PILE FOUNDATIONS : KNOW-HOW

How To Determine Pile Depth Of Embedment

This third in the series of articles covering Pile Foundations: Know-How *discusses the need for accurate predetermination of required depth of pile embedment by adequate soil investigation, laboratory testing and field verification by load tests.*

by Clyde N. Baker, Jr.

THE PROBLEM of how to determine pile depth of embedment can be restated as an exercise in how to determine pile capacity for a given depth of embedment. The predetermination of pile capacity is usually based upon an anticipated level of driving resistance (the piles shall be driven to a certain blow count) or based upon an analysis of the soil conditions and soil properties combined with a knowledge of the pile bearing area and the perimeter surface area. The problem with the former procedure is that the required pile lengths cannot be predetermined with sufficient accuracy except in certain situations and by engineers with considerable experience.

In this presentation different pile situations are separated into appropriate categories and pile capacity or depth of embedment is determined for that category. The importance of the proper selection of critical parameters is discussed and certain "red lights" to watch out for are described.

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Pile Categories

The possible pile-foundation support combinations can be grouped into four main categories as shown in Figure 1. These categories consist of A-bearing pile, B-friction pile (cohesive soil), C-friction pile (cohesionless soil), and D-combination friction-bearing piles.

With regard to case A, the pile is driven through soft or loose soils to a very dense bearing stratum such as rock or hardpan where negligible penetration into the stratum is contemplated and the pile tip rests on the bearing stratum.

In case B, the pile is driven in clay soils to some predetermined depth but the material in which the pile tip rests is not appreciably denser or stronger than the material above the tip.

In case C, the pile is driven in cohesionless silts, sands and gravels and the soil at the tip is not appreciably denser than the soils above.

In case D, the pile is driven through softer layers into a hard layer and the penetration into the hard layer is sufficient to be significant. Case D represents a combination of the other cases.

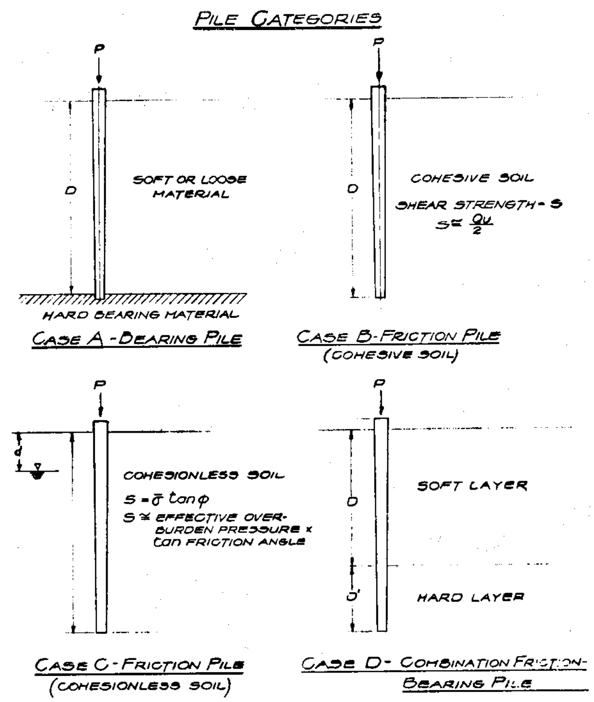


Fig. 1

Determining Pile Capacity Or Depth Of Embedment

Formulas which can be used to determine the pile capacity and consequently the required depth of embedment for a given pile load for the different pile-soil support cases are shown in Figure 2.

Case A, a strict bearing pile situation: determination of the required pile depth simply involves the determination of the depth to the rock or hard bearing stratum. This can usually be determined by sufficient borings, although in certain areas the elevation of the bearing surface can vary drastically over a relatively short distance so that the number of borings required for a project must vary with the amount of anticipated or observed variations in the bearing surface. The pile capacity for this case is based either on the bearing capacity of the bearing stratum or on the allowable stresses in the pile material.

No credit is given to any strength in the overburden soils because of the much greater compressibility of these soils relative to the bearing stratum. A firm seat on the bearing stratum is desired but care must be taken not to overdrive and damage the pile. The pile is usually driven to a certain blow count based on an appropriate pile driving formula. We have found satisfactory experience using a modification of the *Engineering News Record* formula, $R = \frac{2E}{S+.2}$ where E = rated energy and S = set in inches per blow.

Case B, piles driven into clay soils and intended to achieve their support through side friction: the analysis for pile capacities or required pile depths is based upon the perimeter shear concept. Accurate information must be available on the shear strength of the soil and on the adhesion factor between the soil and pile material. For clay soils, it is common to assume that the shear strength is equal to one-half the unconfined compressive strength. This appears reasonable for mos.

PILE CAPACITY DETERMINATION

KEY: -- Q = Ultimate Failure Load
- P = Allowable Load
- a = Pile Cross-Sectional Area
- p = Av. Pile Perimeter Over Length Involved
- D = Depth of Pile Penetration
- D' = Depth of Pile Penetration into Bearing Stratum
- d = Depth to Water Table
- q = Bearing Capacity of Material
- FS = Factor of Safety
- n = Adhesion Factor
- K = Earth Pressure Coef.
- γ_f = Unit Wt. Above Water Table
- γ_b = Submerged Unit Wt.
- r = Pile Radius
- N_c, N_q, N_γ = Bearing Capacity Factors
- D_L = Limited to $\leqq 2or$

CASE A

Pile Capacity Q = Bearing Capacity (q) of Bearing Stratum X Pile Tip Area $\leqq$ Allowable Stress in Pile Material X Tip Area

$$P = \frac{Q}{FS}$$

CASE B

$$\text{Pile Capacity } Q = D \times p_{AV} \times n \times S_{AV} = D \times p_{AV} \times n \times \frac{Qu_{AV}}{2}$$

$$P = \frac{Q}{FS}$$

CASE C

$$\text{Pile Capacity } Q = d \times p \times K \times \gamma_f \times \tan\phi \times \frac{d}{2} + (D-d) \times p \times K \times (\gamma_f \times d + \gamma_b \times \frac{D-d}{2}) \tan\phi + q \times a$$

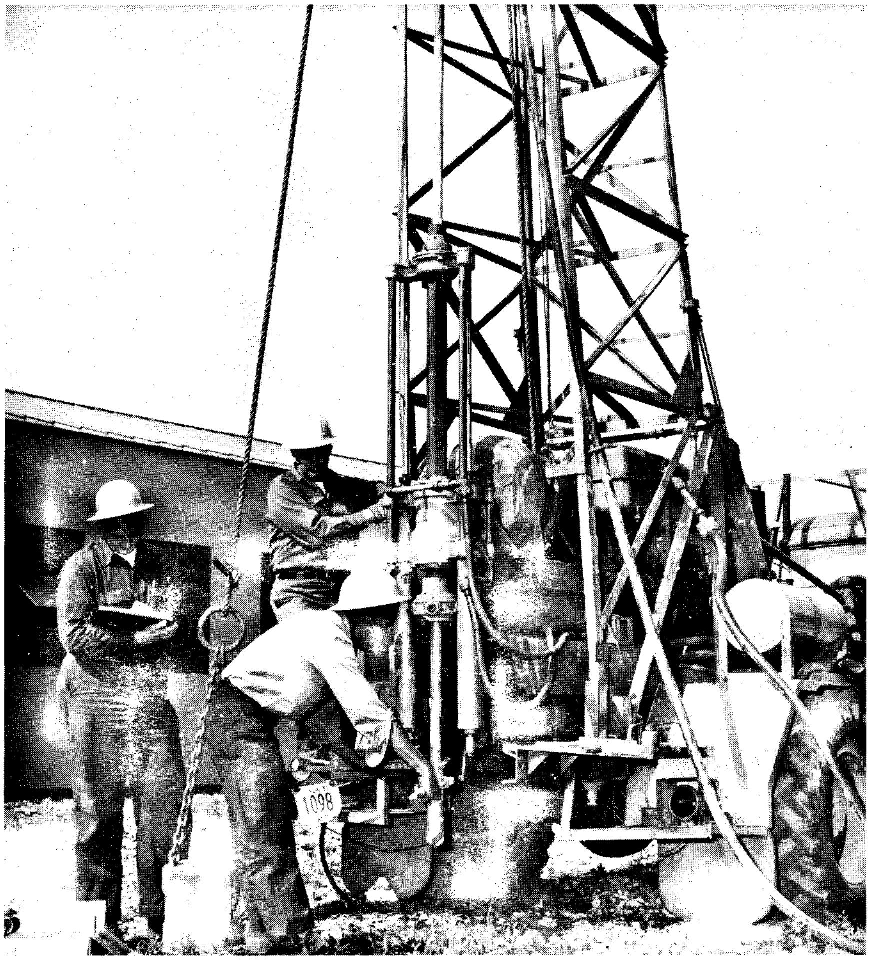
Where $q = 1.3_c N_c + \gamma D_L N_q + 0.6\, \gamma r N_\gamma$

For Cohesionless Soils $C = 0$

CASE D

$$\text{Pile Capacity } Q = qa + D' \times p \times (K \text{ or } n) \times S_{AV}$$

Fig. 2



Typical drill rig boring operation used to obtain information for pile foundations.

clay soil situations but may not be sufficiently accurate for particularly sensitive clays or unusually preconsolidated clays.

Case C, friction pile in cohesionless soil: the perimeter shear concept is again used but in this case it is also necessary to include the bearing capacity of the pile tip since the bearing capacity of granular soils increases markedly with confinement and may be significant even considering the small pile tip area. The available side friction will be a function of the shearing strength of the soil and the lateral earth pressure coefficient between the soil and pile. The shear strength is a function of the effective overburden pressure, which must take into account the presence of the water table, if any, and the angle of internal friction of the soil.

The bearing capacity of the soil at the tip of the pile can be estimated by several methods. The most common methods are those developed by Terzaghi, Meyerhoff, and Housel. The reader is referred to the references at the end of this paper for a more complete description of the procedures. One simple approach is to use the Terzaghi-Peck formula for bearing capacity of shallow foundations (this is the formula shown on Figure 2) and to modify it appropriately for deep piles. The main variable which needs to be modified is the depth variable since it has been observed that there is a limiting depth beyond which the additional depth is of no benefit. The pile can reach effective failure by compression in the soil rather than by forming a failure surface extending up to ground surface. This limiting depth factor is probably no more than 10 times the pile diameter for most soil conditions.

Case D, the combination friction-bearing pile: it is necessary to consider both the pile tip capacity and the available skin friction in that part of the pile embedded in the hard stratum. If there is a marked difference in compressibility between the upper strata and the stratum in which the pile is embedded, available skin friction in the upper soils is usually ignored since it may dissipate with time and transfer load down to the denser soil. If the soil strata properties are not appreciably different with regard to compressibility, it is reasonable to consider the available side friction throughout the entire length of the pile. The soils may be either cohesive or cohesionless and the determination of maximum available side friction would be as indicated for cases B or C.

If the compressibility of the softer upper soils is only moderately greater than the compressibility of the lower stronger soils in which the pile is embedded, it may be desirable to consider a portion of the available side friction in the upper softer soils as acting. The proportion used can be based on the relative compressibility of the soils (relative estimated modulus of deformation). This should be considered only if there is no chance of negative skin friction developing. The problem of negative skin friction is discussed briefly later in this article.

Selection Of Soil Parameters

The previously described procedures are relatively straightforward and non-controversial. The main problem is in the correct selection of the parameters to use in the formulas. The critical variables are the adhesion factor, the lateral earth pressure coefficient, and the shear strength. With regard to the adhesion factor, it has been observed that the adhesion between cohesive soil and timber piles is excellent and that an adhesion factor of 1, or close to 1, can be assumed. With regard to steel and concrete piles, the adhesion is often not as good and an adhesion factor of somewhat less than 1 is advisable.

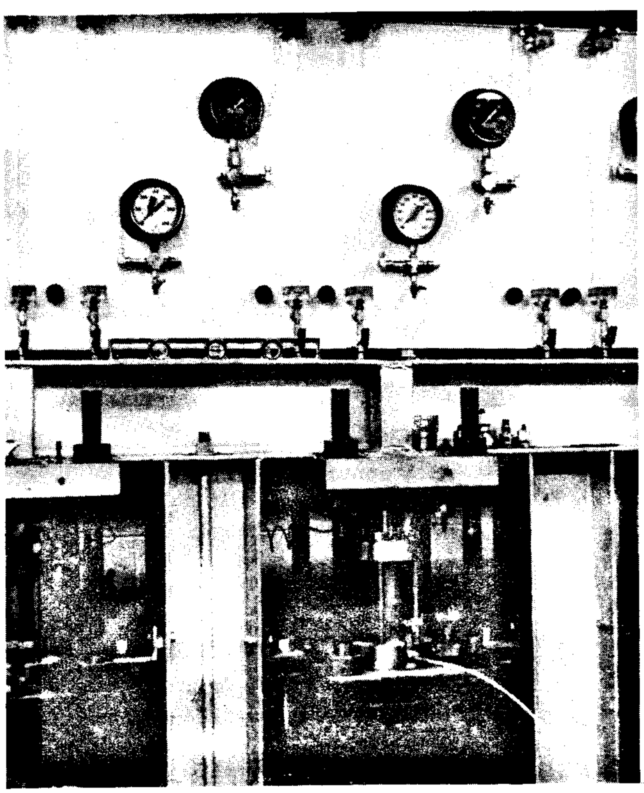
With regard to the coefficient of lateral earth pressure in cohesionless or frictional-type soil, the selection of the proper value depends on the density of the deposit before and after driving, pile spacing, and the taper of the pile. The initial at rest lateral earth pressure coefficient in most granular soils is in the range of .4 to .7. Driving piles into the deposit will tend to increase the lateral earth pressure coefficient. However, for straight sided piles, the coefficient may still be less than 1, but for tapered piles, even with a rather small taper (such as for timber piles) a lateral earth pressure coefficient of at least 1.5 can be assumed.

For cohesive soils, it is essential to select the appropriate average shear strength for the stratum in question. This involves knowledge of the unconfined compressive strength of the clay soil and knowing or being able to estimate with reasonable accuracy the angle of internal friction for cohesionless soil. Objections to using one-half the unconfined compressive strength of clay soil as the appropriate shear strength value can be raised, but, given the usual variations in soil conditions and other factors, and considering the total factor of safety, use of one-half of the unconfined compressive strength as the shearing strength is still considered reasonable.

The factor of safety to be used in pile foundation design depends in part upon the reasonableness and reliability of the foregoing soil parameters. It is quite normal to use a factor of safety of 2.5 when predetermining pile lengths.

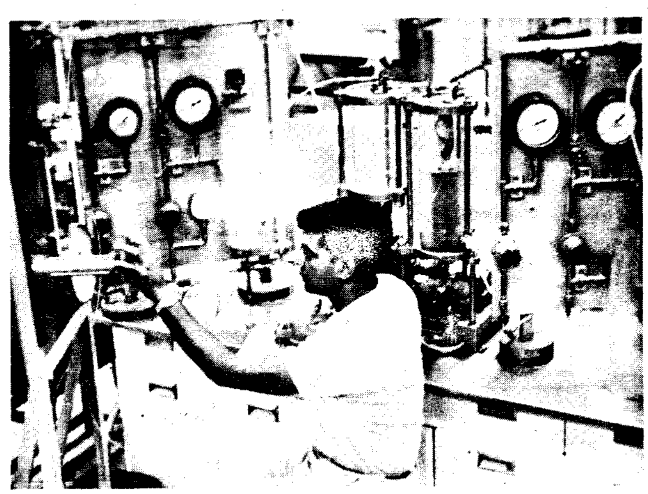
Use Of Pile Load Tests

To verify the suitability of predetermined pile depths of embedment by one or more representative load tests is sound foundation design practice. This is particularly desirable on large projects where undue conservatism may be excessively costly. Conversely,



High pressure triaxial compression testing equipment used to determine shear strength parameters required for bearing capacity predictions of deep bearing stratum.

Laboratory unconfined compression tests used to obtain indication of maximum available skin friction for timber piles in clay soil.



Laboratory triaxial compression equipment used to determine shear strength parameters required for estimating skin friction available in soils having measurable angle of internal friction.

where it is known in advance that no pile load tests will be performed, it is prudent to remain conservative in the selection of the soil parameters used in the formulas for determining the pile depth of embedment. Load tests should be run a sufficient length of time after pile driving so that any disturbance effects have dissipated or at least have been minimized or otherwise taken into account. A minimum time of two days is recommended for any case except rock bearing and a minimum of one week is recommended for any friction or combined friction bearing case in clays. In fact, a fair test might require waiting one month or more after driving in sensitive clay soil. Such a long wait might not be practical except by load testing in advance of construction.

Danger Signals

In every soil engineer's experience there are usually projects that did not turn out exactly as anticipated and from which much can be learned. It has been rightly said that we learn more from our failures than we do from our successes because we seldom know whether we have been too conservative in most cases where all goes well. Experience has taught us that there have been several soil condition situations which required pile lengths considerably different from those initially predicted.

A particular condition to watch out for is dense sand below the water table where it is anticipated that the dense sand will serve as a bearing stratum and only slight penetration into the stratum is predicted. If the stratum has a density less than the critical void ratio (and the critical void ratio can be quite low and the sand quite dense), excess pore pressures may develop during driving. This can greatly reduce the driving resistance and permit the piles to penetrate deep into the dense bearing stratum prior to reaching any agreed-upon blow count. On the other hand, if the sand is denser than the critical void ratio, negligible penetration will be achieved. Unfortunately, it is often difficult to determine in advance which way the dense sand will behave. In the case of excessive penetration into the bearing stratum, a pile load test can be used to verify that the pile can carry the load (once pore pressures dissipate) at a less than anticipated blow count.

If a pile load test is performed in sand or other cohesionless soils, it is desirable to use a dead load reaction type load test rather than using uplift resistance on anchor piles to provide the reaction. The reason being that the uplift action on the anchor piles actually reduces the confining pressure on the test pile and reduces the bearing capacity of the test pile to much less than would be anticipated in a pile group. However, a load test using a dead load reaction wherein the pile is jacked against actual dead load represents a condition closer to that prevailing in a pile group where the pile cap and the load from exterior piles act to confine interior piles.

Another condition which may cause surprises is that of driving piles in very tough or hard clay and predetermining their pile lengths. This is particularly true for straight-side piles where the pile tip may create a hole slightly larger than the pile diameter and there is little adhesion between the pile and the clay. If too high a value is assumed in side friction, the piles may not pass the load test. In our experience it is unsafe to assume greater than 2,500 psf in side friction for straight sided piles in clay, no matter how hard the clay. The condition is less critical for tapered piles such as timber piles, but prudence is still required in estimating the available side friction.

Other Considerations

Two other factors which may affect required pile depth of embedment are the effects, if any of group action, and negative skin friction. These factors will be discussed in detail later in the series. A brief comment should be sufficient at this time.

Pile group action may reduce the safe working load for piles by either increasing settlement above some tolerable limit or reducing available side friction on individual piles because of stress interference between piles. Negative skin friction results when the soil adjacent to a pile consolidates and settles more than the pile itself (pile tip in less compressible stratum) and the soil-pile skin friction exerts a down drag on the pile.

Conclusions

It can be concluded that reasonably accurate predetermination of required depth of pile embedment can be made for most soil-pile foundation situations provided that an adequate soil investigation and laboratory testing program has been performed and provided that predictions are verified in the field by properly conducted load tests. ■

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