20.2.1.1 Cohesionless Soils

For cohesionless soil

\[
\begin{align*}
q_0 &= \sigma'_v K M \tan \phi' = \beta \sigma'_v \\
\sigma'_v &= N \sigma_v
\end{align*}
\]

where \( \beta \) = a combined shaft resistance factor
\( K_M \) = coefficient of lateral earth pressure
\( \sigma'_v \) = vertical effective stress adjacent to the pile
\( M \) = a factor accounting for the friction at the pile-soil interface
\( N \) = bearing capacity factor
\( \sigma_v \) = vertical effective stress at the pile base.

The value of \( K_M \) is influenced by the angle of shearing resistance, the method of installation, the compressibility and original state of stress in the ground, and the size and shape of the pile. It increases with the in situ density and angle of shearing resistance of the soil and with the amount of displacement. It is higher for displacement-type piles than for low-displacement-type piles such as H-piles. For bored piles, \( K_M \) is usually assumed equal to the coefficient of earth pressure at rest, \( K_0 \). For driven displacement-type piles, \( K_M \) is normally assumed to be twice the value of \( K_0 \). For tapered piles the value of \( K_M \) may be increased by 30% to 50%.

The value of \( M \) ranges from 0.7 to 1.0, depending on the pile material (steel, concrete, wood) and method installation (Bozozuk et al., 1978b).

The combined shaft resistance coefficient \( \beta \) generally ranges from 0.20 to 1.5 as indicated in Table 20.1 - see Poulos and Davis (1980) for further discussion.

### Table 20.1 - Range of \( \beta \) Coefficients

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>CAST-IN-PLACE PILES</th>
<th>DRIVEN PILES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>0.2 - 0.30</td>
<td>0.3 - 0.5</td>
</tr>
<tr>
<td>Loose sand</td>
<td>0.2 - 0.4</td>
<td>0.3 - 0.8</td>
</tr>
<tr>
<td>Medium sand</td>
<td>0.3 - 0.5</td>
<td>0.6 - 1.0</td>
</tr>
<tr>
<td>Dense sand</td>
<td>0.4 - 0.6</td>
<td>0.8 - 1.2</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.4 - 0.7</td>
<td>0.8 - 1.5</td>
</tr>
</tbody>
</table>

The toe bearing capacity factor \( N \) depends on soil composition in terms of grain size distribution, angularity and mineralogy of the grains, natural soil density, density changes due to pile installation, and other factors. Typical ranges of values for \( N \) are given in Table 20.2.
Table 20.2 - Range of N, Factors

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>CAST-IN-PLACE PILES</th>
<th>DRIVEN PILES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>10 - 30</td>
<td>20 - 40</td>
</tr>
<tr>
<td>Loose sand</td>
<td>20 - 30</td>
<td>30 - 80</td>
</tr>
<tr>
<td>Medium sand</td>
<td>30 - 60</td>
<td>50 - 120</td>
</tr>
<tr>
<td>Dense sand</td>
<td>50 - 100</td>
<td>100 - 120</td>
</tr>
<tr>
<td>Gravel</td>
<td>80 - 150</td>
<td>150 - 300</td>
</tr>
</tbody>
</table>

In the absence of test loading, a factor of safety of at least 3 should be applied to any theoretical computation.

**Remark:** Consistent with research results and field observations reported in the literature, earlier versions of the Canadian Foundation Engineering Manual recommended to apply the concept of ‘critical depth’ when designing piles in cohesionless soils. According to this concept, the unit shaft resistance and point resistance would increase linearly with depth only down to the critical depth below this critical depth both the unit shaft resistance, \( r \), and the vertical effective stress at the pile base, \( a'_v \), would remain essentially constant. According to Meyerhof (1976) the critical depth would depend on the pile diameter and the soil density and would be in the range of \( 8 \) to 20 pile diameters.

However, more recent investigations suggest that the apparent absence of increase of \( r \) or \( a'_v \) below a certain depth may be the result of having ignored the effect of ‘locked-in’ stresses in test piles before the beginning of load tests. The evidence available to date is sufficient to cast some doubt on the relevance of the critical depth concept. Unfortunately, this evidence is still not sufficient to reach a conclusive answer on the real variations of unit shaft resistance and point resistance with depth for piles in sand. Caution is thus advised in the design of long piles in cohesionless soils.

### 20.2.1.2 Cohesive Soils

Design methods for piles in fine-grained soils are in some cases of doubtful reliability. This is particularly so for the bearing capacity of shaft-bearing piles in clays of medium- to-high shear strength. Because of this, pile test loading should be carried out where economically justified or, alternatively, an adequate factor of safety be used.

Piles in cohesive soils generally derive their capacity from tills, substantial toe resistance may be mobilised, which, for large-diameter bored piles, may represent the usable capacity of the pile.

**20.2.1.2(1) Total Stress Versus Effective Stress Approach**

Until recent times, it was the general practice to evaluate the capacity of piles in clay from a total stress approach, i.e., on the basis of the undrained shear strength, \( \tau_u \), of the clay. Empirical correlations between \( \tau_u \), and the toe-and-shaft resistance on a pile
have been developed, but these have not proved reliable, particularly for $r_u$ in excess of about 25 kPa. Therefore, analysis in terms of effective stresses is more rational, i.e., the same method as used for piles in cohesionless soils, and the method given in Subsection 20.2.2.2 applies in all details. Burland (1972) provides a detailed discussion on relevant values of $\beta$; Skempton (1951) and Ladanyi (1963) present discussion and values of $N_r$. The relationship in Subsection 20.2.1.1 may be used in design with the following values:

$$\beta = 0.25 - 0.31$$

$$N_r = 3 - 10$$

For tapered piles, Bianchetti et al., (1980) suggest applying a factor of 2 to $\beta$.

**20.2.1.2(2) Shaft Resistance in Clays with $r_u < 100$ kPa**

A pile driven in clay with an undrained shear strength smaller than 100 kPa derives its capacity almost entirely from shaft resistance. It is still common practice to determine the ultimate shaft resistance of a single pile using total stress analysis from the formula:

$$r_u = c \cdot f$$

where $c$ = adhesion coefficient ranging from 0.5 to 1.0

Figure 20.1 shows the shaft resistance as a function of the undrained shear strength of the clay. However, the actual resistance depends significantly on the geometry of the foundation, the driving method and sequence, the properties of the clay, and time effects. The capacity of piles determined from the above formula should be confirmed by wet loading.
20.2.1.2(4) Toe Resistance

The ultimate toe resistance may be estimated from:

\[ R_t = N_t t_s A_t \]

where:
- \( R_t \) = toe resistance
- \( A_t \) = cross-sectional area of pile at toe
- \( t_s \) = minimum undrained shear strength of the clay at pile toe
- \( N_t \) = a bearing capacity coefficient, which is a function of the pile diameter, as follows:

<table>
<thead>
<tr>
<th>Pile toe diameter</th>
<th>( N_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>smaller than 0.3 m</td>
<td>9</td>
</tr>
<tr>
<td>0.3 m to 1 m</td>
<td>7</td>
</tr>
<tr>
<td>larger than 1 m</td>
<td>5</td>
</tr>
</tbody>
</table>

In very stiff clays and silts, where samples are difficult to retrieve and the undrained shear strength is not easily measured, the pressuremeter may be used to evaluate the strength of the soil.

20.2.1.2(6) Safety factors and allowable loads on bored piles

The allowable loads on bored piles are determined from a combination of shaft resistance and toe resistance, after the application of appropriate factors of safety on the calculated resistance values. The relative contribution of the shaft resistance and the toe resistance is a function of the rigidity of the pile and the compressibility of the clay around the shaft and below the base of the pile.

If the soil below the toe has the same or greater compressibility than the soil around the shaft, the allowable load, \( Q_w \), on the pile may be taken as:

\[ Q_w = \frac{1}{3} (R_t - R_s) \]

If the soil below the toe is less compressible than the soil around the shaft, the movements of the shaft relative to the soil will generally be too small to mobilize the full shaft resistance. In such a case, the allowable load on the pile should be taken as:

\[ Q_w = \frac{1}{2.5} R_s \]

While the above formulae may be considered as limiting cases, the decision to consider shaft resistance in addition to toe resistance must be made with care and only after properly planned and interpreted testing is carried out. Such tests should indicate whether or not the resistance available is commensurate with strain both around the shaft and at the toe, and consider any possibility of reduction in shaft resistance with
time. The final selection of the allowable load should be based upon permissible pile movement.

20.2.1.3 Stratified Deposits

The relative contribution of the various strata penetrated by a pile to the capacity of that pile is primarily a function of the relative stiffness of these layers and of the type of pile. Static analysis for total axial capacity essentially involves calculating contributions of various unit shaft resistance values $\tau$ associated with the different strata which the pile penetrates, and the end bearing associated with the stratum containing the pile point. Care must be taken, however, to measure the critical depth mentioned in 20.2.1.1 from the upper ground surface (this conservative approach must be adopted given the lack of field test data indicating critical depth for multilayered deposits). Furthermore, it is important to install the toe of the pile a distance of at least four diameters into any stiffer clay stratum so that the full value $N_t = 9$ can be used, and to watch for the presence of a weaker stratum below the tip which could reduce the end resistance.

20.2.2 Pile Groups - Static Analysis

It is common practice to define the axial capacity on a pile group relative to the sum of the capacities of the individual piles in the group. Group 'efficiency' is defined as the ratio of the group capacity to this sum of the individual pile capacities.

20.2.2.1 Cohesionless Soils

Driven piles in cohesionless soils develop larger individual capacities when installed as a group (group efficiency $>1$) since lateral earth pressure and sand density increase with the driving of additional piles. Therefore, it is conservative to use the sum of the individual pile capacities as an estimate of the pile group capacity.

For bored pile groups, the individual pile capacity is reduced by the addition of the extra piles, since the boring process reduces sand density and lateral earth pressures (efficiency is $<1$). Therefore, for bored pile groups a reduction factor (Meyerhof (1976) suggests 0.67) may need to be applied to the sum of individual pile capacities.

20.2.2.2 Cohesive Soils

In addition to the possibility that individual piles in a pile group act independently to support the applied load, a closely spaced pile group, can act as a 'block' whereby the soil between adjacent piles is dragged down between them, shaft resistance develops around the perimeter of the group only, and end-resistance develops under the whole of the pile-soil block. A rational approach to estimating pile group capacity is to use the minimum of a) the sum of the individual pile capacities and b) the capacity of the pile-soil block analyzed as an equivalent single pile. For this block capacity calculation, an average unit shaft resistance $\tau$ must be calculated since for zones where there is soil-soil contact $\tau = c_s$, and for zones where there is soil-pile contact $\tau = c_p$. The block perimeter is the circumference $C$ of the equivalent pile, and the area of the block base is taken as the base area $A$, of the equivalent pile.

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20.3.2.2 Empirical Methods for Piles in Sand

The settlement of a pile group is evaluated on an empirical basis and the methods are less reliable than those used for single piles because of the limited reference data available. For pile groups in cohesionless soil, two empirical methods are available:

**Vesic’s Method**

The ratio of the settlement of the pile group with width, B, to that of the individual pile with diameter, b (Vesic, 1972) is:

\[ \frac{s_{p,\text{group}}}{s_{p,\text{individual}}} = \left( \frac{b}{B} \right)^{0.5} \]

**Meyerhof’s Method**

The settlement of a pile group, \( s_{p,\text{group}} \), in millimeters, may be related to the standard penetration \( N \) of the soil (Meyerhof, 1974) by:

\[ s_{p,\text{group}} = 0.92 \left( \frac{q}{N} \right) B \]

where \( q \) = equivalent net vertical foundation pressure, in kPa, determined from \( q = 0.07Q/B \), where \( Q \) is total load transferred to piles and \( L \) and \( B \) are the length and width, respectively, of the plan area of the pile group.

\( B \) = pile group width, in meters

\( l \) = an influence factor ranging from 0.5 to 1.0. (Refer to Meyerhof, 1976)

20.3.2.3 Empirical Method for Piles in Clay

For the evaluation of the settlement of pile groups in homogeneous clay, Terzaghi and Peck (1967) assumed that the load carried by the pile group is transferred to the soil through an equivalent footing located at one-third of the pile length up from the pile toe (Figure 20.7). The load is assumed to spread into the soil at a slope of 2V:1H under the assumption that the equivalent footing is the top of the foundation of a pyramid. The settlement calculation for the equivalent footing then follows the methods described in Chapter 12. The Terzaghi and Peck method usually results in settlement values that greatly overestimate the actual values. Therefore, where settlement considerations govern the design, the method may result in unconservative pile lengths.

Recent field tests and long-term settlement observations of piles in the sensitive clays of the St. Lawrence Valley suggest that the assumption of an equivalent footing placed at the lower third point is not representative of the actual settlement behavior of a pile group. Blanchet et al., (1986) report that the settlement of a pile group is due mainly to reconsolidation of the clay after driving and to shear creep deformation with little if any consolidation settlement.
Figure 20.7: Stress distribution beneath a pile group in homogeneous clay using the equivalent footing concept (after Terzaghi and Peck, 1967).

All piles have a neutral plane located at some level in the soil, where an equilibrium exists between the loads on the pile above the neutral plane and the sub-grade resistance below the neutral plane. The loads consist of the service load (dead load, only) and downgrading due to negative skin friction. The negative skin friction is caused by shear creep deformation in combination with the large stiffness difference between the soil and the pile (Fellenius, 1984a). Accordingly, the settlement calculation of a pile group, or of a single pile, in a soil not undergoing consolidation settlement from other causes than from the service load, follows the same approach as given for piles in soil where consolidation settlement from other causes does occur in the soil around the piles. (See Subsection 20.2.5.)

In clay soils, the reconsolidation can take an appreciable time — for large pile groups more than a year — and the pore-pressure dissipation occurring during the reconsolidation will cause settlement. Therefore, the settlement analysis must include the effect of the reconsolidation of the soil around the piles after the pile driving.

20.3.2.4 General Settlement Equation

Piles in close proximity interact, so that load $P_i$ on one pile with settlement $S_i$ results in settlement $e_b$, of an adjacent pile where $e$ is called the "interaction factor".

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Total settlement of a pile \( j \) in a group of \( n \) piles:

\[
S_j = S_0 + \sum_{i=1}^{n} F_i a_i
\]

where:
- \( S_j \) = the settlement of pile \( j \)
- \( S_0 \) = the settlement of a pile under unit load, evaluated using one of the procedures from 20.3.1
- \( F_i \) = the load on pile \( i \)
- \( a_i \) = the interaction factor relating settlement of pile \( j \) to load on pile \( i \). These are found using elastic theory, provided in Figure 20.8 for floating piles from Poulos and Davia (1980). Other solutions are available for end-bearing piles.

20.3.2.5 Pile Cap Conditions

Two simplified pile cap conditions can be examined using the general settlement equation shown above:

- Rigid pile cap, where all piles settle an equal amount but loads on individual piles are not known.
- Flexible pile cap, where the loads on each pile are known and each pile has different settlement.

The flexible pile cap problem is solved simply using the settlement equation directly.

The rigid pile cap problem is solved using the general equations (one for each pile) and the known total load applied to the pile group which is the sum of the individual loads:

\[
P_{\text{total}} = \sum_{i=1}^{n} F_i
\]

There are then \( n-1 \) equations with \( n-1 \) unknowns \((P_1, P_2, ..., P_{n-1}, S)\). In addition to the group settlement \( S \), the individual pile loads are evaluated.

20.4 Lateral Capacity of Piles in Soil

Vertical piles resist lateral loads or moments by deflecting until the necessary reaction in the surrounding soil is mobilized. The behaviour of the foundation under such loading conditions depends essentially on the stiffness of the pile and the strength of the soil.
Figure 20.8a: Interaction factors for floating piles, L/d = 10. (After Poulos and Davis, 1980).

Figure 20.8b: Interaction factors for floating piles L/d = 25. (After Poulos and Davis, 1980).
Figure 20.3a: Interaction factors for floating piles, $L/d = 50$. (After Fouad and Davis, 1980).

Figure 20.3b: Interaction factors for floating piles $L/d = 100$. (After Fouad and Davis, 1980).
The horizontal load capacity of vertical piles may be limited in three different ways:

- the capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
- the bending moments may generate excessive bending stresses in the pile material, resulting in structural failure of the piles; or
- the deflections of the pile heads may be too large to be compatible with the superstructure.

All three methods of failure must be considered in design.

There is much room for improvement of these design methods, and often the best method is still that based on well-planned and well-executed lateral test loading.

20.4.1 Broms' Method

Various static analyses of lateral load capacity have been reported, including those of Brinch-Hansen (1961). Broms (1964a and b) has presented solutions in graphical form. Figures 20.9 and 20.10, for uniform clay and sand strata. In each case, two types of pile failure are examined:

- "short" pile failure where the lateral capacity of the soil adjacent to the pile is fully mobilized; and
- "long" pile failure where the bending resistance of the pile is fully mobilized.

The solutions are based on a number of simplifying assumptions covering the magnitude of the lateral soil pressures and their distribution along the pile. Results are given for:

- pile of diameter d;
- embedded length, L;
- lateral load capacity, $H_l$;
- yield moment of pile, $M_{yim}$
- clay cohesion, $c_c$;
- coefficient of passive sand resistance, $K_s$;
- height of lateral load above groundwater, $h$; and
- soil unit weight, $\gamma$.

Recently, Poulos (1985) has extended Broms' solutions to consider lateral load capacity for piles in layered clay soils.

20.4.2 Pressuremeter Method

Considering the close analogy between the behaviour of soils around a horizontally loaded pile and around a pressuremeter probe, an empirical method for determining horizontal resistance $R_h$ from pressuremeter test results has been proposed by Menard
The horizontal load capacity of vertical piles may be limited in three different ways:

- the capacity of the soil may be exceeded, resulting in large horizontal movements of the piles and failure of the foundation;
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All three methods of failure must be considered in design.

There is much room for improvement of these design methods, and often the best method is still that based on well-planned and well-executed lateral test loading.

20.4.1 Bruns' Method

Various static analyses of lateral load capacity have been reported, including those of Briach-Hansen (1941). Bruns (1964a and b) has presented solutions in graphical form, Figures 20.3 and 20.10, for uniform clay and sand strata. In each case, two types of pile failure are examined:

- 'short' pile failure where the lateral capacity of the soil adjacent to the pile is fully mobilized; and
- 'long' pile failure where the bending resistance of the pile is fully mobilized.

The solutions are based on a number of simplifying assumptions covering the magnitude of the lateral soil pressures and their distribution along the pile. Results are given for:

- pile of diameter $d$;
- embedded length $L$;
- lateral load capacity $H_L$;
- yield moment of pile, $M_{yld}$;
- clay cohesion $c$;
- coefficient of passive sand resistance, $K_p$;
- height of lateral load above groundwater, $e$, and
- soil unit weight $y$.

Recently, Poulos (1983) has extended Bruns' solutions to consider lateral load capacity for piles in layered clay soils.

20.4.2 Pressuremeter Method

Considering the close analogy between the behaviour of soils around a horizontally loaded pile and around a pressuremeter probe, an empirical method for determining horizontal resistance $R_h$ from pressuremeter test results has been proposed by M boundary
(1962). According to this method, the ultimate horizontal resistance of a short head-restrained pile may be expressed by:

\[ R_h = p_1 b (D - b) \]

where \( R_h \) = ultimate horizontal resistance of pile
\( p_1 \) = limit pressure from pressuremeter test
\( b \) = diameter of pile
\( D \) = embedment depth of pile

20.5 Lateral Pile Deflections

For the subgrade reaction models, it is assumed that the soil around a pile can be simulated by a series of horizontal springs, each spring representing the behaviour of a layer of soil of unit height. When the pile is forced against the soil under the action of horizontal loads, the soil deforms and generates an elastic reaction assumed to be identical to the force that would be generated by the simulating spring subjected to the same deformation. With the further assumption that the soil is homogeneous, i.e., all springs are identical, the soil’s behaviour can be estimated if the equivalent spring constant is known. This spring constant is called the coefficient of subgrade reaction \( k_s \) (dimension: force/volume).

20.5.1 Coefficient of Subgrade Reaction

Though simple in its definition, the coefficient of subgrade reaction has proved to be a very difficult parameter to evaluate. This is because it cannot be measured in laboratory tests, but must be back-calculated from full-scale field tests.

Investigations have shown it to be variable not only with the soil type and mechanical properties, but also with stress level and the geometry of the pile.

In the absence of better information, the coefficient of horizontal subgrade reaction may be estimated by the following method:

1) In cohesionless soil (Terzaghi, 1955)

\[ k_s = n_0 \frac{d}{D} \]

where \( k_s \) = coefficient of horizontal subgrade reaction (force per volume)
\( z \) = depth
\( d \) = pile diameter
\( n_0 \) = coefficient related to soil density as given in Table 20.3.
CHAPTER 21

STRUCTURAL DESIGN AND INSTALLATION OF PILES

21.1 Introduction

This chapter provides information on the use of different types of deep foundations, including special features of structural design and considerations in the installation of such foundations.

21.1.1 Capacity of Deep Foundations

21.1.1.1 Structural Capacity

The structural strength of a deep foundation unit, determined in accordance with the National Building Code of Canada, represents the load which the unit can support as a structural member. In most cases, the bearing capacity of a deep foundation unit is governed by geotechnical considerations, rather than by the structural strength of the unit.

The installation and inspection of a deep foundation unit is generally less controllable than for a similar superstructure member. Moreover, the environment of the deep foundation unit may be potentially more damaging structurally than the environment of the superstructure member.

In determining the structural capacity, it is important to note that permissible deviations in alignment and location of the unit be established and considered in the design. Normally, it is not possible to install deep foundations closer than 70 mm to the specified position, and, therefore, the design should allow for this location limitation. When the off-location effect is considered, the restraining influence of the pile cap, the beams, and soil should be included. The effects of moments and lateral loads must also be considered in the design, and it should be noted that some of the design procedures stipulated in the National Building Code are left to the designer.

21.1.1.2 Geotechnical Capacity of a Driven Pile

The geotechnical capacity of a driven pile is a function of the dynamic response of the pile, the so-called dynamic impedance £A/c, where £ is the modulus of elasticity, A is the cross-sectional area of the pile, and c is the speed of the strain wave in the pile. The strength of the pile material has no influence beyond a minimum value, which mostly is smaller than about 250 MPa. Therefore, the geotechnical capacity of a driven pile differs from that of the structural capacity. The potential geotechnical capacity of two piles with the same importance is the same, be the piles of the same material or different, e.g., steel or concrete, whereas the structural capacity may differ.
The allowable geotechnical stress of a driven pile should be limited to a factor times E/c of the pile material. In the absence of field verification of the «stress» and magnitude of soil setup, or soil relaxation (see Subsection 20.2.7.3), the factor is suggested to be 2.0 (until = 12/s). Yield verification by means of test loading or dynamic monitoring (Subsection 21.1.9) will supersede this suggestion. The value of 3.0 for steel piles is relatively constant and equal to 60 MPa. For prestressed piles, ordinary reinforced, as well as prestressed—both the elastic modulus, E, and the wave speed, c, vary and E/c is not constant. However, 21.9 E/c is usually within the relatively narrow range of 12 to 15 MPa, and 15 MPa is suggested for use in design. (For discussion, see Taliaferro, 1984.)

The dynamic impedance of a closed-toe steel pile can be substantially improved by increasing the pile before driving. The resulting increased dynamic impedance (new combined value of E/A/c) will enable the pile to be driven to a higher geotechnical capacity and for verify the existence of soil setup. To avoid having the steel tube receiving the hammer impact, the concrete should be finished with a slightly convex upper surface protruding above the steel tube. Also, it is advisable to add some reinforcing bars to the concrete within a zone of four pile diameters from the pile head.

Soil setup can be verified in the field by a load test or by dynamic measurements during repositioning. While restricting abuse is a highly recommended method of quality control and will verify soil relaxation, it does not provide sufficiently reliable information on soil setup on a pile driven to refusal, unless the pile impedance is increased or a heavier hammer is used that can develop more force and driving energy per blow (as opposed to the hammer used in initial driving) and, therefore, move the pile to a penetration larger than about 3 to 4 mm per blow. In driving composite piles, ideally, the design should ensure that the impedance, E/A/c, of the sections of the pile are the same. If the impedances differ by more than a factor of 2, serious damage or driving difficulties can arise. Composite piles are concrete piles with long steel H-pile ends, or steel pipe and steel H-piles combined, or two sections of different size concrete piles combined, etc.

When driving a pile with a follower made of the same material as the pile, the areas should be equal. If the pile and the follower are of different materials, e.g., a concrete pile and a steel follower, the impedances should be equal. This means that the steel area should be about 20% of the concrete area. (For additional comments, see Fellenius, 1984.)

21.1.2 Wave Equation Analysis

The one-dimensional wave equation analysis is the application of longitudinal wave transmission in the pile driving process, which provides a mathematically accurate expression describing the mechanics of strain wave travel along a pile after it has been hit by the ram of the pile hammer. This method takes into account the characteristics of:

- the hammer (mass of the ram or piston, height of fall of the ram, rated energy, and impact velocity).
the driving cap or helmet (mass, stiffness, and coefficient of restriction of the hammer cushion or capblock, and the pile cushion, when used);

the pile (material, dimensions, mass and stiffness), and

the soil (assumed deformation characteristics represented by quake and damping factors for shaft and toe resistance).

The wave-equation analysis can be used with great advantage to assist in the selection of hammers and capblocks, in the design of cushions, in the prediction of driving stresses and bearing capacities, and in the choice of driving criteria.

The wave-equation analysis is fundamentally correct. It can provide qualitative information to use in, for instance, the comparison between two hammers. However, the results of the analysis are no more quantitatively correct than the data used as input in the analysis. When no direct measurements or observations are available for reference (calibration), it will be fortuitous if the results are quantitatively relevant to the real situation. In the absence of calibration data from the analysis of dynamic monitoring (Subsection 21.1.3), the wave-equation analysis should be limited to use for providing a range of results established with due consideration to the possible variations of the hammer-pile-soil system.

The wave-equation analysis should be recognized as one of the major advances of the current state-of-the-art. Its use is highly recommended. However, it should be considered as a tool among many and not used by a person well-experienced not only in the wave-equation analysis, but also in the overall art of pile installation and pile-soil analysis.

21.1.3 Dynamic Monitoring

By monitoring the effect of the hammer impact on a pile in terms of force (stress, strain) and velocity (acceleration) by means of special instrumentation and analyzing the obtained force and velocity wave traces, information can be obtained as to the proper functioning of the hammer, the impact force, the transferred energy, and the soil response to the impact on the pile.

The dynamic monitoring method has been used in Canada since 1976 and is a well-established method. For details on the instrumentation and method, see Gobe et al., (1970), Rasmieh et al., (1972), Tellier et al., (1976), and Auslioli and Felicius (1983).

The soil response may be related to the pile static capacity by a method called Case Method Estimate (CMES). This method is fast and produces a value for each impact as the driving proceeds. For more accurate capacity determination, a more time-consuming computer treatment of the data is required, called CAPWAP. Representative blows are selected for analysis, when required. The CAPWAP analysis provides not only a value of the static capacity, which closely agrees with the capacity, obtained by means of a static load test, it also provides data suitable for input in a wave equation analysis.

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The advantage of the dynamic monitoring is that several piles can be tested for the cost of one static load test to account for the natural variability of capacity between piles, and apart from the capacity, the method also provides a control of hammer efficiency, and determines the energy and driving stresses developed in the pile. In addition, by means of dynamic monitoring, the integrity of a pile can be ascertained. No redesign pile test driving should be performed without inclusion of dynamic monitoring into the programme.

Dynamic monitoring and analysis should be carried out by an experienced person and the data should be related to other important geotechnical information from the site.

21.1.4 Dynamic Pile Driving Formulas

The dynamic driving formulas, e.g., Hiley, Engineering News, and Jardus formulas, and more than 100 others, are derived by equating the nominally available energy, that is, the rated energy, not the actual energy, with work performed by the pile, calculated as the static capacity of the pile times the penetration for the blow.

The approach is fundamentally incorrect. However, the static pile capacity predicted by dynamic formulas in particular cases and in local areas can be close to the real values because the smaller the penetration of a pile for a hammer blow, the larger the static capacity. Nevertheless, quantitative agreement is only accidental and cannot be relied upon.

Since the wave-equation analysis is far superior and as easy to perform, there is now little reason to continue using the dynamic formulas.

21.2 Wood Piles

21.2.1 Use of Wood Piles

Wood piles are:
- best suited for use as friction piles in sands, silts, and clays;
- not recommended for driving through dense gravel or till, or for toe-bearing piles in rock, since they are vulnerable to damage both at head and toe in hard driving;
- difficult to splice; and
- commonly used for depths of 6 to 15 m, for diameters of 200 to 400 mm, corresponding to the natural dimensions of available tree trunks, and for design loads of 100 to 500 kN.
21.2.2 Materials

Wood piles must conform with the requirements of Subsection 4.2.3 of the National Building Code (1990). They may be used untreated where they are entirely located below the permanent water table. In this condition, they are resistant to decay, irrespective of the quality of groundwater.

Where untreated wood piles are exposed to soil or air above the permanent water table, and, in particular, when they are subjected to intermittent submergence, they are vulnerable to decay. In such an environment, only treated piles should be used for permanent structures.

21.2.3 Structural Design

The structural design of wood piles must conform with the requirements of Subsection 4.2.7 of the National Building Code (1990). No special considerations need be given to handling stresses, but special precautions must be taken to protect the pile toe and head from damage due to driving stresses.

21.2.4 Installation of Wood Piles

When driving wood piles, low-velocity hammer blows should be used. For example, drop hammers and single-acting steam/air hammers should have relatively small heights of fall, and incorporate a soft cushion at the copblock. The size of the hammer to choose for the driving depends on the number of piles, among them the weight of the pile, its size (diameter of head and toe) and impedance, and the soil properties. As an approximate guide, and in the absence of hard experience, the hammer-ratiod energy should not exceed a value equal to 150 kJ (Newton meters) times the pile head diameter.

Where hard driving occurs, the pile head should be protected with provision in the form of a steel ring, and the pile toe should be protected with a special steel shoe.

Wood piles cannot withstand hard driving. Over-driving will generally lead to the destruction of the pile. To avoid this, driving must be stopped when high resistance to penetration is encountered (set of about 1 to 2 mm per blow).

21.2.5 Common Installation Problems

The potential problems associated with driving of wood piles are the splitting and breasting of the pile toe or head, the splitting or bowing of the body of the pile, and the breaking of the pile during driving.
21.3 Precast and Prestressed Concrete Piles

21.3.1 Use of Precast and Prestressed Concrete Piles

Because of their structural strength and wide choice of possible dimensions, precast and prestressed concrete piles can have a wide range of loading. They are:

- suitable for use as shaft bearing piles when driven in sand, gravel, or clay;
- suitable for use as toe bearing piles;
- suitable for resisting uplift forces, when designed for it; and
- suitable for driving in soils containing boulders, when correctly designed.

They have been used for depths up to 15 m for precast concrete piles and up to 40 m for prestressed concrete piles without splicing devices, and to greater depths with splicing devices.

Typical cross-sections are square, hexagonal, and octagonal, with face-to-face diameters of 300 to 650 mm, or cylindrical, with diameters up to 1400 mm. (The larger diameter cylinders are usually hollow and prestressed.)

Design loads vary over a wide range depending on geometry, concrete strength, and reinforcing.

21.3.2 Materials and Fabrication

Concrete piles must conform to the requirements of Subsection 4.2.3 of the National Building Code (1960). High-quality concrete piles, ordinary reinforced or prestressed, should utilize concrete with a 28-day strength of at least 50 MPa. For additional comments on materials and fabrication, see "Recommendations for Design, Manufacture, and Installation of Concrete Piles" prepared by the American Concrete Institute Committee 543.

21.3.3 Pile Splices

Since the length of precast concrete piles is limited by handling conditions, special mechanical splices have been developed to allow the construction of very long precast concrete piles. Quality requirements for concrete pile splices are stringent because of the determining influence that splices have on the integrity and durability of concrete piles. Pile splices are now produced by specialized manufacturers and have been subjected to extensive design review and testing. General requirements for splices are as follows:

- the strength of the splice must be comparable to that of the pile in compression, tension, and bending;

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the splice must be designed and positioned so as to ensure and
maintain the alignment of the joined pile segments; therefore, the
splices must be cast square with the pile segment, and the
maximum permissible deviation (out of squareness) of a pile
segment end is 1:100; and
the splice must be designed so that the tolerance play (deck)
between two joined pile segments is less than 0.5 mm in either
compression or tension (values in excess of this amount will
impair the drivability of the pile.)

Additional requirements have been imposed as to the bending stiffness, etc., of a spliced
pile.

Splices can be obtained with a central tube, usually I.D. 40 mm, cast in each pile
segment, providing inspection access through the entire pile after driving. (See
Subsection 24.3.3.)

21.3.4 Structural Design

The structural design of precast and prestressed concrete piles must conform with the
requirements of Subsections 4.2.4 and 4.5.7 of the National Building Code (1990). The
effects of moments or lateral loads must be considered in the structural analysis of
the pile.

Temporary stresses resulting from handling and driving may be significant factors in the
structural design. For driven piles, common practice is to select a pile having an
adequate factor of safety against structural and/or geotechnical failure under service
loading, and to select the driving equipment, hammer, and pile cushion on the basis of
the structural capacity of the selected pile.

The tolerance on placing reinforcing steel and thickness of concrete cover are important
because too-wide tolerances will result in a too-thick concrete cover on one side (and,
potentially, a too-thin cover on the other). The concrete cover for high-quality piles
should be as thin as possible and about equal to 1.5 times the diameter of the largest
reinforcing bar or prestressing strand, or equal to the specified largest stone aggregate,
whichever value is the larger.

The longitudinal reinforcing bars or prestressing strands should have a minimum
cross-sectional area determined by the condition that the maximum axial tension stress
in the pile, when calculated on the steel area alone, must not exceed the value of 0.5
times the steel yield strength. Practice has shown that, normally, this value is adequate
gainst tension failure including additional tension stresses caused by forces due to
bending and torsion induced during driving. Increasing the prestress will not increase
the tensile capacity of the pile, unless the steel area and/or the steel yield is increased
also.
21.3.5 Installation

Toe driving of precast or prestressed concrete piles requires care. Tensile stresses are high when the penetration resistance is low. On the other hand, when the penetration resistance is high, the compression force reflected at the toe is superimposed on the impact force, and the combined stress may exceed the compression strength of the pile.

21.3.5.1 Required Quality of Pile

21.3.5.1(1) Structural Integrity Before Driving

Piles should be designed and fabricated according to the recommendations in Subsections 21.3.3 to 21.3.4. However, all piles should be carefully inspected before driving and those that have become severely flanged, spalled, or otherwise damaged, should be rejected. Minor flaws having a width of 0.5 mm or less, may be acceptable.

21.3.5.1(2) Pile Head

It is essential that the pile head be perpendicular to the pile axis in order to avoid uneven distribution of impact forces. For anticipated hard driving, it is good practice to protect the pile head by means of a steel plate, which should be at least 15 mm thick. The plate should be anchored into the pile by means of separate reinforcing bars. The pile head should be encased with a steel collar connected to the head plate and extending to a depth equal to half the pile diameter. The plate and collar should be cast with the pile.

When easy driving conditions are expected, the pile head needs only be chamfered at the edges and corners. It is important to ensure that no reinforcing steel or prestressed strands protrude from the head.

21.3.5.1(3) Pile Toe

In most cases, the pile toe needs only to be chamfered at the edges and corners. However, when hard driving conditions are expected, and, in particular, when piles are driven to toe bearing, a special steel shoe should be attached to the pile toe.

Generally, the approach should be that pile shoes should be used if there is no previous experience available from the site, or from a representative nearby site, or from a special study made before finalizing the design indicating that pile shoes are not necessary.

21.3.5.1(4) Centre Tubes for Inspection

When precast or prestressed piles are longer than about 25 m, the integrity of the pile after driving can be very difficult to assess from driving records alone. However, it is easy to cast a centre tube in the pile through which access to the full length of the pile is achieved for inspection purposes. The cost of the centre tube is offset by the quality assurance gained, and by the increase of allowable load that this increased assurance justifies. (See Subsection 24.3.3.) However, precast piles with centre tubes are normally
not available from stock and may require a substantial lead time for supply, as opposed to piles without center tubes.

21.3.5.2 Driving Hammers

21.3.5.2(1) Types of Hammers

Drop hammers and diesel hammers are commonly used for driving precast or prestressed concrete piles. Single acting and differential acting air/hydraulic hammers may also be used. Vibratory hammers are not recommended for prestressed concrete piles because of the high tension stresses they generate.

21.3.5.2(2) Size of Hammer

The selection of the appropriate hammer is important. In the absence of experience, a wave equation analysis should be used to guide the selection of the hammer.

21.3.5.2(3) Height of Fall and Impact Velocity

To avoid the formation of tension cracks, the ram velocity, or drop height, should be reduced during early driving when little soil resistance is encountered, and, in general, when driving through soft soils. With reduced ram velocity, the tensile stresses reflected from the pile toe can be kept within acceptable limits.

21.3.5.3 Driving Helmet

21.3.5.3(1) Dimensions

To avoid the transmission of tension or bending forces, the driving cap or helmet should fit loosely, but not too loosely as to prevent the proper alignment of hammer and pile.

21.3.5.3(2) Capblock

A capblock must be placed inside the driving helmet to eliminate the damage that would be caused by direct impact. The most common material for a capblock is a hardwood block, the grain of which is parallel to the pile axis, and which is enclosed in a tightly fitting steel sleeve. A typical thickness is 125 mm. Note that the hard-wood changes its properties during driving and should not be used once it is crushed or burned, since damage to the pile may result.

21.3.5.3(3) Cushion

To avoid damage to the head of concrete piles and to assist in controlling tensile stresses, as the result of direct impact from the steel driving helmet, it is essential that a cushion be provided between the helmet and the pile head. A typical cushion is made of compressible material such as plywood with a minimum thickness of 50 mm. The cushion should be changed before excessive deformation or charring takes place. The cushion must fit snugly inside the helmet. Too loose a fit will result in rapid destruction of the cushion and an undesirable increase in its stiffness.
21.3.6 Common Installation Problems

Two installation problems commonly arise. Rigid horizontal tension cuts may form in the early stages of driving, when the resistance to penetration is low. Such cuts, where visible above ground, frequently indicate severe damage below ground, sometimes even loss of a portion of the pile. In hard driving, the plumb toe or pile head may be crushed in compression.

21.4 Steel H-Piles

21.4.1 Use of Steel H-Piles

Steel H-piles are:

- suitable for use as shaft-bearing piles, toe-bearing piles, and combinations of these two;
- commonly used for any depth, since splicing is relatively easy, and for loads of 350 kN to 1800 kN. Optimum full lengths are 12 to 21 m.

21.4.2 Materials

Steel H-piles must conform to the requirements of Subsection 4.2.3 of the National Building Code (1990). To minimize damage during driving, it is advantageous to use a pile with a high-yield strength. However, it is not always possible to utilize the high-yield stress to obtain higher pile capacity (see Subsection 21.1.1.2). Experience indicates that corrosion is seldom a practical problem for steel piles driven into natural soil. However, in fill ground at or above the water table moderate corrosion may occur. Where these conditions exist, steps should be taken to protect the piles. Among these are the application of coatings (such as coal tar epoxy) before driving, encasement by cast-in-place concrete jackets, cathodic protection, specification of copper content in the steel, or combinations of these. Increasing the steel section to provide an allowance for corrosion is also a common practice, a typical value being 1.5 mm.

21.4.3 Splices

Common Canadian practice is to splice by making full strength butt welds on H-piles. Although this is not always necessary to carry the design loads, it is good practice in order to accommodate the high driving stresses that can be developed in steel. Sufficient time should be allowed for heating the pile segment ends before welding and for the welded pile to cool below 300°C before driving is resumed. Special pile splices available from the industry greatly reduce the splicing (welding) time in the field.
21.4.4 Structural Design

The structural design of steel H-piles must conform to the requirements of Subsection 4.2.7 of the National Building Code (1980) and Subsection 21.1.1 of the Manual. Because of the high strength of steel, handling conditions are usually not considered in the design of steel H-piles. Uneven stresses of up to about 18 MPa can exist between the flanges and the web, owing to the different cooling conditions for the flanges as opposed to the web in manufacture.

21.4.4.1 Driving Conditions

Experience indicates that the maximum stresses developed in a pile occur during the driving process. The impact stress delivered by any hammer does not normally exceed the value of about 200 MPa, which is smaller than the yield strength of ordinary steel used in steel piles. However, because of additional stresses imposed by bending, eccentric blows and, above all, reflected forces from the pile toe, superimposed on the impact force, higher-than-ordinary steel yield strength may be called for. This increased structural capacity, however, does not increase the geotechnical capacity of the pile (see Subsection 21.1.1.3).

Figure 21.1 shows measurements of impact stress developed in a large number of steel piles driven to refusal by different hammer types under different conditions in different parts of Canada. The measurements illustrate the high variability of driving conditions and the necessity of quantifiable control of the developed stresses in actual pile driving.

21.4.4.2 Working Conditions

The pertinent design method and details given in CSA Standard S16 'Steel Structures for Buildings' are those applicable to laterally supported compression members. For piles subjected to moments or horizontal loads in addition to vertical loads, the effects of such loads, as described in Section 20.5, must be taken into account in the structural design of the piles.

21.4.5 Installation and Common Installation Problems

Driving of steel H-piles is generally easy. However, problems can arise when driving H-piles through dense gravel, or through cliffs containing boulders. If driven unsupported under these conditions, the pile toe may deform to an unacceptable extent, and separation of the flanges and web may occur. To avoid problems, when hard driving is expected, the tips of the H-pile should be protected. This can be done by using cast steel drive shoes. Welding steel plates to the flanges at the toe of the pile is an inadequate method. Chambering of flanges is also used to prevent toe damage, when driving H-piles through or into hard material.

File shoes should be used if there is no previous experience available from the site, or from a representative nearby site, or from a special study made before finishing the design, indicating that they are not necessary.
Long H-piles are prone to bending and doglegging, and the straightness of the H-pile cannot be inspected after driving (Hanna, 1968). Therefore caution is recommended when using long H-piles.

All kinds of driving hammers may be used to drive steel H-piles. A wave equation analysis is recommended for use in selection of the appropriate hammer. As a guide and in the absence of local experience, the rated energy of the hammer should be limited to a value of $6 \times 10^8$ (Newton metre) times the cross-sectional area of the pile. (For recommendations for driving cap and capblock, see Subsection 21.3.5.3.) Pile couplings are not used when driving steel H-piles.

Driving H-piles with a poorly fitting helmet, or with the helmet misaligned with the pile and/or non-concetric, may damage the pile head. The damaged pile head may then act as a cushion on the pile and adversely affect the penetration resistance.

Commonly, installation problems with H-piles originate with the use of too small a section of pile.
21.5 Steel Pipe Piles

21.5.1 Use of Steel Pipe Piles

Steel pipe piles may be driven with the lower end of the pipe open or closed. They may be left open or filled with concrete. They can be used as shaft-bearing piles, ice-bearing piles, or isolated piles.

Steel pipe piles are commonly used for:

- variable lengths, since cuts and splicing are easily made;
- diameters up to 60 in.; and
- loads up to 1800 kips.

21.5.2 Materials

21.5.2.1 Steel

The materials to be used for steel pipe piles must conform to the requirements of the National Building Code (1970). Experience indicates that corrosion is not a practical problem for steel piles driven into natural soil. However, in fill soils above the groundwater table moderate corrosion may occur. Where these conditions exist, steps should be taken to protect the piles. Among these are the application of coatings, such as coal tar epoxy before driving, encasement by cast-in-place concrete jackets, cathodic protection, inclusion of copper-content in the steel, or combinations of these. Increasing the wall thickness to provide a margin for corrosion is also a common practice.

For detailed information on corrosion on steel piles, see National Bureau of Standards Monograph 127 (1972) and Bjerrum (1967).

21.5.2.2 Concrete

Steel pipe piles may or may not be filled with concrete. When concrete is used, it must conform to the requirements of Subsection 4.3.3 of the National Building Code (1970). However, the concrete must be 120 min slump or greater, and it should be injected through a short funnel ("elephant trunk"), which causes the concrete to fall down the centre of the pile, allowing air to escape and eliminating the risk that voids will develop owing to entrapped air.

Although allowance is seldom made for the phenomenon, concrete in a confined state (such as when it is confined by a heavy wall steel tube) develops a higher compressive strength than is does without confinement. Similarly, the modulus of elasticity of confined concrete is higher than that of unconfined concrete, such as in a precast pile or in a thin-walled pipe pile (Confined modulus versus Young’s modulus).
21.5.3 Structural Design

The structural design of steel pipe piles must conform to the requirements of Subsection 4.3.4 of the National Building Code (1990). Because of the properties of steel, handling conditions are not normally considered in design.

21.5.3.1 Driving Conditions

Experience indicates that the maximum stresses developed in a pile occur during the driving process. The impact stress delivered by any hammer does not normally exceed a value of about 200 MPa, which is less than the yield strength of ordinary steel used in steel piles. However, because of additional stresses imposed by bending, eccentric blows, and, above all, reflected forces from the pile toe superimposed on the impact force, higher-than-ordinary steel yield strength may be called for. This increased structural capacity, however, does not increase the geotechnical capacity of the pile (see Subsection 21.1.1.1).

Figure 21.1 shows measurements of impact stress developed in a large number of steel piles driven by different hammers under different conditions in different parts of Canada. The measurements illustrate the high variability of driving conditions and the necessity of quantifiable control of the developed stresses in actual pile driving.

21.5.3.2 Working Conditions

The structural capacity of steel pipe piles is determined according to the requirements of the National Building Code (1990) and to Subsection 21.1.1.1 of this Manual.

21.5.4 Installation

Installation of steel pipe piles is generally easy. Problems can arise when driving pipe piles that are closed at the toe through materials containing obstructions, or when driving open pipe piles through dense material, or when using thin-wall pipes. In the first case, piles may deflect and deviate from their design alignment to an unacceptable extent. In the second case, the pipe may be deformed, in particular, at the pile toe, but also by local buckling along the pile.

21.5.4.1 Protection of the Pile

21.5.4.1.1 Piles Driven with Closed Toe

A flat plate of 10- to 20-mm thickness is normally used to close the toe of the pile. The diameter of the toe plate is normally 20 mm larger than that of the pile. When pipe piles are driven into weathered rock or through bouldery soil, a special shoe of cast iron is often needed.

The shoes should be included in the design of all piles if no previous experience is available from the site or from a representative nearby site, or unless a special study made before finalizing the design indicates that pile shoes are not necessary.
Pile shoes are not normally necessary, but the damage and the difficulties arising from not using them when they are necessary more than justify the foregoing requirement. On projects large enough to make an index pile driving programme economical, piles may be pulled for visual inspection of the pile toe. Sometimes, dynamic monitoring observations can clarify the need for pile shoes. If pile shoes are found to be unnecessary, they can then be omitted from the piles still to be installed. To take advantage of this approach, appropriate financial conditions need to be included in the contract specifications.

21.5.4.12) Piles Driven with Open Toe

No special protection is necessary for soft or medium driving. When hard driving is expected, as in dense gravel, a special driving shoe should be provided, which is made of special steel or steel alloy. Regular checks on the level of soil within the pile are necessary to recognize the formation of a soil plug at the pile toe. If the soil within the pipe is cleaned out, the integrity and the straightness of the pile can be inspected.

21.5.4.2 Driving Equipment

All kinds of driving hammers may be used to drive steel pipe piles. A wave-equation analysis should be used in selection of the appropriate hammer. As a guide and in the absence of local experience, the rated energy of the hammer should be limited to a value of $6 \times 10^6$ joules (j) (Newton meters) times the cross-sectional area of the pile. (For recommendations for driving cap and tamp, see Subsection 21.3.5.3.) Pile cushions are not used when driving steel pipe piles.

A pile head damaged by local buckling, will act as a cushion and reduce the effect of the driving. Such damage is normally not caused by too large an impact force, but from misalignment of the pile in the hammer, non-constant impacts, and/or a poorly fitting helmet.

21.5.5 Common Installation Problems

Common installation problems are:

- high pressures may develop in fine-grained soils due to pile driving. This pressure may cause thin-walled piles to buckle locally;

- the impedance of this wall pipe may not be sufficient to allow the development of sufficient force in the pile to achieve adequate pile capacity;

- failure to use backing rings when splicing and to ensure that full butt welds are properly made, which may cause rupture or leaks.
21.6 Compacted Expanded-Base Concrete Piles

21.6.1 Use of Compacted Concrete Piles

Compacted expanded-base concrete piles (also called "pressure injected footings") require the use of special installation equipment handled by persons experienced in the installation work. Compacted base concrete piles develop their bearing capacity primarily from the densification of soil around the expanded base. They are:

- suited for piles in granular soils, in particular in loose sands, where high capacities can be developed at shallow depths, and for piles subjected to uplift forces provided they are structurally designed for this condition;
- usually unsuited in loose granular soils containing more than about 15 to 20% of fine-grained soil, where special measures are needed to ensure the integrity of the base and shaft; and
- commonly used with shaft diameters of 300 to 600 mm, for loads of 500 to 1600 kN, and for lengths of 3 to 15 m.

21.6.2 Materials

Materials used for compacted concrete piles must conform with the requirements of Subsection 4.2.3 of the National Building Code (1990). Because of the installation techniques, however, damp concrete must be used in the compacted base and in the compacted shaft unless an encased shaft is not used. (Damp concrete is concrete with zero slump, and containing about 15 litres of water per cement bag.)

The strength of the concrete should be checked on special compacted samples, although there is currently no standard method for such sampling and testing. When test cylinders are made in a conventional way, water may be added to alleviate difficulties in ensuring a homogeneous product. However, adding water may prevent the discovery of cases where too dry concrete was used in the base.

Compacted concrete piles are sometimes built with an encased shaft. The casing is generally made of light-gauge steel tubing and is intended only to provide protection against intrusion of water or soil during concreting operations. Temporary casing is recommended to protect shafts in soft soil.

21.6.3 Structural Design

The structural design of compacted concrete piles must conform with the requirements of Subsection 4.2.7 of the National Building Code (1990) and to Subsection 21.1.1 of this Manual.
21.6.3.1 Working Conditions

21.6.3.1(1) Piles with Compacted Shaft

Piles with compacted shafts are made of damp concrete compacted against the soil and may be reinforced. The area of concrete effective in load carrying is limited to a value equal to the nominal area of pile shaft corresponding to the outside diameter of the driving tube, unless local experience and measurements of test piles indicate that a larger diameter is achieved on the compaction of the shaft.

21.6.3.1(2) Piles with Encased Shaft

Piles with encased shafts may be reinforced. Where compacted concrete piles have to resist uplift forces, the structural strength of the shaft must be determined accordingly. Consideration must be given to a proper continuity of reinforcing at the junction of the shaft with the base.

21.6.4 Installation

Installation of compacted concrete piles requires the use of special equipment and should be done by an experienced contractor.

For piles with compacted shafts, care must be exercised in order to maintain at all times a sufficient height of dry concrete within the driving tube. If the tube is withdrawn too rapidly, or if too much concrete is expelled, a void may be created between the top of the compacted concrete column and the bottom of the tube. Water and soil may fill this void and produce a reduction (heaving), or even a complete gap in the concrete shaft. To avoid this problem, a constant control is necessary on the quantities of concrete placed, the evaluation of the base of the tube, and the elevation of the top of the compacted concrete.

Under some soil conditions, such as when piles have to be driven through a clay layer into a lower sand deposit, existing piles may heave as the result of driving new piles adjacent to them. Typical cases are discussed by Bresciani et al., (1979) and Clark (1978).

The capacity of compacted concrete piles is related empirically to the volume of concrete and energy imparted to the compacted base. Problems with insufficient load capacities may occur when such piles are used in areas of soil conditions where little or no experience is available.

21.6.5 Common Installation Problems

Poor-quality concrete, resulting from inadequate design of the mix, or from using concrete that is too dry, frequently causes problems. Segregation of aggregates, or loss of cement by adherence to the walls and fins of a drum-type mixer may occur. Premature setting of concrete is of concern, particularly, when heated concrete or casting is used in hot weather.
Serious problems have been caused by poorly made reinforcing cages, or by the use of brittle steel. A reinforced compacted shaft can have its concrete badly damaged by the motion of a broken cage as subsequent charges of concrete are rammed.

Attempts to form compacted shafts under conditions where high pore-water pressures exist, or have been induced in the soil by the driving, may lead to reeking or contamination of the shaft. Inadequate compaction may result from having too much concrete in the drive tube at the time of the ramming.

Heave and displacement caused by driving of nearby piles cause many failures of compacted shafts. This problem should be checked carefully during the installation process.

21.7 Bored Piles

21.7.1 Uses of Bored Piles

Bored piles can be made in different shapes and dimensions. Although cylindrical piles are the most commonly used type, in recent years elements of diaphragm walls have been used in various combinations (I, M, X) as deep-foundation units. Bored piles are being used increasingly because of their high load capacities. Bored piles are:

- best suited for toe-bearing, high-capacity piles in rock or dense till;
- successfully used in stiff clays; and
- commonly used for variable lengths (bored piles excavated with bentonite slurry have been installed at depths in excess of 100 m, for diameters in excess of 1 m and up to 3 m, using up to 4 m bells, and for loads up to 18 000 kN).

Where deposits of loose cohesionless materials have to be penetrated, or where artesian groundwater conditions prevail, it may be necessary to resort to the use of bentonite slurry or temporary casing.

21.7.2 Materials

Materials to be used for bored piles must conform with the requirements of Subsection 4.2.3 of the National Building Code (1990), except where concrete is placed by tunnel or pump, when the requirements of CSA A23.1 concerning maximum slump cannot be met. Stamps of about 180 mm are normally used. The concrete mix should be designed by a person competent in this field of work.

When bored piles are provided with structural steel casings, the appropriate considerations discussed in Sections 11.3 of this chapter also apply.
When bored piles are excavated with bentonite slurry (premixed), the quality of the slurry (density, viscosity, etc.) should be determined and documented, and it should be kept under constant control to ensure that it performs satisfactorily.

21.7.3 Structural Design

The structural design of bored piles must conform to the requirements of Subsection 4.2.7 of the National Building Code (1990).

21.7.4 Installation

21.7.4.1 Excavation

The excavation for a bored pile may be made:

- by using a large diameter auger or bucket drill to remove the soil above the founding level;
- by driving, vibrating, or pushing down a heavy casing to the proposed founding level, and by removing the soil from the casing either continuously as driving proceeds or in one sequence after the casing has reached the founding level;
- by using a damshall mounted on a Kelly bar to remove the soil, and by keeping the excavation open by use of a bentonite slurry; and
- by drilling, coring, or chopping when penetration into rock is specified (blasting could adversely affect the properties of surrounding rock and soil).

Selection of the excavation procedure depends upon the soil and prevailing groundwater conditions.

Regardless of the procedure used for excavation, it is essential that the base be cleaned to the sound founding material, and that groundwater be controlled to that excess uplift pressure do not act below the founding level, and water and soil do not flow over the prepared base. It is also essential that the walls of a socket in rock be cleaned of loose rock or stone, when loads are designed to be transferred to the founding rock by bond of the concrete to the walls of the socket (see Section 20.5).

21.7.4.2 Placing Concrete

After the excavation has been completed, inspected, and accepted, concrete may be placed during one continuous operation. Steel reinforcement, steel struts, or core sections should be accurately placed and adequately supported. Should the method of pile construction specify removal of the casing, care should be exercised to ensure that the reinforcement is not disturbed or exposed to surrounding soil during the removal process. Where the excavation is dry, concrete may be placed by buckets, funnel,
Fuses, or 'elephant trucks', so that segregation does not occur. Free-falling concrete must be poured through a centering chute, making it fall down the center of the hole, well clear of the walls of the shaft. This results in adequate compaction below the upper 1.5 m. Vibration of the concrete is then required for the upper 1.5 m to produce a concrete of uniform strength. Concrete slump equal to or exceeding 125 mm must not be vibrated, but gently troweled only.

If ground conditions are such that the casing may be removed during the concreting of the pile, the procedure used should ensure that the concrete will not be disturbed, pulled apart, pinched off by earth movement, or contaminated by the entry of water or mud. Usually, the level of the concrete should be maintained at a height above the bottom of the casing sufficient to counter the head of any water or mud outside the casing. If the method of pulling has created an annulus around the casing, mud may fill this annulus and mix with the casing as it is withdrawn, creating a temporary head, which may cause intrusion if the cutoff level of the pile is below ground level.

Placing of concrete is best done by pumping, although a tremie may be used with adequate safeguards. With either method, the pour must be fast and continuous. Use of a tremie is best restricted to piles of shorter than 15 m cored length, which can be handled with a single length of pipe, i.e., with no joint to be uncoupled. A tremie should be withdrawn slowly and should have a uniform and smooth cross-section to minimize disturbance after placement. Employment of a retardent is desirable.

21.7.5 Common Installation Problems

Some common problems associated with the installation of bored piles are:

- Inadequate precautions to control ground-water flow during excavation, resulting in loss of ground and potential long-term undermining of floor areas;
- The flow of concrete through water when the tremie pipe is pulled out of the concrete during placing (the result is a layer or pocket of sand and gravel and a concentration of cement or lime at cut-off levels);
- The introduction of soil in the theoretical concrete section (rocking), caused by too-rapid withdrawal of the temporary casing;
- The temporary casing becomes stuck and is withdrawn after partial set of concrete has taken place, causing cracking of the shaft;
- Use of concrete that is too old when placed (a retarder should be specified when delays are possible); and
- Use of low-slump concrete without vibration, causing the formation of voids.
CHAPTER 22

STATIC LOAD TESTING OF PILES

22.1 Use of a Static Load Test

As indicated in Chapter 19, test loading of piles is the most positive method of determining load capacity of a pile. Depending upon the type and size of the foundation, such tests may be performed at different stages during design and construction.

Static load tests of piles should be carried out according to ASTM D-1149. This standard is reviewed and revised about every five years, and the latest standard should be used. It should be noted that the ASTM D-1149 specifies what is required and how to arrange and document a test. It does not specify a particular method, but gives several optional methods.

22.1.1 Load Tests During Design

Results of static load tests are indispensable aids to the design of pile foundations. The number of tests, type of piles tested, method of driving or of installation and loading should be selected by the engineer responsible for the design. The following points should be considered:

- a detailed soil investigation should be carried out at the test location;
- the piles, and the installation equipment and procedure should be those intended to be used in the construction of the foundation;
- the pile installation must be observed and documented in detail;
- the piles should be loaded to at least twice the proposed working load, and preferably to failure;
- the arrangement, execution, and reporting of the test should closely follow the appropriate ASTM Standards (D-1143, push test; D-5669, pull test; and D-3996, lateral test); and
- where possible, measurements using inclinometers should be made of the toe movement of the pile to allow for a separate evaluation of shaft and toe resistance.

22.1.2 Load Test During Construction

It is recommended practice to perform proof tests on representative units at early stages of construction. The purpose of such tests is to ascertain that the allowable loads used for design are appropriate, and that the installation procedure is satisfactory.
The selection of the test piles should be made by the engineer responsible for the design. The selection should be made on the basis of observed installation behaviour.

22.1.3 Routine Lead Tests for Quality Control (Inspection)

Where full advantage of Sentences 4.2.4.1.(1)(a) and 4.2.7.2.(2) of the National Building Code (1990) is to be taken, a sufficient number of tests must be carried out on representative units to assess and verify the uniformity of the allowable loads and the proper behaviour of the constructed foundation. Test loading for control should be performed on one of every 50 piles, or portion thereof, of the same type and capacity. Tests should also be performed on one out of each group of units, where driving records or other observations indicate that the soil conditions differ significantly from those normally prevailing at the site. Selection of the deep foundation units to be tested is the responsibility of the design engineer.

Static load testing is expensive, and while it is not practical to attempt a statistically representative number of tests, usually more than one test is necessary. However, the necessary number of static tests can be significantly reduced if combined with dynamic testing and monitoring, where the static testing will serve essentially as a calibration of the dynamic testing. Dynamic monitoring can be performed much more frequently without loss of technical reliability, and at the same or less cost. (See Subsection 21.1.3.)

22.2 Test Arrangement

A static load test must be arranged in conformity with the ASTM D-1143, if the minimum distance and accuracy values recommended by the ASTM standard are reduced, the reliability and usefulness of the test results could be impaired. For instance, the specified clear distances between the measuring beams, the platform supports, etc., and the test pile, are minimum values, which actually mean that some — usually negligible — erratic influence on the test data is accepted. When performing other than routine tests, it is advisable to increase these values.

In a routine test, the load is generally applied by means of a hydraulic jack, which is also used as a load gauge measuring the applied load. This system may have an apparent high accuracy, because of the use of a high-precision manometer. Nevertheless, because of many influencing factors not evident in a laboratory calibration, an actual sudden error, which can be as high as 20% of the applied load, often affects the load values. This error is usually on the underside, where a higher accuracy and confidence in the test results are needed, i.e., where potential errors of up to 20% cannot be accepted. A separate load cell has to be used as the main gauge for determining the load. The jack pressure gauge should then be kept as a back-up.

The load cell must be suitable for field use, i.e., have a low sensitivity toward inclined and eccentrically applied loads, and toward temperature variations.

In order to ensure reasonably accurate load values in the field, the ASTM D-1143 recommendation to use a thick steel plate on both sides of the jack and load cell, and the use of a spherical bearing plate (swivel plate) must be observed.
The measuring of the pile compression by means of a telltale to the pile toe, and consequently, also the movement of the pile toe, will greatly enhance the value of a loading test. Whenever possible, it should be considered also for routine tests.

22.3 Load Testing Methods

22.3.1 Methods According to the ASTM Standard

The ASTM Designation D-2143 contains seven separate methods for performing a load test. Reference to the ASTM method, therefore, implies acceptance of any one method unless the desired one is specified. The methods are as follows:

1) ‘Standard Loading Procedure’—a slow maintained-load method using eight equal-load increments to twice the design load. Total test duration is 85 to 72 hours, or more.

2) ‘Cyclic Loading’—the ‘Standard Loading Procedure’ method with unloading and reloading cycles added.

3) ‘Loading in Excursion Standard Test Load’—after finishing the ‘Standard Loading Procedure’ the method, the pile is reloaded until failure or to a predetermined maximum load.

4) ‘Constant Time Interval Loading’—a maintained-load method, in ten equal increments of load until twice the design load. The increments are applied every 60 min, regardless of settlement. The method is similar in all other aspects to the ‘Standard Loading Procedure’ method.

5) ‘Constant Rate of Penetration Method’ (C.R.P.)—requires the use of a special pump that can provide a constant flow of 50 to the jack. Initial penetration rate is between 0.22 and 0.35 mm/min. Total test duration is 2 to 3 hours.

6) ‘Quick Load Test Method’—a maintained-load method using many small load increments applied at constant short time intervals. The test is carried out to failure, or to a predetermined maximum load. Total test duration is 3 to 6 hours.

7) ‘Constant Settlement Increment Loading Method’—a special method, where the applied load increments are varied so as to achieve approximately equal settlements per load increment. The settlement increment is chosen to about 1% of the pile head diameter.

The slow testing methods, (1) to (4), and (7) above, are very time-consuming. When the objective of the test is to determine the bearing capacity of the pile, these methods can actually make the test data difficult to evaluate and displace the pile true load movement behavior, thereby countering the objective of the test. The benefit of the slow
methods lies in the additional soil-pile behaviour information, occasionally obtained, which the interpreting engineer can use in an overall evaluation of the piles.

Settlement cannot be predicted from the results of a static load test. The settlement of a pile group, in particular, cannot be predicted from the results of a test on a single pile, even if the pile test was a long-term one of weeks or months. However, the long-term test on a single pile can occasionally provide valuable information on the pile behaviour and the distribution of load between the shaft and the toe, which can be used as a guide when predicting the group behaviour.

For routine testing and proof testing purposes, the quick methods, (5) and (6) above, are sufficient. Where the objective is to determine the bearing capacity of the pile for a limit states design, the quick test is technically preferable to the slow methods.

On occasion, there could be conflicting reasons and objectives for choosing a quick or a slow testing loading method. The conflict is best resolved by first carrying out a quick test to a soil failure, or a maximum test load, and, thereafter, a slow test, usually according to an elaborate slow-method testing programme. (If the slow method is performed first, the value of the quick-method test is impaired.)

An additional aspect in favour of the quick-testing method is that it is the easiest and also the least costly testing method of the seven listed.

When a quick test is performed, the duration of each load increment should not exceed 15 minutes, nor should it be shorter than 3 minutes unless a well trained crew is at hand. When manually recorded telltale measurements are included, the duration should be not shorter than 10 minutes. The number of load increments to failure, or maximum load, should be more than 25, preferably 35 to 40.

When the pile is unloaded, the oil in the hydraulic pump is first leaked a small amount. This normally shows up as a significant pressure drop on the manometer, indicating a reduced jack load. However, the actual load reduction is small, as can be observed when a load cell is used to obtain the true load on the pile. The load difference between the jack load and the load cell is an indication of the friction in the jack. Then, the oil is leaked at a bit at a time and, after a wait period of about 2 minute readings are taken of load, manometer pressure and deformation is approximately five to seven levels. Note that the pressure in the jack is constantly being reduced. During the unloading of the pile, the pump must not be activated to load up the pile (e.g., to bring the load up to a level that was 'missed').

In particular, when telltale measurements are included, the data from the first couple of load increments are extremely important for the complete evaluation of a test. Once a test is completed, additional valuable data can be quickly and cheaply obtained by a quick reloading of the pile to within three increments of the previous maximum load, followed by the regular quick test method from this load onwards.
22.3.2 Other Testing Methods

It may be necessary to test piles under loading conditions other than the usual axial compressive load; for example, pullout tests (ASTM D-3569) and horizontal (lateral) tests (ASTM D-2966) may be specified.

Pullout testing is similar to compressive testing (push test) but does not require as elaborate a reaction arrangement. It should be recognized that the pile shaft capacity in pull is smaller than the shaft capacity in push. Therefore, in a combination test—push and pull—the difference in capacity will not indicate the correct toe resistance of the pile.

When one designs and performs a lateral test, the following points should be considered:

- the method of applying horizontal loads, for instance, by inserting horizontal jacks between the heads of two adjacent piles in a group or in a row, may not be acceptable in very stiff clay or dense granular soil, unless the spacing between the piles is larger than 10 pile diameters;

- in most cases, it is not sufficient to measure the horizontal displacement of the pile head versus the applied load. To allow for an appropriate evaluation of the elastic behavior of the pile-soil system, and in particular of the coefficient of subgrade reaction, \( k_s \), it is also necessary to instrument the pile for the measurement of bending stress and curvature;

- since horizontal loads applied by the structures are generally of a transient nature (wind loads, earthquakes, etc.), it may be necessary to provide similar cyclic loading conditions in the test; and

- since pile heads are generally confined by the pile cap, the test should preferably be performed on a head-restrained pile.

22.4 Presentation of Test Results

The results of tests performed according to any of the methods described in Subsection 22.3.1 should be presented in a report conforming to the requirements of ASTM D-1143. Presentation of the results should include a load-movement curve and a time-movement curve.

22.4.1 Load-Movement Curve

The load-movement readings taken should be presented in a diagram with the loads on a linear scale on the ordinate, and the observed movements (at both the beginning and the end of each load increment) on a linear scale on the abscissa.
To facilitate the interpretation of the test results, the scales for the loads and the movements should be selected so that the line representing the calculated elastic deformation of the pile will be inclined at an angle of about 20 degrees to the load axis. The elastic shortening, \( \delta \), is computed from:

\[
\delta = \frac{Q L}{A E}
\]

where
- \( \delta \) = calculated elastic compression
- \( Q \) = applied load
- \( L \) = pile length
- \( A \) = cross-sectional area of the pile
- \( E \) = elastic modulus of the pile material

22.4.2 Time-Movement Curve

The time-movement readings should be presented in a diagram with the time on a linear scale on the abscissa, and the observed movements on a linear scale on the ordinate.

22.5 Interpretation of Test Results

There is a wide variety of criteria for interpreting test-loading results, which can be divided into two groups:

- Criteria governing the acceptance of the tested pile. Typical of these is the method that one was specified in the National Building Code (1970 edition). In these methods, no consideration is given to the failure load of the pile. In most cases, a pile is deemed acceptable if the observed settlement of the pile head is within specified limits, which are selected independently of the type and length of the pile. These methods overestimate the capacity of a short pile and underestimate the capacity of a long pile and should not be used.

- Criteria defining the failure load of the tested pile, from which the allowable load may be computed by applying factor of safety. Such methods should be used because they provide a better understanding of pile capacity and behaviour.

Many different failure criteria have been proposed in the technical literature, a number of which have been discussed by Fellenius (1973b, 1980). The failure loads as evaluated from the different criteria show a range of about 30% from the lowest to the highest. The person responsible for the evaluation of the results of a test must choose the criterion of his own technical preference. As a guide for the choice, three methods are briefly described and commented on below:
22.3.1 The Offset Limit Load

The offset limit load, $Q_{o}$ (Davisson, 1977), of a pile is the load that produces a movement of the pile head, which is equal to:

$$\Delta = \delta + (4 + 8b) \times 10^{-3}$$

where $\Delta$ = the movement of the pile head at the offset limit load
$A$ = elastic shortening of the pile
$b$ = diameter of the pile at the toe, or the base of an expanded base pile

When using the offset limit load expanded base piles, the theoretical diameter of the base should be used as the pile diameter.

The offset limit load is intended for driven piles. When applied on bored piles, it becomes impractically conservative.

The offset limit load criterion is represented by a straight line on the load-movement curve (Figure 22.1). The load-movement curve of the test intersects the line at point $P$, the ordinate of which is the limit load of the test.

The offset limit load provides a failure value that is always conservative. A primary advantage is that the actual limit load can be drawn in the load-movement diagram already before starting the test. The limit load can, therefore, be used in an acceptance criterion for proof-tested piles in contrast specifications; for instance, "the pile head movement for 180% of the design load shall not exceed the elastic free-standing column compression of the pile plus $(4 + 8b) \times 10^{-3}$, where $b$ is the pile toe diameter."

The disadvantage of the offset limit load lies in the difficulty of determining Young's modulus, $E$, for concrete piles and corroded pipe piles. Also, the criterion is very sensitive for errors or inaccuracies in both load and movement values.

Furthermore, in an actual case, the interpreted limit load is often more a value representative for the boundary between semi-elastic and semi-plastic ranges of pile-movement behaviour. In other words, it is sometimes too conservative and can deviate significantly from the plunging failure, which, when it occurs, is then considered as the true ultimate failure load.

The offset limit load criterion is primarily intended for interpretation of quick testing methods, but it can also be used when interpreting results from the slow methods. It is not suitable for testing methods that involve unloading and loading cycles.

22.5.2 The Brinch-Hansen Failure Criterion

The Brinch-Hansen criterion (Brinch-Hansen, 1961) assumes that the shape of the pile load-movement curve is such that when the movements, $\Delta$, are plotted along the abscissa in a diagram with $\sqrt{\Delta/Q}$ along the ordinate, the data plot in a straight line.
Figure 22.1: Construction of the Offset Limit Load (after Fellentes, 1980).

having a slope, $C_3$, and a y-intercept, $C_2$ (see Figure 22.2). The criterion is then as follows: the load $Q_0$ for the movement, $\Delta u$, is the failure load, if the load 0.80 $Q_0$ gave the movement 0.80 $\Delta u$.

After determining the slope, $C_3$, and y-intercept, $C_2$ of the plotted line, the ultimate failure load and movement at failure are:

$$Q_u = \frac{1}{k (C_1 - C_2)}$$  and  $$\Delta u = C_2/C_1$$

The Brinch-Hansen criterion has the advantage that it agrees well with the plunging failure and is subjectively conceived as the true failure value.

The criterion is applicable on both quick- and slow-testing methods. However, it is not suitable for methods that include unloading and loading cycles.
Figure 22.2: Ultimate Failure According to the 80%-Criterion (after Fellenius, 1983).

The disadvantage of the Brinch-Hansen 80% criterion, as opposed to the oldest limit load criterion, is that the plot and calculations involved cannot be performed in advance of the test loading.

The Brinch-Hansen failure load is only used if the failure load was reached during the test and, in particular, if the point 0.80Q, 0.25 Q are plotted on the test curve. If these conditions are not fulfilled, plunging failure has not been obtained in the particular test.

When plunging failure has not been obtained in a test, then for design purposes, the failure load must be assumed to be equal to the maximum load applied. The test-loading curve must not be extrapolated to a failure value.

22.3.3 The Chia Failure Criterion

The Chia criterion (Chia, 1977) uses a plot similar to that of Brinch-Hansen, plotting A/Q along the ordinate and A along the abscissa. When failure is approached, these data point plot in a straight line, and the inverse slope of the line is defined as the failure load.

The Chia criterion always results in a failure load that is greater than the maximum test load applied and is therefore less useful. The value of the 'Chia line' lies in that it can be used to indicate potential damage to the pile, as can be shown by sudden changes (i.e., curves or kinks) in the line.
CHAPTER 23

FACTORs OF SAFETY AND ULS FACTORS FOR PILES

23.1 Factor of Safety

To obtain the allowable pile load, $Q_a$, the capacity of the pile, $Q$, should be divided by an appropriate factor of safety. Large factors of safety may be required:

- for friction piles in clay;
- where only a limited number of tests are performed and where soil conditions are variable;
- for piles in loose sands and silts where the capacity may decrease with time.

The factor of safety is a function of the assurance obtained from the method of determining the capacity, its representativeness, etc. For instance, in a testing programme early in the design work, using piles that are not necessarily the same piles as will be used for the final project, the safety factor applied could be 2.5. In the final design phase, when additional information has become available, the factor could be reduced to, say, 2.2. Then, when a contractor has been selected and testing is carried out on the actual pile used for the project, the factor could be reduced to 2.0. Provided conditions are shown to be favourable, the factor to apply for proof testing could become 1.8. Finally, at one point, the design loads on the piles will stay the same, but the pile lengths become shorter. Such a stepwise reduction of the safety factor only applies to large and well-engineered projects, where cost savings without compromising assurance is emphasized. For an instructive discussion on level of safety factor versus effort of pile investigation see O'Brien and Lovell (1983).

23.2 Ultimate Limit States Design

In analysis according to the ultimate limit states design, ULS, the factored pile capacity, $R_u$, is computed from the factored bearing capacity coefficients, which are obtained from insertion of the factored soil-strength parameters, $f_s$ and $f_r$, into the formulae given in Section 20.2.

When the capacity of a pile is determined from field testing, the factored capacity is obtained by multiplying the ultimate failure load on the pile with a performance factor, $f_p$.

The performance factors, $f_p$, recommended for use in pile design are given in Table 23.1. (See also Chapter 7.)
<table>
<thead>
<tr>
<th>ITEM</th>
<th>ULS-FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downdrag loads (negative skin friction)</td>
<td>$\psi = 1.25$</td>
</tr>
<tr>
<td>Bearing Capacity: In Situ Testing</td>
<td>Performance Factor $\psi = 0.5$</td>
</tr>
<tr>
<td>Static penetrometer test</td>
<td>2.3</td>
</tr>
<tr>
<td>Standard penetration test</td>
<td>0.8</td>
</tr>
<tr>
<td>Static test loading (routine test)</td>
<td>0.6</td>
</tr>
<tr>
<td>Static test loading (high technical level test)</td>
<td></td>
</tr>
<tr>
<td>Dynamic analysis using measured data of strain and acceleration</td>
<td>0.5</td>
</tr>
</tbody>
</table>

High technical level test is a test that includes a number of features not normally employed in a routine push test, thereby increasing the confidence of the reliability and the representativeness of the conclusions drawn from the result of the testing. For instance, testing more than one pile, using telltales to evaluate the load distribution in the pile, combining static testing with dynamic monitoring, and/or other aspects aimed to improve the quality of the test.