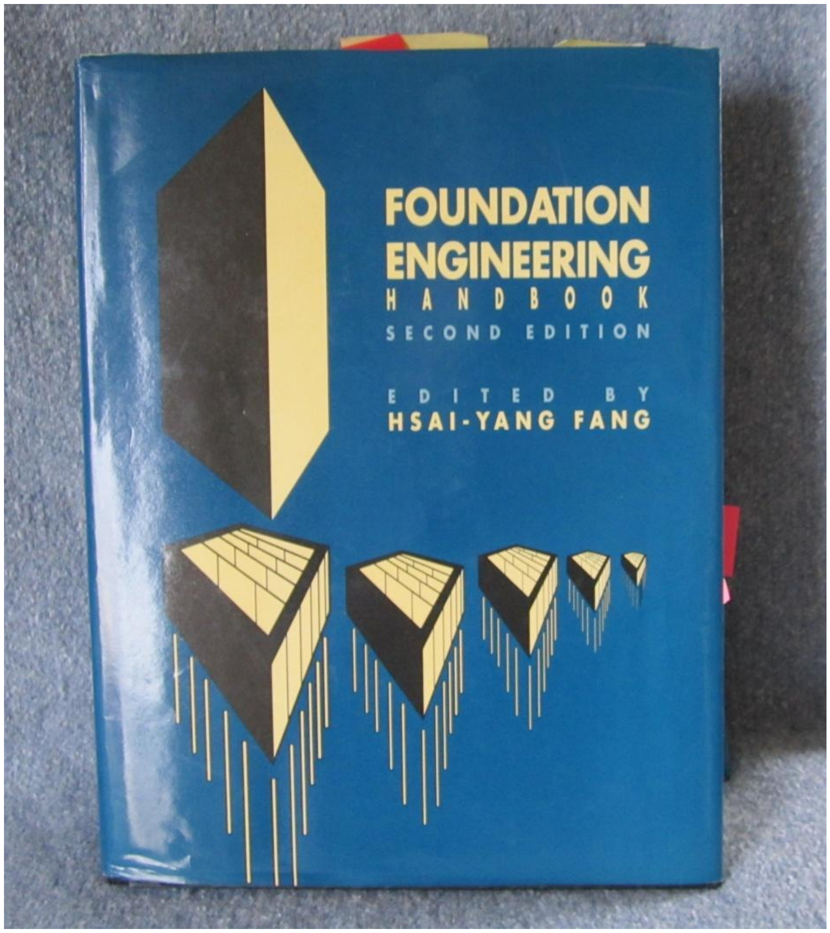

Piled Foundations

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13 PILE FOUNDATIONS

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13.1 INTRODUCTION AND BACKGROUND

Piles are vertical or slightly inclined, relatively slender structural foundation members. They transmit loads from the superstructure to competent soil layers. Length, method of installation, and way of transferring the load to the soil can vary greatly.

Piles are used for a variety of reasons, as follows:

- A competent soil layer can only be found at depth.
- The soil layers immediately below the structure, while competent, are subject to scour.
- The structure transmits large concentrated loads to the soil that cannot be spread out horizontally by means of a wide, shallow foundation.
- The structure is very sensitive to differential settlement.
- The site has a very high water table or artesian water conditions and the soil is sensitive to the construction of even shallow excavations required for mat or footing foundations.

In some cases, the piles serve only to improve the bearing capacity, density, or stiffness of the surrounding soil without directly carrying the load of the structure. Figure 13.1 gives a few examples of the use of piles, including a schematic display of group arrangements of piles.

The connection between the superstructure and the pile is called the *pile cap*. The upper end of the pile (the end connected to the pile cap) is called the *pile head*, and the bottom end is called the *pile toe*. What lies between the pile head and the pile toe is called the *pile shaft*. In older terminology, the term "skin" was used to refer to the surface of the pile shaft.

Piles can be cylindrical or conical. Conical piles can be smooth-tapered or step-tapered. The cross section of a pile can be circular, octagonal, hexagonal, and even triangular; it can be H-shaped, solid, or hollow. The surface of the shaft can be smooth or grooved. The pile toe can be blunt or pointed, or equipped with a shoe with a blunt or pointed end. The shoe can be of the same diameter as the pile shaft or enlarged; it can be equipped with a separate dowel of hardened steel, a rock point. Figure 13.2 provides a small selection of various cross sections and shapes of piles.

The pile material can be wood, concrete, or steel, or any combination thereof. Wood is naturally a very variable material. The proportioning of concrete and the amount and type of reinforcement differs with intended use and with geographic location (mostly because engineering tradition differs both between and within countries). For steel piles, the yield strength of the steel varies considerably between different countries and different sources.

Piles can be used singly or in groups. Mostly, piles are placed in groups. The behavior of a single pile is different from that of an individual pile in a group. A pile group can consist

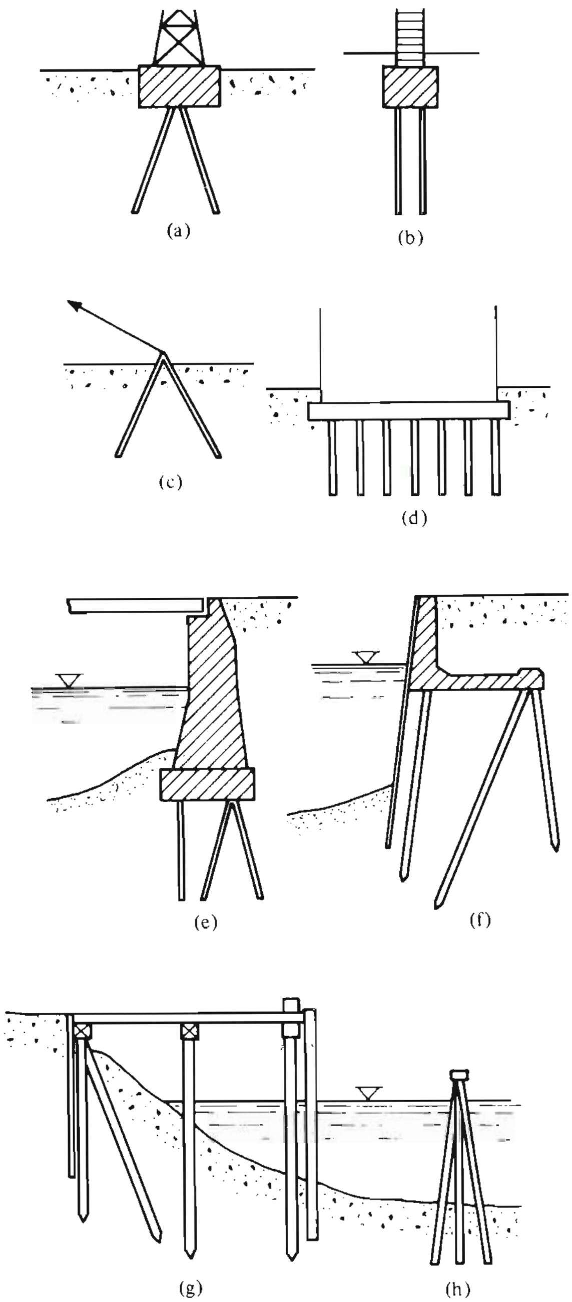

Fig. 13.1 Examples of the use of piles. (*After Kezdi, 1975.*)

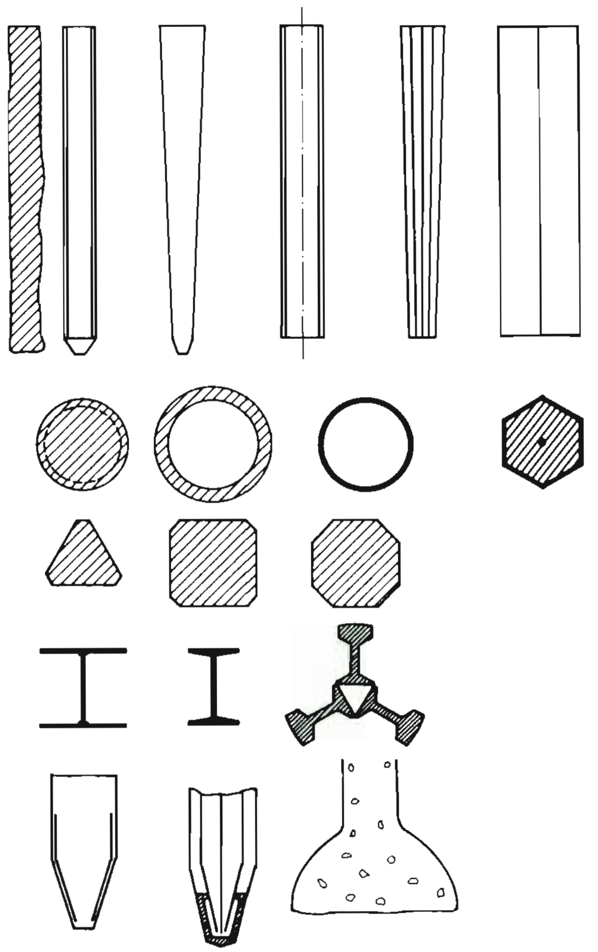

Fig. 13.2 Varous shapes and cross sections of piles. (*Modified after Kezdi, 1975.*)

of a cluster of piles, where the group effect is governing in all directions of load and movement, or consist of a row of piles (a pile bent) where the pile behavior is governed by the group effect in one direction of load and movement, while in the orthogonal direction the piles are independent of the group and behave as single piles.

The list of installation methods can be made very long: Piles can be installed by means of driving or be made in-situ (bored), or be installed by combination of driving and in-situ methods (concrete-filled steel-pipe piles). Driven piles can be head-driven, that is, forced into the ground by means of a pile driving hammer creating a compression force by impacting on the pile head. They can be toe-driven, that is, installed by means of a hammer impacting the pile toe from inside of the pile creating a tensile force in the pile to pull the pile down. Head-driving techniques can be used to toe-drive pipe piles by the use of a mandrel acting on the bottom of the pipe. Piles can be installed by means of vibration hammers with or without combination of jetting techniques.

Last, but far from least, piles are installed in all kinds of soils, even soft rock. For many years, more than 100 metre long slender piles have been installed in clays. Even greater depths are common for offshore piles. Short, stubby, driven piles are frequently used in ablation tills.

It is impossible to cover all aspects of piling within the space allotted to this text. Therefore, this text is not a comprehensive documentation of the state-of-the-art of pile design, but a brief presentation of basics of pile analysis and a few simple approaches that can assist the practising foundation engineer in routine work.

13.2 ASPECTS FOR GENERAL CONSIDERATION

All of the variations mentioned in the introduction will to a greater or smaller degree influence how piles support a structure. For single piles, the dependency on the more important factors is reasonably well known. About group piles, on the other hand, very little is known.

According to Vesic (1977), there were in 1977 extremely few papers reporting full-scale tests with pile groups: Five pile groups were tested in clay and six in sand. The words "full-scale" are used generously in this context. In all but one case, the full-scale was limited to a diameter of 100 mm and a length of 2 m and the maximum number of piles in a group was nine. Despite some recent full-scale tests (O'Neill et al., 1982), so little is known and less verified of pile group behavior that every general method of analysis and statement of group effect should be considered hypothetical and unproven. Therefore, the practising foundation engineer, working under time constraint and having limited data for input into an analysis, is well advised to consider the quantitative result of all analysis to be uncertain, whether the analysis is based on a simplified theoretical approach, or performed according to the latest advancement in theory and laboratory testing. Also, in the light of the ever increasing liability of the designer, verification in the field is imperative in all pile design.

Verification of the analysis can consist of simply relating the design to the old design and past performance of piles as well as of pile-supported structures in the same general area. Naturally, when such experience is lacking or inadequate, various methods of field testing and observations are required. Then, it is necessary to first know the degree of error involved in the data and second to minimize this degree as much as technically and economically possible. The data should be subjected to a rational analysis and related to a theoretical model of the behavior of the pile(s). Otherwise, conclusions drawn and recommendations made may be erroneous.

No design work is complete unless combined with an educated quality inspection and control program aimed to verify that the construction procedures and revelations agree with the design assumptions.

The bearing capacity of a pile consists of a combination of fully developed shaft and toe capacities. For shaft capacity, a distinction is made between, on the one hand, the mobilization of shear caused by the transfer to the soil of load applied to the pile [positive shaft resistance is mobilized for compression loading (the pile is pushed into the ground) and negative shaft resistance for tension loading (uplift; pull)] and, on the other hand, shear mobilized by the soil moving relative to the pile [negative skin friction when the soil settles relative to the pile and positive skin friction when the soil expands (piles in swelling soil), respectively].

The capacity (the ultimate resistance; the ultimate load) is often difficult to assess even by means of a static loading test. The oldest approach is simply to state that the ultimate load in a test is equal to the applied load when the movement of the pile head is 10 percent of the pile toe diameter. Vesic (1977) listed this definition and some others based on the movement of the pile head. Fellenius (1975b, 1980) presented a comparison of nine methods based on the shape of the load–movement curve (see below).

In the simplest principle, design for capacity consists of determining the allowable load (the service load; the upper limit of the applied load) on the pile by dividing capacity with a factor of safety.

When the service load on the piles stresses the soil, the soil will consolidate and compress and the pile will settle. Usually, the methods of settlement calculation are very simple. In estimating the settlement of piles in sand, for instance, a common approach is to consider the settlement to be equal to 1 percent of the pile head diameter plus the "elastic" compression of the pile under the load.

For calculating the settlement of an essentially shaft-bearing pile group in homogeneous clay soil, Terzaghi and Peck (1967) recommended taking the settlement of the group as equal to that calculated for an equivalent footing located at the lower third point of the pile embedment length and loaded to the same stress and over the same area as the pile group plan area (see Fig. 13.3).

More sophisticated methods for calculation of settlement may be used that employ elastic half-sphere analysis and/or finite-element techniques. Vesic (1977) and Poulos and Davis (1980) presented several such analytical approaches toward calculating settlement on single piles and pile groups. Also in the sophisticated methods, the overly simplified assumptions are made that no stress is present in the pile or piles before load is applied to the pile foundation, the piles are of equal length and have equal capacities, and the soil surrounding the piles is assumed to be homogeneous with depth as well as across the group.

The obvious discrepancy between the conditions in the field on the one hand and the too general and/or ideal conditions (assumed in both the simple and the sophisticated approaches) is very frustrating to the foundation engineer. As a consequence, prediction and analysis of settlement of pile groups in engineering design practice leaves much to be desired.

For the case of piles installed through a multilayered soil deposit, where upper layers settle owing to, for instance, a surcharge on the ground surface or to a general groundwater lowering, a settlement calculation of the pile group is often not performed. The design practice seems to trust that the settlement will somehow be taken care of by deducting dragload from the allowable load. (Dragload is the accumulation of negative skin friction in the settling layers.) When a settlement calculation is carried out, sometimes the load generating settlement is taken

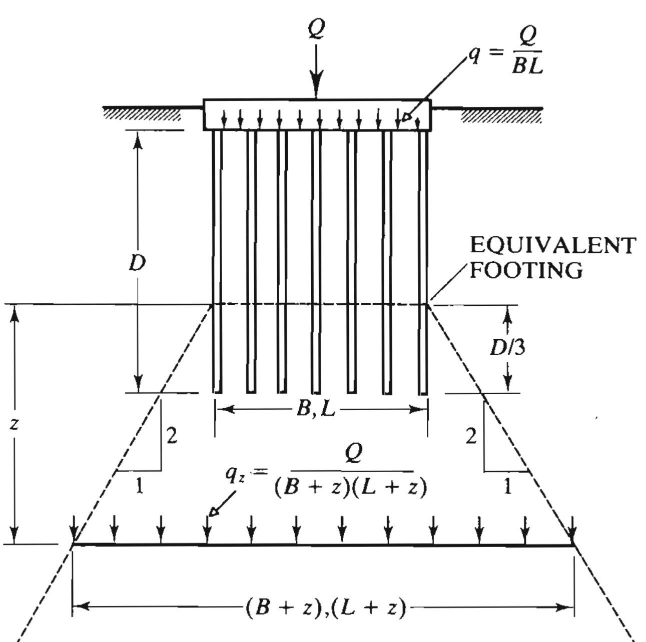

Fig. 13.3 The equivalent footing concept for calculation of settlement of a pile group. (*After Terzaghi and Peck, 1967.*)

to consist of service load and dragload combined! This practice is very unsatisfactory, because capacity and settlement interact and they cannot be treated separately and independently of each other. However, methods do exist by which capacity and settlement can be calculated. One of the most straightforward methods will be described in the following sections.

13.3 THE SHAFT RESISTANCE

For the analysis of shaft resistance, Johannessen and Bjerrum (1965) and Burland (1973) established that the unit resistance is proportional to the effective overburden stress in the soil surrounding the pile. The constant of proportionality is called beta-coefficient, β, and is assumed to be a function of the earth pressure coefficient in the soil, K_s, times the soil internal friction, $\tan \phi'$, and times the quotient of the wall friction, M (Bozozuk, 1972). Thus, the unit shaft resistance, r_s, follows the following relations:

$$r_s = \beta \sigma'_z \tag{13.1}$$

$$\beta = M K_s \tan \phi' \tag{13.2}$$

where

r_s = unit shaft resistance at depth z
β = Bjerrum–Burland beta-coefficient
σ'_z = effective overburden stress at depth z
M = quotient of wall friction = $\tan \delta' / \tan \phi'$
δ' = effective soil–pile friction angle
ϕ' = effective soil friction angle
K_s = earth pressure coefficient

The terms and symbols are also explained in Figure 13.4.

One can develop a wide range of beta-coefficients from a combination of possible earth pressure coefficients, friction angles, and wall friction quotients. However, it appears that the variation of the beta-coefficient is smaller than the variation of its parts would suggest.

In analyzing measurements on piles subjected to downdrag, Bjerrum et al. (1969) found that the beta-coefficient in a soft silty clay ranged between 0.20 and 0.30. This range can be considered the low boundary of the beta-coefficient. While the theoretical upper boundary obviously can be very large, there is a practical limit governed by the density and strength of the soil in which the pile is driven or otherwise installed. For piles in very dense soil, the upper boundary can approach and exceed a value of 1.0, but usually an upper limit of 0.8 is assumed. Table 13.1 suggests a relative range of beta-values. The ranges shown are very wide and very approximate.

When shaft resistance is mobilized by a compression load (push load) applied to a pile, it is defined as "positive shaft resistance". When mobilized by uplift (pull or tension load), the term is "negative shaft resistance". For reasons of reduction of diameter (Poisson's ratio effect) and unloading of the effective overburden stress when loading in pull as opposed to loading in push, the negative shaft resistance is smaller than the positive shaft resistance. Ratios of 0.50 have been proposed.

It has been suggested that the above effective stress relation (Eq. 13.1) ceases to be valid at a certain critical depth equal to about 10 to 20 pile diameters. Below the critical depth, the unit shaft resistance would be constant and equal to the value at the critical depth. However, the concept of critical depth is unproven and in question. It should therefore be applied with caution, if at all.

Sometimes, particularly in cemented soils and more so for cast in-situ piles than for driven piles, the shaft resistance is a

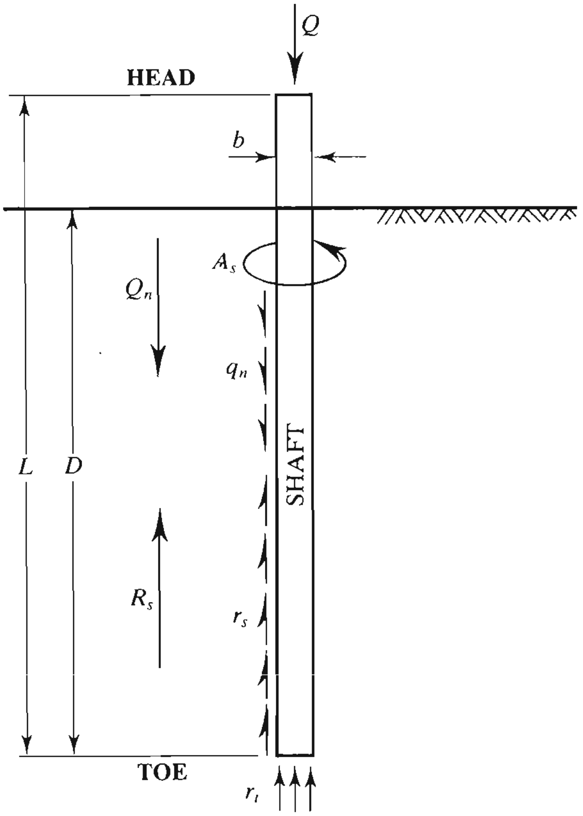

Fig. 13.4 Terms and symbols for pile analysis. Q_d = dead load; Q_l = live load; Q_n = drag load; Q_u = ultimate load (= capacity); R_s = shaft resistance; R_t = toe resistance; R_u = ultimate resistance (= capacity); L = pile length; D = embedment depth; b = pile diameter; A_s = circumferential area; A_t = toe cross-sectional area; N_t = toe bearing capacity coefficient.

$$R_t = A_t r_t - A_t \sigma'_{z=0} N_t$$

$$R_s = \Sigma A_s r_s = \Sigma A_s \beta \sigma'_z \quad \text{or} \quad \Sigma A_s (c' + \beta \sigma'_z)$$

function of both friction and cohesion. Equation 13.1 then changes to:

$$r_s = c' + \beta \sigma'_z \qquad (13.3)$$

where c' = effective cohesion intercept.

Although it has been proven conclusively that the transfer of load from a pile to the soil by means of shaft resistance is governed by the effective stress, for piles in clay, a total stress analysis can be useful in site-specific instances. Also, enough information is often not available to support a reliable design based on effective stress analysis. A total stress analysis may then be used, which means that the shaft resistance is equal to the undrained shear strength of the soil and independent of the overburden stress:

$$r_s = \alpha \tau_u \qquad (13.4)$$

TABLE 13.1 RANGES OF BETA-COEFFICIENTS.

Soil Type	*phi*	*beta*
Clay	25–30	0.23–0.4
Silt	28–34	0.27–0.5
Sand	32–40	0.30–0.8
Gravel	35–45	0.35–0.8

where

τ_u = undrained shear strength
α = proportionality coefficient

However, the total stress analysis can only lead so far and effective stress analysis according to Equations 13.1 and 13.3 provides the better means for analysis of test data and for putting experience to use in a design. Of course, more sophisticated effective stress theories for unit shaft resistance exist. However, in contrast to most of these, the effective stress approach according to Equations 13.1 and 13.3 is not restricted to homogeneous soils, but applies equally well to piles in layered soils and it can easily accommodate non-hydrostatic pore pressures.

The proportionality coefficient is equal to unity in soft and firm clays, but smaller than unity in stiff and hard clays, especially if they are overconsolidated. A useful qualitative reference is illustrated in Figure 13.5, showing that for wood and concrete piles the proportionality coefficient is equal to unity up to a shear strength of about 30 kPa, whereupon it becomes progressively smaller. For steel piles, the coefficient is indicated as smaller than unity even for soft clays.

Equation 13.5 gives the total shaft resistance as the integral of the unit shaft resistance over the embedment depth:

$$R_s = \int_0^D r_s dz = \int_0^D A_s (c' + \beta \sigma'_z) dz \qquad (13.5)$$

where

R_s = total shaft resistance (fully mobilized)
A_s = pile unit circumferential area
D = pile embedment depth

It is important to realize that even simple axial loading of a single pile can be made in several different ways. Figure 13.6 illustrates six cases, A through F, of axial loading. Case A shows a pile loaded with a compression load (push load) at the pile head. The transfer of load to the soil increases the effective stress in the soil and produces compression stress in the pile. The increased stress in the pile causes an increase of pile diameter (Poisson's ratio effect; a minimal increase, of course). These aspects are symbolically indicated in the figure.

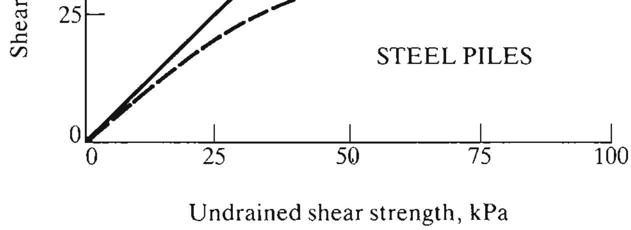

Fig. 13.5 Shaft resistance in clay as a function of undrained shear strength. (*After Tomlinson, 1957.*)

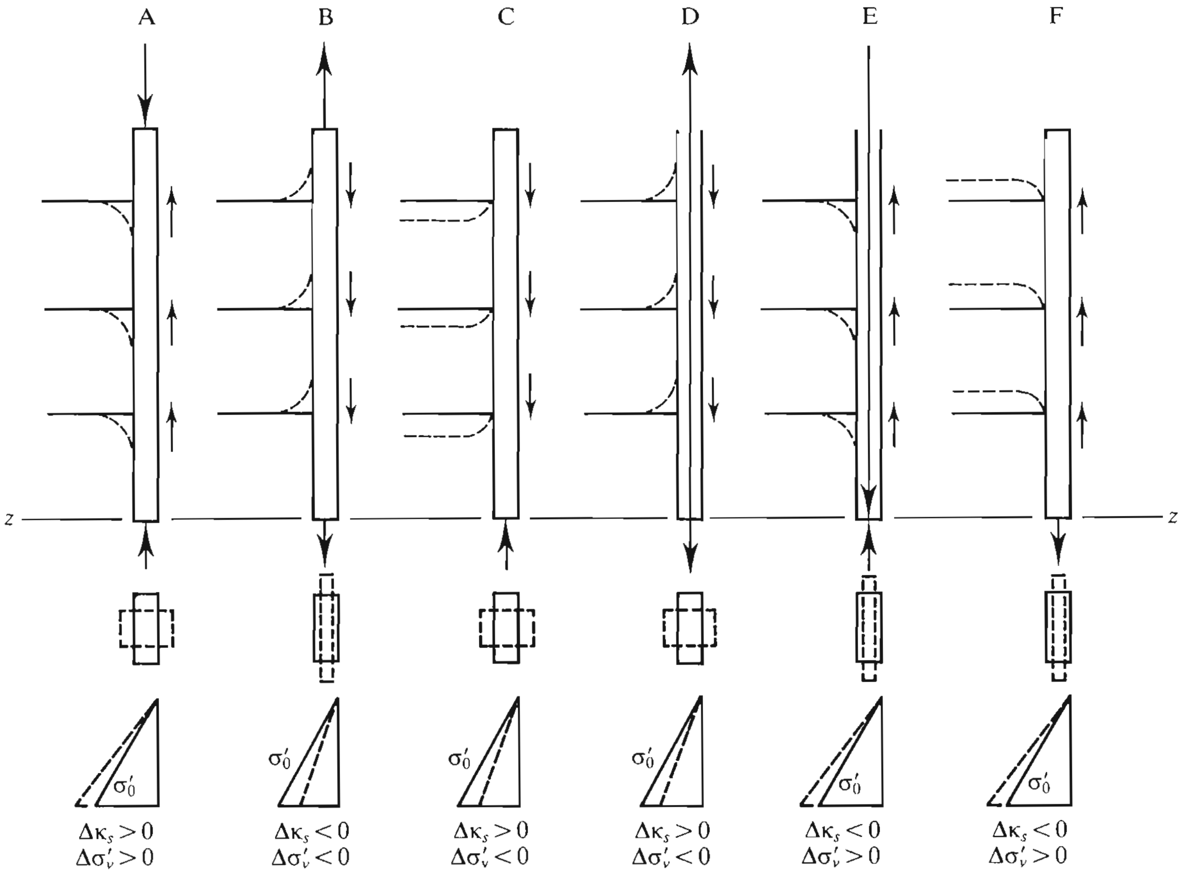

Fig. 13.6 Behavior modes of a pile subjected to six different axial loading conditions. (*After Fellenius, 1984a.*)

Case B shows the effect of an axial tension load (uplift load), when the effective stress in the soil is relieved, the pile is in tension, and the diameter reduces. Case C shows the effect of loading the pile by negative skin friction, when the pile effective stress is relieved, but, in contrast to Case B, the stress in the pile is increased and the diameter is increased. To test for the effect of negative skin friction, the pile would have to be pulled from the toe, as is illustrated in Case D.

Case E shows a pile tested by applying a push load at the toe, which simulates the effect of a pile in swelling soil as modeled in Case F.

The differences between the loading cases shown in Figure 13.6 are slight and the relevance of the distinctions can be questioned. However, it has been observed that the shaft resistance in pull (Case B) is smaller than the shaft resistance in push (Case A).

In contrast to most of the more sophisticated theories for unit shaft resistance in existence, the effective stress approach according to Equation 13.1 and Figure 13.6 is not restricted to homogeneous soils, but equally well applicable to piles in layered soils.

The analysis of load in a pile makes use of mathematical expressions called "transfer functions". Thus, for the case of fully mobilized shaft resistance, the following simple relation is obtained for the load in pile loaded to its full capacity:

$$Q_u = R_s + R_t \tag{13.6}$$

where

R_s = shaft resistance
R_t = toe resistance

and

$$Q_z = Q_u - \int_0^z r_s dz = Q_u - \int_0^z A_s(c' + \beta\sigma_z')dz \tag{13.7}$$

where

Q_z = the load in the pile at depth z
Q_u = the ultimate load

Equation 13.7 is the equation for a curved line which curvature increases progressively with increasing effective overburden stress, that is, increasing depth. Notice that a transfer function in a homogeneous soil resulting in a load distribution at failure that does not decrease progressively with depth is not correct.

Figure 13.7 indicates a few commonly suggested transfer functions (Q) with the corresponding distributions of unit shaft resistance (r_s). Of the five examples shown, only the first two, Cases (a) and (b), are reasonable in a homogeneous soil. The others are merely examples of little interest to the practitioner. In the literature, sometimes, evaluations of field test data are presented that show load distributions similar to Cases (c) and (d). Such evaluations should be considered with considerable scepticism. Case (e) does not exist in the real world.

The movement of the pile surface relative to the soil required to mobilize the ultimate shaft resistance is very small. Even at failure, the movement is not a slip, but occurs as a shear deformation in a zone at and extending out from the pile surface. Long-term measurements of load transfer and movements in piles in a slowly settling clay, where movements at depths were measured about one pile diameter away from the pile surface, showed that relative movements of about 1 mm were all that

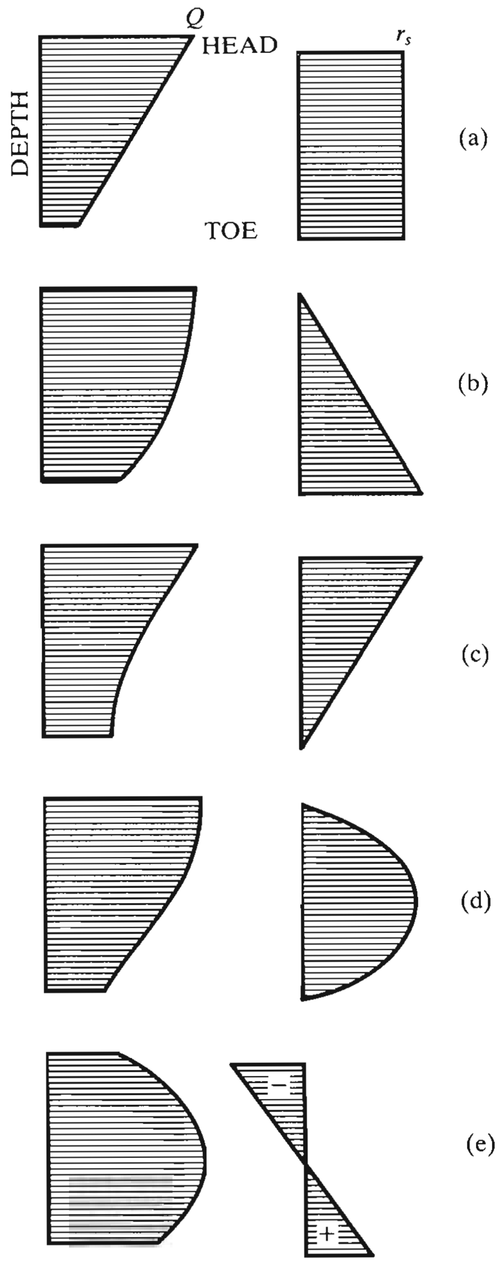

Fig. 13.7 Load transfer functions for five distributions of shaft resistance. (*After Vesic, 1970.*)

was necessary to mobilize the ultimate shaft resistance (Fellenius, 1972; Bjerin, 1977).

13.4 TOE RESISTANCE

The toe resistance is governed by the effective overburden stress according to the following simple relation:

$$R_t = A_t N_t \sigma'_{z=D} \qquad (13.8)$$

where

R_t = the toe resistance = $r_t A_t$
A_t = the pile toe cross-sectional area
N_t = toe bearing capacity coefficient
D = embedment depth
$\sigma'_{z=D}$ = effective overburden stress at the toe

As in the case for the shaft resistance, the unit toe resistance dependency on the effective overburden stress has been claimed to cease at a critical depth—conveniently close to the critical depth of the shaft resistance. For reasons similar to those given for the critical depth with regard to shaft resistance, one can question the existence of the critical depth also in the case of mobilizing toe resistance.

The toe bearing capacity coefficient, N_t, is considered to be related to the N_q factor in the bearing capacity formula for a conventional shallow foundation, that is, it is a function of the effective internal friction angle. Meyerhof (1976) and the *Canadian Foundation Engineering Manual* (1985) suggest that N_t be taken as approximately equal to $3N_q$. Given that, in engineering practice, concern for the toe capacity of a pile only occurs for pile toes placed in dense soils, where the internal friction angle is relatively high, at least about 38°, and that the uncertainty of the N_q relation for a friction angle in the range above 38° is considerable, the suggested relation to N_q is very approximate.

The N_t coefficient depends on soil composition in terms of grain size distribution, angularity and mineralogical origin of the grains, original soil density, density changes due to installation techniques, and other factors. For sedimentary cohesionless deposits, N_t ranges from a low of about 30 to a high of about 120. In very dense, non-sedimentary soils, such as glacial base tills, the N_t coefficient can be considerably higher, but also approach the lower boundary given above. Table 13.2 suggests a relative range of N_t coefficients. The ranges shown are very wide and very approximate.

The movement of the pile toe against the soil necessary to mobilize the ultimate toe resistance is considerably larger than that necessary for mobilizing the shaft resistance. The magnitude required for piles of large diameter is greater than for small-diameter piles. Driven piles, having densified the natural soil below the pile toe and/or preloaded it, require smaller movement as opposed to bored and cast-in-situ piles, not having densified the soil below the pile toe, but, potentially, having disturbed and loosened the soil instead.

For a driven pile, the necessary movement lies in the range of about 3 to 10 percent of the pile toe diameter. However, the load–movement relation is not a straight line. For instance, at a movement of half of that necessary to mobilize the ultimate toe resistance, more than half the toe resistance may be developed. For bored piles, the magnitude is more variable and less predictable.

Although the effective overburden stress governs the toe resistance also in clay, a total stress analysis is sometimes (traditionally) employed wherein the unit toe resistance is set to a factor times the undrained shear strength of the soil at the pile toe. In overconsolidated clays, the factor is sometimes set to a value of 3. Mostly, however, the value is 9. Considering that the undrained shear strength often is in the range of 20 to 30 percent of the effective overburden stress, the N_t factor in these clays becomes equal to about 3, which is the lower boundary of the range given in Table 13.2 for clays.

TABLE 13.2 RANGES OF N_t COEFFICIENTS.

Soil Type	*phi*	N_t
Clay	25–30	3–30
Silt	28–34	20–40
Sand	32–40	30–150
Gravel	35–45	60–300

13.5 CAPACITY DETERMINED FROM IN-SITU FIELD TESTING

Before the capacity of a pile and/or its required embedment depth can be determined, site information obtained by means of a site exploration program must be obtained. The site investigation includes identification of soil layers and classification of soil properties and strength parameters by sampling and testing of samples. In addition, and most important for determining the soil profile, in-situ testing is performed. Usually, the in-situ testing consists of all or at least one of standard penetration testing, vane shear testing, and cone penetrometer testing. More recently, pressuremeter testing and dilatometer testing are included in site exploration programs.

For many years, the N-index of standard penetration test has been used to calculate capacity of piles. Meyerhof (1976) compiled and rationalized some of the wealth of experience available and recommended that the capacity be a function of the N-index, as follows:

$$R = R_t + R_s = mNA_t + nNA_sD \qquad (13.9)$$

where

m = a toe coefficient
n = a shaft coefficient
N = N-index at the pile toe
N = average N-index along the pile shaft
A_t = pile toe area
A_s = unit shaft area; circumferential area
D = embedment depth

For values inserted into Equation 13.9 using base SI-units, that is, R in newton, D in meters, and A in square meters, the toe and shaft coefficients, m and n, become:

$m = 400 \times 10^3$ for driven piles and 120×10^3 for bored piles ($\mathrm{N/m^2}$)

$n = 2 \times 10^3$ for driven piles and 1×10^3 for bored piles ($\mathrm{N/m^3}$)

The standard penetration test (SPT) is a subjective and highly variable test. The test and the N-index have substantial qualitative value, but should be used only very cautiously for quantitative analysis. The *Canadian Foundation Engineering Manual* (1985) includes a listing of the numerous irrational factors influencing the N-index. However, when the use of the N-index is considered with the sample of the soil obtained and related to a site and area-specific experience, prediction by the crude and decried SPT test does not come out worse than predictions by other methods of analysis.

The vane shear test provides a value of undrained shear strength that may be applied to Equation 13.4, above. While the vane shear test appears to be useful in soft clays, it is less reliable when used in silts. It should be recognized that no sample is obtained in the test and that not all vanes are alike. Again, when applied with knowledge of the soil layer tested and related to local experience of its prior use, the vane shear strength can be useful for judging pile capacity.

The static cone penetrometer resembles a pile. There is shaft resistance in the form of so-called local friction measured immediately above the cone point, and there is toe resistance in the form of the directly applied and measured cone-point pressure.

When applying cone penetrometer data to a pile analysis, both the local friction and the point pressure may be used as direct measures of shaft and toe resistances, respectively. However, both values can show a considerable scatter. Furthermore, the cone-point resistance, the cone-point being small compared to a pile toe, may be misleadingly high in gravel and layered soils. Schmertmann (1978) has indicated an averaging procedure to be used for offsetting scatter, whether caused by natural (real) variation in the soil or inherent in the test.

The piezocone, which is a cone penetrometer equipped with pore pressure measurement devices at the point, is a considerable advancement on the static cone. By means of the piezocone, the cone information can be related more dependably to soil parameters and a more detailed analysis can be performed. Soil is variable, however, and the increased and more representative information obtained also means that a certain digestive judgment can and must be exercised to filter the data for computation of pile capacity. In other words, the designer is back to square one; more thoroughly informed and less liable to jump to false conclusions, but certainly not independent of site-specific experience.

The pressuremeter and dilatometer have yet to provide the field verification necessary as reference before the devices become useful to the foundation engineer dealing with pile design.

13.6 INSTALLATION CONSIDERATIONS

It is difficult to determine the magnitude of the shaft and toe resistances before the disturbance from the pile installation has subsided. For instance, presence of dissipating excess pore pressures causes uncertainty in the magnitude of the effective stress in the soil and on-going strength gain due to reconsolidation is hard to estimate. Such installation effects can take a long time to disappear, especially in clays. Fellenius and Samson (1976) and Bozozuk et al. (1978) reported observations inside pile groups installed in silty clay where the reconsolidation period was greater than 6 months. For single piles in soft clay, Fellenius (1972) reported a reconsolidation period of 5 months.

There are indications that piles subjected to static loading tests in a soil before it has fully recovered from the installation effect require considerably large movements before the full ultimate resistance is mobilized (Fellenius et al., 1983, 1989) as opposed to piles tested after full recovery.

13.7 RESIDUAL COMPRESSION

Reconsolidation after installation of a pile imposes compression loads in the pile as a result of negative skin friction developing in the upper part of the pile. The induced load, called *residual load* or *residual compression*, is resisted by positive shaft resistance in the lower part of the pile and some toe resistance. If the residual load is not recognized in the evaluation of results from a static loading test, totally erroneous conclusions will be drawn from the test.

Presenting results from laboratory testing of slender model piles in sand, Hanna and Tan (1973) demonstrated in a pioneering paper the effect of not-considering versus considering residual compression when evaluating data from pile testing. Because the sand was deposited around the pile after the pile had been placed in the testing apparatus, it was ensured that the pile was subjected to a compression load before the start of the static loading test. Figure 13.8, modified from Hanna and Tan (1973), shows load-transfer functions evaluated when not considering the residual compression with load distribution curves shown for five applied loads, the largest being the load at induced soil failure, that is, with the ultimate shaft and toe resistances mobilized. Studying the slope of the load distribution curve at failure, it would appear from the upper diagram in

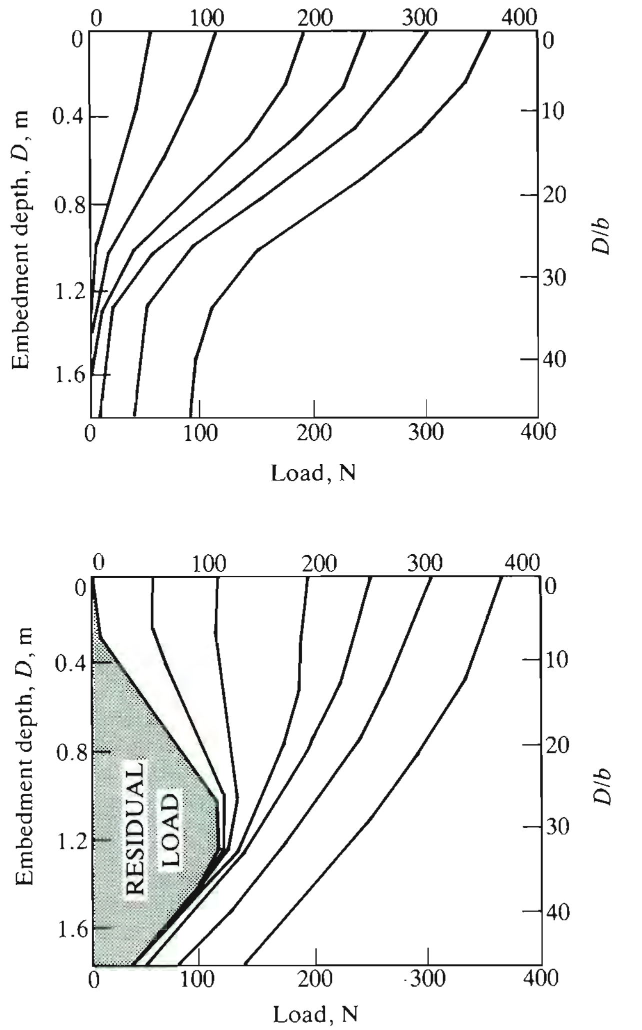

Fig. 13.8 Load distribution in a pile without and with consideration of residual load. (*After Hanna and Tan, 1973.*)

Figure 13.8 that the shaft resistance was the largest along the upper portion of the pile and that it reduced in magnitude with increasing depth to become first constant (straight-line load distribution) and then almost zero near the pile toe. This is seemingly in support of the critical depth concept. However, when, as is shown in the lower diagram in Figure 13.8, the residual compression evaluated to load was included in the analysis, the appearance changed dramatically. The load-distribution line curves progressively in accordance with Equation 13.7 and there is no critical depth or other anomaly shown. It should be noticed that the quantitative effect of including, as opposed to omitting, the residual load in the analysis is that the shaft resistance becomes smaller and the toe resistance becomes larger.

For a discussion in principle of the influence of residual loads on the analysis of piles in compression and tension testing, see Holloway et al. (1978) and Briaud and Tucker (1985).

Residual load in a pile is caused during the installation of a pile, by reconsolidation of the soil after the installation, and by a previous loading cycle. It manifests itself as a compression in the pile caused by negative skin friction in the upper portion of the pile, which is balanced by positive shaft resistance in the lower portion plus some toe resistance.

While the effect of residual loads on the analysis of results from a static loading test have now been realized by the profession, the fact that every pile, whether loaded by a structure or not, is subjected to a similar interaction with the soil is not, as yet, readily recognized.

13.8 THE NEUTRAL PLANE

As stated earlier, only extremely small relative movements between the pile and the soil are required to mobilize a shaft resistance (in positive as well as negative direction). In combination with the additional fact that the difference in stiffness between the pile material and the soil is considerable and that there are movements and strains in a natural soil, make it clear that, for every pile, a stress transfer exists between the pile and the soil. In other words, there are movements in any and every soil, which are restrained by the pile. The restraint builds up force in the pile, and as there is force but no accelerating movement of the pile, the forces must be balanced—must be in equilibrium. Thus, negative skin friction is induced in the upper portion of the pile, resulting in a load in the pile that increases from zero at the pile head to a maximum at the depth of equilibrium. Below the equilibrium depth, the load decreases by being transferred to the soil by means of positive shaft resistance in combination with the small toe resistance.

Every pile develops an equilibrium of forces between the sum of the dead load (the sustained load) applied to the pile head and the dragload, and the sum of the positive shaft resistance and the toe resistance. The location of the equilibrium is called the *neutral plane* and it is the depth at which the shear stress along the pile changes from negative skin friction to positive shaft resistance. This depth is also where no relative displacement occurs between the pile and the soil.

The key aspect of the foregoing is that the development of a neutral plane and negative skin friction always occurs in piles without any appreciable settlement of the soil around the piles.

Normally, the neutral plane lies below the mid-point of a pile. The extreme case is for a pile on rock, where the location of the neutral plane is at the bedrock elevation. For a dominantly shaft-bearing pile "floating" in a homogeneous soil with linearly increasing shear resistance, the neutral plane lies at a depth which is about equal to the lower third point of the pile embedment length.

The larger the toe resistance, the deeper the elevation of the neutral plane, and, the larger the dead load, the shallower the elevation of the neutral plane.

The load transfer in a pile above the neutral plane during long-term conditions is expressed by Equation 13.10 (as opposed to the load in the pile during a static loading test to failure, where the load transfer follows Eq. 13.7):

$$Q_z = Q_d + \int_0^z r_s dz = Q_d + \int_0^z A_s(c' + \beta\sigma'_z)dz \quad (13.10)$$

where

z = depth above the neutral plane
Q_z = the load in the pile at depth z
Q_d = the dead load applied to the pile head

Below the neutral plane, the load is expressed by Equation 13.7 (provided that the toe resistance is fully mobilized).

It is usually assumed that the unit skin friction, q_n, is equal to the unit positive shaft resistance, r_s, an assumption that is debatable, but any error results in an overestimation of the dragload and places the neutral plane higher than if q_n is smaller than r_s. That is, the assumption gives results on the conservative side.

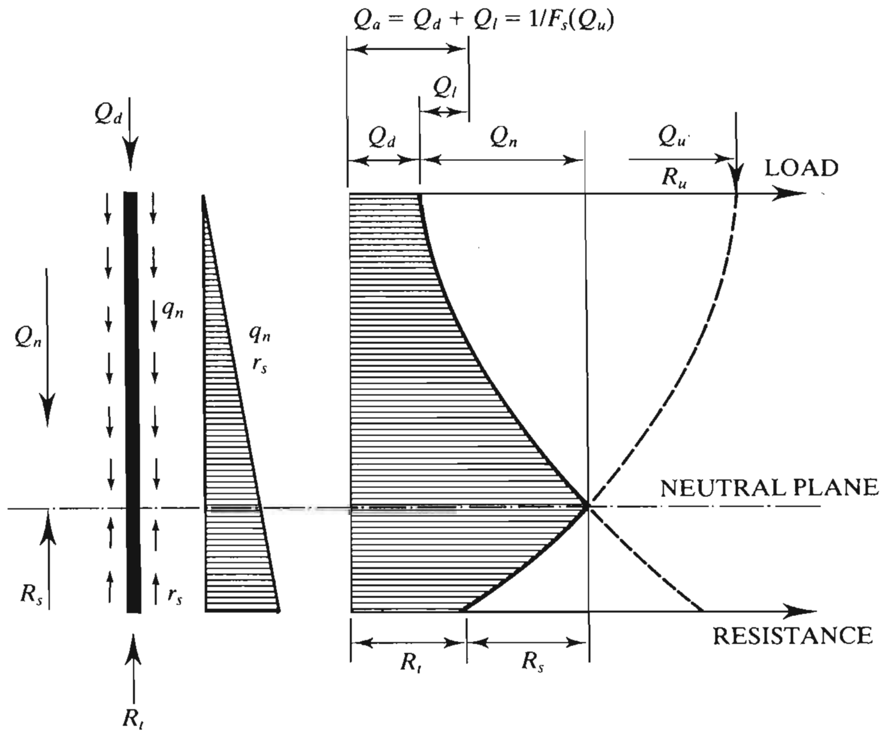

Fig. 13.9 Construing the neutral plane. For terms and symbols, see Figure 13.4. (*After Fellenius, 1989a.*)

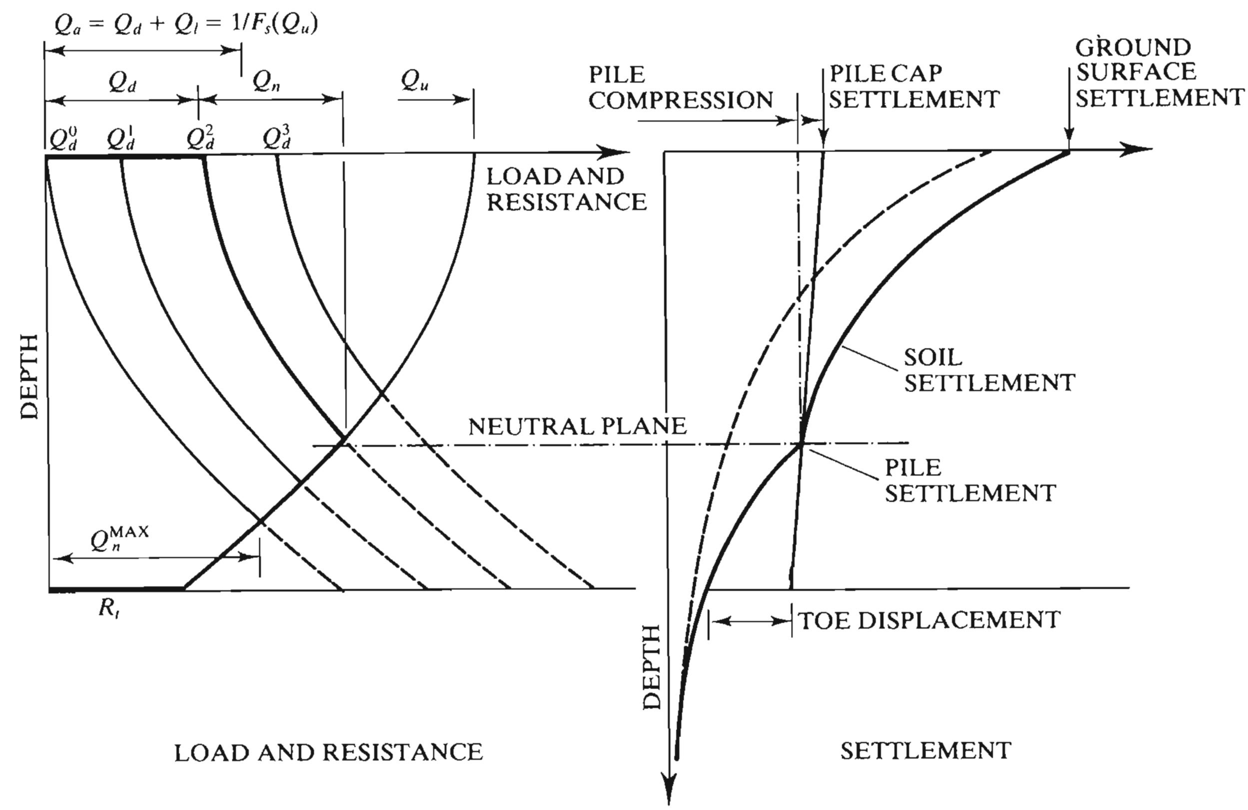

Fig. 13.10 Diagrams of load and resistance and of settlement showing the dependence of the settlement on the location of the neutral plane. The unified design for capacity, negative skin friction, and settlement. (*After Fellenius, 1989a.*)

With larger toe resistance, the elevation of the neutral plane lies deeper into the soil. If an increased dead load is applied to the pile head, the elevation of the neutral plane moves upward.

Figure 13.9 illustrates how to construe the location of the neutral plane. The figure shows the distribution of load in a pile subjected to a service load, Q_d, and installed in a relatively homogeneous soil deposit, where the shear stress along the pile is proportional to the effective overburden stress (for explanations of terms and symbols, see Fig. 13.4).

For reasons of clarity, several simplifying assumptions lie behind Figure 13.9: (a) that any excess pore pressure in the soil caused by the pile installation has dissipated and the pore pressure is hydrostatically distributed; (b) that the shear stress along the pile is independent of the direction of the relative movement, that is, the magnitude of the negative skin friction, q_n, is equal to the magnitude of the unit positive shaft resistance, r_s; and (c) that the toe movement induced is large enough to mobilize some toe resistance, R_t.

As shown, a dragload, Q_n, develops above the neutral plane. The magnitude of the dragload is calculated as the sum (the integral) of the unit negative skin friction. Correspondingly, the total shaft resistance below the neutral plane, R_s, is the sum of the unit positive shaft resistance.

In Figure 13.10, the left-hand diagram illustrates how the elevation of the neutral plane changes with a change in the load, Q_d, applied to the pile head. Notice also that the magnitude of the dragload changes when Q_d changes. The right-hand diagram illustrates the distribution of settlement in the soil as caused by a surcharge on the ground, and/or lowering of the groundwater table, etc., and by the dead load on the pile(s).

Figure 13.10 indicates that the settlement of the pile and the settlement of the soil are equal at the neutral plane. The "kink" in the curve at the neutral plane represents the influence of the dead load on the pile that starts to stress the soil at the neutral plane. If the dead load is zero, the settlement distribution curve has no "kink"and follows the dashed line.

13.9 CAPACITY OF A PILE GROUP

In extending the approach to a pile group, it must be recognized that a pile group is made up of a number of individual piles that have different embedment lengths and that have mobilzed the toe resistance to a different degree. The piles in the group have two things in common, however. They are connected to the same stiff pile cap and, therefore, all pile heads move equally, and the piles must all have developed a neutral plane at the same depth somewhere down in the soil (long-term condition, of course).

Therefore, it is impossible to achieve that the neutral plane is common for the piles in the group, with the mentioned variation of length, etc., unless the dead load applied to the pile head from the cap differs between the piles. Thus, the Unified Method extended to a pile group can be used to discuss the variation of load within a group of stiffly connected piles.

A pile with a longer embedment below the neutral plane or one having mobilized a larger toe resistance as opposed to other piles will carry a greater portion of the dead load on the group. On the other hand, a shorter pile, or one with a smaller toe resistance, as opposed to other piles in the group, will carry a smaller portion of the dead load. If a pile is damaged at the toe, it is possible that the pile exerts a negative—pulling—force at the cap and thus increases the total load on the pile cap. Remember, a dragload will occur without any appreciable settlement in the soil around the piles.

An obvious result of the development of the neutral plane is that no portion of the dead load is transferred to the soil via the pile cap. Unless, of course, the neutral plane lies right at the pile cap and the entire pile group is failing.

13.10 SUMMARY OF DESIGN PROCEDURE FOR CAPACITY AND STRENGTH

The design of a pile or a pile group follows four steps:

a. Compiling and assessing all site and soil information
b. Calculating capacity and distribution of shaft and toe resistances
c. Calculating load-transfer curves determining the neutral plane location
d. Checking that the structural strength is adequate

The calculations are interactive inasmuch as change of the load applied to a pile will change the location of the neutral plane and the magnitude of the maximum load in the pile.

13.10.1 Compiling Site and Soil Information

Compile first into a table all available data useful for reference when determining shaft and toe resistances, while noting the elevation of the groundwater table and the distribution of pore pressures and identifying soil layers of similar properties and expected behavior. Values, such as unit weights, water contents, shear strengths, N-values, etc., should be tabulated.

Then, use the tabulated data to estimate the beta-coefficients, cohesion intercepts (or undrained shear strength values), and N_t factors, as well as appropriate ranges of such values.

13.10.2 Capacity and Allowable Load

Calculate the bearing capacity, Q_u, of a single representative pile as a sum of the shaft and toe resistances, R_s and R_t, according to Equations 13.5 and 13.8 and determine the load distribution curve for a single pile according to Equation 13.7.

Determine the allowable (or factored) load by dividing the pile capacity with a Factor of Safety, F_s, governed by the degree of uncertainty in the given case, or use the applicable Resistance Factor.

In the beginning of a design process, a range of 2.5 through 3.0 is usually chosen for F_s. Later, as more information becomes available, such as capacity determined by means of static or dynamic tests, the value of F_s may be reduced to the range of 1.8 through 2.0. For a discussion on the factor of safety, see Chapter 23 in the *Canadian Foundation Engineering Manual* (1985).

The allowable load, or—in the ULS design—the factored load, includes both permanent (dead; sustained; Q_d) loads and temporary or transient (live; transient; Q_l) loads. It does not include the dragload. (The magnitude of the dragload only affects the structural strength of the pile, not the bearing capacity.)

13.10.3 Load-Transfer Curve, Neutral Plane, and Structural Strength

Starting with the dead load, Q_d, and increasing the load in the pile by adding effect of negative skin friction, q_n, in accordance with Equation 13.10, the long-term load distribution in the pile above the neutral plane is determined. The neutral plane is where the transfer curve according to Equation 13.7 intersects the curve determined according to Equation 13.10. The construction of the neutral plane is illustrated in Figure 13.9.

The maximum load in the pile is the dead load plus the dragload and it occurs at the neutral plane. The maximum load must not be larger than a certain portion of the structural strength of the pile. The limit is governed by considerations different to those applied to the structural strength at the pile cap. It is recommended that for *straight and undamaged piles*, the allowable maximum load at the neutral plane be limited to 70 percent of the pile strength. For composite piles, such as concrete-filled pipe piles, the load should be limited to a value that induces a maximum of 1.0 millistrain into the pile with no material becoming stressed beyond 70 percent of its strength.

13.11 SETTLEMENT OF PILE FOUNDATIONS

13.11.1 Introduction

Settlement occurs as a consequence of a stress increase causing a volume reduction of the subsoil. It consists of the sum of "elastic" compression of the soil skeleton and free gas present in the voids, which occurs quickly and is normally small, and of consolidation, that is, volume change due to the expulsion of water, which occurs quickly in coarse-grained soils, but slowly in fine-grained soils.

Consolidation settlement is due to the fact that the imposed stress, initially carried by the pore water, is transferred to the soil skeleton, which compresses in the process until all the imposed stress is carried by effective stress. In some soils, creep adds to the compression of the soil skeleton. Creep is compression occurring without an increase of effective stress.

Soil materials do not show a linear relation between stress and strain, and settlement is a function of the relative stress increase. The larger the existing stress before an additional stress is applied, the smaller the induced settlement. Cohesive soils, in particular, have a distinct non-linearity. Of course, these statements are given with due consideration to any pre-consolidation pressure.

When analyzing piles, it is important that settlement is not confused with the movement occurring as a result of the transfer of load to the soil, that is, the movement necessary to build up the resistance to the load. In the case of shaft resistance, this movement is small, but substantial movement of the pile toe into the soil may occur before full toe resistance is mobilized.

13.11.2 Conventional Approach

Settlement is calculated as compression due to increase in stress—that is, the difference between the original and the final effective stresses. The increase is normally not constant throughout the soil volume, but a function of the vertical distribution (spreading) of stress. In engineering practice, the distribution under the mid-point of a footing is usually calculated by the 2:1 method according to Equation 13.11:

$$q_z = q_0 \frac{BL}{(B+z)(L+z)} \tag{13.11}$$

where

B = footing width (breadth)
L = footing length
q_0 = applied stress (beneath the footing; at the pile cap)
q_z = applied vertical stress at depth z

The settlement is calculated by dividing the soil profile into layers, calculating for each layer the compression caused by the stress increase. The settlement is then equal to the sum of the compressions of the individual layers. Traditionally, the settlement calculation is treated differently in cohesionless and cohesive soils, as follows.

Cohesionless Soil

In cohesionless soil, the calculation of the settlement is carried out according to Hooke's law, as follows:

$$\varepsilon = \frac{1}{E} q_z \tag{13.12}$$

and

$$S = \Sigma s = \Sigma(\varepsilon h) \tag{13.13}$$

where

ε = strain induced in a soil layer
E = modulus of elasticity
h = thickness of soil layer
s = compression of soil layer
S = settlement for the footing as a sum of the compressions of the soil layers

The "elastic" modulus method for settlement calculation is an over-simplification and results in a highly inaccurate settlement value and use of the method is discouraged. The tangent modulus method described below is a considerably better approach.

Cohesive Soil

For settlement calculation in cohesive soils, it is generally realized that the elastic modulus approach cannot be used. Instead, conventional calculation makes use of a compression index, C_c, and the original void ratio, e_0, to determine the strain, ε, induced in a layer.

Cohesive soils, however, may be consolidated for a higher stress than the actual effective stress. This higher stress is called the preconsolidation stress, σ'_p. The compression of such soils is much smaller for stresses below the preconsolidation stress and it can be calculated using a compression index, C_{cr}. When in overconsolidated soil and with the final stress larger than the preconsolidation stress, strain, ε, is calculated according to Equation 13.14:

$$\varepsilon = \frac{1}{1+e_0}\left[C_{cr}\ln\frac{\sigma'_p}{\sigma'_0} + C_c\ln\frac{\sigma'_1}{\sigma'_p}\right] \tag{13.14}$$

A weakness of Equation 13.14 is that the calculation requires three parameters, C_c, C_{cr}, and e_0, and too often in a project design the compression indices and the void ratio value are incompatible. Again, the tangent modulus method described below is a considerably better approach.

13.11.3 The Janbu Tangent Modulus Approach

Stress–strain relation in a soil is non-linear. For a stress increase from where the original stress in the soil is small, the corresponding increase of strain is larger than where the original stress was larger. That is, the slope of the line, the tangent modulus, M_t, increases with increasing original stress. According to a tangent modulus approach proposed by Janbu (1963, 1965), as referenced by the *Canadian Foundation Engineering Manual* (1985), the relation between stress and strain depends on two non-dimensional parameters that are unique for a soil: a stress exponent, j, and a modulus number, m. For most cases, the stress exponent can be assumed to be either 0, which is representative of cohesive soils, or 0.5, which is representative of cohesionless soils.

In cohesionless soils, $j > 0$, the following simple formula governs:

$$\varepsilon = \frac{1}{mj}\left[\left(\frac{\sigma'_1}{\sigma_r}\right)^j - \left(\frac{\sigma'_0}{\sigma_r}\right)^j\right] \tag{13.15}$$

where

ε = the strain induced by the increase of effective stress
σ'_0 = the original effective stress
σ'_1 = the new effective stress
j = the stress exponent
m = the modulus number, which is determined from testing in the laboratory and/or in the field
σ_r = a reference stress, a constant, = 100 kPa (1 tsf)

In an essentially cohesionless, sandy, silty soil, the stress exponent is close to a value of 0.5. By inserting this value and considering that the reference stress is equal to 100 kPa, the formula is simplified to:

$$\varepsilon = \frac{1}{5m}(\sqrt{\sigma'_1} - \sqrt{\sigma'_0}) \tag{13.16}$$

Notice, Equation 13.16 is not independent of the choice of units. The stress values must be inserted in kPa. That is, a value of 2 MPa is to be inserted as the number 2000 and a value of 300 Pa as the number 0.3. In English units Equation 13.16 becomes:

$$\varepsilon = \frac{2}{m}(\sqrt{\sigma'_1} - \sqrt{\sigma'_0}) \tag{13.16a}$$

Again, the equation is not independent of units. Because the reference stress is 1.0 tsf, Equation 13.16a requires that the stress values are inserted in units of tsf.

If the soil is overconsolidated and the final stress exceeds the preconsolidation stress, Equations 13.16 and 13.16a change to:

$$\varepsilon = \frac{1}{5m_r}(\sqrt{\sigma'_p} - \sqrt{\sigma'_0}) + \frac{1}{5m}(\sqrt{\sigma'_1} - \sqrt{\sigma'_p}) \tag{13.17}$$

$$\varepsilon = \frac{2}{m_r}(\sqrt{\sigma'_p} - \sqrt{\sigma'_0}) + \frac{2}{m}(\sqrt{\sigma'_1} - \sqrt{\sigma'_p}) \tag{13.17a}$$

where

σ'_0 = the original effective stress (kPa or tsf)
σ_p = the preconsolidation stress (kPa or tsf)
σ'_1 = the new effective stress (kPa or tsf)
m = the modulus number (dimensionless)
m_r = the recompression modulus number (dimensionless)

Equation 13.17 requires stress units in kPa and Equation 13.17a in tsf.

In cohesive soils, the stress exponent is zero, $j = 0$. Then, in a normally consolidated cohesive soil:

$$\varepsilon = \frac{1}{m}\ln\left(\frac{\sigma'_1}{\sigma'_0}\right) \tag{13.18}$$

and in an overconsolidated soil:

$$\varepsilon = \frac{1}{m_r}\ln\left(\frac{\sigma'_p}{\sigma'_0}\right) + \frac{1}{m}\ln\left(\frac{\sigma'_1}{\sigma'_p}\right) \tag{13.19}$$

By means of Equations 13.15 through 13.19, settlement calculations can be performed without resorting to simplifications such as that of a constant elastic modulus. Apart from knowing the original effective stress and the increase of stress plus the type of soil involved, without which knowledge no settlement analysis can ever be made, the only soil parameter required is the modulus number. The modulus numbers to use in a particular case can be determined from conventional laboratory testing, as well as in-situ tests. As a reference, Table 13.3 shows a range of conservative values typical for various soil types, which is quoted from the *Canadian Foundation Engineering Manual* (1985).

In a cohesionless soil, where previous experience exists from settlement analysis using the elastic modulus approach (Eq. 13.12), a direct conversion can be made between E and m, which results in Equation 13.20 when using SI-units—stress and E-modulus in kPa:

$$m = \frac{E}{5(\sqrt{\sigma'_1} + \sqrt{\sigma'_0})} = \frac{E}{10\sqrt{\sigma'}} \tag{13.20}$$

When using English units and stress and E-modulus in tsf, Equation 13.20a applies:

$$m = \frac{2E}{(\sqrt{\sigma'_1} + \sqrt{\sigma'_0})} = \frac{E}{\sqrt{\sigma'}} \tag{13.20a}$$

Notice, most natural soils have aged and become overconsolidated with an overconsolidation ratio, OCR, that often exceeds a value of 2. The recompression modulus, m_r, is often five to ten times greater than the virgin modulus, m, listed in the table.

In a cohesive soil, unlike the case for a cohesionless soil, no conversion is required as the traditional and the tangent modulus approaches are identical, although the symbols differ. Thus, values from the C_c and e_0 approach are immediately transferable via Equation 13.21:

$$m = \ln 10\left(\frac{1 + e_0}{C_c}\right) = 2.30\left(\frac{1 + e_0}{C_c}\right) \tag{13.21}$$

In cohesive soils, the Janbu tangent modulus approach is much preferred to the C_c and e_0 approach because, when m is determined directly from the testing, the commonly experienced difficulty is eliminated of finding out what C_c value goes with what e_0 value.

TABLE 13.3 TYPICAL AND NORMALLY CONSERVATIVE MODULUS NUMBERS.

Soil Type	*Modulus Number*	*Stress Exponent, j*
Till, very dense to dense	1000–300	1
Gravel	400–40	0.5
Sand		
Dense	400–250	0.5
Compact	250–150	0.5
Loose	150–100	0.5
Silt		
Dense	200–80	0.5
Compact	80–60	0.5
Loose	60–40	0.5
Clays		
Silty clay and clayey silt		
Hard, stiff	60–20	0.5
Stiff, firm	20–10	0.5
Soft	10–5	0.5
Soft marine clays and organic clays	20–5	0
Peat	5–1	0

13.11.4 Calculation of Pile Group Settlement

The neutral plane is, as mentioned, the location where there is no relative movement between the pile and the soil. Consequently, whatever the settlement in the soil is in terms of magnitude and vertical distribution, the settlement of the pile head is equal to the settlement of the soil at the neutral plane plus the compression of the pile caused by the applied dead load and the dragload combined.

The simplest method for calculating the settlement of the pile group at the location of the neutral plane is by calculating the settlement for a footing equal in size to the pile cap, placed at the location of the neutral plane, and imposing a stress distribution equal to the permanent (dead) load on the pile cap divided by the footing area. The settlement calculation must include the effect of all changes of effective stress in the soil, not just the load on the pile cap. Notice that the load giving the settlement is the permanent load acting on the pile cap and that neither the live load nor the dragload are included in the settlement calculation.

For a dominantly shaft-bearing pile "floating" in a homogeneous soil with linearly increasing shear resistance, the neutral point lies at a depth which is about equal to the lower third point of the pile embedment length. It is interesting to note that this location is also the location of the equivalent footing according to the Terzaghi–Peck approach illustrated in Figure 13.3. (The assumptions behind the third-point location are that the unit negative skin friction is equal to the positive shaft resistance, that the toe resistance is small, and that the load applied to the pile head is about a third of the bearing capacity of the pile.)

Assume that the distribution of settlement in the soil around the pile is known and follows the "settlement" diagram in Figure 13.10. As illustrated in the diagram for the case of the middle service load, by drawing a horizontal line from the neutral plane to intersection with the settlement curve, the settlement of the pile at the neutral plane can be determined and, thus, the settlement of the pile head. The construction in the figure is valid both for a small settlement that diminishes quickly with depth and for a large settlement that continues to be appreciable well below the pile toe.

The construction in Figure 13.10 has assumed that the induced toe movement (toe displacement) is sufficiently large to fully mobilize the toe resistance. As stated, the movement between the shaft and the soil is always large enough to mobilize the shaft shear—negative skin friction or positive shaft resistance—but if the soil settlement is small, it is possible that the toe movement is not large enough to mobilize the full toe resistance. In such a case, the neutral plane moves to a higher location as determined by the particular equilibrium condition.

In a pile group connected with a stiff cap, all piles must settle an equal amount and the elevation of the neutral plane must be equal for the piles in the group. (The individual capacities may vary, and, therefore, the permanent load actually acting on an individual pile will vary correspondingly.) Then, according to Fellenius (1984, 1989), the settlement of the group is determined as the settlement of an equivalent footing located at the elevation of the neutral plane with the load spreading below the equivalent footing calculated by the 2:1 method.

13.11.5 Summary of Settlement Calculation

Step 1. The soil profile is assessed and divided into layers for calculation, which requires that pertinent soil parameters are assigned to each layer.

Step 2. Calculation of settlement of a pile group requires the prior calculation of bearing capacity including the distribution of load and resistance along the piles, which determines the location of the neutral plane.

Step 3. The pile group is replaced by an equivalent footing at the neutral plane and the increase of stress below the equivalent footing caused by the dead load on the pile group is calculated using the 2:1 method. This stress is added to the change of effective stress caused by other influences, such as fill, excavation, and groundwater lowering.

Step 4. The settlement of each soil layer below the neutral plane as caused by the change of effective stress is determined using the tangent modulus approach and the values are summed to give the soil settlement at the neutral plane. The settlement of the pile group is this value plus the compression of the pile for the dead load and the dragload.

Step 5. Inasmuch as the determination of the neutral plane made use of a fully developed toe resistance, a check is made of the magnitude of settlement calculated below the pile toe. If this value is smaller than about 5 percent of the pile diameter, Step 2 is repeated using an appropriately smaller toe resistance to arrive at a new location of the neutral plane (higher up) and followed by a repeat of Steps 3 through 5, as required.

13.11.6 Special Aspects

The dragload must not be included when considering bearing capacity, that is, the analysis of soil bearing failure. Consequently, for bearing capacity consideration, it is incorrect to reduce the dead load by any portion of the dragload.

The dead load should only be reduced owing to insufficient structural strength of the pile at the location of the neutral plane, or by a necessity to lower the location of the neutral plane in order to reduce the amount of settlement.

Normally, when the pile capacity is reliable, that is, it has been determined from results of a static loading test or analysis of data from dynamic monitoring, a factor of safety of 2 ensures that the neutral plane is located below the mid-point of the pile.

In the design of a pile foundation, provided that the neutral plane is located deep enough in the soil to eliminate settlement concerns for the piles, the settlement of the surrounding soils (and the negative skin friction) are of no concern directly for the pile group. However, where large settlement is expected, it is advisable to avoid inclined piles in the foundation, because piles are not able to withstand lateral or sideways movement and the settlement will bend an inclined pile.

Piles that are bent, doglegged, or damaged during the installation will have a reduced ability to support the service load in a downdrag condition. Therefore, the unified design approach postulates that the pile installation is subjected to stringent quality control directed toward ensuring that the installed piles are sound and that bending, cracking, and local buckling do not occur.

When the design calculations indicate that the settlement could be excessive, increasing the pile length or decreasing the pile diameter could improve the situation. When the calculations indicate that the pile structural capacity is insufficient, increasing the pile section, or increasing the strength of the pile material could improve the situation. When such methods are not practical or economical, the negative skin friction can be reduced by the application of bituminous coating or other viscous coatings to the pile surfaces before the installation, as demonstrated by Bjerrum et al. (1969). (See also Fellenius, 1975a, 1979; Clemente, 1981.) For cast-in-place piles, floating sleeves have been used successfully. It should be recognized, however, that measures such as bitumen coating and sleeves are very expensive, and they should only be considered when other measures for lowering the neutral plane have been shown to be impractical.

The unified design approach shares one difficulty with all other approaches to pile group design, viz., that there is a lack of thorough and representative full-scale observations of load distribution in piles and of settlement of pile foundations. For settlement observations, the lack is almost total with respect to observations of settlement of both the piles and the soil adjacent to pile foundations.

In a typical design case, the shaft and toe resistance for a pile can only be estimated within a margin. To provide the profession with reference cases for aid in design, it is very desirable that sturdy and accurate load cells be developed and installed in piles to register the load distribution in the pile during, immediately after, and with time following the installation. Naturally, such cells should be placed in piles subjected to static loading tests, but not exclusively in these piles (see Dunnicliff, 1982, 1988).

The greatest perceived need lies in the area of settlement observations. It is paradoxical that although pile foundations are normally resorted to for reasons of excessive settlement, the design is almost always based on a capacity rationale, with disregard of settlement. To improve this situation, full-scale and long-term field observation cases are needed. Actual pile foundations should be instrumented to determine both the settlement of the piles and the distribution of settlement in the soil near the piles. No instrumentation for study of settlement should be contemplated without the inclusion of piezometers.

13.12 STATIC TESTING OF PILES

The axial compression testing of a vertical single pile is the most common test performed. However, despite the numerous tests that have been carried out and the many papers that have reported on such tests and their analyses, the understanding of static pile testing in current engineering practice leaves much to be desired. The reason is that engineers have concerned themselves with mainly one question, only—"does the pile have a certain least capacity?"—finding little of practical value in analyzing the pile–soil interaction. However, considerable engineering value can be gained from routine elaboration on a pile test, during the actual testing in the field, as well as in the analysis of the results.

13.12.1 Testing Methods

A static loading test is performed by loading a pile with a gradually or stepwise increasing force, while monitoring the movement of the pile head. The force is obtained by means of a hydraulic jack reacting against a loaded platform or anchors.

The American Society for Testing and Materials, ASTM, publishes three standards, D-1143, D-3689, and D-3966 for static testing of a single pile in axial compression, axial uplift, and lateral loading, respectively.

The ASTM standards detail how to arrange and perform the pile test. Wisely, they do not include how to interpret the tests, because this is the responsibility of the engineer in charge, who is the only one with all the site- and project-specific information necessary for the interpretation.

The most common test procedure is the slow maintained load method referred to as the "standard loading procedure" in the ASTM Designation D-1143 and D-3689 in which the pile is loaded in eight equal increments up to a maximum load, usually twice a predetermined allowable load. Each load level is maintained until zero movement is reached, defined as 0.25 mm/h (0.01 in/h). The final load, the 200 percent load, is maintained for a duration of 24 h. The "standard method" is very time consuming, requiring from 30 to 70 h to complete. It should be realized that the words "zero movement" are very misleading: the "zero" movement rate is equal to a movement of more than 2 m (7 ft) per year!

Each of the eight load increments is placed onto the pile very rapidly; as fast as the pump can raise the load, which usually takes about 20 seconds to 2 minutes. The size of the load increment in the "standard procedure", 12.5 percent of the maximum load, means that each such increase of load is a shock to the pile and the soil. Smaller increments that are placed more frequently disturb the pile less, and the average increase of load on the pile during the test is about the same. Such loading methods provide more consistent, reliable, and representative data for analysis.

Tests that consist of load increments applied at constant time intervals of 5, 10, or 15 minutes are called Quick Maintained-Load Tests or just "Quick Test". In a Quick Test, the maximum load is not normally kept on the pile longer than any other load before the pile is unloaded. Unloading is done in about five steps of no longer duration than about 1 minute. The Quick Test allows for attempting to apply one or more load increments beyond the minimum number that the particular test is designed for, that is, making use of the margin built into the test. In short, the Quick Test is from technical, practical, and economical points of view superior to the "standard loading procedure".

A Quick Test should aim for 25 to 40 increments with the maximum load determined by the amount of reaction load available or by the capacity of the pile. For routine cases, it may be preferable to stay at a maximum load of 200 percent of the intended allowable load. For ordinary test arrangements, where only the load and the pile head movement are monitored, time intervals of 10 minutes are suitable and allow for the taking of two to four readings for each increment. When testing instrumented piles, where the instruments take a while to read (scan), the time interval may have to be increased. To go beyond 20 minutes, however, should not be necessary. Nor is it advisable, because of the potential risk of the influence of time-dependent movements, which may impair the test results. Usually, a Quick Test is completed within 3 to 6 h.

For a description of constant-rate-of-penetration and cyclic methods, see Fellenius (1975b, 1980) and references contained therein.

In routine tests, cyclic loading, or even single unloading and loading phases must be avoided, as they do little more than destroy the possibility of a meaningful analysis of the test results. There is absolutely no logic in believing that anything of value on load distribution and toe resistance can be obtained from an occasional unloading or from one or a few "resting periods" at certain load levels, when considering that we are testing a unit that is subjected to the influence of several soil types, is subjected to residual stress of unknown magnitude, exhibits progressive failure, etc., and when all we know is what is applied and measured at the pile head.

13.12.2 Interpretation of Failure Load

For a pile that is stronger than the soil, the failure load is reached when rapid movement occurs under sustained or slightly increased load (the pile plunges). However, this definition is inadequate, because plunging requires very large movements and it is often less a function of the capacity of the pile–soil system and more a function of the man–pump system.

To be useful, a definition of failure load must be based on some mathematical rule and generate a repeatable value that is independent of scale relations and the opinions of the individual interpreter. Furthermore, it has to consider the shape of the load–movement curve or, if not, it must consider the length of the pile (which the shape of the curve indirectly does).

Davisson (1972) proposed a load limit defined as the load corresponding to the movement that exceeds the elastic compression of the pile by an offset of 4 mm (0.15 in) plus a value equal to the diameter of the pile divided by 120. For example, the offset value for a pile toe diameter of 300 mm (12 in) is 6 mm (0.25 in).

Brinch-Hansen (1963) proposed an 80 percent criterion defining the ultimate load as the load that gives four times the movement of the pile head as obtained for 80 percent of that load. Usually, the 80 percent criterion agrees well with the perceived "plunging failure."

In applying the general work by Kondner (1963), Chin (1970, 1971) proposed a method in which each applied load is divided by its corresponding movement and the resulting number is plotted against the movement. After some initial variation, the plotted values fall on a straight line. The inverse slope of this line is the Chin failure load.

The three methods mentioned for determining failure load are included in the *Canadian Foundation Engineering Manual* (1985). Details on the application of the methods in engineering practice have been presented by Fellenius and Rasch (1990). Figure 13.11 illustrates the variation of the three and six other methods (Fellenius, 1980), when applied to the results of a static loading test on a 40 m (130 ft) long 300 mm (12 in) diameter pile in clay and silt. As shown, the Davisson limit of 2130 kN (240 tons) is lower than all the others and the Chin value of 2930 kN (330 tons) is the highest. The other seven values are grouped more or less together around an average of 2400 kN (270 tons).

It is difficult to make a rational choice of the best criterion to use, because the preferred criterion depends strongly on a person's past experience. In the case of an engineering report, the preference and experience of the receiver of the report may also influence what criterion to choose.

The Davisson limit has the merit of allowing the engineer, when proof testing a pile for a certain allowable load, to determine *in advance* the maximum allowable movement for this load with consideration of the length and size of the pile. Thus, as proposed by Fellenius (1975b), contract specifications can be drawn up including an acceptance criterion for piles proof tested according to quick testing methods. The specifications can simply call for a test to at least twice the design load, as usual, and declare that at a test load equal to a factor F times the design load the movement shall be smaller than the Davisson offset from the elastic column compression of the pile. Normally, F would be chosen within a range of 1.8 to 2.0. The acceptance criterion could be supplemented with the requirement that the safety factor should also be smaller than a certain minimum value calculated on pile bearing failure defined according to the 80 percent criterion or other preferred criterion.

The Brinch-Hansen 80 percent criterion usually gives a Q_u value that is close to what one subjectively accepts as the true

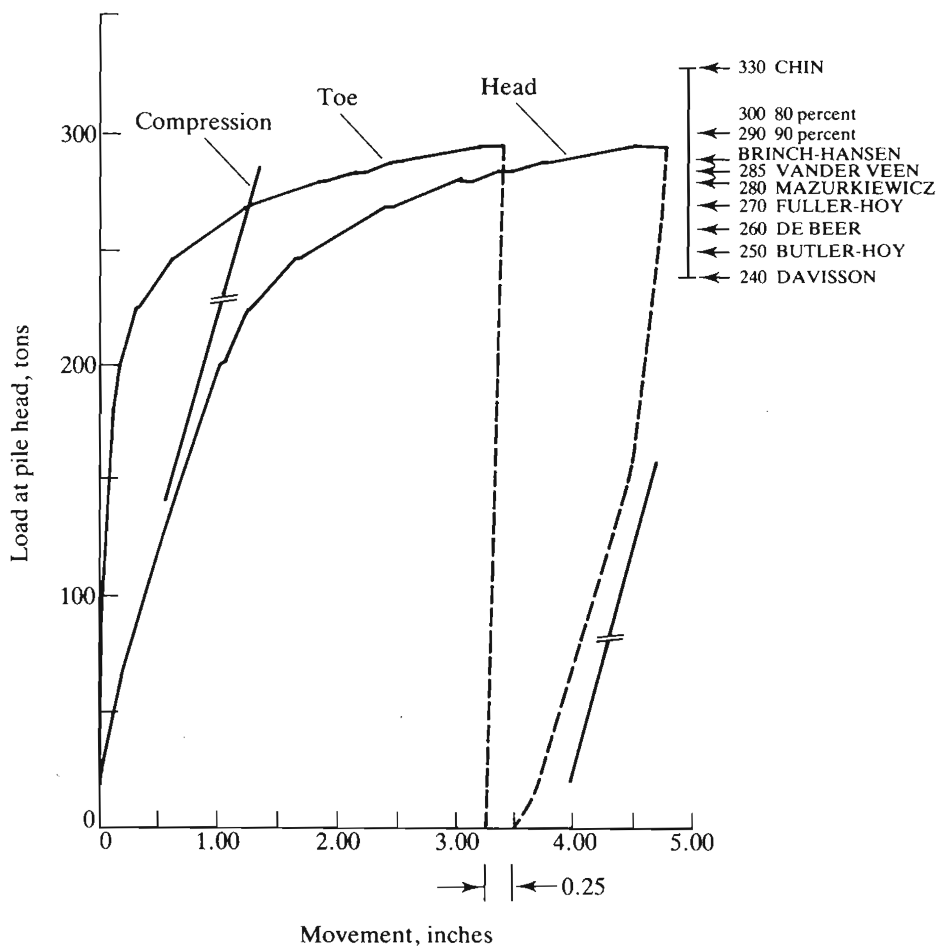

Fig. 13.11 Load–movement diagram from a quick maintained-load static loading test with measurement of pile compression and toe movement. (*After Fellenius, 1980.*)

ultimate failure value. However, the criterion is very sensitive to inaccuracies of the test data.

The Chin method allows a continuous check on the test, if a plot is made as the test proceeds, and a prediction of the maximum load that it will be possible to apply during the test. Sudden kinks or slope changes in the Chin line indicate—give an early warning—that something is amiss with either the pile or with the test arrangement (Chin, 1978).

13.12.3 Influence of Errors

A static loading test is usually considered a reliable method for determining the capacity of a pile. However, even when using new manometers and jacks that have been calibrated together, the applied loads are usually substantially overestimated. The error is usually about 10 to 15 percent of the applied load. Errors as large as 30 to 40 percent are not uncommon. The diagram in Figure 13.12 is from an actual field test and it is representative of the error commonly encountered in a routine static loading test.

The reason for the error is that the jacking system is required to do two things at the same time, that is, both to provide the load and to measure it, and load cells with moving parts are considerably less accurate than those without moving parts. For instance, when calibrating testing equipment in the laboratory, it is ensured that no eccentric loading, bending moments, or temperature variations influence the calibration. In contrast, all of these adverse factors are at hand in the field and influence the test results to an unknown extent, unless a load cell is used.

It is inconceivable that the foundation engineering practice can continue to perform static loading tests with potential errors as large as those that usually occur. Therefore, it is absolutely essential that a load cell be used in all tests. Of course, the jack pressure should still be measured and be used as a back-up.

The above deals with the error of the applied load. The error in movement measurement can also be critical. Such errors do not originate in the precision of the reading (the usual precision is more than adequate) but in undesirable influences, such as heave or settlement of the reference beam during unloading the ground when loading the pile. For instance, one of the greatest villains known to spoil a loading test is the sun: the reference beam must be shielded from sunshine at all times.

It must be remembered that the minimum distances from the supports of the reference beam to the pile and the platform, etc., as recommended in the ASTM standards, really are minimum values, which most often do not give errors of much concern for ordinary testing but which are too short for research or investigative testing purposes.

13.12.4 The Analysis of Results Using Telltale Data

In the routine static loading test, measurements are taken at the pile head only. Yet, in the interpretation of the test results, what is of interest is the distribution of load in the pile and especially at the pile toe. It is impossible to estimate with any worthwhile accuracy the mobilized toe resistance from load–movement data obtained at the pile head.

The test can be substantially enhanced by placing telltales in the pile. A telltale is a rod with its lower end connected to the pile, usually at the toe, and free from the pile along its overall length by means of a guidepipe arrangement. By attaching a dial gage at the upper end of the rod and measuring the change of distance between the rod top and the pile head, the shortening of the pile during the test is monitored. The

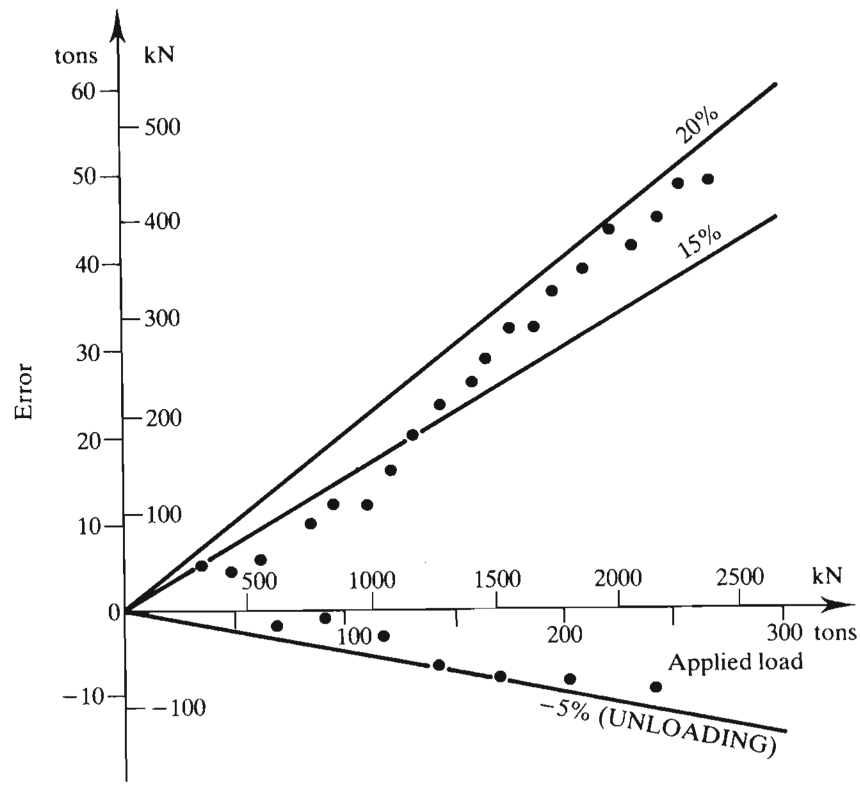

Fig. 13.12 Example of error of load encountered in a routine static loading test. (*After Fellenius, 1984b.*)

movement of the pile toe is obtained as the measured pile shortening subtracted from the movement of the pile head.

In the static loading test on a precast concrete pile, for which results are presented in Figure 13.11, a guidepipe for a telltale had been cast in the pile, allowing a telltale to be inserted to the pile toe after the driving to monitor the compression of the pile and the pile toe movement. With use of some foresight and planning, telltales can be installed rather easily and cheaply in all types of piles. The mentioned ASTM standards include suggestions for telltale arrangements.

The measured compression divided by the telltale length gives the strain of the pile for the applied load, which when combined with the Young's modulus of the pile material results in the average load in the pile over the telltale length. In the case of a constant unit shaft resistance, the average load is equal to the load in the middle of the pile, or the middle of the telltale length. In the case of a linearly increasing unit shaft resistance, the average load is equal to the load at a level lying somewhere between the mid-point and the upper third point. Obviously, knowledge of the distribution of the shaft resistance is essential for the evaluation of the load distribution.

However, an estimation of the toe resistance can be made from the measured shortening of a telltale to the pile toe. The following relations were given by Fellenius (1980) and build on the assumption of constant unit shaft resistance acting along the full length of the pile (the telltale length):

$$Q_{ave} = AE\frac{\Delta L}{L} \tag{13.22a}$$

$$R_t = 2Q_{ave} - Q_{head} \tag{13.22b}$$

$$R_s = Q_h - R_t \tag{13.22c}$$

where

Q_{ave} = average load in the pile
A = cross-sectional area of the pile
ΔL = measured shortening of the pile
L = pile or telltale length
R_t = toe resistance
Q_{head} = load applied to the pile head
R_s = shaft resistance

For linearly increasing unit shaft resistance, the relation for the toe resistance becomes:

$$R_t = 3Q_{ave} - 2Q_{head} \tag{13.23}$$

By means of Equations 13.22 and 13.23, a range of toe resistance values can be bounded by the two extremes of constant and linearly increasing unit shaft resistance. Furthermore, by means of adding a small value to the measured shortening of the pile, one can include an analysis of the effect on the calculated toe and shaft resistances of a residual load acting on the pile before the static loading test was started.

Having several telltales in a pile results in several values of average load, because three average values of load result from measurement taken by two telltales; the third load value is obtained from the difference in compression measured over the distance between the two telltale ends connected to the pile. Correspondingly, having three telltales results in six load values. There is a practical limit, because from primarily practical considerations of accuracy, it is not worthwhile having telltale lengths and distances shorter than about 10 m (30 ft).

Leonards and Lovell (1978) presented an analysis method for determining the load distribution in a pile instrumented with one telltale, where only the relative distribution of unit shaft resistance needs to be known, or the upper and lower ranges of it in the case of a boundary-type analysis. The shaft resistance does not need to be uniform, but can be of any irregular distribution. Lee and Fellenius (1990) developed the Leonard–Lovell method to include a simultaneous analysis of two or three telltales, which allows a computation of residual load in the pile and its influence on the test results.

When using telltales, the accuracy of the compression measurements must be several times better than the accuracy usually accepted for movement measurements. The nominal precision of measurements of movement using dial gages is usually only 0.025 mm (0.001 in). On special occasions for recording compression using telltales, dial gages with a ten times finer reading precision are used. The actual accuracy of the values is, of course, smaller than the precision, even when neglecting influences on the measuring beam. At best, when using mechanical gages, the error is about 0.1 mm (0.005 in) or larger. The ten times finer gages will have a smaller error, but not ten times smaller.

It is necessary to have dial gages with stems that are long enough to allow the telltale records to be taken during the entire test without having to reset the gages or to shim them, otherwise, errors are introduced that will destroy the value of the records.

Even if movement and compression measurements are complementary readings from the view of mathematics in an analysis, when using telltales, the telltales should always measure compression directly, not movement, because obtaining compression as difference between two measurements of movement introduces large errors.

Apart from the obvious fact that results of an analysis of telltale measurements depend primarily on the accuracy of the measurements, the analysis introduces the modulus of the pile material and the results depend also on how accurately the modulus is known. Steel has a constant modulus and steel piles are very suitable for telltale instrumentation. Concrete, however, does not have a constant modulus over the stress range considered in a static loading test. Therefore, telltale measurements in concrete piles and concreted steel pipe piles are difficult to analyze.

In the analysis of telltale compression measurements from a pile having a stress (or strain) dependent modulus, a diagram should be made showing increment of load over increment of strain plotted against strain, that is, observed tangent (chord) modulus versus strain (Fellenius, 1989b). In Figure 13.13 is shown such a plot, which is from the test of a long prestressed concrete pile tested in a quick maintained-load test to a load exceeding 4500 kN (500 tons). The upper diagram shows load–strain as measured in two telltales and the lower diagram shows the modulus versus strain determined from the data.

As can be seen, after an applied load of about 2000 kN (200 tons), the tangent modulus plots become approximately linear and sloping toward reducing values. The tangent modulus in the beginning of the test is about twice that at the end of the test (from about 38 GPa to about 16 GPa; 5500 ksi to 3900 ksi). The tangent modulus can be obtained from linear regression of the straight line, which solves the coefficients a and b in the following equation:

$$E_t = A\frac{d\sigma}{d\varepsilon} = a\varepsilon + b \tag{13.24}$$

where

E_t = tangent modulus
A = cross-sectional area of the pile
σ = stress in the pile
ε = strain in the pile
a = slope of the tangent modulus line
b = initial tangent modulus; y intercept

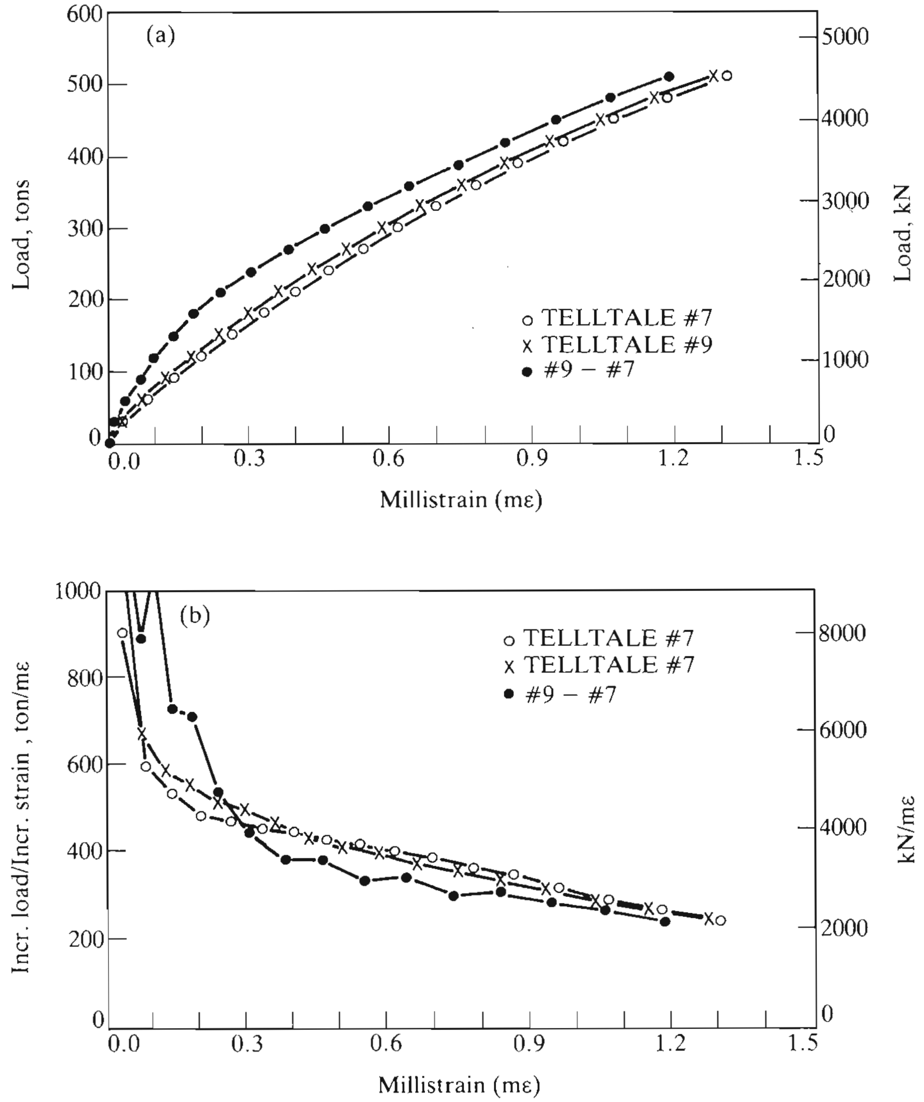

Fig. 13.13 Diagrams of (a) load versus strain and (b) tangent modulus versus strain. (*After Fellenius, 1989b.*)

The initial tangent modulus, term b in Equation 13.24, is best determined from the strain measured over an upper portion of the pile, where the load is affected the least by shaft resistance. Alternatively, and better still, strain should be measured over a free-standing (sleeved off) portion of the pile, because the smaller the shaft resistance, the smaller the error in term b.

On integrating the line, an equation for the stress–strain curve of the pile as a free-standing column is obtained:

$$\sigma = \frac{1}{2} a\varepsilon^2 + b\varepsilon \tag{13.25}$$

Equation 13.25 gives the stress–strain relation for the pile as a column, that is, without the influence of shaft resistance, and can be used for evaluating the average load over the telltale length directly from the measured strain. Alternatively, the secant modulus is determined, as follows:

$$E_s = \frac{1}{2} a\varepsilon + b \tag{13.26}$$

and the load in the pile for a certain induced strain is:

$$Q = AE_s\varepsilon \tag{13.27}$$

The tangent modulus plot can also be used to evaluate the shaft resistance acting on the pile by making use of the fact that the plot becomes linear when all shaft resistance has been mobilized.

It will be obvious for anyone trying the tangent modulus plot in an analysis that only eight load levels to calculate from are too few and the values are too far apart; the "standard procedure" is not suitable for analysis.

The primary value of telltales is for measuring the toe movement. For evaluating the load in the tested pile, the accuracy of the compression measurements must be very high. This means that mechanical type dial gages, even those with high-precision gradation, are not suitable. The use of linear voltage displacement transducers, LVDTs, is to be preferred.

The tangent modulus analysis shown above is equally suitable to direct measurement of strain, of course. Using strain gages in a test requires more knowhow than using telltales. Therefore, an experienced instrumentation specialist should be included in the project team early in the planning of the test.

When planning a static loading test and considering the inclusion of telltales to provide data for use in the analysis of load distribution, it is recommended that the telltales be limited to one to the toe and one back-up placed, say 10 m (30 ft) above the toe. The rest of the instrumentation for measuring strain should be electrical strain gages. Furthermore, telltale data intended for load distribution analysis must be obtained with an accuracy much greater than normally used for measurements during a static loading test. The minimum precision of the dial gages is 0.01 mm (0.0004 in).

13.13 PILE DYNAMICS

13.13.1 Wave Mechanics

The penetration resistance of driven piles provides a direct means of determining bearing capacity of a pile. In impacting a pile, a short-duration force wave is induced in the pile, giving the pile a downward velocity and resulting in a small penetration of the pile. Obviously, the larger the number of blows necessary to achieve a certain penetration, the stronger the soil. Using this basic principle, a large number of so-called pile driving formulae have been developed for determining pile bearing capacity. All these formulae are based on equalizing potential energy available for driving in terms of weight of hammer times its height of fall (stroke) with the capacity times penetration (set) for the blow. The set often includes a loss term.

The principle of the dynamic formulae is fundamentally wrong as wave action is neglected along with a number of other aspects influencing the penetration resistance of the pile. Nevertheless, pile driving formulae have been used for many years and with some degree of success. However, success has been due less to the theoretical correctness of the particular formulae used and more to the fact that the users possessed adequate practical experience to go by. It is mainly when applied to single-acting hammers that use of a dynamic formula may have some justification. Dynamic formulae are the epitome of an outmoded level of technology and they have been replaced by modern methods, such as the wave equation analysis and dynamic measurements, which are described below.

Pile-driving formulae or any other formula applied to vibratory hammers are based on a misconception. Vibratory driving works by eliminating resistance to penetration, not overcoming it. Therefore, records of penetration combined with frequency, energy, amplitudes, etc., can only relate to the resistance not eliminated, not to the static pile capacity after the end of driving.

Pile-driving hammers are rated by the maximum potential energy determined as the ram weight times the maximum ram travel. However, diesel hammers and double-acting air/steam hammers, but also single-acting air/steam hammers, develop their maximum potential energy only during favorable combinations with the pile and the soil. Then, again, the energy actually transferred to the pile may vary due to variation in cushion properties, pile length, toe conditions, etc. Therefore, a relation between the hammer rated energy and measured transferred energy provides only very little information on the hammer. More information is obtained by relating the energy ratio to the actual potential energy, either measured directly (Likins and Rausche, 1988; Hannigan and Webster, 1988) or—for single-acting diesel hammers—determined from the blow rate. Figure 13.14 illustrates the relation between the blow rate and the hammer stroke. With the stroke known, the potential energy of the hammer in actual operation is obtained by multiplying the stroke by the weight of the ram.

About ten years ago, the wave equation analysis had developed sufficiently that it became a tool for general use by the profession. This constituted a big leap in the understanding of pile driving, because, in contrast to the pile-driving formula, the wave equation is theoretically correct and an analysis includes all aspects influencing the pile driving and penetration

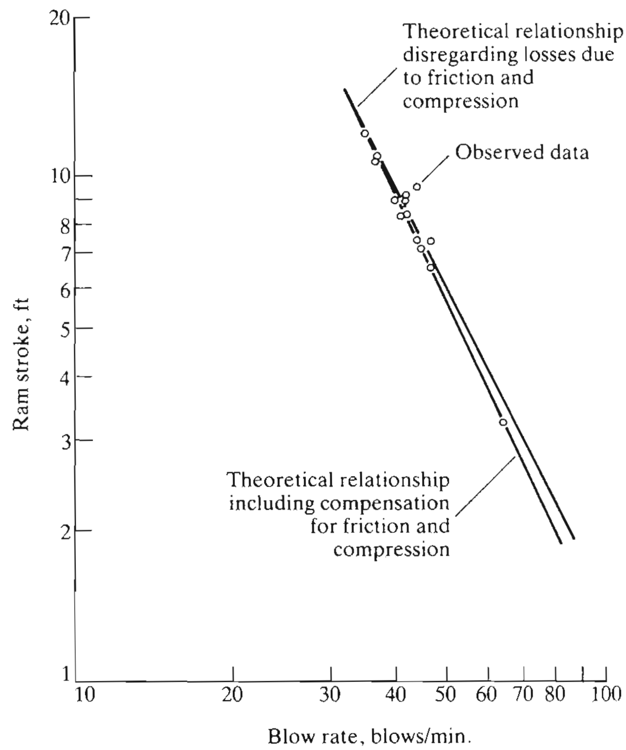

Fig. 13.14 Relationship between ram stroke and blow rate for a single-acting diesel hammer. (*After Fellenius et al., 1978.*)

resistance: hammer mass and travel, combustion in a diesel hammer, helmet mass, cushion stiffness, hammer efficiency, soil strength, viscous behavior of the soil, elastic properties of the pile, to mention some. The commercially available wave equation programs are simple and fast to use and there is, therefore, not even a justification of convenience in continued use of dynamic formulae.

However, the parameters used as input into a wave equation program are really variables with certain ranges of values and the number of parameters included in the analysis is large. Therefore, the result of an analysis is only qualitatively correct, and not necessarily quantitatively correct, unless it is correlated to observations.

The full power of the wave equation analysis is only realized when combined with dynamic measurements during pile driving by means of transducers attached to the pile head. The impact by the pile-driving hammer produces strain and acceleration in the pile which are picked up by the transducers and transmitted via a cable to a data acquisition unit (the Pile Driving Analyzer), which is placed in a nearby monitoring station. The acquisition translates strain and acceleration measurements into force and velocity, displaying these graphically on an oscilloscope. Simultaneously and blow for blow, the energy transferred to the pile is calculated and an estimation is obtained of the bearing capacity of the pile. Impact force, maximum compression force, and maximum tensile force are determined and printed out on a strip chart. The complete procedure is described in the American Society for Testing and Materials Standard for Dynamic Measurements, ASTM D-4945.

The immediate results are usually presented in the form of a "wave trace", which shows the measured force and velocity drawn against time as illustrated in Figure 13.15. As is the convention, the time unit is given in units of L/c, that is, the time it takes for the wave to travel the length of the pile. At time $2L/c$, therefore, the traces show the reflections originating

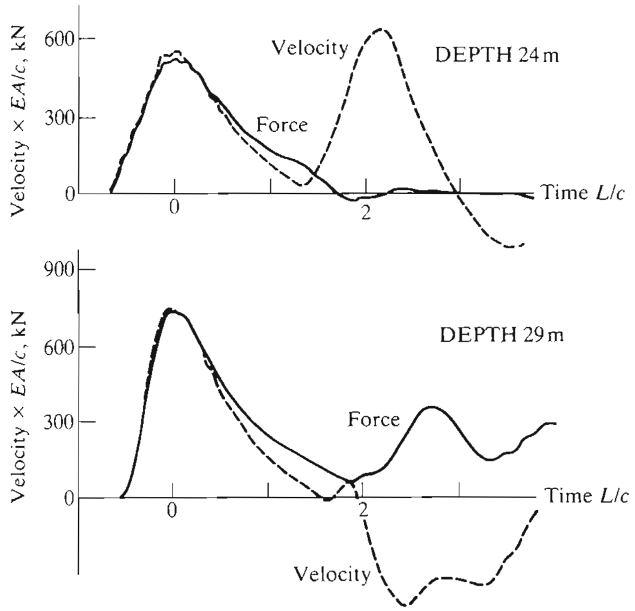

Fig. 13.15 Force and velocity wave traces during easy and hard driving. (*After Authier and Fellenius, 1983.*)

from the pile toe. The peak at zero time (zero L/c) is defined as the point of impact. Because of the wave action, hammer impact force is transmitted to the pile before this time, as well as a considerable time thereafter.

At first, force and velocity are proportional by the so-called acoustic impedance, a material constant (equal to AE/c; the product of the cross-sectional area and the elastic modulus over the wave propagation velocity). Therefore, when velocity and force are plotted to scale of the ratio of impedance, the force and velocity traces at first plot on top of each other. When the impact wave (stress-wave or strain-wave) traveling down the pile meets soil resistance, a reflection of the wave occurs that travels back up the pile. This reflected wave will superimpose on the downward wave, which has the effect of increasing stress at the location of the monitoring transducers and decreasing the velocity. Thus, the two traces will separate with the amount of separation proportional to the soil resistance encountered: at first shaft resistance and, finally, at time $2L/c$, also toe resistance. If there is no or only little resistance at the pile toe, the reflected wave will be in tension and have the effect that the measured velocity increases and the force decreases.

The wave traces shown in Figure 13.15 are taken from the driving of a 43 m long, 460 mm diameter, octagonal shape, concrete pile. The upper set of traces, depth 24 m, is from relatively easy driving. The lower set, depth 29 m, is from when the pile toe entered dense soils. Before time $2L/c$, both sets of traces show only little separation between the force and velocity traces, which is indicative of small shaft resistance acting on the pile. For the upper, "easy-driving" diagram, the traces at time $2L/c$ indicate a velocity peak and essentially zero force, that is, a tension reflection from the pile toe. The lower, "hard-driving" diagram shows a force reflection at time $2L/c$ and a negative velocity, the pile "bounces up" which is indicative of toe resistance. Thus, the traces provide valuable qualitative information on the distribution and magnitude of the soil resistance.

The dynamic measurements can also provide quantitative information of pile static capacity. In the mid-1960s, Dr. G. G. Goble and coworkers of Case Western Reserve University derived a simple relation for calculating capacity from the values of force and velocity at times $0L/c$ and $2L/c$. In words, the resistance to penetration is equal to the mean of the forces at the two times plus the velocity change between the two times multiplied by the impedance (EA/c) of the pile (Rausche et al., 1985). The static capacity is obtained by subtracting a velocity-dependent portion calculated using a damping factor. The static capacity value is called the Case Method Estimate (of capacity). The method has of course been substantially developed since its first derivation and it is today the mainstay of the capacity determination of dynamically monitored piles.

The dynamic measurements can also be used to investigate damage and defects in the pile, such as voids, cracks, spalling, local buckling, etc. (Rausche and Goble, 1978; Rausche et al., 1988; Middendorp and Reiding, 1988).

13.13.2 CAPWAP Analysis

Dynamic records are routinely stored using a tape recorder (or similar unit) or digitized to a computer for renewed analysis. This enables a more time-consuming and detailed analysis to be performed called the CAPWAP signal matching analysis (Rausche et al., 1972). The CAPWAP analysis provides, first of all, a calculated static capacity and the distribution of resistance along the pile. However, it also provides several additional data, for example, the movement necessary to mobilize the full shear resistance in the soil (the quake) and damping values for input in a wave equation analysis. The principle and the procedure of the CAPWAP analysis signal matching are as follows.

As mentioned, the force and velocity induced by the hammer are proportional via the pile impedance (EA/c) and force and velocity react differently to the reflected wave: force increases and velocity decreases. The resulting separation of the two plots is, therefore, an indication of the size of the resistance: dynamic and static together.

The two measurements, force and velocity, are independent of each other. However, they are caused by the same impact from the hammer and affected by the same soil resistance and they have to follow the same physical laws of wave propagation. The CAPWAP analysis makes use of this situation by taking the input from one measurement, usually the velocity, moderating it by reflections computed from an assumed distribution of damping, quake, and soil resistance, and transferring it to force by means of wave mechanics computations. In a trial and error procedure, the input data are adjusted until the computed force plots on top of the measured force throughout the impact event. The signals have been matched and the CAPWAP analysis has then calibrated the site conditions and provided the static bearing capacity of the pile as well as indicated dynamic parameters.

13.13.3 Pile Integrity Tester

Low-strain integrity testing is used on all types of piles, but in particular on bored piles, caissons, piers, or piles that cannot be subjected to driving. It is performed using a high-sensitivity accelerometer placed on the pile head, an amplifier–receiver, a special small-impact device, and a portable computer with digitizing and graphics capability.

In testing for integrity, a "low-strain" compressive impact wave is generated and the acceleration and velocity records (traces on the screen) of the impact are studied. Damage or defects in the pile will show up on the acceleration and velocity traces, which are graphically displayed and stored on disk for later reprocessing. A special computer program processes the records and includes special effects such as averaging of records

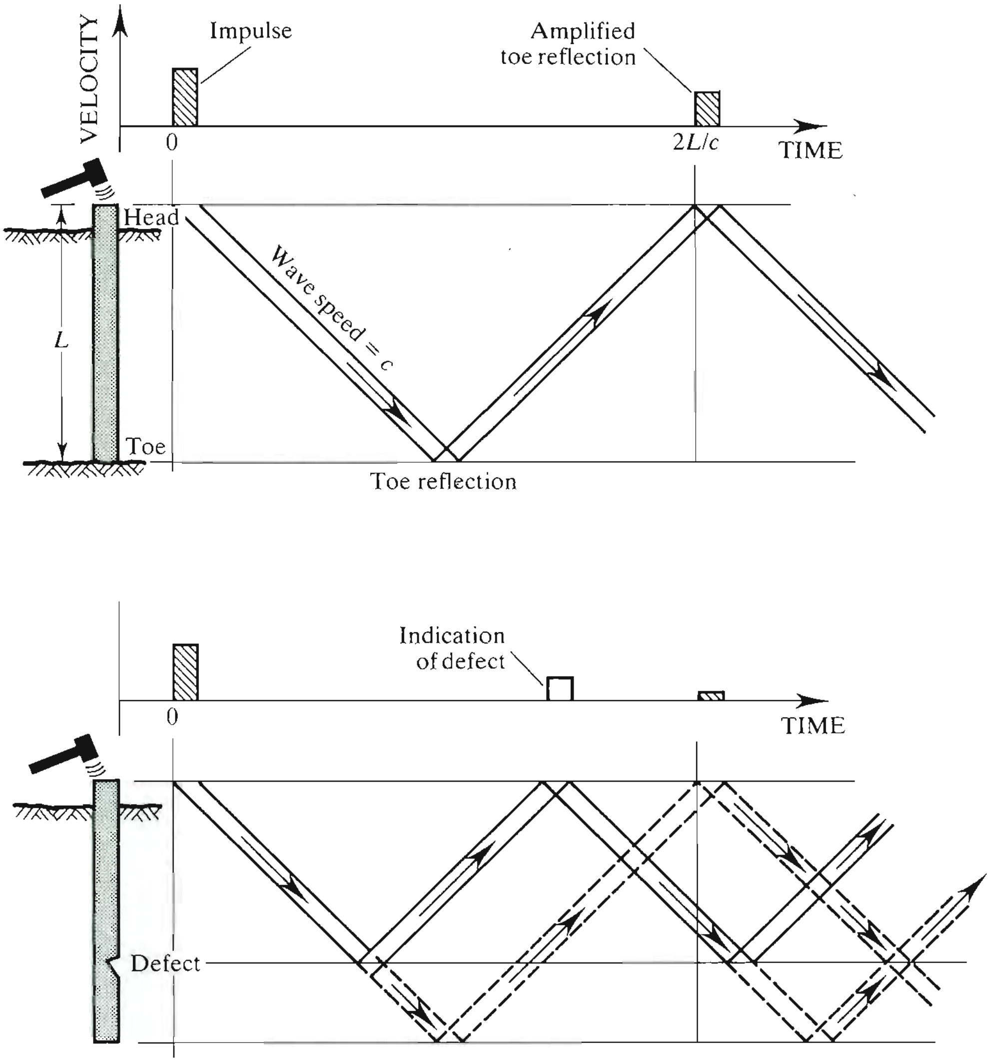

Fig. 13.16 Graphic illustration of how an impulse travels through a sound pile as opposed to a pile with a defect.

from several blows, gradual or exponential amplification of the reflections from down the pile, etc., to enable separation of random reflections from important reflections such as those from cracks, discontinuities, voids, etc. A signal matching procedure similar to the CAPWAP method is also available.

In testing a pile, a slight blow is delivered to the pile head by means of a hand-held hammer. The impact of the hand-held hammer initiates a small strain wave that travels down the pile at the speed of sound. A highly sensitive accelerometer is used to pick up the impact (sonic impulse) and the faint reflection (echo) of the impact from below the pile.

The principles of low-strain testing are illustrated in Figure 13.16, which shows two vertical piles, one having a defect and one being sound. To the right of each pile is shown an upper diagram of velocity integrated from the acceleration measured at the pile head and plotted versus time, and a lower diagram indicating the downward travel of the impulse in the pile. When the strain wave reaches a crack, or a void, in the pile, a reflection in the form of a tensile wave is sent back up to the pile head. If there is no such defect in the pile, the wave travels unimpeded to the pile toe (bottom end of the pile) and reflects from there.

The acceleration measurement is picked up by a signal amplifier and sent to a computer, which digitizes, integrates, and processes the record for display on a screen.

The existence of soil shear resistance along the pile shaft dampens the strain wave. Without amplification, the reflected velocity trace would lie very close to the zero axis (Fig. 13.16). The computer processing allows an amplification of the signal that progressively offsets the shear resistance. At times, however, the faint signal can be overshadowed by randomly varying impulses, that is, electronic noise. Then, to filter such random impulses, several contiguous blows may be processed and averaged, which eliminates the noise and allows an indication of damage or defect to stand out in the record.

13.14 HORIZONTALLY LOADED PILES

Because foundation loads act in many different directions, depending on the load combination, piles are rarely loaded in true axial direction only. Therefore, a more or less significant lateral component of the total pile load always acts in combination with an axial load. The imposed lateral component is resisted by the bending stiffness of the pile and the shear resistance mobilized in the soil surrounding the pile.

An imposed horizontal load can also be carried by means of inclined piles, if the horizontal component of the axial pile load is at least equal to and acting in the opposite direction to the imposed horizontal load. Obviously, this approach has its limits as the inclination cannot be impractically large. It should, preferably, not be greater than 4(vertical) to 1(horizontal). Also, only one load combination can provide the optimal lateral resistance.

In general, it is not correct to resist lateral loads by means of combining the soil resistance for the piles (inclined as well as vertical) with the lateral component of the vertical load for the inclined piles. The reason is that resisting an imposed lateral load by means of soil shear requires the pile to move against the soil. An inclined pile will rotate owing to such movement and either push against or pull away from the pile cap, which will substantially change the axial load in the pile. Such combination requires sophisticated computer analysis by means of a suitable program, such as GROUP1 developed by Reese et al. (1990).

In design of vertical piles installed in a homogeneous soil and subjected to horizontal loads, an approximate and usually conservative approach is to assume that each pile can sustain a horizontal load equal to the passive earth pressure acting on an equivalent wall with depth $6b$ and width $3b$, where b is the pile diameter, or face-to-face distance (*Canadian Foundation Engineering Manual*, 1985).

Similarly, the lateral resistance of a pile group may be approximated by the soil resistance on the group calculated as the passive earth pressure over an equivalent wall with depth equal to $6b$ and width equal to:

$$L_e = L + 2B \tag{13.28}$$

where

L = the length, center-to-center, of the pile group in plan perpendicular to the direction of the imposed loads
B = the width of the equivalent area of the group in plan parallel to the direction of the imposed loads

The lateral resistance calculated according to Equation 13.28 must not exceed the sum of the lateral resistance of the individual piles in the group. That is, for a group of n piles, the equivalent width of the group, L_e, must be smaller than n times the equivalent width of the individual pile, $6b$. For an imposed load not parallel to a side of the group, calculate for two cases, applying the components of the imposed load that are parallel to the sides.

The very simplified approach expressed in Equation 13.28 does not give any indication of movement. Nor does it differentiate between piles with fixed heads and those with heads free to rotate, that is, no consideration is given to the influence of pile bending stiffness. As the governing design aspect with regard to lateral behavior of piles is lateral displacement and the lateral capacity or ultimate resistance is of secondary importance, the usefulness of the simplified approach is very limited in engineering practice.

The analysis of lateral behavior of piles must consider two aspects.

- *The pile response.* The bending stiffness of the pile, how the head is connected (free head, or fully or partially fixed head).
- *The soil response.* The input in the analysis must include the soil resistance as a function of the magnitude of lateral movement.

The first aspect is modeled by treating the pile as a beam on an "elastic" foundation, which is done by solving a fourth-degree differential equation with input of axial load on the pile, material properties of the pile, and the soil resistance as a nonlinear function of the pile displacement.

The derivation of lateral stress may make use of a simple concept called "coefficient of subgrade reaction" having the dimension of force per volume (Terzaghi, 1955). The coefficient is a function of the soil density or strength, the depth below the ground surface, and the diameter (side) of the pile. In cohesionless soils, the following relation is used:

$$k_s = n_h \frac{z}{b} \tag{13.29}$$

where

k_s = coefficient of horizontal subgrade reaction
n_h = coefficient related to soil density
z = depth
b = pile diameter

The intensity of the lateral stress, p_z, mobilized on the pile at depth z is then as follows:

$$p_z = k_s y_z b \tag{13.30}$$

where y_z = the horizontal displacement of the pile at depth z.

Combining Equations 13.29 and 13.30:

$$p_z = n_h y_z z \tag{13.31}$$

The relation governing the behavior of a laterally loaded pile is then as follows (Reese and Wang, 1985):

$$Q_h = EI\frac{d^4y}{dx^4} + Q_v\frac{d^2y}{dx^2} - p \tag{13.32}$$

where

Q_h = lateral load on the pile
EI = bending stiffness (flexural rigidity)
Q_v = axial load on the pile

Design charts have been developed that, for an input of imposed load, basic pile data, and soil coefficients, provide values of displacement and bending moment. See, for instance, the *Canadian Foundation Engineering Manual* (1985).

The design charts cannot consider all the many variations possible in an actual case. For instance, the p–y curve can be a smooth rising curve, can have an ideal elastic–plastic shape, or can be decaying after a peak value. As an analysis without simplifying shortcuts is very tedious and time-consuming, resort to charts has been necessary in the past. However, with the advent of the personal computer, special software has been developed that makes the calculations easy and fast. In fact, as in the case of pile driving analysis and wave equation programs, engineering design today has no need for computational simplifications. Exact solutions can be obtained as easily as approximate ones. Several proprietary and public-domain programs are available for analysis of laterally loaded piles. One of the most widely used and accepted is produced by Reese and Wang (1985).

One must not be led to believe that, because an analysis is theoretically correct, the results also predict to the true behavior of the pile or pile group. The results must be correlated to pertinent experience, and, lacking this, to a full-scale test at the site. If the experience is limited and funds are lacking for a full-scale correlation test, then a prudent choice is necessary of input data, as well as of margins and factors of safety.

Designing and analyzing a lateral test is much more complex than for the case of axial behavior of piles. In service, a laterally loaded pile almost always has a fixed-head condition. However, a fixed-head test is more difficult and costly to perform than a free-head test. A lateral test without inclusion of measurement of lateral deflection down the pile (bending) is of limited value. While an axial test should not include unloading cycles, a lateral test should be a cyclic test and include a large number of cycles at different load levels. The laterally tested pile is much more sensitive to the influence of neighboring piles than is the axially tested pile. Finally, the analysis of the test results is much more complex and requires the use of a computer and appropriate software.

13.15 SEISMIC DESIGN OF LATERAL PILE BEHAVIOR

Seismic design of lateral pile behavior is often taken as being the same as the conventional lateral design. A common approach is to assume that the induced lateral force to be resisted by piles is static and equal to a proportion, usually 10 percent, of the vertical force acting on the foundation. If all the horizontal force is designed to be resisted by inclined piles and all piles, including the vertical ones, are designed to resist significant bending at the pile cap, this approach is normally safe, albeit costly. It cannot be used for lateral design of vertical piles, however.

The seismic wave appears to the pile foundation as a soil movement forcing the piles to move with the soil. The movement is resisted by the pile cap, bending and shear are induced in the piles, and a horizontal force develops in the foundation, starting it to move in the direction of the wave. A half period later, the soil swings back, but the foundation is still moving in the first direction, and, therefore, the forces increase. This situation is not the same as the one originated by a static force.

Seismic lateral pile design consists of determining the probable amplitude and frequency of the seismic wave as well as the natural frequency of the foundation and structure supported by the piles. The first requirement is, as in all seismic design, that the natural frequency must not be the same as that of the seismic wave. Then, the probable maximum displacement, bending, and shear induced at the pile cap are estimated. Finally, the pile connection and the pile cap are designed to resist the induced forces.

There is at present a rapid development of computer software for use in detailed seismic design.

13.16 DESIGN EXAMPLE

A heavy column foundation, which is to support a vertical load of 12 MN (dead-load portion is 9.6 MN), will be placed at a site where the soils consist of an upper 6 m thick layer of organic, compressible clay followed by a 4 m thick sand layer below which a 5 m thick normally consolidated clay layer is deposited. Under the silty clay, depth 15 m, lies a 25 m thick layer of silty sand deposited on bedrock at a depth of 40 m. The groundwater table is located at the ground surface and the pore water pressure in the soil is hydrostatically distributed. The site has been thoroughly investigated and the geotechnical parameters of the soil layers have been determined. The column foundation is to be placed level with the ground surface in the center of a 100 m by 100 m area over which a 2 m thick fill will be placed. Table 13.4 presents geotechnical values for use in this example.

The foundation must be pile-supported and it has been decided to use 12 driven, 300 mm square, precast, prestressed, concrete piles and to install them by means of a Vulcan 010 single-acting air hammer. The piles will have to go well into the silty sand layer, but they will not reach bedrock. The 12 piles will each require a capacity of 1000 kN. The concrete 28-day strength is 50 MPa and the prestressing is by eight 11-mm (7/17-in) strands of yield 1860 MPa (270 ksi). They will be placed at the minimum spacing recommended by the *Canadian Foundation Engineering Manual* (1985), that is, 2 percent of the embedment depth plus 2.5 diameters, which results in a spacing center-to-center of 1.2 m. The piles will be placed in four rows of three piles and the foundation size is, therefore, 4.0 m by 3.0 m.

TABLE 13.4 GEOTECHNICAL PARAMETERS.

Parameter	*Units*	*Fill*	*Clay*	*Silty Sand*	*Silty Clay*	*Sand*
Thickness	m	2.0	6.0	4.0	5.0	25.0
Density	kg/m^3	1800	1600	1900	1850	2000
w	%	—	50	20	36	15
β	—	—	0.25	0.45	0.35	0.50
c'	kPa	—	12.0	0	0	0
N_t	—	—	—	—	—	60
m	—	—	15	200	30	300
m_r	—	—	120	500	450	900
j	—	—	0	0.5	0	0.5
OCR	—	—	1.8	2.0	—	2.0
$\sigma_1' - \sigma_p'$	kPa	—	—	—	40	—

Given the particular conditions imposed by the site conditions, the pile, and the hammer to use, the design effort includes calculations of what embedment length of the pile to consider by means of a static analysis of axial capacity, analyzing the drivability of the pile, and checking that the calculated settlement of the pile foundation is acceptable.

Embedment Length

A static analysis begins by determining the distribution of unit shaft resistance in the soil and the toe resistance according to Equations 13.3 and 13.5. When applying the values given in Table 13.4 and stipulating a factor of safety of 3.0 on the total load of 1000 kN, which requires a total pile capacity of 3000 kN, an embedment depth of 23 m is obtained. For the design example, the analysis is carried out by means of the UNIPILE program (Goudreault and Fellenius, 1990) that also yields the depth to the neutral plane and the load in the pile at the neutral plane, 19 m and 1900 kN, respectively. The 1900 kN load translates to a stress of about 20 MPa in the concrete pile, which is much smaller than 70 percent of the 50-MPa concrete cylinder strength and acceptable.

During initial driving, excess pore pressures will develop in the clay strata and, to a degree, also in the silty sand stratum, reducing the effective stress. When excess pore pressures are introduced in the UNIPILE program, the computed static resistance at depth 24 m becomes about 1800 kN, which is the static resistance that the hammer has to overcome in the initial driving. The balance of about 1200 kN will be derived from soil set-up occurring during the reconsolidation after driving and from consolidation of the soils under the weight of the fill.

Drivability Analysis

Drivability analysis combines analyses by the wave equation and static methods. The most common approach is to perform wave equation analysis to produce a so-called bearing graph, which shows the capacity against the penetration resistance. To account for variability of input values, such as hammer efficiency, soil quakes, cushion properties, etc., it is prudent to present a range of curves within an envelope or band. Figure 13.17 presents the band applicable to the design example and calculated by means of the GRLWEAP program (Goble Rausche Likins and Associates, 1988). The bearing graph shows that the required static capacity of about 1800 kN at the end-of-initial-driving (EOID) will be reached at a penetration resistance of about 400 ± 100 blows/m, which corresponds to

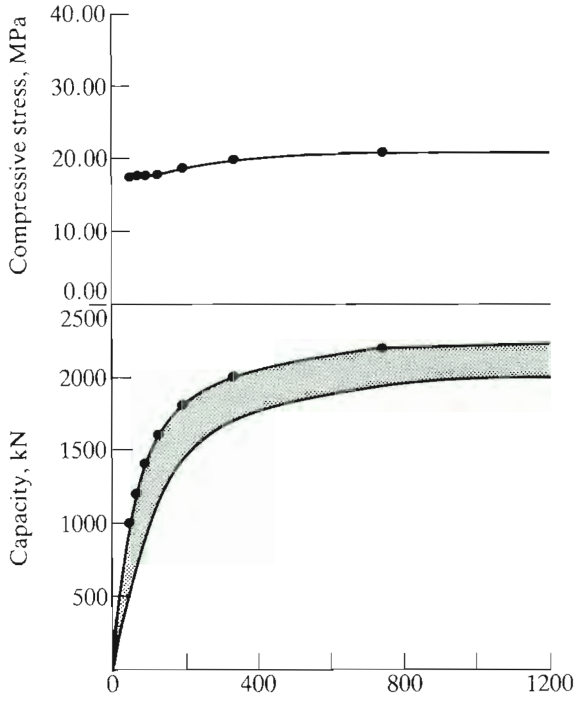

Fig. 13.17 Example design case. Bearing graph from a range wave equation analysis.

about 10 $\pm$ 2 blows/25 mm. Continuing to drive beyond a resistance at EOID exceeding 10 blows/25 mm will not be productive. The analysis also shows that the maximum compressive stress induced by the driving is about 22 MPa, which is smaller than two-thirds of the concrete strength of 50 MPa and therefore acceptable.

The bearing graph only presents the conditions toward the end of the initial driving. In a drivability study, one must answer many additional questions, such as what is the accumulated penetration resistance and the maximum tensile stress, which usually occurs before the full embedment depth has been reached. This study can be made by means of GRLWEAP's option of "blow count versus depth", the results of which are shown in Figure 13.18.

Figure 13.18 shows that although the static resistance (the capacity) increases linearly with depth, the penetration resistance (the blow count) increases progressively and the maximum depth to which one can reasonably expect to drive the pile with the assigned hammer is about 26 m. The analysis also suggests that the time for driving the pile will be 45 minutes, excluding splicing, but full-strength mechanical splices are completed in a few minutes and the splicing will add less than 5 minutes to the installation time.

Furthermore, the analysis and Figure 13.18 show that the maximum tensile stress is about 3.5 MPa, or 315 kN. The yield strength of the eight 11-mm strands of 1850 MPa results in an ultimate tensile strength of the pile of 1400 kN. The acceptable tensile stress is 70 percent of this value, that is, about 1000 kN. Hence, the analysis suggests that tensile stress will not become a problem in the driving.

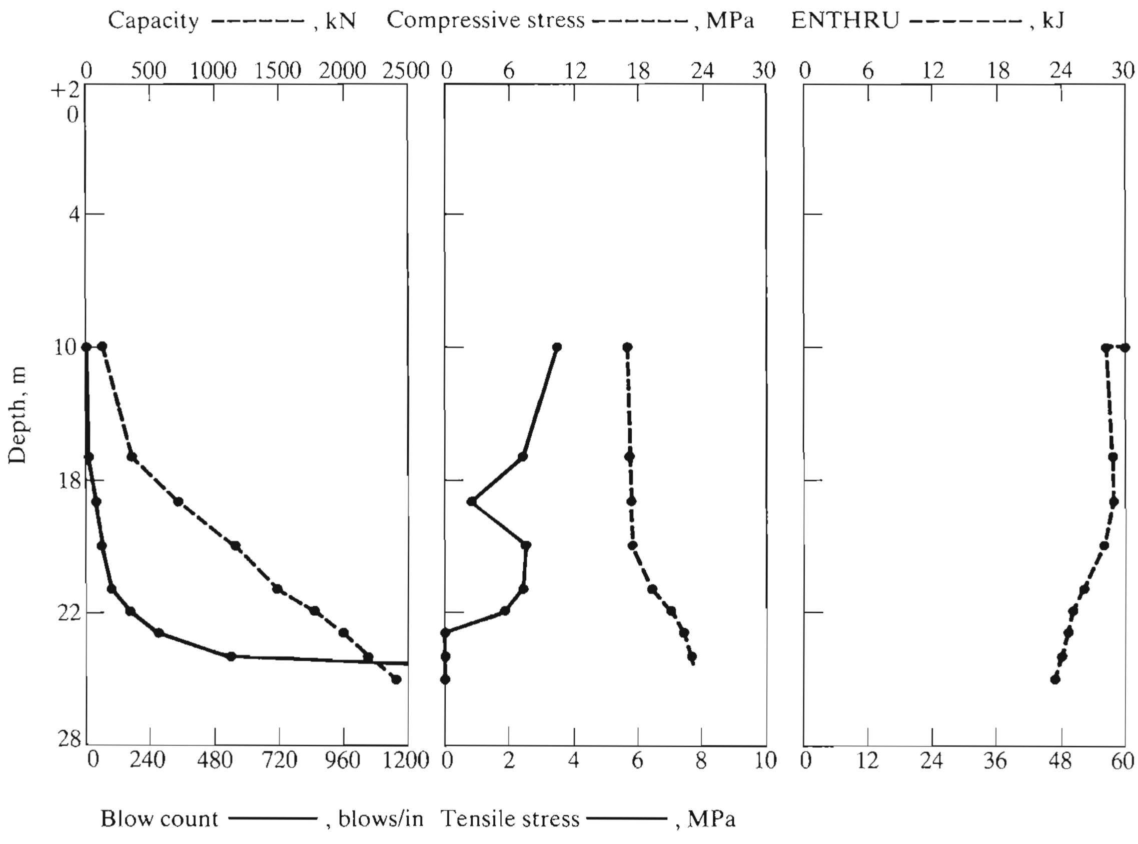

Fig. 13.18 Example design case. Drivability Study.

Settlement Analysis

The static analysis indicated a neutral plane located at an embedment depth of 19 m. By placing an equivalent footing at this depth having the same size as the pile cap (12 m^2), loading it by the dead load on the pile cap (9600 kN), and distributing this load by means of the 2:1 method, the settlement can be calculated from the parameters given in Table 13.4. The UNIPILE program calculates the settlement for the equivalent footing to be 30 mm. The movement of the pile toe into the sand is 20 mm, which is sufficient to activate the full toe resistance for the driven pile.

Acceptance of the Design

The bearing capacity as well as the structural capacity are within acceptable limits. The assigned hammer, although it is a light one, appears to be sufficient for installing the piles. Acceptance of the design now depends on whether the 30-mm settlement can be accepted. It should be recognized that even though piles have been installed underneath a foundation, there will always be settlement. However, in many instances a limit of 25 mm is acceptable. Most structures can actually tolerate considerably higher values.

If found unacceptable, however, the settlement can be reduced by taking the piles deeper into the sand. Changing to a larger size pile or to another type of pile and installing them to the same 23-m depth will not change the settlement appreciably. In the example design case, there is obviously a margin for using a larger hammer that can take the piles deeper. Perhaps, in the extreme, even all the way to the bedrock, which would be a costly proposition, however.

It is difficult to verify settlement calculations in advance. However, to verify pile capacity is not difficult. For a case similar to the example design case, where no large margin of capacity has been demonstrated, it will be wise to verify the design assumptions by means of dynamic monitoring of the initial driving and CAPWAP analysis of the dynamic data.

The bearing graph in Figure 13.17 shows that the hammer will be able to move the pile against a capacity of up to about 2200 kN. Because of soil set-up, the pile capacity will probably increase to about 3000 kN, which will be evidenced by a penetration restrike resistance in excess of about 15 blows/25 mm. To verify the capacity, it will be necessary to bring in a heavier hammer capable of moving the pile against a 3000-kN capacity. Alternatively, other means of capacity verification can be employed, for example, a static loading test or special dynamic methods, such as the STATNAMIC method and device (Bermingham and Janes, 1989).

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