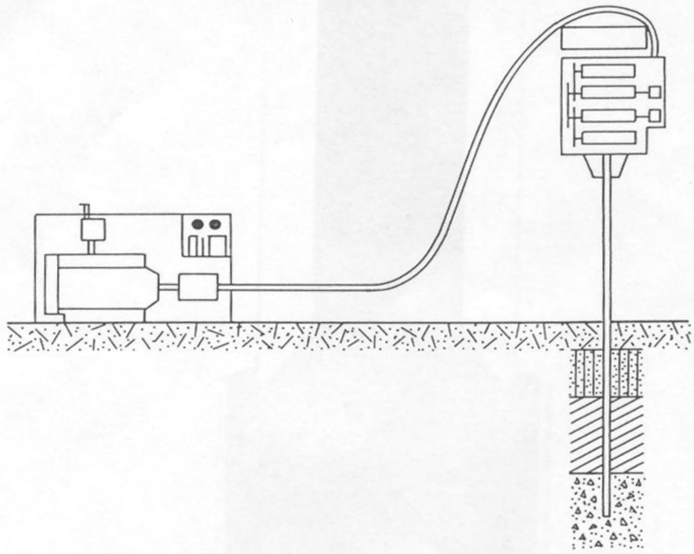
TECHNICAL ENGINEERING AND DESIGN GUIDES
AS ADAPTED FROM THE
US ARMY CORPS OF ENGINEERS, No. 1

DESIGN OF



PILE FOUNDATIONS

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Table 4-5. 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 (a)*Douglas Fir	875	1,700	95	190	1,500,000
Southern Pine (a)(b)*	825	1,650	90	205	1,500,000

d. The allowable stresses for compression parallel to the grain and bending, derived in accordance with ASTM D2899, are reduced by a safety factor of 1.2 in order to comply with the general intent of Paragraph 13.1 of ASTM D2899 (Item 22).

e. For hydraulic structures, the above values, except for the modulus of elasticity, have been reduced by dividing by a factor of 1.2. This additional reduction recognizes the difference in loading effects between the ASTM normal load duration and the longer load duration typical of hydraulic structures, and the uncertainties regarding strength reduction due to conditioning processes prior to treatment. For combined axial load and bending, stresses should be so proportioned that:

$$\left| \frac{f_a}{F_a} + \frac{f_b}{F_b} \right| \leq 1.0$$

where

f_a = computed axial stress

F_a = allowable axial stress

f_b = computed bending stress

F_b = allowable bending stress

E. DEFORMATIONS. Horizontal and vertical displacements resulting from applied loads should be limited to ensure proper operation and integrity of the structure. Experience has shown that a vertical deformation of 1/4 inch and a lateral deformation of 1/4 to 1/2 inch at the pile cap are representative of long-term movements of structures such as locks and dams. Operational requirements may dictate more rigid restrictions and deformations. For other structures such as piers, larger deformations may be allowed if the stresses in the structure and the piles are not excessive. Since the elastic spring constants used in the pile group analysis discussed later are based on a linear load versus deformation relationship at a specified deformation, it is important to keep the computed deformations at or below the specified value. Long-term lateral deformations may be larger than the computed values or the values obtained from load tests due to creep or plastic flow. Lateral deflection may also increase due to cyclic loading and close spacing. These conditions should be investigated when determining the maximum predicted displacement.

F. ALLOWABLE DRIVING STRESSES. Axial driving stresses calculated by wave equation analysis should be limited to the values shown in Figure 4-3.

G. GEOMETRIC CONSTRAINTS.

1. Pile Spacing. In determining the spacing of piles, consideration should be given to the characteristics of the soil and to the length, size, driving tolerance, batter, and shape of the piles. If piles are spaced too closely, the bearing value and lateral resistance of each pile will be reduced, and there is danger of heaving of the foundation, and uplifting or damaging other piles already driven. In general, it is recommended that endbearing piles be spaced not less than three pile diameters on centers and that friction piles, depending on the characteristics of the piles and soil, be spaced a minimum of three to five pile diameters on center. Piles must be spaced to avoid tip interference due to specified driving tolerances. See paragraph 5-2A3 for typical tolerances. Pile layouts should be checked for pile interference using CPGI, a program which is being currently developed and is discussed in paragraph 1-3C6.

2. Pile Batter. Batter piles are used to support structures subjected to large lateral loads, or if the upper foundation stratum will not adequately resist lateral movement of vertical piles. Piles may be battered in opposite directions or used in combination with vertical piles. The axial load on a batter pile should not exceed the allowable design load for a vertical pile. It is very difficult to drive piles with a batter greater than 1 horizontal to 2 vertical. The driving efficiency of the hammer is decreased as the batter increases.

4-3. Pile Capacity

Pile capacities should be computed by experienced designers thoroughly familiar with the various types of piles, how piles behave when loaded, and the soil conditions that exist at the site.

A. AXIAL PILE CAPACITY. The axial capacity of a pile may be represented by the following formula:

$$Q_{ult} = Q_s + Q_t$$
$$Q_s = f_s A_s$$
$$Q_t = q A_t$$

where

Q_{ult} = ultimate pile capacity
Q_s = shaft resistance of the pile due to skin friction
Q_t = tip resistance of the pile due to end bearing
f_s = average unit skin resistance
A_s = surface area of the shaft in contact with the soil
q = unit tip-bearing capacity
A_t = effective (gross) area of the tip of the pile in contact with the soil

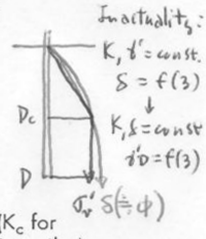
1. Piles in Cohesionless Soil. (β method)

a. Skin Friction. For design purposes the skin friction of piles in sand increase linearly to an assumed critical depth (D_c) and then remain constant below that depth. The critical depth varies between 10 to 20 pile diameters or widths (B), depending on the relative density of the sand. The critical depth is assumed as:

D_c = 10B for loose sands
D_c = 15B for medium dense sands
D_c = 20B for dense sands

The unit skin friction acting on the pile shaft may be determined by the following equations:

$$f_s = K\sigma'_v \tan\delta$$
$$\sigma'_v = \gamma' D \text{ for } D < D_c$$
$$\sigma'_v = \gamma' D_c \text{ for } D \ge D_c$$
$$Q_s = f_s A$$



where

K = lateral earth pressure coefficient (K_c for compression piles and K_t for tension piles)
σ'_v = effective overburden pressure
δ = angle of friction between the soil and the pile
γ' = effective unit weight of soil
D = depth along the pile at which the effective overburden pressure is calculated

Values of δ are given in Table 4-6.

Table 4-6. Values of δ

Pile Material	δ
Steel	0.67ø to 0.83 ø
Concrete	0.90 ø to 1.0 ø
Timber	0.80 ø to 1.0 ø

Values of K for piles in compression (K_c) and piles in tension (K_t) are given in Table 4-7. Table 4-6 and Table 4-7 present ranges of values of δ and K based upon experience in various soil deposits. These values should be selected for design based upon

Table 4-7. Values of K

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

* Note: The above do not apply to piles that are prebored, jetted, or installed with a vibratory hammer. Picking K values at the upper end of the above ranges should be based on local experience. K, δ, and N_q values back calculated from load tests may be used.

Extended Driving and Seating Using WEAP

Pile Material	Allowable Driving Stress
Steel	$0.85\ F_y$
Reinforced Concrete	
Compression	$0.85\ f'_c$
Tension	500 psi
Prestressed Concrete	
Compression	$(0.85\ f'_c - f_{pc})$
Tension	f_{pc}
Timber	3000 psi

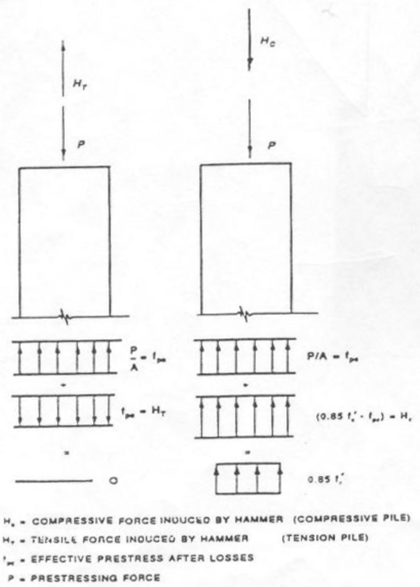


Figure 4-3. Prestressed concrete pile driving stresses

experience and pile load test. It is not intended that the designer would use the minimum reduction of the ø angle while using the upper range K values.

For steel H-piles, A_s should be taken as the block perimeter of the pile and δ should be the average friction angles of steel against sand and sand against sand (ø). It should be noted that Table 4-7 is general guidance to be used unless the long-term engineering practice in the area indicates otherwise. Under prediction of soil strength parameters at load test sites has at times produced back-calculated values of K that exceed the values in Table 4-7. It has also been found both theoretically and at some test sites that the use of displacement piles produces higher values of K than does the use of nondisplacement piles. Values of K that have been used satisfactorily but with standard soil data in some locations are as presented in Table 4-8.

b. End Bearing. For design purposes the pile-tip bearing capacity can be assumed to increase linearly to a critical depth (D_c) and then remains constant. The same critical depth relationship used for skin friction can be used for end bearing. The unit tip bearing capacity can be determined as follows:

$$q = \sigma'_v N_q$$

where

$\sigma'_v = \gamma' D$ for $D < D_c$

$\sigma'_v = \gamma' D_c$ for $D \geq D_c$

For steel H-piles A_t should be taken as the area included within the block perimeter. A curve to obtain the Terzaghi-Peck (Item 59) bearing capacity factor N_q (among values from other theories) is shown in Figure 4-4. To use the curve one must obtain measured values of the angle of internal friction (ø) which represents the soil mass.

c. Tension Capacity. The tension capacity of piles in sand can be calculated as follows using the K values for tension from Table 4-7:

$$Q_{ult} = Q_{s_{tension}}$$

2. Piles in Cohesive Soil. (α method)

a. Skin Friction. Although called skin friction, the resistance is due to the cohesion or adhesion of the clay to the pile shaft.

$$f_s = c_a$$

$$c_a = \alpha c$$

$$Q_s = f_s A_s$$

where

c_a = adhesion between the clay and the pile

α = adhesion factor

c = undrained shear strength of the clay from a Q test

The values of α as a function of the undrained shear are given in Figure 4-5A.

An alternate procedure developed by Semple and Rigden (Item 56) to obtain values of α which is especially applicable for very long piles is given in Figure 4-5B where:

$$\alpha = \alpha_1 \alpha_2$$

and

$$f_s = \alpha c$$

b. End Bearing. The pile unit-tip bearing capacity for piles in clay can be determined from the following equation:

$$q = 9c$$

$$Q_t = A_t q$$

However, the movement necessary to develop the tip resistance of piles in clay soils may be several times larger than that required to develop the skin friction resistance.

c. Compression Capacity. By combining the

Table 4-8. Common values for corrected K

Soil Type	Displacement Piles Compression	Displacement Piles Tension	Nondisplacement Piles Compression	Nondisplacement Piles Tension
Sand	2.00	0.67	1.50	0.50
Silt	1.25	0.50	1.00	0.35
Clay	1.25	0.90	1.00	0.70

Note: Although these values may be commonly used in some areas they should not be used without experience and testing to validate them.

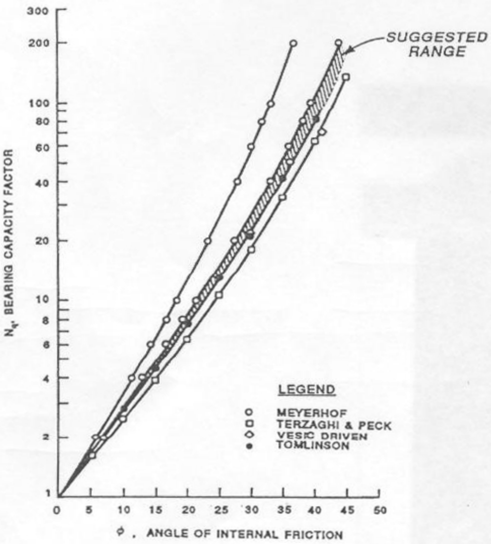


Figure 4-4. Bearing capacity factor

skin friction capacity and the tip bearing capacity, the ultimate compression capacity may be found as follows:

$$Q_{ult} = Q_s + Q_t$$

d. Tension Capacity. The tension capacity of piles in clay may be calculated as:

$$Q_{ult} = Q_s$$

e. The pile capacity in normally consolidated clays (cohesive soils) should also be computed in the long-term S shear strength case. That is, develop a S

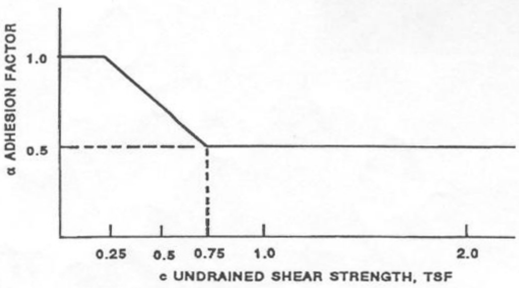


Figure 4-5A. Values of α versus undrained shear strength

case shear strength trend as discussed previously and proceed as if the soil is drained. The computational method is identical to that presented for piles in granular soils, and to present the computational methodology would be redundant. It should be noted however that the shear strengths in clays in the S case are assumed to be $\phi > 0$ and $C = 0$. Some commonly used S case shear strengths in alluvial soils are as reported in Table 4-9.

3. Piles in Silt.

a. Skin Friction. The skin friction on a pile in silt is a two component resistance to pile movement contributed by the angle of internal friction (ϕ) and the cohesion (c) acting along the pile shaft. That portion of the resistance contributed by the angle of internal friction (ϕ) is as with the sand limited to a critical depth of (D_c), below which the frictional portion remains constant, the limit depths are stated below. That portion of the resistance contributed by the cohesion may require limit if it is sufficiently large, see Figures 4-5A and B. The shaft resistance may be computed as follows:

$$K\gamma' D \tan\delta + \alpha c$$

where

$$(D \le D_c)$$

$$Q_s = A_s f_s$$

where

Q_s = capacity due to skin resistance

f_s = average unit skin resistance

A_s = surface area of the pile shaft in contact with soil

K = see Table 4-7

α = see Figures 4-5A and B

D = depth below ground up to limit depth D_c

δ = limit value for shaft friction angle from Table 4-6

b. End Bearing. The pile tip bearing capacity increases linearly to a critical depth (D_c) and remains constant below that depth. The critical depths are given as follows:

D_c = 10 B for loose silts

D_c = 15 B for medium silts

D_c = 20 B for dense silts

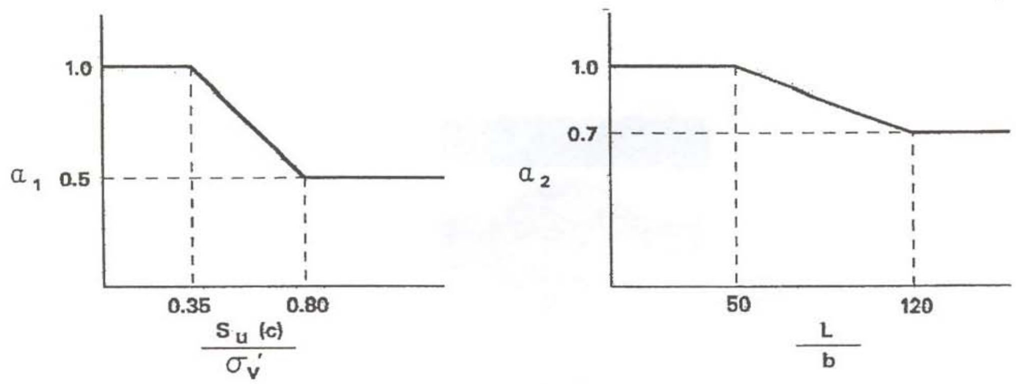


Figure 4-5B. Values of a_1 a_2 applicable for very long piles

The unit and bearing capacity may be computed as follows:

$$q = \sigma'_v N_q$$

$$\sigma'_v = \gamma' D \text{ for } D < D_c$$

$$\sigma'_v = \gamma' D_c \text{ for } D \geq D_c$$

$$Q_t = A_t q$$

where

N_q = Terzaghi bearing capacity factor, Figure 4-4

σ'_v = vertical earth pressure at the tip with limits

A_t = area of the pile tip, as determined for sands

c. Compression Capacity. By combining the two incremental contributors, skin friction and end bearing the ultimate capacity of the soil/pile may be computed as follows:

$$Q_{ult} = Q_s + Q_t$$

d. Tension Capacity. The tension capacity is computed by applying the appropriate value of K_t from Table 4-7 to the unit skin friction equation above.

$$Q_{ult} = Q_{S_{tension}}$$

e. It is recommended that when designing pile foundations in silty soils, considerations be given to selecting a very conservative shear strength from classical R shear tests. It is further recommended that test piles be considered as a virtual necessity, and the possibility that pile length may have to be increased in the field should be considered.

4. Piles in Layered Soils. Piles are most frequently driven into a layered soil stratigraphy. For this condition, the preceding methods of computation may be used on a layer by layer basis. The end bearing capacity of the pile should be determined from the properties of the layer of soil where the tip is founded.

Table 4-9. S case shear strength

Soil Type		Consistency	Angle of Internal Friction ø
Fat clay	(CH)	Very soft	13° to 17°
Fat clay	(CH)	Soft	17° to 20°
Fat clay	(CH)	Medium	20° to 21°
Fat clay	(CH)	Stiff	21° to 23°
Silt	(ML)		25° to 28°

Note: The designer should perform testing and select shear strengths. These general data ranges are from test on specific soils in site specific environments and may not represent the soil in question.