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

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

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

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

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

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

13.9 CAPACITY OF A PILE GROUP

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

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

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

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

13.10 SUMMARY OF DESIGN PROCEDURE FOR CAPACITY AND STRENGTH

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

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

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

13.10.1 Compiling Site and Soil Information

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

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

13.10.2 Capacity and Allowable Load

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

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

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

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

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

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

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

13.11 SETTLEMENT OF PILE FOUNDATIONS

13.11.1 Introduction

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

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

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

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

13.11.2 Conventional Approach

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

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

where

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

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

Cohesionless Soil

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

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

and

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

where

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

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

Cohesive Soil

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

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

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

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

13.11.3 The Janbu Tangent Modulus Approach

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

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

$$\varepsilon = \frac{1}{mj}\left[\left(\frac{\sigma_1'}{\sigma_r}\right)^j - \left(\frac{\sigma_0'}{\sigma_r}\right)^j\right] \tag{13.15}$$

where

ε = the strain induced by the increase of effective stress
σ_0' = the original effective stress
σ_1' = the new effective stress
j = the stress exponent
m = the modulus number, which is determined from testing in the laboratory and/or in the field
σ_r = a reference stress, a constant, = 100 kPa (1 tsf)

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

$$\varepsilon = \frac{1}{5m}(\sqrt{\sigma_1'} - \sqrt{\sigma_0'}) \tag{13.16}$$

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

$$\varepsilon = \frac{2}{m}(\sqrt{\sigma_1'} - \sqrt{\sigma_0'}) \tag{13.16a}$$

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

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

$$\varepsilon = \frac{1}{5m_r}(\sqrt{\sigma_p'} - \sqrt{\sigma_0'}) + \frac{1}{5m}(\sqrt{\sigma_1'} - \sqrt{\sigma_p'}) \tag{13.17}$$

$$\varepsilon = \frac{2}{m_r}(\sqrt{\sigma_p'} - \sqrt{\sigma_0'}) + \frac{2}{m}(\sqrt{\sigma_1'} - \sqrt{\sigma_p'}) \tag{13.17a}$$

where

σ_0' = the original effective stress (kPa or tsf)
σ_p = the preconsolidation stress (kPa or tsf)
σ_1' = the new effective stress (kPa or tsf)
m = the modulus number (dimensionless)
m_r = the recompression modulus number (dimensionless)

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

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

$$\varepsilon = \frac{1}{m}\ln\left(\frac{\sigma_1'}{\sigma_0'}\right) \tag{13.18}$$

and in an overconsolidated soil:

$$\varepsilon = \frac{1}{m_r}\ln\left(\frac{\sigma_p'}{\sigma_0'}\right) + \frac{1}{m}\ln\left(\frac{\sigma_1'}{\sigma_p'}\right) \tag{13.19}$$

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

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

$$m = \frac{E}{5(\sqrt{\sigma_1'} + \sqrt{\sigma_0'})} = \frac{E}{10\sqrt{\sigma'}} \tag{13.20}$$

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

$$m = \frac{2E}{(\sqrt{\sigma_1'} + \sqrt{\sigma_0'})} = \frac{E}{\sqrt{\sigma'}} \tag{13.20a}$$

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

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

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

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

TABLE 13.3 TYPICAL AND NORMALLY CONSERVATIVE MODULUS NUMBERS.

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

13.11.4 Calculation of Pile Group Settlement

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

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

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

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

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

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

13.11.5 Summary of Settlement Calculation

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

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

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

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

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

13.11.6 Special Aspects

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

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

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

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

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

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

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

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

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

13.12 STATIC TESTING OF PILES

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

13.12.1 Testing Methods

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

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

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

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

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

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

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

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

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

13.12.2 Interpretation of Failure Load

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

To be useful, a definition of failure load must be based on some mathematical rule and generate a repeatable value that is independent of scale relations and the opinions of the