CHAPTER 6

PILE FOUNDATIONS

Section I. GROUP BEHAVIOR

6-1. Group action. Piles are most effective when combined in groups or clusters. Combining piles in a group complicates analysis since the characteristics of a single pile are no longer valid due to the interactions of the other group piles. The allowable load of a single pile will not be the same when that pile is combined in a cluster or in a group. There is no simple relationship between the characteristics of a single isolated pile and those of a group. Relationships depend on the size and other features of the group and on the nature and sequence of the soil strata. The ultimate bearing capacity of a group of piles is not necessarily equal to the ultimate bearing capacity of a single isolated pile multiplied by the number of piles in the group.

- Only in certain cases (for example where the group compacts the soil) will the ultimate bearing capacity of the group be greater than the number of piles times the ultimate bearing capacity.

- For end-bearing piles on rock or in compact sand or gravel with equally strong material beneath, the ultimate bearing capacity of the group will be essentially equal to the number of piles times the ultimate bearing capacity.

- For piles which rely on skin friction in a deep bed of cohesive material, the ultimate bearing capacity of a large group maybe substantially less than the number of piles times the ultimate bearing capacity.

6-2. Driving.

a. Effects on the soil. When piles are installed in groups, consideration should be given to their effects on the soil. Heave and lateral displacement of the soil should be limited by the choice of a suitable type of pile and by appropriate spacing. Some soil, particularly loose sands, will be compacted by displacement piles. Piles should be installed in a sequence which avoids creating a compacted block of ground into which additional piles cannot be driven. Similar driving difficulties may be experienced where a stiff clay or compacted sand and gravel have to be penetrated to reach the bearing stratum. This may be overcome by first driving the center piles of a group and working outwards, but it is frequently more convenient to begin at a selected edge and work across the group. In extreme cases, it may be necessary to predrill through a hard upper stratum. If the group is confined by sheet piling which has already been driven, it may be preferable to drive from the perimeter inward to avoid displacemer of the sheet piling.

b. Effects on adjacent structures. When piles are to be driven for a new foundation alongside an existing structure, care must be taken to insure that the existing structure is not damaged by the operation. Settlement or heave caused by pile driving may seriously damage the foundations of nearby structures. For example, piles driven behind a retaining wall can increase the pressure on the wall. This increase in pressure maybe caused by densfication of a granular soil by vibration, or a plastic soil may actually be forced against the wall. To avoid or minimize the effects of vibration, the pile may be driven in a predrilled hole or jetted or jacked into place. The jetting itself could have a detrimental effect upon the soil beneath an existing structure.

6-3. Spacing.

Piles should be spaced in relationship to the nature of the ground, their behavior in groups. and the overall cost of the foundation. The spacing should be chosen with regard to the resulting heave or compaction. Spacing should be wide enough for all piles installed to the correct penetration without damaging adjacent construction or the piles themselves. For piles founded on rock, the minimum center-to-center spacing is 2 times the average pile diameter, or 1.75 times the diagonal dimension of the pile cross section, but not less than 24 inches. An optimum spacing of 3 times the diameter of the pile is often used. This allows both adequate room for driving and economical design of the pile cap.

Section II. GROUND CONDITIONS

6-4. Rock.

Site investigation should establish whether the underlying rock surface is level, inclined, or irregular. It should also determine the thickness of decomposed rock which the pile should penetrate. If the surface is inclined, driven piles may have to be pointed. The upper load limit of a pointed pile embedded in sound rock may be the allowable compressive stress of the material in the pile. If the overlying material is saturated plastic clay, displacement piles and consequent volume changes may heave piles already driven.

6-5. Cohesionless soils.

a. Piles driven into dense sand. Piles are driven through the soft materials and into a dense, deep stratum of sand to develop adequate carrying capacity. If the sand is moderately loose, the required penetration may be deep. If the sand is dense, penetration may be only a few feet. Skin friction of compressible soil is not considered since it will disappear in a period of time. The entire load will then be carried by the firm stratum (figure 5-3).

(1) *Point resistance.* Point resistance can be found using calculations and laboratory tests (chapter 5, section IV). It can also be determined approximately by making a load test on two piles driven about 5 feet apart. One pile is driven to refusal in the firm bearing stratum while the other is driven until its point is 3 feet above the surface of the bearing stratum. If both piles are loaded at equal rates, the effect of time on skin friction can be eliminated. The point resistance is equal to the difference between the ultimate bearing capacities of the two piles.

(2) *Depth estimate.* The depth to which piles must extend into the sand can be estimated on the basis of driving tests combined with load tests or, in the case of small projects, calculations using dynamic or static formulas.

b. Compaction piles. Compaction piles densify the sand. The design load for compaction piles is conservative. The piles are driven to equal penetration with each hammer stroke. The hammer strokes wil1 be progressively shorter as work continues because the sand becomes more compacted by driving the preceding pile. Driving resistance increases as each pile is driven because of the compaction of the soil.

(1) *Driving loads.* On small jobs, loads of 20 tons are usually assigned to compaction piles of timber and 30 tons to precast concrete. The piles should be driven to the capacities indicated by the Engineering News formula (chapter 5, section III). On large jobs, a test group of several piles should be driven. The center pile should be driven first to a capacity indicated by the Engineering News formula. When the entire group of piles has been driven, the center pile should be redriven, and its capacity determined by the formula. The difference between the 2 computed capacities reflects the effects of densification. A load test on the center pile after redriving may be used to check the accuracy of the computed capacity.

(2) *Length.* The length of compaction piles decreases markedly with increasing *taper.* Piles from 20-ton to 30-ton capacity having a taper of 1 inch to 2 ½ feet can seldom be driven more than 25 feet in loose sands.

c. Piles for preventing scour. Scour, which results from currents, floods, or ship-propeller action, will significantly reduce the functional resistance of a pile. The bases of bridge piers located near river channels must be established below the level to which the river bottom is removed by scour during floods. In many cases, the depth of the river increases faster during floods than the crest rises. As bridges are located where the channel is narrow, the depth of scour is likely to be greater than average. Furthermore, the construction of the bridge usually causes additional constriction of the channel and itself increases the depth of scour. Depths of scour can be as much as 4 feet for each 1 foot of rise. For military construction, a reasonable design estimate is a depth of scour equal to 1 foot for each foot of rise of the water. Scour can be minimized by surrounding the pile foundation with sheet piles or providing riprap protection around the base of the pier (refer to TM 5-312).

d. Group behavior. The ultimate bearing capacity of pile groups in cohesionless soil is equal to the number of piles times the ultimate bearing capacity of an individual pile, provided the pile spacing is not less than three pile diameters. A pile group in cohesionless soil settles more than an individual pile under the same load (figure 6-l). Ordinarily, driving to a resistance of 20 tons for timber piles or 30 tons for concrete piles as determined by the Engineering News formula will insure that settlements are within tolerable limits. Piles driven into a thick bearing stratum of dense, cohesionless materials should not settle provided correct safety and engineering analysis have been followed.

e. Uplift resistance. The total uplift resistance of a pile group is the smaller of the following.

- The uplift resistance of a single pile times the number of piles in the group.
- The uplift capacity of the entire pile group as a block (figure 6-2), which is the

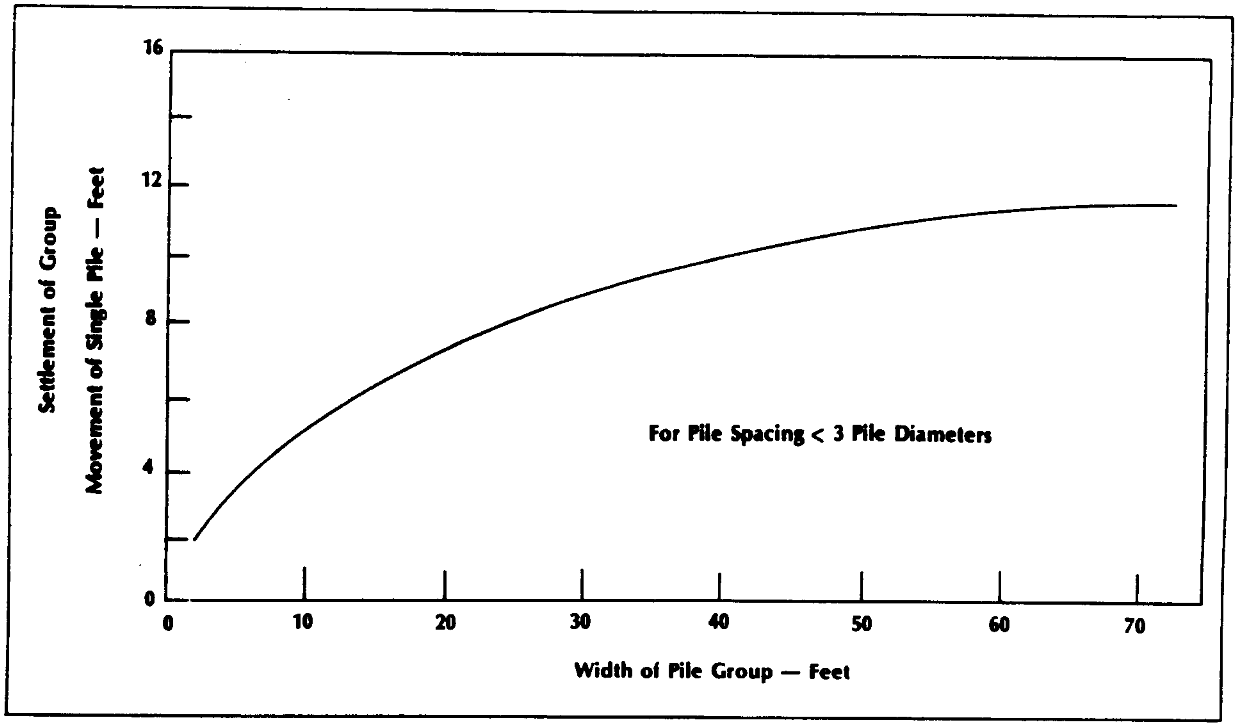


Figure 6-1 Estimated settlement of pile groups in sand

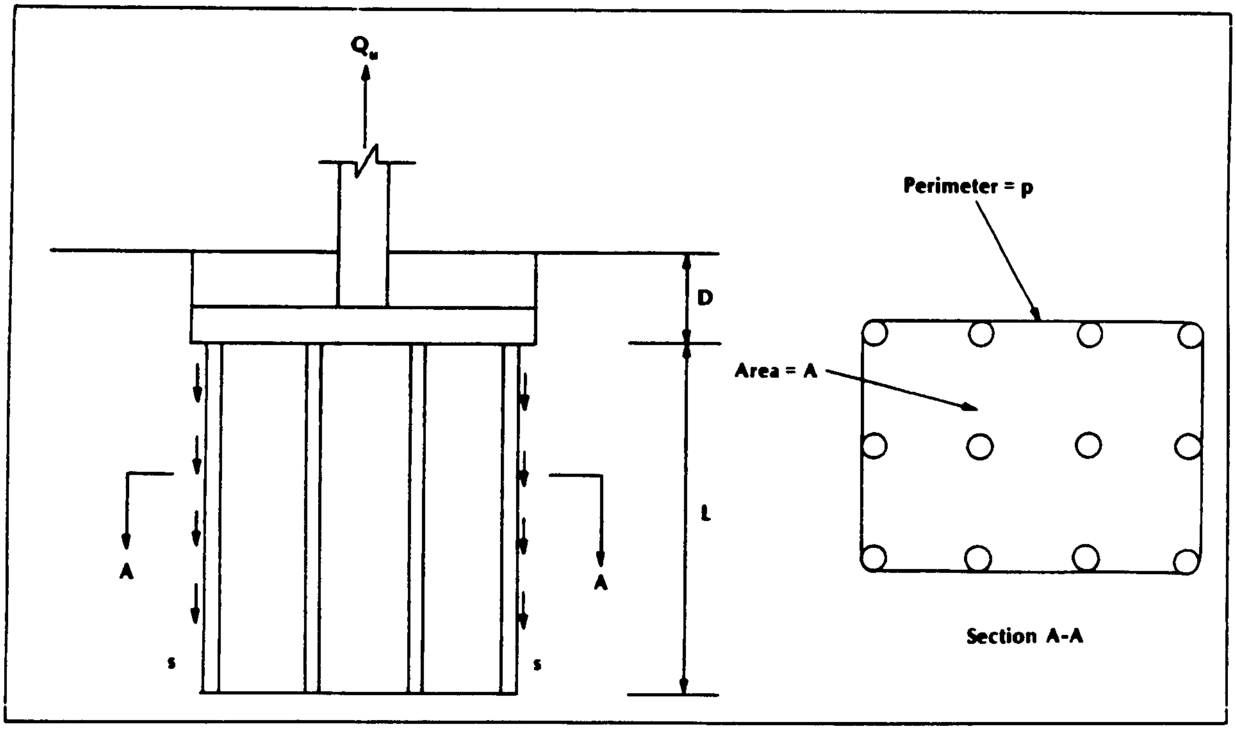
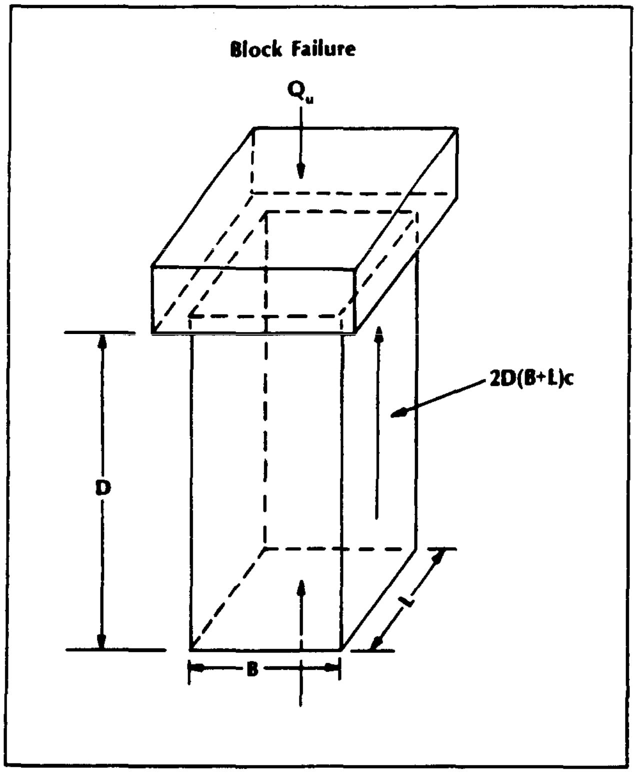


Figure 6-2 Uplift capacity of pile group



$Q_u = qA_p + cdP$

Q_u = Ultimate Bearing Capacity of Group

q = Unit Bearing Capacity

A_p = Plan Area of Block

c = Unit Frictional Resistance

d = Pile Embedment

p = Perimeter of Group

Figure 6-3 Block failure of piles in clay

sum of the weight of the pile cap, the weight of the block of soil (using buoyant weights below the water table), and the frictional resistance along the perimeter of the block.

f. Driving. The driving resistance of sands does not indicate the true resistance of the pile. If the sands are loose, pore pressures allow the piles to penetrate with little resistance. However, with time these pore pressures will dissipate; and redriving or subsequent load tests will indicate a greater soil resistance. If the materials are dense, initial driving may cause negative pore pressure, making driving hard. As these pressures dissipate, both resistance and load values lower. Redrive tests should be performed when excessively high or low driving resistances are encountered.

6-6. clay.

a. Group action. Piles driven in clay derive their capacity from friction. They are commonly driven in groups or clusters beneath individual footings or as single large groups beneath mats or rafts. The bearing capacity of a pile cluster maybe equal to the number of piles times the bearing capacity per pile, or it may be much smaller because of block failure (figure 6-3). The load on a group of piles may be sufficient to cause block failure. Block failure generally can be eliminated if the pile spacing is equal to or greater than three pile diameters.

b. Settlement. The need to limit settlement will govern design of piles in clay. Procedures for computing foundation settlement are presented in TM 5-545. Stress distribution

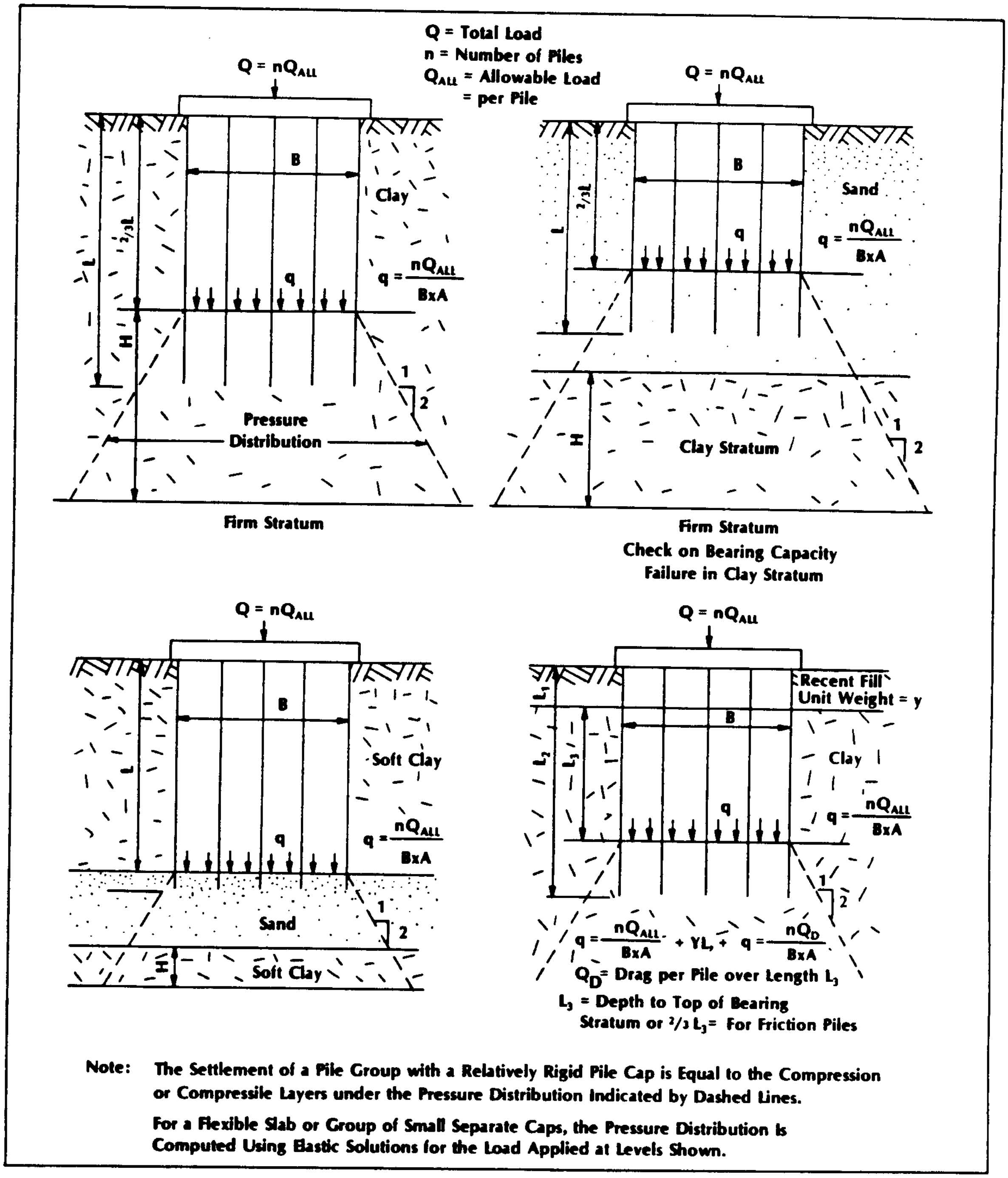


Figure 6-4 Approximate distribution of stress beneath pile foundations

requirements may be found by analyzing the settlement of pile groups (figure 6-4). The reduction in settlement provided by friction piles is generally small, and therefore alternate types of shallow foundations should be considered in lieu of friction piles.

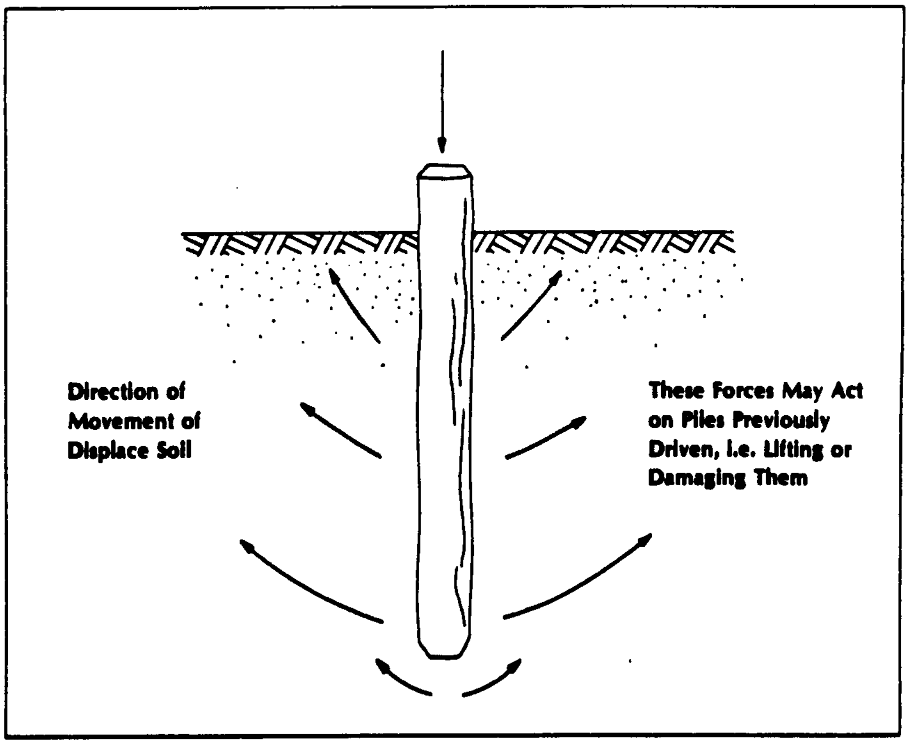


Figure 6-5 Pile action on the soil

Settlement of a group of friction piles will tend to increase as the number of piles in the group increases. Efficiency factors can be used to calculate how to reduce the allowable load to compensate for settlement. For piles spaced wider than three pile diameters, the reduced group capacity can be found by multiplying the sum of the individual capacities times the ultimate bearing capacity times an efficiency factor which varies from 0.7 for a spacing of three pile diameters to 1 for eight pile diameters. Alternatively, the pile groups are proportioned on the basis of computed settlements.

c. Uplift resistance. The resistance to uplift of pile groups in clay is governed by the same considerations that apply to uplift resistance of pile groups in sand.

d. Driving. Clay soils are relatively incompressible under the action of pile driving. Hence, a volume of soil eaual to that of the pile usually will be displaced (figure 6-5). This will cause ground heave between and around the piles.

- Driving a pile alongside those previously driven frequently will cause those already in place to heave upward.

- In the case of piles driven through a clay stratum to firm bearing beneath, the heave may be sufficient to destroy the contact between the tip of the pile and the firm stratum. This may be detected by taking level readings on the tops of piles previously placed. Raised piles should be redriven to firm bearing.

- The displacement of soil by the pile may cause sufficient lateral force to move previously driven piles out of line or damage the shells of cast-in-place concrete piles of the shell-less type. This problem may be solved by predrilling.

6-7. Negative friction (down drag).

a. Cohesive soils. After a pile is installed through a stratum of cohesive soil, the downward movement of the consolidating and overlying soils will cause a drag on the pile. The consolidation may be caused by the weight of the deposit, by the imposition of a surcharge such as a fill, or by remolding during pile installation. The downward drag may cause excessive settlement. Coating the pile with a bitumen compound will reduce drag. The magnitude of the drag per unit of area cannot exceed the undrained shearing strength of the compressible soil (table 5-l). The drag acts on the vertical surface area of the entire pile foundation. Methods of analysis for drag on piles in clay are illustrated in figure 6-6.

b. Sensitive clays. When piles are driven through sensitive clay, the resulting remolding may restart the consolidation process. The downward force due to negative friction may then be estimated by multiplying the cohesion of the remolded clay by the surface area of the pile shaft. Particular care should be given to the design of friction pile foundations if the soil is sensitive. In such circumstances it maybe preferable not to use piles.

c. Design allowance. If drag will develop, the point resistance of the piles should be evaluated separately by means of analysis or load tests. The drag load should be added to the load earned by the bearing stratum. When drag causes an overload, the allowable load may be reduced by 15 percent if a safety factor from 2.5 to 3 is provided for the working load.

6-8. Permafrost.

a. Suitability. Piles are extremely satisfactory as foundations in arctic regions. Their use is discussed in detail in TM 5-349. Since the bearing value of frozen ground is high, piles in permafrost will support a tremendous load. However, because freezing of the active zone creates uplift, piles maybe installed at least twice as deep as the thickness of the active zone. To reduce uplift, piles are installed butt down. Loads are not placed on piling until the permafrost has had a chance to refreeze, unless the normal skin fiction and bearing will support the load.

b. Allowable load. The allowable load can be determined as follows.

- Immediately after construction (figure 6-7, 1).

$$Q_{all} = P \times A$$

where:

Q_{all} = allowable load

P = the compressive strength of permafrost

A = the tip area of the pile

- During the summer season (figure 6-7, 2).

$$Q_{all} = a + P \times A$$

where:

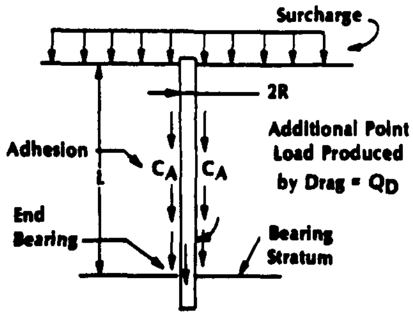
a = the adfreezing strength of the permafrost to the pile

- During the winter season (figure 6-7, 3).

To prevent uplift, a must be greater than b

where:

b = the adfreezing strength of the frost zone to the pile



Single Pile

Sensitive Clay

Maximum Drag Force:

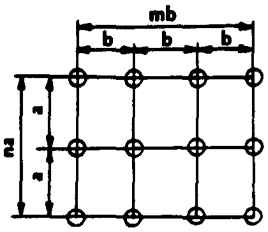
Without surcharge: $Q_D = 2\pi R C_A L$ — With surcharge: $Q_D = 2\pi R C_A L$

Insensitive Clay

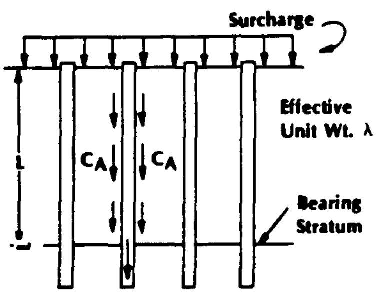
Maximum Drag Force

Without surcharge: $Q_D = 0$ — With surcharge: $Q_D = 2\pi R C_A L$

Adhesion C_A (See Table 5-1)



Pile Cluster



Bearing Piles

Sensitive Clay

End Bearing Piles
Maximum Drag Force:

$$Q_D = ab_A \lambda L$$

But Drag Cannot Exceed the Smaller of the Following Two Forces:

$$Q_D = 2\pi R C_L L$$

or

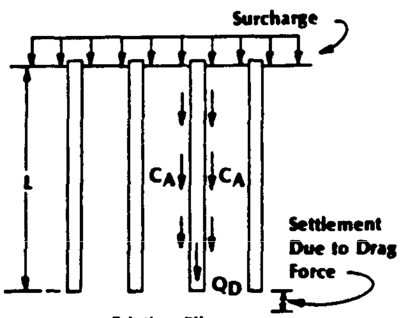
$$Q_D = \frac{2(na + mb)C_A L}{(n+1)(m+1)}$$

$(n+1)(m+1)$ ← Number of Piles

Adhesion C_A (See Table 5-1)

Friction Pile

When Pile Relies on Side Friction for Support and the Direction of Side Friction Is Changed To Drag by Consolidation, then Drag Force that Can Develop Is Limited by Point Bearing Resistance of Pile But Could Equal the Maximum Drag for End Bearing Piles if This Value Is Less than Point Resistance.



Friction Piles

Insensitive Clay

End Bearing and Friction Piles

Condition	Without Surcharge	With Surcharge
	Because Only a Limited Volume of Remolded Soil Is Consolidating, Drag Is Small Compared to Point Bearing Capacity of Pile.	Drag Force Approaches the Upper Limit of the Values for Piles in Sensitive Clay.

Figure 6-6 Analysis of drag on piles in clay

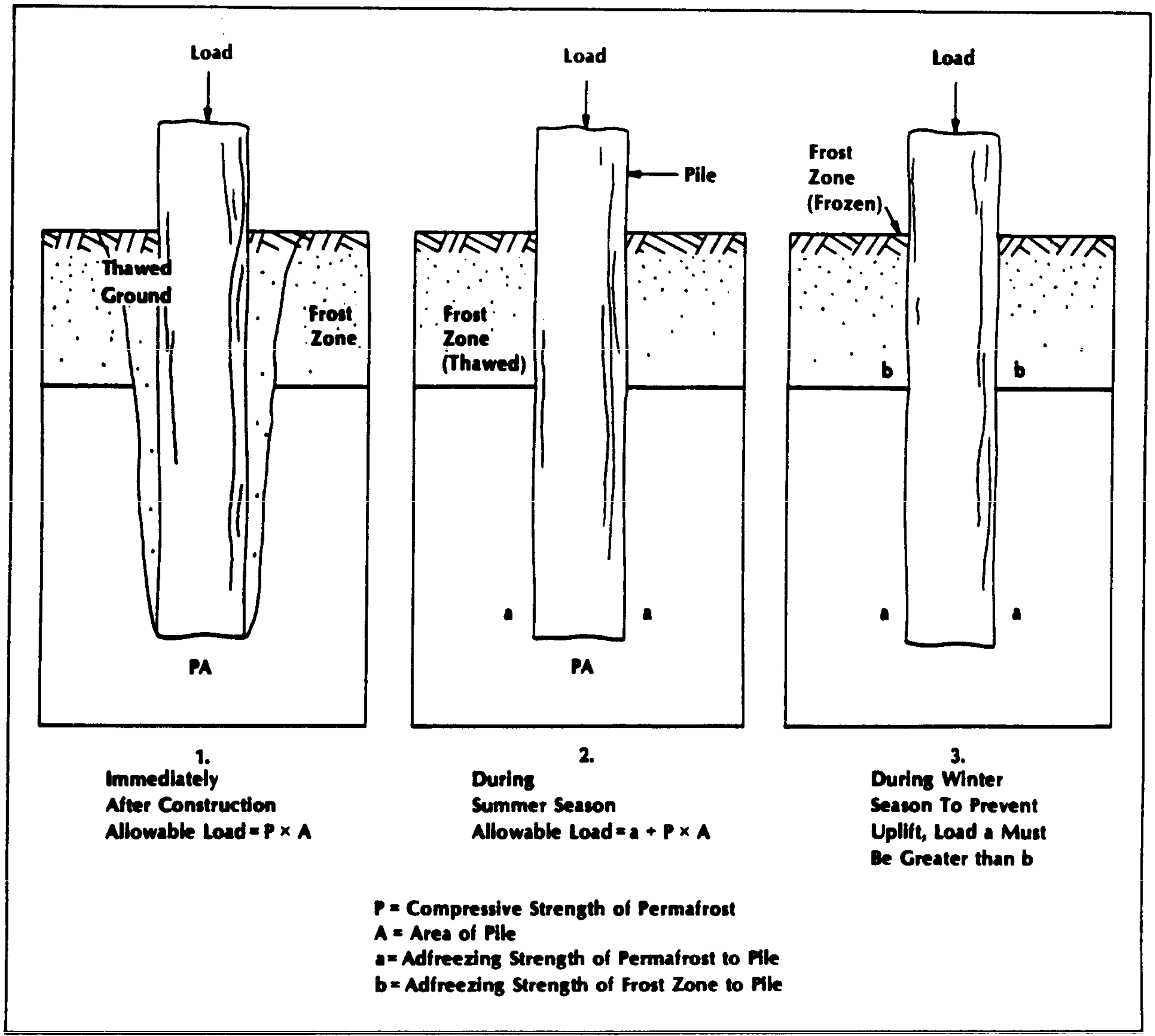


Figure 6-7 Forces acting on and supporting capacity of piling in permafrost

c. Spacing. A minimum spacing of 6 feet is used if the piles are placed in holes thawed by a steam or water jet. Normal design should space piles 10 to 14 feet apart. For very heavy construction, excellent results have been obtained by using 8-inch diameter, standard-weight steel pipes, placed in holes drilled without steam or water jetting, and spaced 6 feet center-to-center.

d. Installation seasons. The best season to install piles in the arctic is autumn, as soon as the ground surface has frozen sufficiently to support equipment. If working conditions permit, winter is an equally good season.

Section III. DESIGN EXAMPLES

6-9. Point-bearing piles in sand.

a. Task. Design a pile structure for soft soils over a thick stratum of sand. Determine the number of 15-inch timber piles required to support an isolated column footing which carries a vertical load of 180 tons, including the weight of the pile cap.

b. Conditions. The soil consists of 10 feet of soft organic clay underlain by sands. The groundwater table is at ground surface. The submerged unit weights of the clay and sands are 40 and 62 pounds per cubic foot respectively. A split spoon boring indicates that the penetration resistance of the sand is 30 blows per foot. A test pile has been driven through the organic clay, penetrating 5 feet into the sand. Time is not available to perform a pile load test.

c. Dynamic formula. A 3,000-pound hammer with a drop of 6 feet is used to drive the test pile. The average penetration of the pile during the last 6 blows of the hammer is 0.25 inch. Using the Engineering News formula applicable for drop hammers, the estimated allowable load for the pile is as follows.

$$Q_{all} = \frac{2 \times 3{,}000 \times 6}{0.25 + 1.0}$$

$= 28{,}800$ pounds
$= 14.4$ tons

d. Static formula. The allowable load **on a** single pile also maybe estimated by means of the static formula (figure 5-4, 2). Based on the penetration resistance of 30 blows per foot, the sand stratum can be assumed to be in a medium dense condition with an angle of internal friction of 36 degrees.

$$Q_u = qA_p + fA_s = P_o N_q A_p + K_c P_o \tan\delta A_s$$

(ultimate bearing capacity)

where:

$P_o = \gamma D$ (effective overburden pressure at tip of pile)

P_o = 10 feet x 40 pcf (clay) + 5 feet x 62 pcf (sand) = 710 psf

N_q = 50 (bearing capacity factor from chart)

K_c = 1.5 (earth pressure coefficient)

$P_o = \gamma D$ (effective overburden pressure at midpoint)

P_o = 10 feet x 40 pcf + 2.5 feet x 62 pcf
= 555 psf (at midpoint of embedment in sands)

$\delta = 29°$ (angle of shaft resistance from chart)

$\tan\delta = \tan(29°) = .554$

$A_p = \pi r^2 = 1.23$ square feet (cross-sectional area of pile)

$A_s = \pi r h$ = (shaft area)

$A_s = 2(3.14)(.625)(5)$
= 19.6 square feet

$Q_u = 710 \times 50 \times 1.23 + 1.5 \times 555 \times 0.554 \times 19.6$
$= 43{,}665 + 9{,}040$
$= 52{,}705$ pounds
$= 26.4$ tons

$Q_{all} = Q_u/FS$
$= \dfrac{26.4}{1.5}$
$= 17.6$ tons

where:

FS=factor of safety

e. Allowable load and spacing. Both the dynamic and static formulas indicate that an allowable load of 15 tons per pile is reasonable. The number of piles required to support the load is 180/15 = 12 piles. As the piles are founded in sand, no reduction for group action is necessary. The piles should be spaced 3 feet (three times the pile diameter) center-to-center and could be arranged in 3 rows of 4 piles each. Piles should be 17 feet long, providing an additional 2 feet required for embedment and for differences in driving resistances. If a concrete cap is used, allowance must be made for embedment of piles into the cap.

6-10. Point-bearing piles in sands with deep clay stratum.

a. Task. Design a pile structure for soft soils over a thick stratum of sand. Determine the number of 15-inch timber piles required to support a load of 180 tons including the weight of the pile cap.

b. Conditions. Foundation conditions are similar to those in paragraph 6-9 except that the sand stratum is of limited thickness and underlain by clay. The soil profile and available soils data are shown in figure 6-8.

c. Allowable load. The allowable load per pile, based on either the dynamic or static formula, is determined to be 15 tons, as noted in paragraph 6-9.

d. Settlement. The clay layer underlying the sand stratum could result in undesirable settlement of the pile foundation. Settlement caused by consolidation is a matter of concern if the structure is not temporary. Consolidation settlement can be estimated using the stress distribution based on figure 6-4 and the approximate method of settlement analysis explained in TM 5-545.

(1) *Basic equation.* The basic equation for settlement due to consolidation of a normally loaded clay of low sensitivity is as follows.

$$\Delta H = [HC_c / (1 + e_o)] [\log_{10} (p_1 / p_o)]$$

where:

ΔH = ultimate decrease in thickness of a confined clay layer due to consoldiation in feet (also, settlement of the structure)

H = thickness of clay layer in feet

C_c = compression index
$C_c = 0.009 (W_L - 10)$
W_L = the liquid limit

e_o = initial void ratio

p_o = initial pressure in tons per square foot

p_1 = final pressure in tons per square foot

(2) *Pressure calculations.* .All pressure calculations will be referred to the center of the clay layer (elevation 268).

$$\begin{aligned} P_o &= 10(40) + 12(62) + 10(48) \\ &= 400 + 744 + 480 \\ &= 1.624 \text{ psf} \\ &= 0.81 \text{ tons per square foot} \end{aligned}$$

where:

P_0 = existing overburden pressure

For simplicity, it is assumed that the piles are arranged in a square pattern of 4 x 4. The increase in pressure, Δp, is obtained by assuming that the load is spread at angle of 2 vertical to 1 horizontal, starting at the lower third point of the pile embedment in sands.

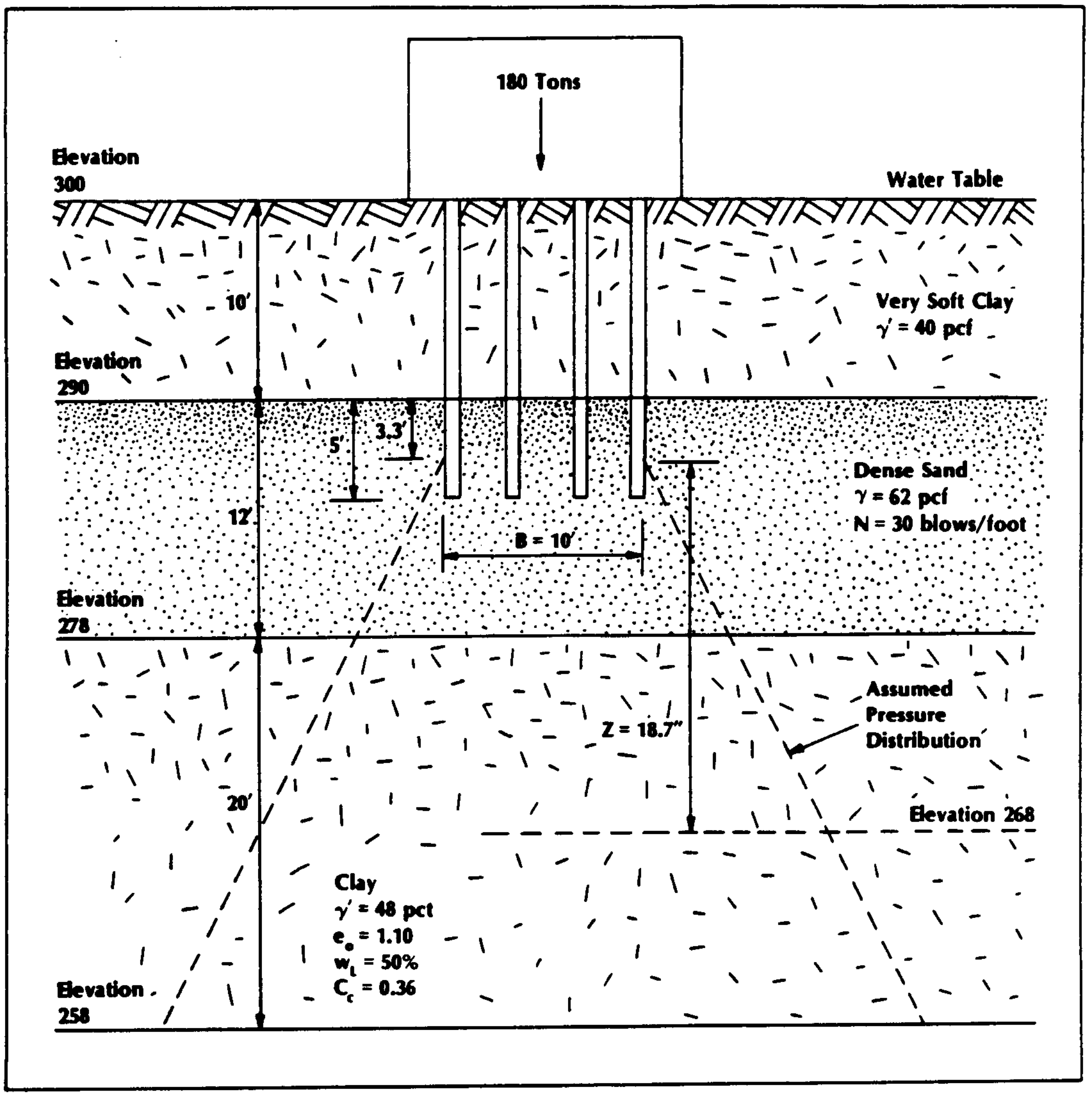


Figure 6-8 Design of pile foundation in dense sand underlain by clay

(3) *Settlemet formula.* Estimated settlement as follows.

$$\Delta p = \frac{Q}{(B + z)^2} = \frac{180}{(10 + 18.7)^2}$$

$$\Delta p = \frac{180}{824} = 0.22 \text{ tons per square foot}$$

$$\Delta p_1 = p + p_o = 0.22 + 0.81$$
$$= 1.03 \text{ tons per square foot}$$

(4) *Settlement estimate.* The settlement may now be estimated as follows.

$$\Delta H = \frac{20\,(0.36)}{(1 + 1.10)}\left[\log_{10} \frac{1.03}{0.81}\right]$$

$$\Delta H = \frac{7.2}{2.1}\left[\log_{10} 1.27\right]$$

$$\Delta H = 3.43\,(0.104) = 0.36 = 4.3 \text{ inches}$$

(5) *Other considerations.* This foundation may be expected to settle approximately 4 inches. Settlement may be reduced slightly by increasing the spacing between the piles. An increase in pile spacing above four times the pile diameter frequently results in uneconomical design of the pile cap. If this amount of settlement is excessive, longer timber piles may be driven through the sand and clay to bedrock. Jetting may be necessary to get the piles through the sand layer. If this is done, the load on each pile maybe increased to 20 tons or more, and the number of piles may be reduced from 16 to 9. Piles 42 feet long will be required.

6-11. Friction piles in clay.

a. Task. Design a pile foundation in a location where borings indicate a uniform clay deposit to a depth of 80 feet (figure 6-9). Determine the number of 12-inch timber piles (readily available in 45 foot lengths) required to support a load of 120 tons.

b. Conditions. The clay is medium stiff, with an average unconfined shear strength of 600 pounds per square foot (0.3 tons per square foot). The allowable load on a single pile may be estimated by using the soil test results and other information (figure 5-3). Time is not available to perform a pile load test.

c. Required embedment. The ultimate load on a single pile using the analysis shown in figure 5-3 is as follows.

$$Q_u = qA_p + ca_z\, dP$$

where:

$$q = 9c = 9 \times 0.3$$
$$= 2.7 \text{ tons per square foot}$$

$$A_p = \pi r^2 = 3.14 \times (0.5)^2$$
$$= 0.785 \text{ square foot}$$

$$c = 0.3 \text{ tons per square foot}$$

$$a_z = 0.92 \text{ (figure 5-3)}$$

$$d = \text{unknown}$$

$$P = 2\pi r = (2)(3.14)(.5) = 3.14 \text{ feet}$$

Based on a safety factor of 2.0, the required embedment to provide an allowable load per pile of 20 tons is as follows.

$$Q_u = 2 \times 20\ qA_p + ca_z\, dP$$

$$40 = 2.7 \times 0.785 + 0.3 \times 0.92 \times 3.14d$$

$$d = 44 \text{ feet}$$

d. Pile spacing and group action. If the piles are arranged in 3 rows of 3 piles each with a spacing of 3 feet 6 inches, center-to-center, the pile group can carry a gross load of 9 (20) = 180 tons. To accommodate the larger settlement expected from a group of piles compared to a single pile, the capacity should be multiplied by an efficiency factor, E, which (as previously noted) is equal to 0.7 for a pile spacing of 3 pile diameters. Thus, the group capacity corrected for settlement to 0.7 x 180 tons (126 tons) is a value greater than the actual load of 120 tons.

e. Block failure. To check for block failure (figure 6-3) the bearing capacity of the pile group is computed as follows.

$$Q_u = qA_p + cdP$$

where:

Q_u = ultimate bearing capacity of group

$$q = 9c = 9 \times 0.3$$
$$= 2.7 \text{ tons per square foot}$$

A_p = plan area of block
= 8 x 8 = 64 square feet

c = 0.3 tons per square foot

d = pile embedment = 44 feet

P = perimeter of group
= 4 x 8 = 32 feet

Q_a = 2.7 x 64 + 0.3 x 44 x 32
= 173 + 422 = 595 tons

With a safety factor of 3, the allowable load on the pile group is 595/3 = 198 tons. Since this is greater than the load which the group will carry (120 tons), the design is satisfactory from the standpoint of block failure.

f. Settlement. The settlement of the pile group may be estimated using the approximate method described in paragraph 6-10, assuming that the pile loads are applied on a plane located one-third of the length of the piles above their tips. Using the data shown in figure 6-9, the following calculations are made with pressures calculated at elevation 125.

P_o = 55(52) = 2,860 pounds per square foot
= 1.43 tons per square foot

$$\Delta p = \frac{Q}{(b + z)^2} = \frac{120}{(8 + 25)^2}$$

= 0.11 tons per square foot

P_1 1.43 + 0.11 = 1.54 tons per square foot

$$\Delta H = \frac{50\ (0.32)}{1 + 1.05} \log_{10} \frac{1.54}{1.43}$$

ΔH = 0.25 foot = 3 inches

Assuming that a long-term settlement of 3 inches is acceptable, the design is considered satisfactory. If the computed settlement is excessive, the amount could be reduced by using greater pile spacings or longer piles. It should be noted that long-term settlements exceeding 1 inch can cause serious problems for rigid structures. This is particularly true when differential settlements occur.

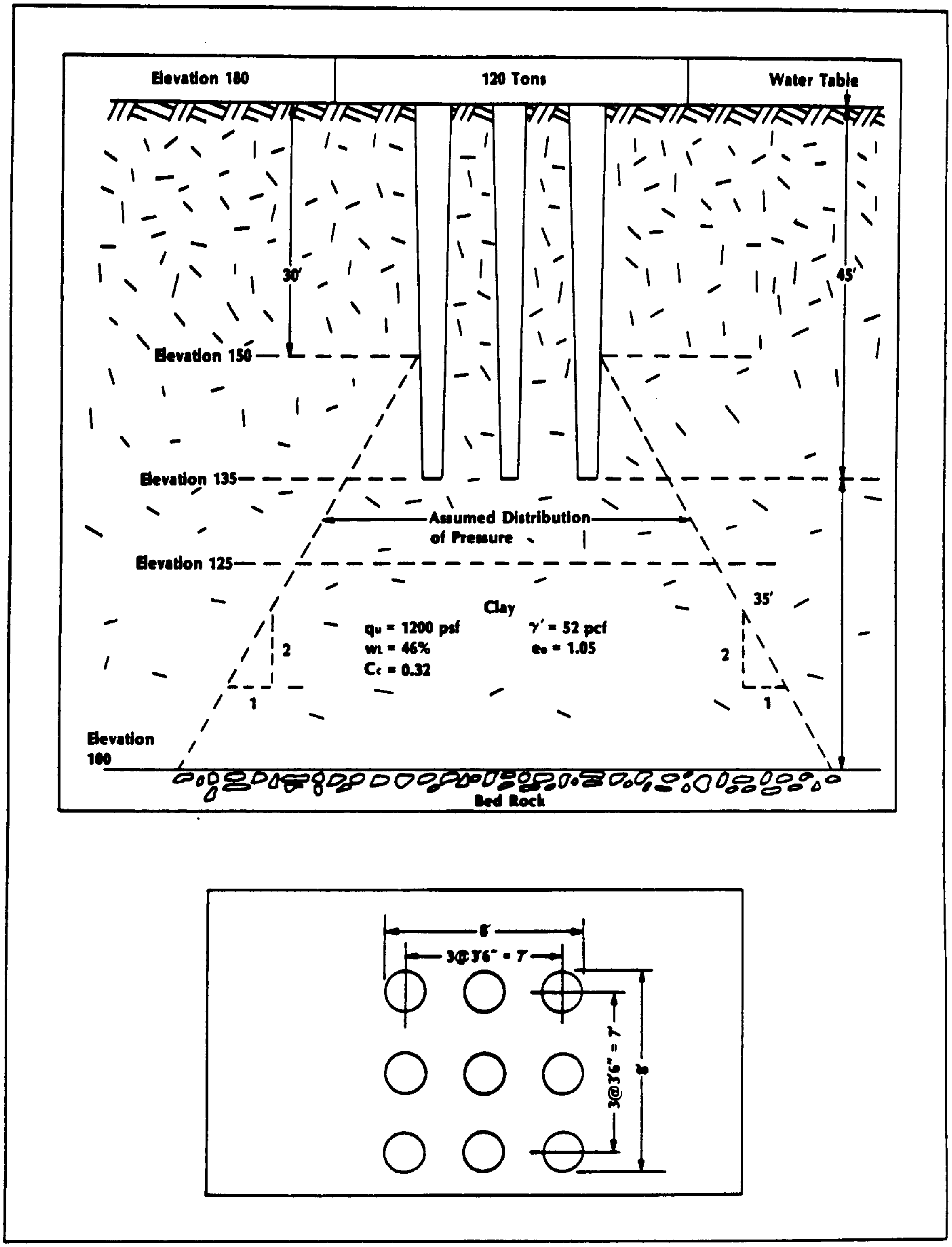


Figure 6-9 Design of friction pile foundation in a deep deposit of clay