers (ASCE, 1993). The pile loading capacity is discussed in Sec. 4C.3. The allowable stresses presented in this section may be increased by one-third to account for unusual loading such as maintenance, infrequent floods, barge impact, construction, or hurricanes.

4C.2.1 Concrete Piles

Concrete piles can be prestressed, precast-reinforced, cast in place, or mandrel-driven.

4C.2.1.1 Prestressed Concrete Piles

Prestressed concrete piles are formed by tensioning high-strength steel cable having an ultimate stress f_{pu} of 250 to 270 ksi with a prestress of about 0.5 to $0.7f_{pu}$ before casting the concrete. Af-

TABLE 4C.1 Allowable Concrete Stresses for Prestressed Concrete Piles

Uniform axial tension	0
Bending (extreme fiber)	
Compression	$0.40f'_c$
Tension	0

Source: ASCE, 1993.

ter casting the concrete, and only when it develops adequate strength, the prestress cables are cut. Due to the bond between steel and concrete, the cables will apply a compressive stress on the concrete pile as they attempt to return to their original length. When designing prestressed concrete piles, both strength and serviceability requirements must be satisfied. Strength design should be conducted in accordance with the American Concrete Institute code (ACI, 1989), except that a strength reduction factor ϕ of 0.7 is to be used for all failure modes and a load factor of 1.9 for both dead and live loads. The use of these factors will result in a factor of safety of 2.7 for all dead and live load combinations. The axial strength to be used in design is the least of: (1) 80% of the concentric axial strength or (2) the axial strength corresponding to an eccentricity equal to 10% of the pile diameter or width. Cracking control is achieved by limiting the actual concrete compressive and tensile stresses resulting from working conditions to the values presented in Table 4C.1. For the combined condition of axial force and bending, the concrete stresses should satisfy the following:

$$f_a + f_b + f_{pc} \leq 0.4f'_c \tag{4C.la}$$

$$f_a - f_b + f_{pc} \geq 0 \tag{4C.lb}$$

where f_a = computed axial stress (tensions negative)
f_b = computed bending stress (tensions negative)
f_{pc} = effective prestress
f'_c = concrete compressive strength

The allowable stresses for hydraulic structures are limited to 0.85% of the values recommended by ACI Committee 543 for improved serviceability (ACI, 1986). Permissible stresses in the prestressing steel cables should be in accordance with the ACI code requirements (ACI, 1989). In cases where the pile is free-standing or when the soil is too weak to provide a reliable lateral support, the pile capacity should be reduced due to slenderness effects. The moment magnification method of ACI as modified by PCI can be used to perform such design (PCI, 1988). Figure 4C.2 illustrates typical prestressed concrete piles.

4C.2.1.2 Precast-Reinforced Concrete Piles

Precast-reinforced concrete piles are designed in accordance with the ACI code (ACI, 1989). For hydraulic structures, the ultimate load is to be increased by a hydraulic load factor H_f. The hydraulic load factor is taken as (1) 1.30 for reinforcement calculations in flexure or compression, (2) 1.65 for reinforcement in direct tension, and (3) 1.30 for reinforcement in shear. When performing shear reinforcement design, the calculations should exclude the shear carried by the concrete prior to application of the hydraulic load factor. The axial strength limitations are taken as in the case for prestressed piles. The slenderness effects are accounted for according to the ACI code moment magnification method (ACI, 1989). Figure 4C.3 shows typical precast-reinforced concrete piles.

4C.2.1.3 Cast-in-Place and Mandrel-Driven Piles

Figure 4C.4 illustrates various types of cast-in-place piles. The depths indicated are for the usual ranges for the different piles. These piles are mostly used when continuous lateral support is present. Cast-in-place and mandrel-driven piles are designed such that working stresses are limited to the al-

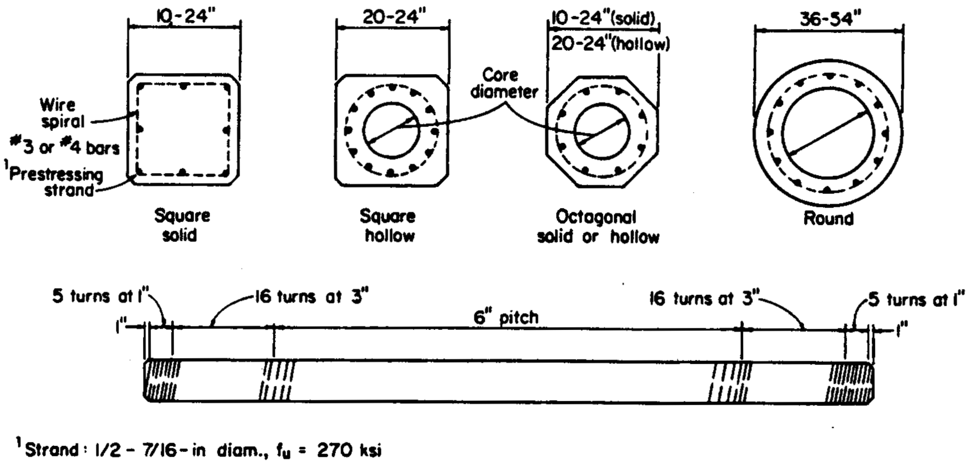
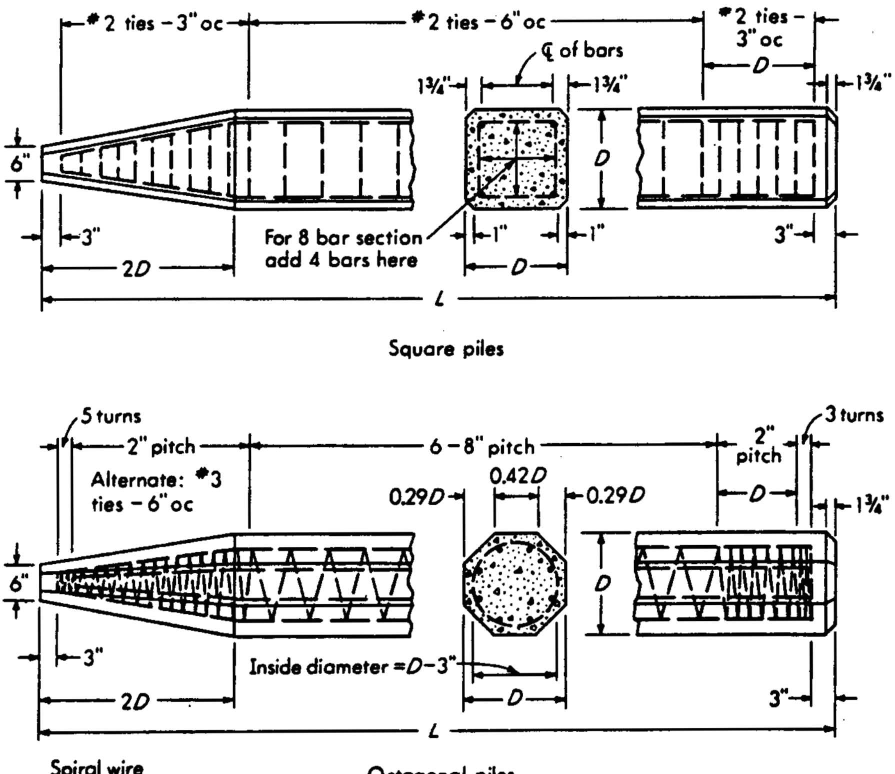


FIGURE 4C.2 Typical prestressed concrete pile. (*From Bowles, 1982.*)



D, in.	16	20	24
Size	#5	#4	#3

FIGURE 4C.3 Typical precast concrete pile. (*From Bowles, 1982.*)

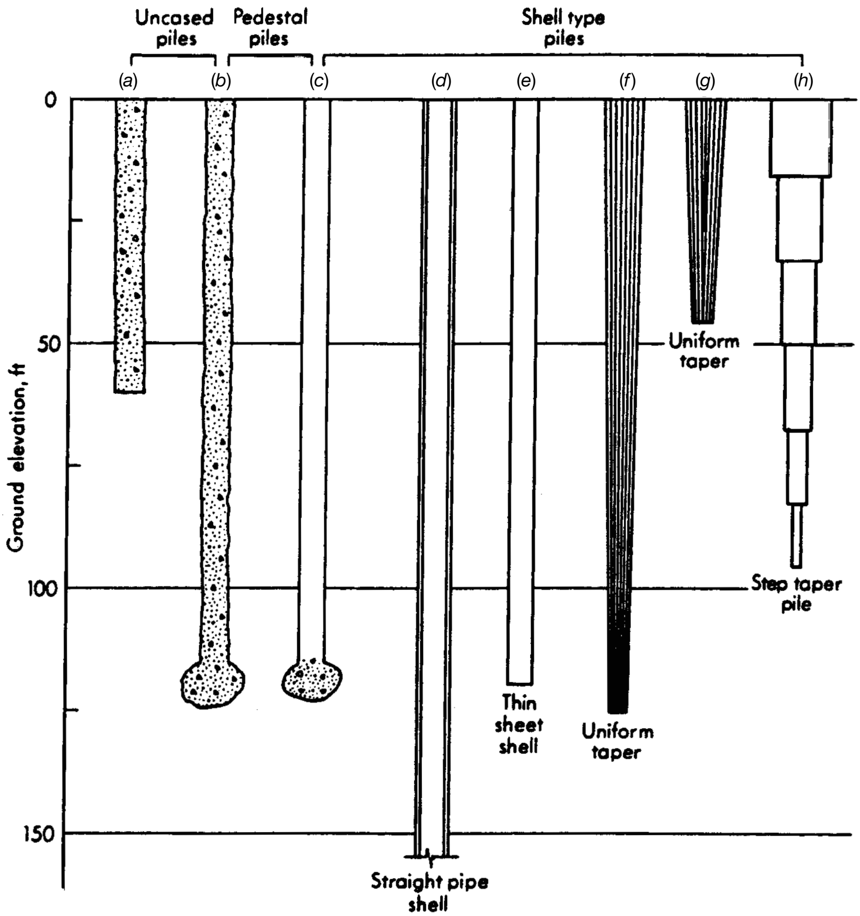


FIGURE 4C.4 Typical cast-in-place concrete pile. (*a*) Western uncased pile. (*b*) Franki uncased-pedestal pile. (*c*) Franki cased-pedestal pile. (*d*) Welded or seamless pile. (*e*) Western cased pile. (*f*) Union or Monotube pile. (*g*) Raymond standard. (*h*) Raymond step-taper pile. (*From Bowles, 1982.*)

lowable stresses shown in Table 4C.2. In case of axial load combined with bending, the concrete stresses are such that

$$\left| \frac{f_a}{F_a} + \frac{f_b}{F_b} \right| \leq 1.0 \qquad (4C.2)$$

where f_a = computed axial stress
F_a = allowable axial stress
f_b = computed bending stress
F_b = allowable bending stress

TABLE 4C.2 Allowable Concrete Stresses for Cast-in-Place and Mandrel-Driven Piles

Uniform axial compression	
Confined	$0.33f'_c$
Unconfined	$0.27f'_c$
Uniform axial tension	0
Bending (extreme fiber)	
Compression	$0.40f'_c$
Tension	0

Source: ASCE, 1993.

4C.2.2 Steel Piles

Steel piles are usually rolled, H-shaped, or pipe piles. The lower region of these piles could be subjected to damage during driving. This is why the U.S. Army Corps of Engineers uses allowable stresses with a high factor of safety for that region, as shown in Fig. 4C.5. Pile shoes are usually used when driving piles in the dense layer. Table 4C.3 shows the allowable stresses for fully supported piles when using pile shoes. The upper portion of the pile is designed as a beam-column where

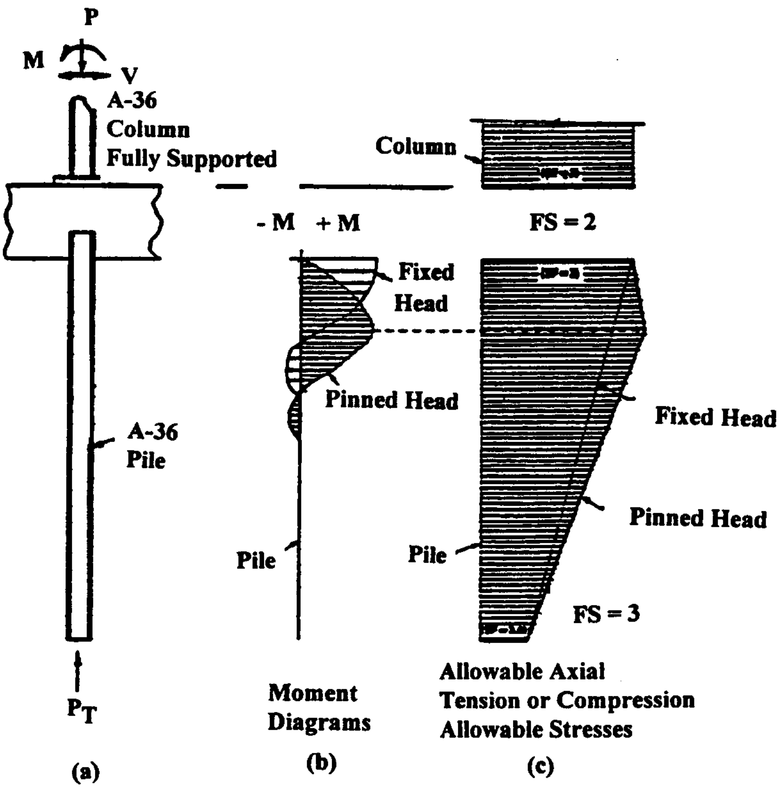


FIGURE 4C.5 Behavior of steel piles. (*From ASCE, 1993.*)

TABLE 4C.3 Allowable Stresses in Lower Pile Region for Steel Piles

Concentric axial tension or compression only 10 ksi (⅓ × F_y ⁵⁄₆)	10 ksi for A-36 material
Concentric axial tension or compression only with driving shoes (⅓ × F_y)	12 ksi for A-36 material
Concentric axial tension or compression only with driving shoes, at least one axial load test and use of a pile driving analyzer to verify pile capacity and integrity (l/2.5 × F_y)	14.5 ksi for A-36 material

Source: ASCE, 1993.

the lateral support conditions are accounted for. Bending moments are, however, negligible in the lower portion of the pile. The moment diagram along the pile is shown in Fig. 4C.5. For laterally unsupported piles the allowable stress should be five-sixths of the values the American Institute of Steel Construction code gives for beam columns (AISC, 1989). For combined axial compression and bending conditions, the stress should be

$$\left| \frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \right| \leq 1.0 \tag{4C.3}$$

where f_a = computed axial stress
F_a = allowable axial stress, = $0.5F_y$
f_{bx}, f_{by} = computed bending stress
F_b = allowable bending stress, 0.5 F_y for noncompact section and ⁵⁄₉ F_y for compact section

3.D.2.3 Timber Piles

Timber piles are cut from tree trunks and driven with the smaller cross section down. Representative allowable stress values for pressure-treated round timber piles are presented in Table 4C.4. These stresses have been adjusted to account for treatment. For untreated piles, or piles that were either air- or kiln-dried before pressure treatment, the allowable stress shown in Table 4C.4 should be increased by dividing each value by 0.9 for Pacific Coast Douglas fir and by 0.85 for southern pine. To account for combined axial load and bending moment effects, the stresses should satisfy

$$\left| \frac{f_a}{F_a} + \frac{f_b}{F_b} \right| \leq 1.0 \tag{4C.4}$$

TABLE 4C.4 Allowable Stresses for Pressure-Treated Round Timber Piles

Species	Compression parallel to grain, psi F_a	Bending, psi F_b	Horizontal shear, psi	Compression perpendicular to grain, psi	Modulus of elasticity, psi
Pacific Coast Douglas fir	875	1700	95	190	1,500,000
Southern pine	825	1650	90	205	1,500,000

Source: ASCE, 1993.

where f_a = computed axial stress
F_a = allowable axial stress
f_b = computed bending stress
F_b = allowable bending stress

4C.3 PILE LOADING CAPACITY

4C.3.1 General

The mechanism of the load transfer from piles to the soil layer is illustrated in Fig. 4C.6. The horizontal earth pressures act on the shaft surface area, creating vertical frictional reactions that increase with depth. If enough displacement occurs, adhesion could also contribute to these reactions. The sum of these reactions is referred to as the mantle friction or skin resistance. In addition vertical reactions occur at the tip of the pile, mobilizing tip-bearing resistance. The ratio of the mantle friction to the tip-bearing resistance varies according to the physical properties and profile of the soil, the pile dimensions, and the method of installation.

4C.3.2 Axial Single-Pile Capacity

The pile loading capacity consists of the sum of the skin resistance and the tip-bearing resistance. Hence it may be represented by the equation

$$Q_{\text{ult}} = Q_s + Q_t = f_s A_s + q A_t \tag{4C.5}$$

where Q_{ult} = ultimate pile capacity
Q_s = shaft resistance of pile due to skin friction
Q_t = tip-bearing resistance of pile
f_s = average skin resistance stress
A_s = surface area of shaft in contact with soil
q = tip-bearing stress
A_t = effective area at tip of pile in contact with soil

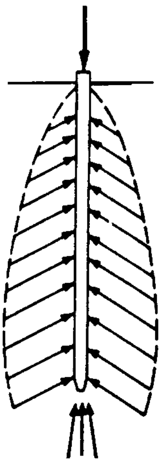


FIGURE 4C.6 Load transfer mechanism using skin friction resistance and tip-bearing resistance.

TABLE 4C.5 Typical K values* for Piles in Compression and in Tension

Soil type	K_c	K_t
Sand	1.00 to 2.00	0.50 to 0.70
Silt	1.00	0.50 to 0.70
Clay	1.00	0.70 to 1.00

*Values do not apply to piles that are prebored, jetted, or installed with a vibratory hammer. Picking K values at the upper end of these ranges should he based on local experience. K, δ, and N_q values hack-calculated from load tests may he used.

Source: ASCE, 1993.

4C.3.2.1 Piles in Cohesionless Soil

The skin resistance of piles in cohesionless soil is assumed to increase linearly up to a critical depth d_c. For design purposes, the critical depth is taken as 10B for loose sand, 15B for medium dense sand, and 20B for dense sand, where B is the pile diameter or width. Below the critical depth, the skin resistance is taken to be a constant value (equal to the critical-depth skin resistance stress). The average skin resistance stress at a particular depth may be calculated using the equation

$$f_s = K\sigma_v' \tan \delta \tag{4C.6}$$

where K = coefficient for lateral earth pressure (see Table 4C.5 for K values for piles in compression and tension)

σ_v' = effective overburden pressure at particular depth d

$= \gamma' d$ for $d < d_c$

$= \gamma' d_c$ for $d < d_c$ using γ' as the effective unit weight for soil

δ = friction angle between soil and pile material (see Table 4C.6 for typical δ values)

It must be emphasized that the K and δ values presented should be selected based on experience and site conditions and could be replaced with better representative values if such are available to the designer. When using steel H piles, the value of δ should be the average friction angle of steel against soil and soil against soil (ϕ value). Also, the value of A_s for steel H piles is to be taken as the block perimeter of the pile.

The tip-bearing stress, q can be determined from the expression

$$q = \sigma_v' N_q \tag{4C.7}$$

where σ_v' is as defined earlier, and the bearing capacity factor N_q is obtained from Fig. 4C.7. When using steel H piles, the area A_t is taken as the area included within the block perimeter. The pile tension capacity in cohesionless soil is obtained by solely calculating the shaft resistance of the pile due to skin friction Q_s using the corresponding K values in Table 4C.5.

TABLE 4C.6 Typical δ Angles in terms of ϕ

Pile material	δ
Steel	0.67 ϕ to 0.83 ϕ
Concrete	0.90 ϕ to 1.0 ϕ
Timber	0.80 ϕ to 1.0 ϕ

Source: ASCE, 1993.

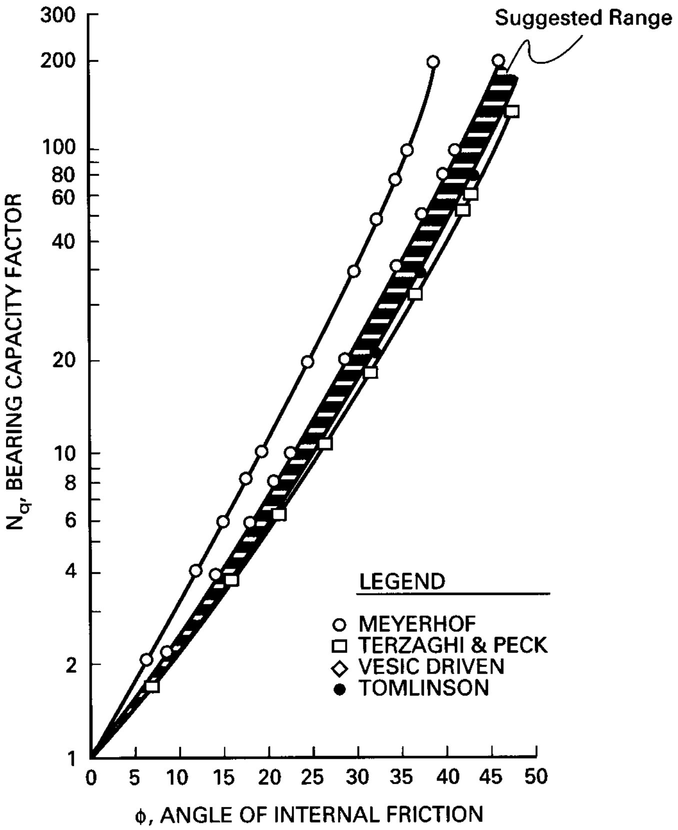


FIGURE 4C.7 Bearing capacity factor N_q versus angle of internal friction ϕ. (*From Terzaghi and Peck, 1967.*)

Example 4C.1 Find the allowable compression capacity of a 12-in-diameter (305 mm) reinforced concrete pile with a total length of 45 ft (13.7 mm) driven in medium dense sand. The K and δ values are found to be 1.5 and 0.9ϕ, respectively. The soil profile is shown in Fig. 4C.X. 1. The soil angle of friction ϕ is 30°. Use a factor of safety of 3.0.

Solution The critical depth is

$$d_c = 15B = 15\left(\frac{12}{12}\right) = 15 \text{ ft } (4.57 \text{ m})$$

The effective overburden pressure σ_v' at water table level is

$$\sigma_v' = 110 \times 10 = 1100 \text{ psf } (53.647 \text{ kPa})$$

The effective overburden pressure σ_v' at the critical depth d_c is

$$\sigma'_{v(-15)} = 1100 + 5(125 - 62.4) = 1413 \text{ psf } (68.91 \text{ kPa})$$

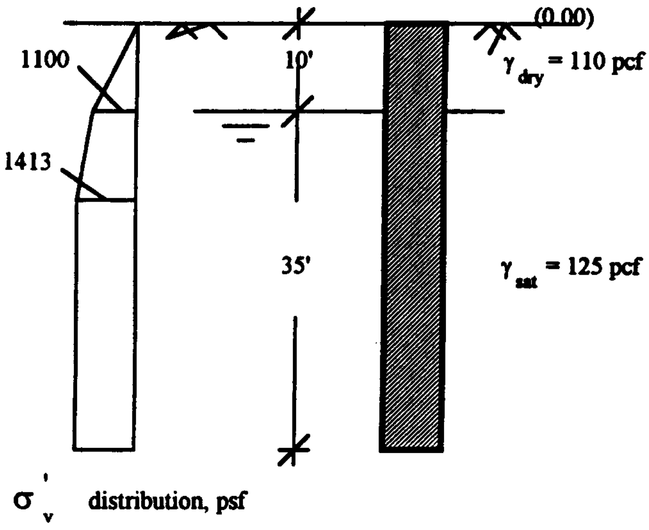


FIGURE 4C.X.1

The shaft resistance due to skin friction from level (0.00) to (–10.00) (3 m) is

$$\left[1.5\left(\frac{1100+0}{2}\right)(\tan 0.9\times 30°)\right]\left[\pi\left(\frac{12}{12}\right)10\right] = 13{,}185 \text{ lb } (58.65 \text{ kN})$$

The shaft resistance due to skin friction from level (–10.00) (3 m) to (–15.00) (4.6 m) is

$$\left[1.5\left(\frac{1100+1413}{2}\right)(\tan 0.9\times 30°)\right]\left[\pi\left(\frac{12}{12}\right)5\right] = 15{,}061 \text{ lb } (66.99 \text{ kN})$$

The shaft resistance due to skin friction from level (–15.00) (–4.6 m) to (–45.00) (13.7 m) is

$$[1.5\times 1413(\tan 0.9\times 30°)]\left[\pi\left(\frac{12}{12}\right)30\right] = 101{,}625 \text{ lb } (452 \text{ kN})$$

The total shaft resistance due to skin friction is

$$Q_s = 13{,}185 + 15{,}061 + 101{,}625 = 129{,}871 \text{ lb } (582 \text{ kN})$$

The tip-bearing resistance is, using $N_q = 18$,

$$Q_t = \sigma_v' N_q A_t$$

$$= 1413 \times 18 \times \frac{\pi\times 1^2}{4} = 19{,}965 \text{ lb } (88.8 \text{ kN})$$

The allowable compression capacity is

$$Q_{\text{all}} = \frac{Q_{\text{ult}}}{FS} = \frac{129{,}871 + 19{,}965}{3} = 49{,}945 \text{ lb} \approx 50 \text{ kips (222 kN)}$$

4C.3.2.2 Piles in Cohesive Soil

The skin resistance is due to adhesion of the cohesive soil to the pile shaft. The following expression can be used to estimate the average skin resistance stress:

$$f_s = \alpha c \tag{4C.8}$$

where c = undrained shear strength of soil from a Q test
α = adhesion factor (see Fig. 4C.8 for values of a in terms of undrained shear strength)

An alternate method proposed by Semple and Rigden (1984) is especially applicable for long piles. It consists of obtaining the adhesion factor α as the product of two factors α_1 and α_2. These two factors can be obtained using Fig. 4C.9.

The tip-bearing stress q is obtained from the expression

$$q = 9c \tag{4C.9}$$

To develop such tip-bearing stress the required pile movement may have to be larger than that necessary to mobilize skin resistance. The pile tension capacity in cohesive soil can be calculated using only the shaft resistance due to skin friction.

Example 4C.2 Find the allowable compression capacity of a 12-in-diameter (305 mm) reinforced concrete pile with a total length of 45 ft (13.7 m) driven in clay layers as shown in Fig. 4C.X.2. Use a factor of safety of 2.5.

Solution The shaft resistance due to skin friction from level (0.00) to (–10.00) [3 m] is

$$1 \times 400\left[\pi\left(\frac{12}{12}\right)10\right] = 12{,}560 \text{ lb (55.9 kN)}$$

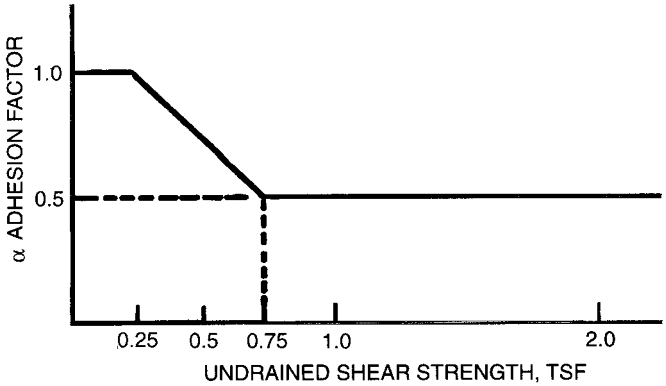


FIGURE 4C.8 Values of α versus undrained shear strength. (*From ASCE, 1993.*)

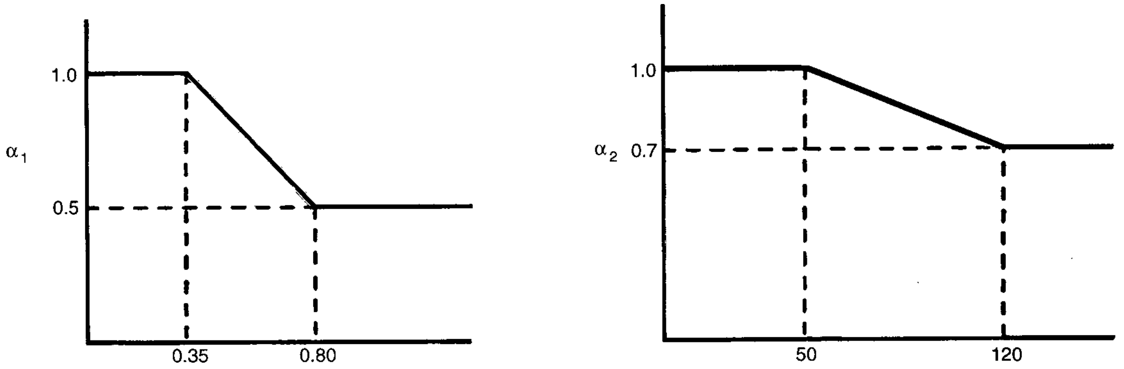


FIGURE 4C.9 Values of α_1 and α_2 applicable for long piles. (*From Semple and Rigden. 1984.*)

The shaft resistance due to skin friction from level (–10.00) [–3 mm] to (–15.00) [–4.5 m] is

$$0.95 \times 600\left[\pi\left(\frac{12}{12}\right)5\right] = 8949 \text{ lb } (39.8 \text{ kN})$$

The shaft resistance due to skin friction from level (–15.00) [–4.5 m] to (–30.00) [–9 m] is

$$0.9 \times 700\left[\pi\left(\frac{12}{12}\right)15\right] = 29{,}673 \text{ lb } (131.98 \text{ kN})$$

The shaft resistance due to skin friction from level (–30.00) [–9 m] to (–45.00) [–13.7 m is

$$0.85 \times 800\left[\pi\left(\frac{12}{12}\right)15\right] = 32{,}028 \text{ lb } (143.5 \text{ kN})$$

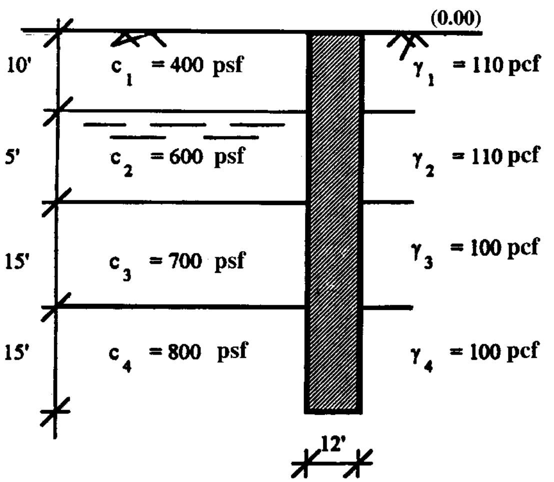


FIGURE 4C.X.2

The total shaft resistance due to skin friction is

$$Q_s = 12{,}560 + 8949 + 29{,}673 + 32{,}028 = 83{,}210 \text{ lb (370.1 kN)}$$

The tip-bearing resistance is

$$Q_r = 9c\frac{\pi(d)^2}{4} = 9 \times 800\frac{\pi(1)^2}{4} = 5652 \text{ lb (25.14 kN)}$$

The allowable compression capacity is

$$Q_{\text{all}} = \frac{Q_{\text{ult}}}{FS} = \frac{83{,}210 + 5652}{2.5} = 35{,}545 \text{ lb} \approx 36 \text{ kips (158.1 kN)}$$

4C.3.2.3 Piles in Silt

The skin resistance of piles in silt is generated from two sources—friction along the pile shaft and adhesion of soil to the pile shaft. The portion of the friction resistance increases linearly up to the critical depth d_c, below which the frictional resistance remains constant. In design, the critical depth is assumed as $10B$ for loose silts, $15B$ for medium silts, and $20B$ for dense silts, where B is the pile diameter or width. The portion of the adhesion is controlled by the undrained shear strength of the soil. The combined average skin resistance stress f_s, can be determined using the equation

$$f_s = K\sigma_v' \tan \delta + \alpha c \tag{4C.10}$$

where all variables are as defined in Secs. 4C.3.2.1 and 4C.3.2.2.

The tip-bearing stress q can be calculated from Eq. (4C.7). The pile tension capacity in silt soil is obtained by excluding the tip-bearing stress calculation and including only the effect of the skin resistance stress f_s along the pile shaft.

Example 4C.3 Determine the allowable compression capacity of a 12-in-diameter (304 mm) reinforced concrete pile with a total length of 50 ft (15 m) driven in medium dense silt. The K and δ values are found to be 1.0 and 1.0ϕ, respectively. The soil angle of friction ϕ is 20°, and its undrained shear strength c is 200 psf (9.5 kPa). The soil profile is shown in Fig. 4C.X.3. Use a factor of safety of 3.0.

Solution The critical depth is

$$dc = 15B = 15\left(\frac{12}{12}\right) = 15 \text{ ft (4.57 m)}$$

The effective overburden pressure σ_v' at water table level is

$$\sigma'_{v(-b)} = 110 \times 10 = 1100 \text{ psf (52.67 kPa)}$$

The effective overburden pressure σ_v' at the critical depth d_c is

$$\sigma'_{v(-15)} = 1100 + 5(110 - 62.4) = 1334 \text{ psf (63.87 kPa)}$$

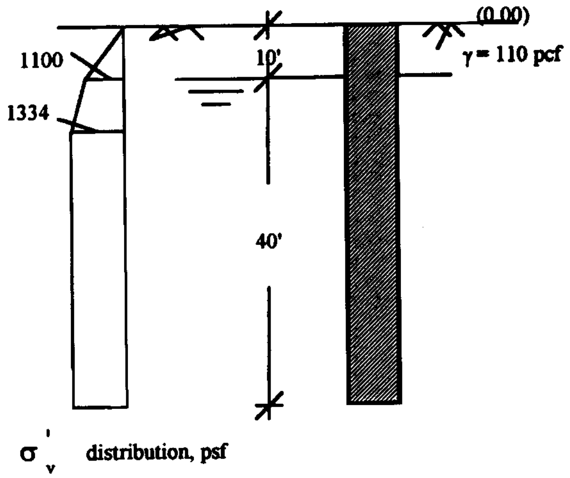


FIGURE 4C.X.3

The shaft resistance due to skin friction from level (0.00) to (–10.00) [3 m] is

$$\left[\left(\frac{1100+0}{2}\right)\tan 20° + 200\right]\left[\pi\left(\frac{12}{12}\right)10\right] = 12{,}566 \text{ lb } (55.89 \text{ kN})$$

The shaft resistance due to skin friction from level (–10.00) [–3 m] to (–15.00)[–4.57 m] is

$$\left[\left(\frac{1100+1334}{2}\right)\tan 20° + 200\right]\left[\pi\left(\frac{12}{12}\right)5\right] = 10{,}095 \text{ lb } (44.9 \text{ kN})$$

The shaft resistance due to skin friction from level (–15.00) [–4.57 m] to (–50.00) [–15 m] is

$$[1334(\tan 20°) + 200]\left[\pi\left(\frac{12}{12}\right)35\right] = 75{,}340 \text{ lb } (335.1 \text{ kN})$$

The total shaft resistance due to skin friction is

$$Q_t = 12{,}566 + 10{,}095 + 75{,}340 = 98{,}001 \text{ lb } (435.9 \text{ kN})$$

The tip-bearing resistance is, using $N_q = 8$,

$$Q_t = \sigma_v' N_q A_t$$

$$= 1334 \times 8 \frac{\pi(1)^2}{4} = 8378 \text{ lb } (37.26 \text{ kN})$$

The allowable compression capacity is

$$Q_{\text{all}} = \frac{Q_{\text{ult}}}{FS} = \frac{98{,}001 + 8378}{3} = 35{,}460 \text{ lb} \approx 35 \text{ kips } (157.7 \text{ kN})$$

4C.3.3 Pile Group Capacity

In actual construction applications it is rare to encounter a foundation consisting of a single pile. Rather, a group of piles is used to transmit the load of the superstructure to the soil mass. Figure 4C.10 presents some typical pile groupings. When several piles are placed with a distance s from each other, it is expected that both the skin resistance stress and the tip-bearing stresses developed in the soil will overlap. Due to this overlap it is reasonable to expect that the pile group capacity could be less than the sum of the individual pile capacities. The pile group efficiency η is the ratio of the pile group capacity to the sum of the individual pile capacities. Several equations have been proposed to determine the numerical value of η. Of these equations, the Converse-Labarre equation seems to be one of the most accepted. According to this equation, the ratio η is equal to

$$\eta = 1 - \frac{\theta}{90°} \frac{(n-1)m + (m-1)n}{mn} \tag{4C.11}$$

where m = number of rows
n = number of piles in a row
θ = arctan (d/s)
d = pile diameter
s = center-to-center spacing of piles

The drawback of using the η equation is neglecting the beneficial effects of the pile-driving operations on increasing the relative density and the friction angle of the sand. Another approach to determine the pile group capacity was presented by Terzaghi and Peck (1967). They assumed the group of piles with the enclosing soil to form a rigid pier which behaves as a unit (see Fig. 4C.11). The bearing capacity of the pier is calculated as the sum of the tip-bearing resistance of the area $a \times b$ and the skin resistance on the perimeter of the pier. Thus,

$$Q_{\text{ult(group)}} = f_s[2(a \times b)L] + q(a \times b) \tag{4C.12}$$

where f_s and q are obtained as presented in Sec. 4C.3.2 for different soil types.

For design purposes the pile group capacity of driven piles in sand not underlain by a weak layer is to be taken as the sum of the single-pile capacities. For other conditions the pile group capacity is the least of either the sum of the single-pile capacities or the group capacity as determined from Eq. (4C.12). The pile spacing is generally taken not less than three times the pile diameter on centers for bearing piles, and a minimum of three to five times the pile diameter on centers for friction piles, depending on the characteristics of the soil and the piles.

4C.4 PILE DYNAMICS

4C.4.1 Dynamics Equations for Pile Capacity

It is well known that the pile load capacity is affected by the method of installation. There are several installation methods based on dynamic processes such as vibration and ramming. In order to estimate the capacity of the pile while it is being driven at the site, many driving formulas have been proposed. These formulas are all based on the following relation:

$$\text{Energy input} = \text{energy used} + \text{energy lost} \tag{4C.13}$$

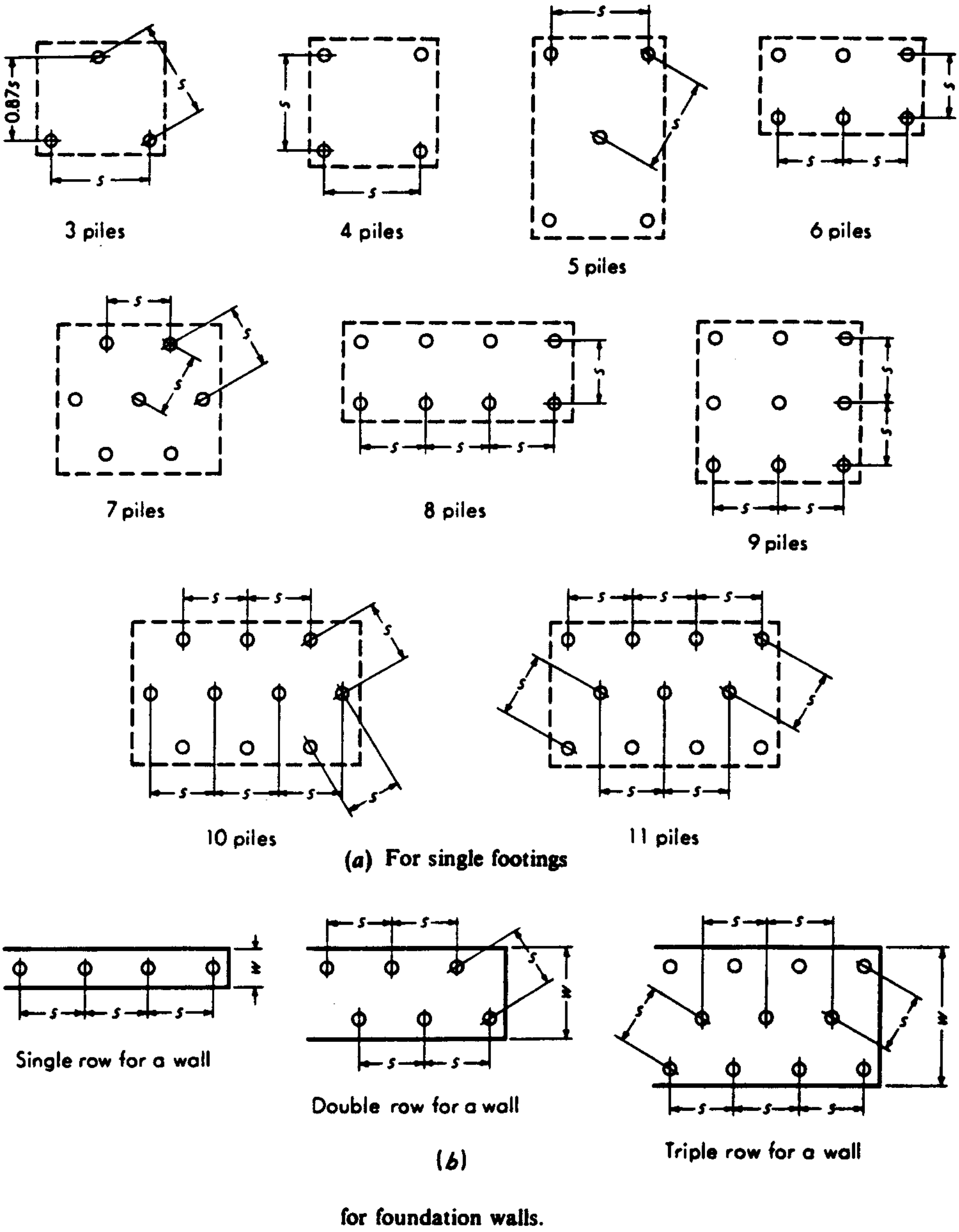


FIGURE 4C.10 Typical pile group patterns. (*a*) Single footings. (*b*) Foundation walls. (*From Bowles, 1982*.)

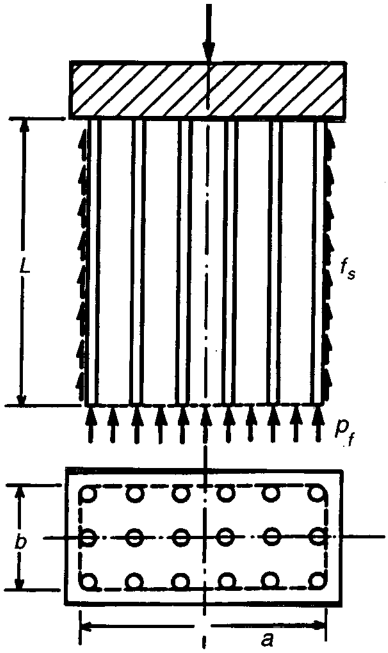


FIGURE 4C.11 Load capacity of a pile group. (*From Terzaghi and Peck, 1967.*)

Unfortunately these equations are not consistently reliable. The main reasons for this are:

1. The pile elastic compression is calculated using a static approach.
2. A portion of the input energy is used in displacing the soil laterally.
3. The total resistance is assumed acting on the pile tip.
4. The effects of driving velocity and duration of intermissions on the dynamic penetration are neglected.

In spite of these limitations, application of the driving formula could be beneficial to compare the dynamic resistance of piles driven in a site. This would give the engineer a way to judge the uniformity of the site subsoil. The most commonly used driving formulas are the *Engineering News* formula,

$$R = \frac{1.25 e_h E_h}{S + 0.1} \frac{W_r + (n \cdot n) W_p}{W_r + W_p} \tag{4C.14}$$

and the Danish formula

$$R = \frac{e_h E_h}{S + C_1} \tag{4C.15}$$

where R = load capacity of pile (just after driving)
e_h = hammer efficiency
E_h = hammer energy rating
S = amount of point penetration per blow

W_r = weight of ram
W_p = weight of pile
n = coefficient of restitution
$C_1 = (e_h E_h / 2AE)^{1/2}$ with AE and L being the pile cross section, modulus of elasticity, and length.

Currently the best means for estimating the pile capacity dynamically consists of recording the pile-driving history and then load testing the pile. It would be a reasonable assumption to expect that piles with similar driving history will develop the same load capacity.

4C.4.2 Dynamics Considerations

The vibration attenuation and the liquefaction potential due to construction dynamic loading should be investigated. The U.S. Army Corps of Engineers requires the removal or densification of liquefiable soil (ASCE, 1993). Also the first few natural frequencies of the structure-foundation assemblage should be determined and compared to the construction operations frequencies in order to avoid resonance.

4C.5 PILE LOAD TEST

As mentioned previously, load testing a pile is considered the most dependable way of determining its carrying capacity. The pile load test consists of applying a series of increasing load values and measuring the corresponding settlements to obtain a pile load-settlement curve (Fig. 4C.12). Many empirical methods have been proposed to determine the pile capacity from the pile load-settlement data. Table 4C.7 includes a list of most of these methods. The methods used by the U.S. Army Corps of Engineers consists of taking the average of three load values obtained from the load test data as the pile load capacity. These three load values are

1. The load causing a movement of 0.25 in on the net settlement curve
2. The load corresponding to the point on the curve with a significant change in slope
3. The load matching the point on the curve that has a slope of 0.01 in per ton

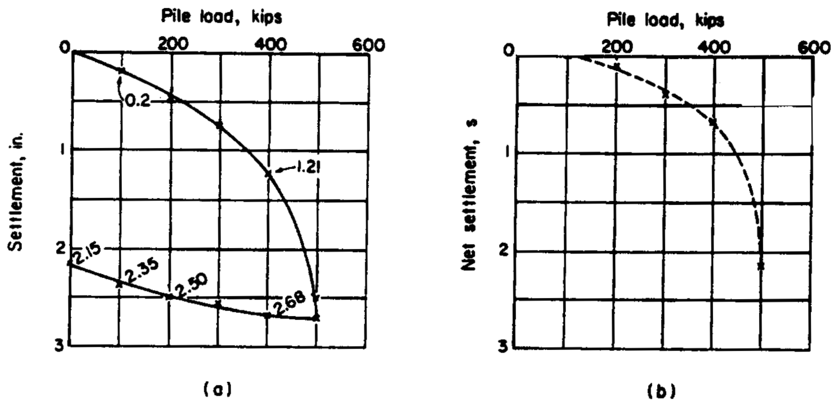


FIGURE 4C.12 Typical pile load test data. (a) Total settlement curve. (b) Net settlement curve.

TABLE 4C.7 Methods of Pile Load Test Interpretation

1. Limiting total butt settlement
   a. 1.0 in (Holland)
   b. 10% of tip diameter (United Kingdom)
   c. Elastic settlement+D/30 (Canada)
2. Limiting plastic settlement
   a. 0.25 in (AASHTO, N.Y. State, Louisiana)
   b. 0.5 in (Boston) [complete relaxation of pile assumed]
3. Limiting ratio: plastic/elastic settlement 1.5 (Christiani and Nielson of Denmark)
4. Limiting ratio: settlement/unit load

a. Total	0.01 in/ton (California, Chicago)
b. Incremental	0.03 in/ton (Ohio)
	0.05 in/ton (Raymond International)

5. Limiting ratio: plastic settlement/unit load

a. Total	0.01 in/ton (N.Y. City)
b. Incremental	0.003 in/ton (Raymond International)

6. Load-settlement curve interpretation
   a. Maximum curvature: Plot log total settlement versus log load; choose point of maximum curvature.
   b. Tangents: Plot tangents to general slopes of upper and lower portions of curves; observe point of intersection.
   c. Break point: Observe point at which plastic settlement curve breaks sharply; observe point at which gross settlement curve breaks sharply (Los Angeles).
7. Plunge Find loading at which pile "plunges" (i.e., the load increment could not be maintained after pile penetration was greater than 0.2B).
8. Texas quick load Construct tangent to initial slope of load versus gross settlement curve; construct tangent to lower portion of the load versus gross settlement curve at 0.05 in/ton slope. The intersection of the two tangent lines is the ultimate bearing capacity.

Source: ASCE, 1993.

4C.6 NEGATIVE SKIN FRICTION

In some cases, piles are driven into compressible soil before its consolidation is complete. If a fill is placed on this compressible soil, the soil will move downward against the pile. This relative movement creates a skin friction between the pile and the moving soil that increases the axial load acting on the pile. This mechanism is known as negative skin friction (Fig 4C.13). In excessive soil consolidation cases a gap may form between the bottom of the pile cap and the fill. This results in the full cap weight being transferred directly to the piles and could alternate the stresses in the pile cap. The value of the negative skin friction can be computed as follows:

$$Q_{NF} = cLP' \tag{4C.16}$$

where c = shear strength of soil
L = length of pile in contact with compressible layers
P' = perimeter of pile

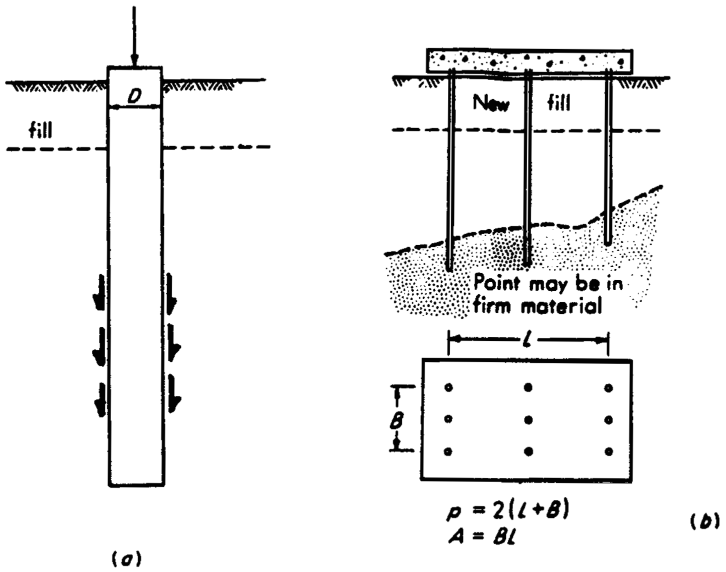


FIGURE 4C.13 Negative skin friction. (*a*) Single pile. (*b*) Pile-group effect.

Thus the total applied load on the pile becomes

$$Q_{\mathrm{TOTAL}} = Q + Q_{\mathrm{NF}} \tag{4C.17}$$

where Q is the load transmitted from the superstructure.

If piles are spaced a small distance apart, the developed negative skin friction may also be represented as the dragging force acting on the perimeter of the pier formed by the group of piles and the enclosed soil. In these situations, two modes of negative skin friction require investigation:

1. $$Q_{\mathrm{NF}} = N(Q_{\mathrm{NF}}/\mathrm{pile}) \tag{4C.18a}$$

where Q_{NF}/pile is as given as by Eq. (4C.16) and N is the number of piles.

2. $$Q_{\mathrm{NF}} = AL\gamma + cLP \tag{4C.18b}$$

where A = area bounded by pile group
L = length of pile in contact with compressible layers
γ = unit weight of compressible layer
c = shear strength of soil
P = perimeter of area A.

In the presence of expansive soils the negative skin friction phenomenon can generate upward tension stresses in the pile. These stresses could be larger if no or an insufficient gap is left between the expansive soil and the bottom of the pile cap.

4C.7 LATERALLY LOADED PILES

Piles are slender vertical members that have only limited capability to resist nonvertical loads. Therefore batter piles are used to resist large inclined or horizontal loads when acting on a structure. Beresantsev et al. (1961) suggested the following practice to transmit inclined loads in terms of the angle α, where α is the angle of the force to be transmitted with the vertical (Fig. 4C.14):

1. Vertical piles $\alpha < 5°$
2. Batter piles $5° \leq \alpha < 15°$
3. Deadman $\alpha < 15°$

Brooms (1965) presented charts that give the limit lateral load to act on a vertical pile versus the ratio of the pile embedment length to its diameter. These diagrams, applied to short piles, are presented in Fig. 4C.15 for cohesive and cohesionless soils. The term "short piles" refers to rigid piles where the lateral capacity is dependent mainly on the soil resistance. Long piles are those whose lateral capacity is primarily dependent on the yield moment of the pile itself. Figure 4C.16 shows the relationship between the limit lateral load and the yield bending moment of the pile for cohesive and cohesionless soils. In these figures the dashed lines represent the case of a fixed pile head, whereas the full lines indicate different e/l ratios, where e is the height of the line of action of the force P above ground surface and l is the length of pile in the ground.

The symbols used in these figures are

d = diameter of pile
γ = soil bulk unit weight
k_p = coefficient of passive earth pressure
H_u = limit value of horizontal load
c_u = undrained shear strength
L = pile embedment length

Very few test results are available for inclined forces acting on vertical piles or on a batter pile. Petrasovits and Awad (1968) conducted model tests on piles having a length of 50cm and a diameter of 1.3 to 3.5cm embedded in a soil with an angle of internal friction $\phi = 37.5°$. Figure 4C.17 gives

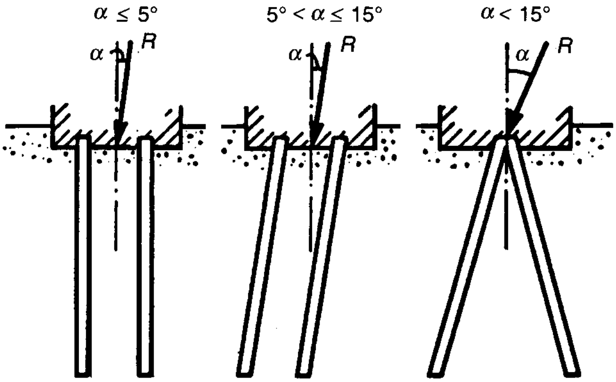


FIGURE 4C.14 Recommended practice to transmit inclined loads to soil mass. (*From Beresantsev et al.*, 1961.)

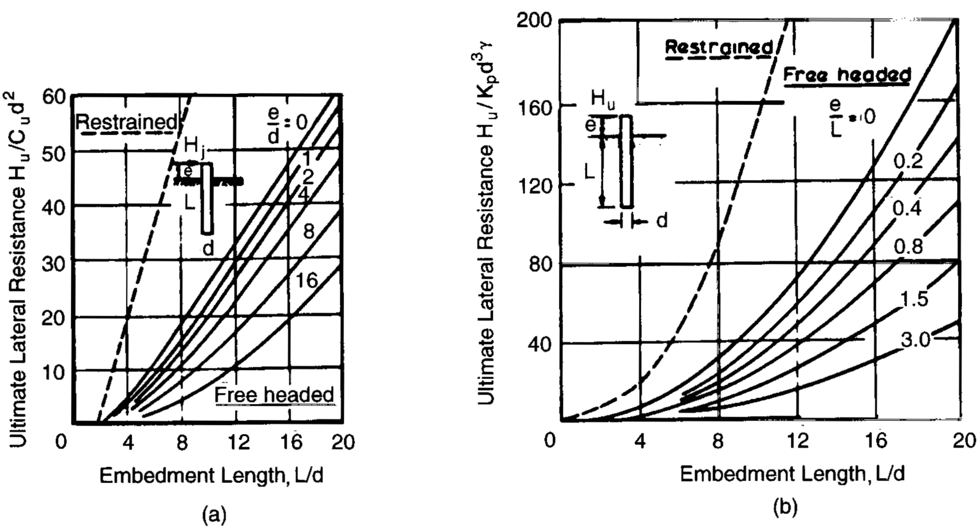


FIGURE 4C.15 Ultimate lateral resistance of short piles. (a) Cohesive soils. (b) Cohesionless soils. (*From Brooms, 1965.*)

the percentage of increase for the applied load for different cases when the inclination angle β of the pile is varied.

4C.8 PILE CAP DESIGN

Pile caps are used to distribute the loads and moments acting on the column to all of the piles in the group. The pile cap is usually made of reinforced concrete and rests directly on the ground, except when the soil underneath is expansive. The design considers the effects of a number of concentrated reactions due to the column load, surcharge load, fill weight, and pile cap weight. For the design of a rigid pile cap it is usual to assume that each pile carries an equal amount of concentric axial load and that a planar stress distribution is valid for nonconcentric loading. This assumption is justified when the pile cap is resting on the ground, the piles are vertical, the load is applied at the center of the pile group, and the pile group is symmetrical. The structural design of a reinforced concrete pile cap requires consideration of the following critical conditions:

1. Punching shear failure at sections located at a distance $d/2$ from the face of the column and around each pile
2. Beam shear failure at sections at a distance d from the face of the column
3. Bending failure at the sections located at the face of the column

where d is the effective depth at which the steel layer is placed.

The rules followed during the design are essentially the same as the ones used for spread footings. When deciding on the piles causing shear, attention is drawn to Chap. 15 of ACI 318 (1989), which states that

> Computation of shear on any section through a footing supported on piles shall be in accordance with the following: a) Entire reaction from any pile whose center is located $d_p/2$ (d_p is the pile diameter at the up-

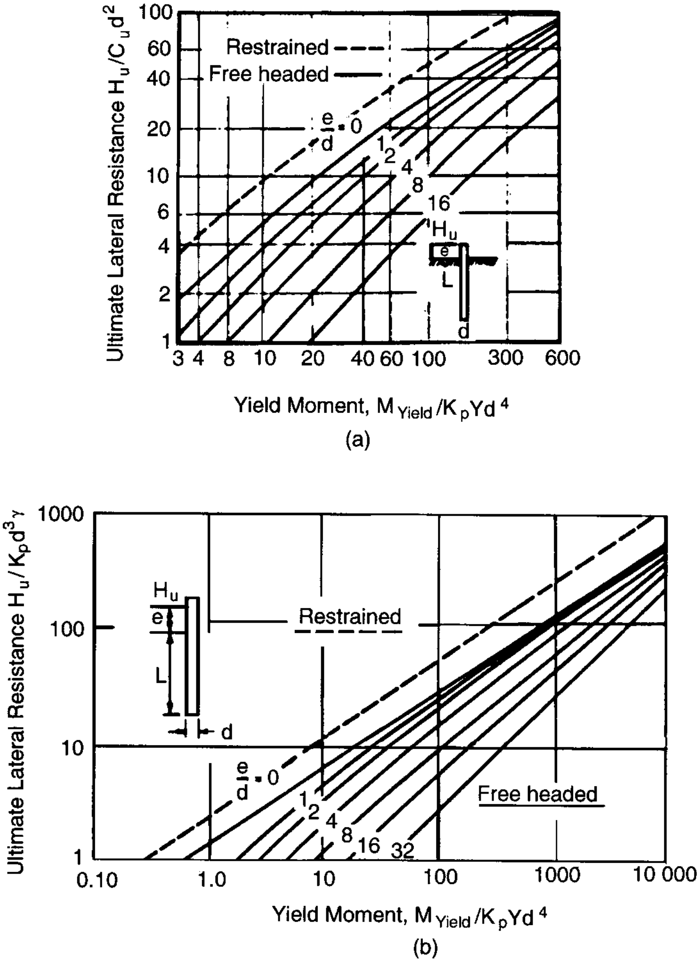


FIGURE 4C.16 Ultimate lateral resistance of long piles. (*a*) Cohesive soils. (*b*) Cohesionless soils. (*From Brooms, 1965.*)

per end) or more outside the section shall be assumed as producing shear on the section, b) reaction from any pile whose center is located $d_p/2$ or more inside the section shall be assumed as producing no shear on the section, c) for intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line interpolation between full value at $d_p/2$ outside the section and zero at $d_p/2$ inside the section.

The designer is urged to keep the pile cap design on the safe side because the individual actual pile reaction may differ from the value used in design due to group action and possible differences between layout on drawings and driven piles.

Example 4C.4 A 28-in-square (7 10-mm^2) column carries the following loads: P_D = 500 kips (2224 kN), P_L = 700 kips (3114 kN), M_D = 200 ft · kips (271 kN · m), and M_L = 300 ft · kips (406.8 kN · m). The column

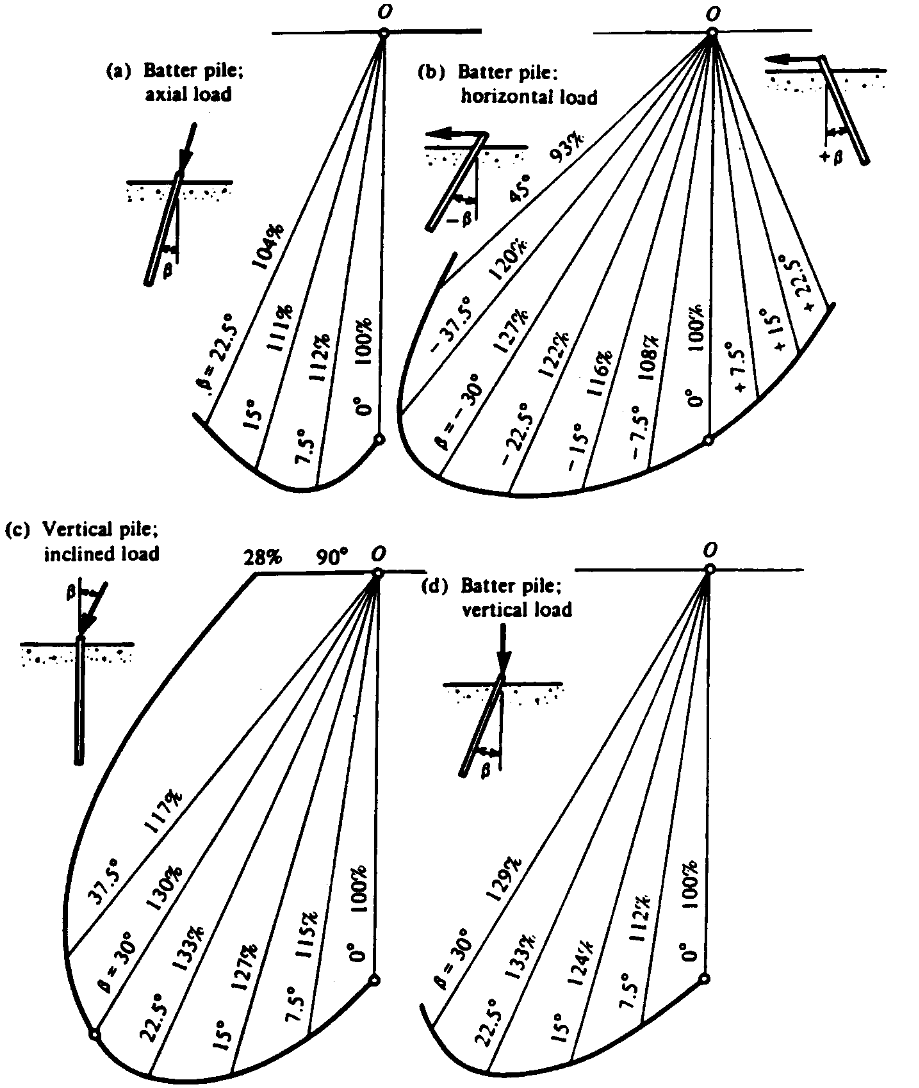


FIGURE 4C.17 Relative bearing capacity for batter piles or vertical piles subjected to inclined forces. (*From Petrasovits and Awad. 1968.*)

is to rest on a 4-ft-thick (1.2-m) cap supported by piles having an allowable load capacity of 100 kips (445 kN) and a diameter of 12 in (304 mm). The cap is topped with 12 in (304 mm) of fill having a unit weight of 120 lb/ft^3 (1922 kg/m^3) and 6-in concrete (152-mm) slab carrying a surcharge load of 100 psf (4788 Pa) (see Fig. 4C.X.4a). Design the pile cap using $f'_c = 4000$ psi (27.6 MPa) and $f_y = 60{,}000$ psi (414 MPa).

Solution The total vertical load is

$$P_{\text{total}} = 500 + 700 = 1200 \text{ kips } (5338 \text{ kN})$$

To account for the bending effects as well as the surcharge and cap weight, choose a total number of 15 piles spaced at 3 ft on centers (Fig. 4C.X.4*b*).

The surcharge load per pile is

$$P_s/\text{pile} = 3^2(0.5 \times 150 + 100 + 1 \times 120 + 4 \times 150) = 8055 \text{ lb} \approx 8.1 \text{ kips (36 kN)}$$

$$\Sigma x^2 = 3(3^2 + 6^2 + 3^2 + 6^2)144 = 38{,}880 \text{ in}^2 \ (250 \times 10^3 \text{ cm}^2)$$

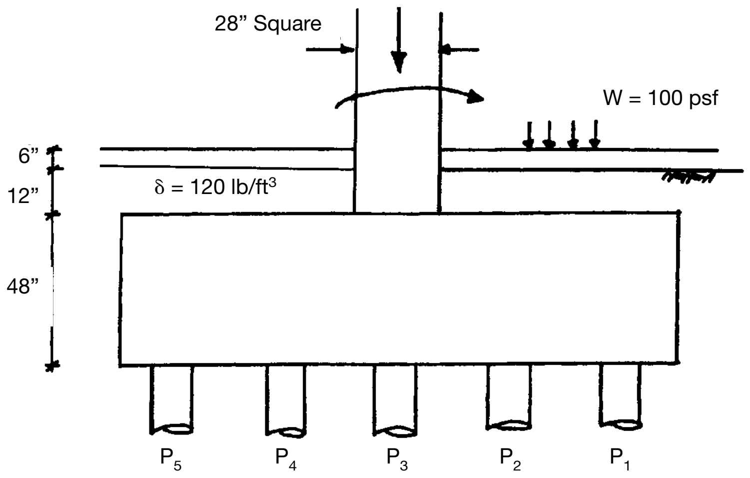


FIGURE 4C.X.4.*a*

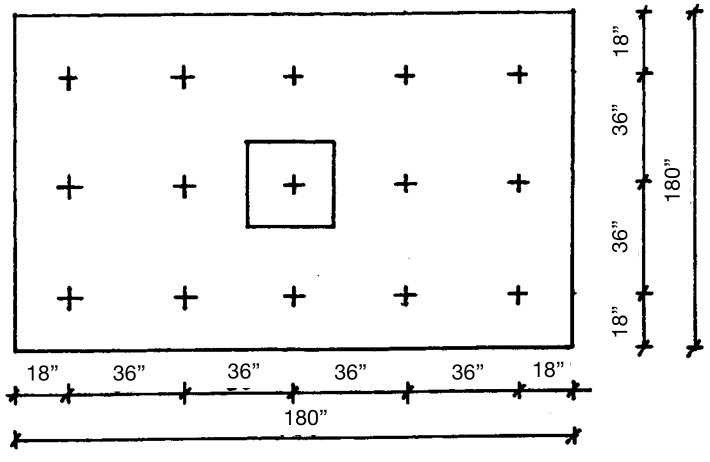


FIGURE 4C.X.4*b*

The net allowable capacity of a pile is

$$P_{net} = 100 - 8.1 = 91.9 \text{ kips } (408.8 \text{ kN})$$

The maximum axial force of a pile is

$$\frac{1200}{15} + \frac{500 \times 12 \times 72}{38{,}880} = 91.1 \text{ kips } (405.2 \text{ kN}) < 91.9 \text{ kips } (408.8 \text{ kN}) \qquad \text{O.K.}$$

The factored load on piles p_1 through p_5 is

$$P_1 = \frac{500 \times 1.4 + 700 \times 1.7}{1.5} + \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)72}{38{,}880} = 143.6 \text{ kips } (638.7 \text{ kN})$$

$$P_2 = \frac{500 \times 1.4 + 700 \times 1.7}{15} + \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)36}{38{,}880} = 134.8 \text{ kips } (599.6 \text{ kN})$$

$$P_3 = \frac{500 \times 1.4 + 700 \times 1.7}{15} = 126 \text{ kips } (560.41 \text{ kN})$$

$$P_4 = \frac{500 \times 1.4 + 700 \times 1.7}{15} - \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)36}{38{,}880} = 117.2 \text{ kips } (521.3 \text{ kN})$$

$$P_5 = \frac{500 \times 1.4 + 700 \times 1.7}{15} - \frac{(200 \times 12 \times 1.4 + 300 \times 12 \times 1.7)72}{38{,}880} = 108.4 \text{ kips } (482.16 \text{ kN})$$

Pile punching shear check:

$$V_u = 143.6 \text{ kips } (639 \text{ kN})$$

$$V_c = 4\sqrt{f'_c}\,b_0 d = \frac{4\sqrt{4000} \times \pi \times 3\ 6 \times 44)}{1000} = 1258 \text{ kips}$$

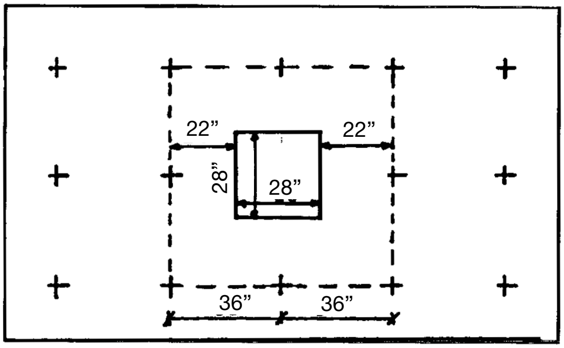
$$\frac{V_u}{\phi} = \frac{143.6}{0.85} = 169 < 1258 \text{ kips } (752 < 5596 \text{ kN}) \qquad \text{O.K.}$$

Two-way shear check (using a conservative approach for computation of applied shear):

$$V_u = 3 \times 143.6 + 3 \times 134.8 + 2 \times 126 + 3 \times 117.2 + 3 \times 108.4 = 1764 \text{ kips } (7846 \text{ kN})$$

$$V_c = 4\sqrt{f'_c}\,b_0 d = \frac{4\sqrt{4000} \times 72 \times 4 \times 44}{1000} = 3206 \text{ kips } (14{,}260 \text{ kN})$$

$$\frac{V_u}{\phi} = \frac{1764}{0.85} = 2075 < 3206 \text{ kips } (9229 < 14{,}260 \text{ kN}) \qquad \text{O.K.}$$

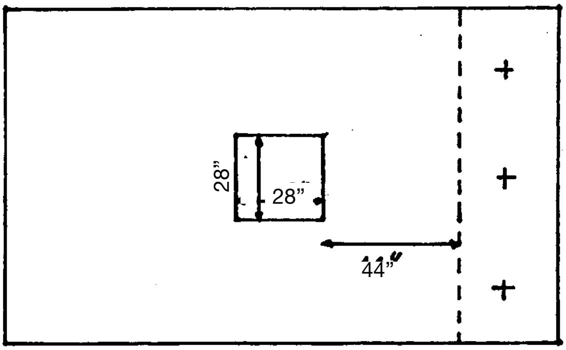


Beam shear check:

$$V_u = 3 \times 143.6 = 430.8 \text{ kips (1916 kN)}$$

$$V_c = 2\sqrt{f'_c}\,b_0 d = \frac{2\sqrt{4000} \times 108 \times 44}{1000} = 601 \text{ kips (2673 kN)}$$

$$\frac{V_u}{\phi} = \frac{430.8}{0.85} = 507 < 601 \text{ kips} \qquad \text{O.K.}$$



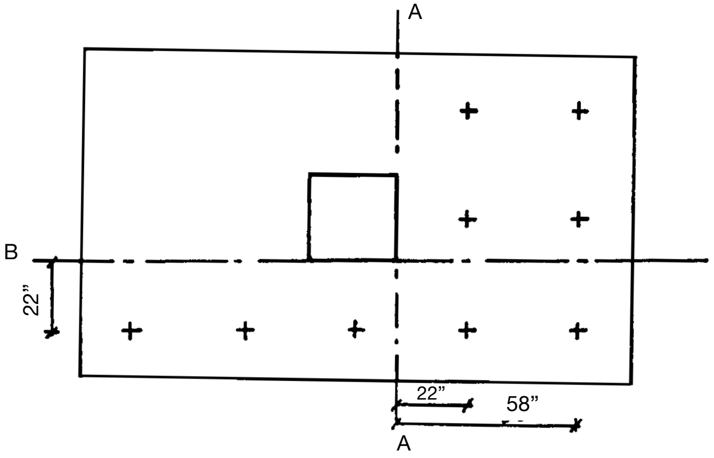
Flexure bending design: Check Sec. 3B.

1. Section *A-A:*

$$M_u = (3 \times 143.6 \times 1000 \times 58 + 3 \times 134.8 \times 1000 \times 22 = 33.9 \times 10^6 \text{ lb} \cdot \text{in (3831 kN} \cdot \text{m)}$$

$$A_s = \frac{M_u}{\phi f_y(0.9d)} = \frac{33.9}{0.9 \times 60{,}000} \times 0.9 \times 44 \times 10^6 = 15.85 \text{ in}^2 \text{ (102.2 cm}^2\text{)}$$

$$(A_s)_{\min} = \left(\frac{200}{f_y}\right)bd = 0.0033 \times 108 \times 44 = 15.68 \text{ in}^2 \text{ (101.2 cm}^2\text{)}$$



Choose A, = 15.85 in^2 (102.2 cm^2)

$$a = \frac{15.85 \times 60{,}000}{0.85 \times 4000 \times 108} = 2.59 \text{ in } (65.8 \text{ mm})$$

$$M_n = 15.85 \times 60{,}000\left(44 - \frac{2.59}{2}\right) = 40.6 \times 10^6 > \frac{33.9 \times 10^6}{0.9} = 37.7 \times 10^6 \text{ 1b} \cdot \text{in } (4260 \text{ kN} \cdot \text{m})$$

Choose A_s = 15.85 in^2 (101.2 cm^2) in the x direction.

2. Section *B-B:*

$$M_u = (143.6 + 134.8 + 126 + 117.2 + 108.4)1000 \times 22 = 11 \times 10^6 \text{ lb} \cdot \text{in } (1243 \text{ kN} \cdot \text{m})$$

$$A_s = \frac{M_u}{\phi f_y(0.9d)} = \frac{11}{0.9 \times 60{,}000 \times 0.9 \times 44} \times 10^6 = 5.18 \text{ in}^2 \ (33.42 \text{ cm}^2)$$

$$(A_s)_{\min} = \left(\frac{200}{f_y}\right)bd = 0.0033 \times 180 \times 44 = 26.1 \text{ in}^2 \ (168.39 \text{ cm}^2)$$

Choose A_s = 26.1 in^2 (168.39 cm^2) in the y direction.

The selection and distribution of the bars as well as the development length checks are performed as presented in Sec. 3B.

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