A REVIEW OF PILE SET-UP

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Abstract

Set-up (time-dependent increase in pile capacity) has long been recognized, and can contribute significantly to long-term pile capacity. Its incorporation into pile design can offer substantial benefits. It is not commonly taken advantage of, resulting in significant economic loss, especially in the transportation industry. This paper presents a review based on an extensive survey of published and unpublished literature gathered by the authors, and provides a detailed case history of evaluation of pile set-up from a Wisconsin project.

Introduction

Set-up (time-dependent increase in pile capacity) has long been recognized, and can contribute significantly to long-term pile capacity. Its incorporation into pile design can offer substantial benefits. It is not commonly taken advantage of, resulting in significant economic loss, especially in the transportation industry. Driven piles comprise a substantial cost for state departments of transportation. For instance, the majority of bridge structures constructed for the Wisconsin Department of Transportation ("WisDOT") are supported on deep foundations consisting of driven piles. In calendar year 2000, the WisDOT bid 106,000 linear feet of driven piling, with a bid value of $1.8 million.

It is known that after installation, pile capacity increases with time. This capacity increase is known as set-up, and was first mentioned in the literature in 1900 by Wendel [Long et al., 1999]. Set-up has been documented in fine-grained soils in most parts of the world [Soderberg, 1961], and has been demonstrated to account for capacity increases of up to 12 times the initial capacity [Titi and Wathugala, 1999]. Set-up rate and magnitude is a function of a combination of a number of factors [Samson and Authier, 1986], the interrelationship of which is not well understood. If it were possible to incorporate the effects of (or predict) set-up during design, it may be possible to reduce pile lengths, reduce pile sections (use smaller-diameter or thinner-walled pipe piles, or smaller-section H-piles), or reduce the size of driving equipment (use smaller hammers and/or cranes). Any one, or a combination, of these reductions should result in cost savings.

The primary objective of this paper is to provide a synthesis through a thorough review of the literature and the state of the practice. The ultimate desire is to be able to estimate set-up reasonably accurately without substantially increasing subsurface exploration costs. The authors suggest an approach that can be adopted to ultimately meet this desire.

Description of Set-Up Phenomenon

As a pile is driven, it displaces soil. Soil is displaced predominately radially along the shaft (some vertical displacement along the shaft may also occur), and vertically and radially beneath the toe. Randolph et al. [1979] states that in clay, pile driving can significantly alter the stress in the soil out to approximately 20 pile radii. Yang [1970] indicates that in clay, soil for a distance from the pile of approximately ½ pile diameter is completely remolded, and for a distance of approximately 1.5 pile diameters exhibits increased compressibility. These phenomena occur with a "displacement" pile (e.g., a closed-end pipe pile, or a non-displacement pile which forms a soil plug), and albeit to a lesser extent, they also occur with a "non-displacement" pile (e.g., an H-pile, or an open-end pipe pile), even absent a soil plug.

As soil around and beneath the pile is displaced and disturbed, excess porewater pressures are generated, decreasing the effective stress of the affected soil. The increase in porewater pressure is constant

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with depth [Soderberg, 1961], and can exceed the existing overburden stress within 1 pile diameter of the pile [Pestana et al., 2002; Randolph et al., 1979]. Decrease in excess porewater pressure is inversely proportional to the square of the distance from the pile [Pestana et al., 2002]. The time to dissipate excess porewater pressure is proportional to the square of the horizontal pile dimension [Holloway and Beddard, 1995; Soderberg, 1961], and inversely proportional to the soil's horizontal coefficient of consolidation [Soderberg, 1961]. Accordingly, larger-diameter piles take longer to set-up than smaller-diameter piles [Long et al., 1999; Wang and Reese, 1989]. Excess porewater pressures dissipate slower for a pile group than for a single pile [Camp et al., 1993; Camp and Parmar, 1999]. As excess porewater pressures dissipate, the effective stress of the affected soil increases, and set-up predominately occurs as a result of increased shear strength and increased lateral stress against the pile.

From this, it can be seen that the majority of soil disturbance, and excess porewater pressure generation and dissipation, occurs along the pile shaft. For this reason, set-up is primarily attributable to an increase in shaft resistance [Axelsson, 2002; Bullock, 1999; Chow et al., 1998]. Based on tests performed on Monotube® (fluted tapered steel) piles, Fellenius et al. [2000] attributed set-up to stiffening of the soil, not to increased shaft resistance. After set-up, a number of studies attribute failure under axial compressive load to the interface between the pile and the soil [Seed and Reese, 1955; Randolph et al., 1979], while others attribute failure to a shear zone within the soil [Karlsrud and Haugen, 1986; Yang, 1956]. Wardle et al. [1992] concluded that no additional increase in capacity (i.e., in addition to set-up) could be attributed to maintained loads.

Mechanisms

Logarithmically Nonlinear Rate of Excess Porewater Pressure Dissipation (Phase 1). Because of the highly disturbed state of the soil, the rate of dissipation of excess porewater pressure is not constant (not linear) with respect to the log of time for some period after driving. During this first phase of set-up, set-up rate corresponds to the rate of dissipation, and so it is also not linear with respect to the log of time for some period after driving. During this phase of non-constant rate of dissipation of excess porewater pressure, the affected soil experiences an increase in effective horizontal stress, consolidates, and gains strength in a manner which is not well-understood and is difficult to model and/or predict. This first phase of set-up has been demonstrated to account for capacity increases in a matter of minutes after installation [Bullock, 1999].

The duration of the logarithmically nonlinear rate of excess porewater pressure dissipation is a function of soil (type, permeability, and sensitivity) and pile (type, permeability, and size) properties. The less permeable the soil and pile, and the greater volume of soil displaced by the pile, the longer the duration of the logarithmically nonlinear rate of dissipation. In clean sands, the logarithmic rate of dissipation may become linear almost immediately after driving. In cohesive soils, the logarithmic rate of dissipation may remain nonlinear for several days.

In clays, during driving and its associated severe remolding, the horizontal effective stress along the pile surface can be close to zero. After reconsolidation, the effective stress ratio (σ'_h/σ'_v) has been shown to equal 1.2, with the water content of the remolded soil lower (up to 13 percent) than the original intact clay [Karlsrud and Haugen, 1986; Soderberg, 1961].

Logarithmically Linear Rate of Excess Porewater Pressure Dissipation (Phase 2). As discussed above, at some time after driving, the rate of excess porewater pressure dissipation becomes constant (linear) with respect to the log of time. In a number of empirical set-up predictive models, this time after driving at which the rate of excess porewater pressure dissipation becomes logarithmically linear (i.e., the time at which set-up rate also becomes logarithmically linear) is referred to as the initial time, t_o.

During this second phase of set-up, set-up rate corresponds to the rate of excess porewater pressure dissipation, and so for most soils is also constant (linear) with respect to the log of time for some period after driving. During the logarithmically constant rate of dissipation, the affected soil experiences an increase primarily in effective horizontal stress, consolidates, and gains shear strength according to conventional consolidation theory.

As with the first phase after driving, the duration of the logarithmically constant rate of excess porewater pressure dissipation is a function of soil (type, permeability, and sensitivity) and pile (type, permeability, and size) properties. The less permeable the soil and pile, and the greater volume of soil displaced by the pile, the longer is the duration of the logarithmically constant rate of dissipation. In clean sands, logarithmically linear dissipation may be complete almost immediately, or may continue for several hours. In fine-grained granular soils (silts or silty fine sands) or mixed soils (a mixture of fine-grained granular and clay soils), logarithmically linear dissipation may continue for several hours, several days, or several weeks. In cohesive soils, logarithmically linear dissipation may continue for several weeks, several months, or even years [Skov and Denver, 1988]. Azzouz et al. [1990] indicate that a 15-inch-diameter pile may require 200 to 400 days for complete consolidation. Whittle and Sutabutr [1999] state that for large-diameter open-end pipe piles, the time for dissipation of excess porewater pressure is controlled by the ratio of the pile diameter to wall thickness.

Aging (Phase 3). Practically speaking, and as with primary consolidation, there is likely a time after which the rate of set-up is so slow as to be of no further consequence, and effective-stress-related set-up is effectively complete. However, as with secondary compression, it has been demonstrated that set-up continues after dissipation of excess porewater pressures. During this third phase of set-up, set-up rate is independent of effective stress. This is related to the phenomenon of aging. Aging refers to a time-dependent change in soil properties at a constant effective stress, has a frictional and mechanical cause, is active for both fine-and coarse-grained soils, and is attributable to thixotropy, secondary compression, particle interference, and clay dispersion [Camp et al., 1993; Long et al., 1999; Schmertmann, 1991]. Aging effects increase the soil's shear modulus, stiffness, and dilatancy, and reduce the soil's compressibility [Axelsson, 1998; Schmertmann, 1981]. Aging results in an increased friction angle at the soil/pile interface [McVay, 1999]. Aging effects can improve soils which have significant organic content, and increase at a rate approximately linear with the log of time [Schmertmann, 1991]. Thixotropic aging effects occur primarily at very low effective stresses under drained conditions in cohesive soils [Schmertmann, 1991]. Aging appears to have a more-significant contribution to set-up in granular soils than in cohesive soils. Schmertmann [1991] indicates that aging may not always occur.

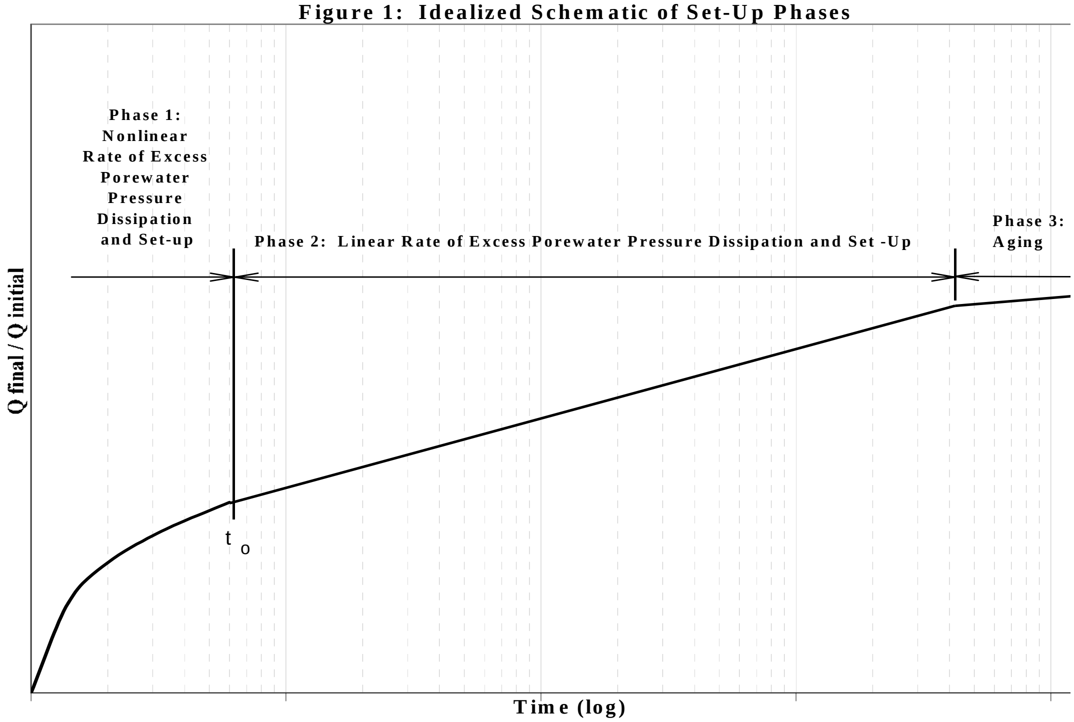
These three phases of set-up are schematically illustrated in Figure 1. For a given soil type at a given elevation along the pile shaft, there is likely some overlap between successive phases, so more than one phase may be contributing to set-up at a time (e.g., aging may begin before essentially complete dissipation of excess porewater pressure). In addition, unless soil conditions are uniform along the entire length of the shaft and beneath the toe, different soils at different elevations will be in different phases of set-up at a given time.

Effect of Soil Type

Set-up is recognized as occurring in organic and inorganic saturated clay, and loose to medium dense silt, sandy silt, silty sand, and fine sand [Åstedt and Holm, 1992; Attwooll et al., 2001; Hannigan et al., 1997]. Holloway and Beddard [1995] observed little or no set-up in very silty low-plasticity cohesive materials. Walton and Borg [1998] indicated that set-up in sand and gravel may not be a significant factor in long-term pile capacity.

Cohesive and Mixed. In cohesive soils (e.g., clay), or a mixture of fine-grained granular and cohesive soils (e.g., clayey silt, or clayey fine sand), driving-induced excess porewater pressure may dissipate slowly. As a result, some set-up is associated with logarithmically nonlinear dissipation (Phase 1), while the majority of set-up is associated with logarithmically linear dissipation (Phase 2). Because of the mechanisms involved, aging (Phase 3) may account for relatively little set-up in these types of soils.

Figure 1: Idealized Schematic of Set-Up Phases



In cohesive soils, the shear strength of the disturbed and reconsolidated soil has been found to be 50 to 60 percent higher than the soil's undisturbed shear strength [Randolph et al., 1979; Seed and Reese, 1955]. At distances from the pile, long-term soil strength decreases with the log of the pile radius, until it equals the soil's initial strength at approximately 10 pile radii [Randolph et al., 1979]. Limiting values of shaft resistance have been found to agree closely with shear strength properties of remolded, reconsolidated clay [Karlsrud and Haugen, 1986]. Randolph et al. [1979] states that stress changes around a pile after installation in clay are nearly independent of the soil's overconsolidation ratio ("OCR"), while Whittle and Sutabutr [(1999] state that reliable set-up predictions for large-diameter open-end pipe piles depend on accurate determination of OCR and hydraulic conductivity. On a percentage basis, soft clays have been found to set-up more than stiff clays [Long et al., 1999].

Fine-Grained Granular. In fine-grained granular soils (silt, or fine sand), driving-induced excess porewater pressure may dissipate relatively rapidly (i.e., almost while driving). As a result, some set-up may occur as a result of logarithmically linear dissipation (Phase 2), while the majority of set-up may be associated with aging (Phase 3) in these soils [Axelsson, 2002]. Either, or both, of these phases may begin almost immediately after driving. A portion of set-up may result from creep-induced breakdown of driving-induced arching mechanisms which results in increased shaft friction, which can be expected to continue for at least several months [Axelsson, 1998; Axelsson, 2002; Chow et al., 1997, 1998; Malhotra, S., 2002]. Loose sands and silts have been found to set-up, some similar to soft clays [Long et al., 1999; Yang, 1970]. Significant set-up has been associated with insensitive clays [Titi and Wathugala, 1999]. Organic silts behave (drain) like clays; inorganic silts behave (drain) like fine sand [Yang, 1970].

To a certain degree, the rate of set-up in granular soils depends on the location of the groundwater table. Above the water table, set-up versus time is a straight-line (arithmetic) relationship. Below the water table, set-up with respect to time is a power function (a function of time raised to some power) [Svinkin et al., 1994]. Capacity increases of approximately 100 percent over 3 months in non-cohesive soil have been observed on some projects, with others demonstrating capacity increases of 20 to 50 percent per log cycle of time [Axelsson, 2002].

Other parameters identified as germane to set-up in non-cohesive soil include pile radius, soil density, soil stiffness (shear modulus), pile-soil dilatancy (which depends on shaft roughness and soil grain characteristics), soil grain characteristics (particle size, shape, and strength), moisture content (saturation), chemical composition of porewater, in-situ stress level, pile geometry, chemical processes, and installation procedure [Axelsson, 2002; Chow et al., 1997, 1998; York et al., 1994; Svinkin et al., 1994]. Set-up is greater for dense sands, and for well-graded sands, than for loose sands, and for uniform sands [Dudler et al., 1968; York et al., 1994]. At present, there is no recognized procedure to identify the conditions which establish the presence, or control the development, of set-up in granular soils [York, 1996].

In dense to very dense silts and fine sands, a decrease in capacity with time, termed relaxation, is also possible [Long et al., 1999; Svinkin, 1994, 2002; Titi and Wathugala, 1999; Yang, 1956; Yang, 1970]. Relaxation is more likely to affect toe resistance than shaft resistance. Weak laminated rocks can also demonstrate relaxation of toe resistance [Hannigan et al., 1997].

Effect of Pile Type

Set-up has been documented for virtually all driven pile types (treated and untreated wood piles, H-piles, open-end pipe piles, closed-end pipe piles, tapered steel piles, fluted steel piles, and pre-stressed concrete piles). Set-up rate decreases as pile size increases [Camp and Parmar, 1999]. Long et al. [1999] offered that there is no clear evidence of difference in set-up between small- and large-displacement piles. A study by Finno et al. [1989] reported that a pipe pile generated higher excess porewater pressures during installation than did an H-pile, but that unit shaft resistances for the 2 piles were approximately equal after 43 weeks.

In sands, a portion of set-up for steel piles has been attributed to corrosion-induced bonding of the sand particles with the steel, and to an increase in volume due to the creation of insoluble ferric-oxide which could lead to increased radial stresses and increased friction [Chow et al., 1997; Chow et al., 1998]. Because of their permeability, wood piles provide an additional conduit for dissipation of excess porewater pressure. Accordingly, wood piles tend to set-up faster than other steel or concrete piles, and more-permeable wood piles set-up faster than less-permeable wood piles [Bjerrum et al., 1958; Yang, 1956]. For piles installed in organic silt, Yang [1956] observed greater set-up for wood piles than for steel H-piles. Pre-stressed concrete piles generally exhibit more set-up than steel piles. This phenomenon has been attributed to a higher soil/pile interface coefficient of friction [Preim and Hussein, 1989].

Measurement of Set-Up

To measure set-up, a minimum of 2 field determinations of a pile's capacity are required. However, the times at which, and the manner in which, such capacity determinations are performed are critical to the value of the information obtained, and the conclusions, which can be drawn from the capacity determinations.

Timing. To maximize measured set-up, the first determination of a pile's capacity should be performed at the end of driving, or as soon after driving as possible, and the second determination should be delayed as long as possible.

Determination of Resistance Allocation. Some determinations of pile capacity measure the overall (combined) pile capacity, without any distinction between shaft and toe resistance. These types of capacity determinations are the least valuable with respect to evaluating set-up, and depending on the relative contributions of shaft and toe resistances, they can provide variable results. For example, consider two piles driven side-by-side, each of which has 50 tons of shaft resistance at the end of initial drive ("EOID"), and each of which experiences set-up which doubles the shaft resistance to 100 tons at the time of retesting. The first pile is driven to a hard layer, and has a toe resistance of 100 tons (at both EOID and at the time of retesting). This first pile had an EOID capacity of 150 tons, and a retested capacity 200 tons, for a set-up ratio (the ratio of a long-term capacity to the end-of-drive capacity) of 1.33. The second pile stops just short of the hard layer, and has a toe resistance of only 25 tons (at both EOID and at the time of retesting). This second pile had an EOID capacity of 75 tons, and a retested capacity of 125 tons, for a set-up ratio of 1.67. The same piles, driven through the same soil deposits and exhibiting the same set-up in both cases, yield different set-up factors. Use of these data in other empirical predictive methods will similarly yield different back-calculated values of other parameters.

Some determinations of pile capacity differentiate between aggregate shaft resistance and toe resistance. These types of capacity determinations are more valuable than those that lump shaft and toe resistances together, but are still limited in determining where set-up is occurring (i.e., in which soil types, at what depths, etc.).

Shaft resistance distribution and toe resistance determination provides the most-valuable determination of pile capacity. It not only differentiates between shaft and toe resistance, but also determines the distribution of the shaft resistance. In this way, set-up can be correlated to soil type, depth, effective stress, soil parameters, etc. This type of determination provides the most flexibility with respect to developing empirical relationships between set-up and various parameters, and the most flexibility with respect to predicting set-up using empirical relationships.

Pile Load Tests. A *top-loaded static load test* is a full-scale proof test, and if carried to geotechnical failure, defines a pile's capacity. A non-internally-instrumented top-loaded static load test determines combined shaft and toe resistance. If internally instrumented with a single tell-tale, or a single strain gage, at the toe, a top-loaded static load test can determine aggregate shaft resistance and toe resistance. If internally instrumented with multiple tell-tales or strain gages, a top-loaded static load test can determine shaft resistance distribution, and toe resistance.

Given the logistics of constructing a reaction system for a top-loaded static load test, it is usually not possible to perform the first test on a pile until several days after driving, during which time some set-up has occurred. For this reason, static load tests are considered impractical to determine initial capacities with reasonable accuracy. If the reaction system is allowed to remain in place, a top-loaded static load test can provide for multiple tests at various times after driving.

Bottom-loaded static load tests are performed using an Osterberg cell ("O-cell") at the toe to load the pile. The cell is a cylindrical hydraulic jack. The soil below the toe of the pile is loaded using the pile's shaft resistance as reaction, and the pile shaft is loaded (upward) using the end-bearing resistance of the soil below the O-cell as reaction. Since shaft and toe resistances are used as reactions to test each other, the test is a full-scale proof test of either the maximum toe resistance, or the maximum shaft resistance, but not the maximum of both (since the maximum of one is reached before the other). Since set-up is primarily attributable to an increase in shaft resistance, the bottom-loaded static load test of most value for evaluating set-up would be one which fails in shaft resistance (i.e., toe resistance exceeds shaft resistance). That limitation notwithstanding, a non-internally-instrumented bottom-loaded static load test can determine aggregate shaft and toe resistance mobilized during the test. If internally instrumented with multiple tell-tales or strain gages, a bottom-loaded static load test can determine the distribution of shaft resistance mobilized during the test, and toe resistance mobilized during the test.

Given the logistics of preparing for an O-cell bottom-loaded static load test (removal of driving equipment, connection of hydraulic lines and instrumentation leads, setting of dial gages, etc.), there is a time lag between end of driving and performing the test, during which time set-up has occurred. For this reason, bottom-loaded static tests are considered impractical to determine initial capacities with reasonable accuracy. Recognizing that set-up is related primarily to an increase in shaft resistance, a bottom-loaded static load test which fails in shaft resistance (toe resistance exceeds shaft resistance) can provide for multiple determinations of shaft resistance at various times after driving.

When a pile is driven, shaft resistance locks residual driving stresses into the pile. If internal instrumentation used in a static load test is installed after driving (and therefore zero or initial instrumentation readings are obtained after driving), these residual stresses must be properly accounted for in reducing the internal instrumentation data. If residual stresses are not properly accounted for, the internal instrumentation data will overpredict shaft resistance at the time of the static load test, in turn overpredicting set-up, which can lead to unconservative design with respect to capacity.

Dynamic Testing. Dynamic testing consists of instrumenting the pile during driving with accelerometers and strain transducers, which are connected to a field-portable digital microcomputer which processes the acceleration and strain signals. The "raw" data as collected in the field is capable of predicting combined shaft and toe resistance. Additional laboratory analysis of field-measured dynamic monitoring data called a CAse Pile Wave Analysis Program (CAPWAP®) analysis is capable of predicting shaft resistance distribution, and toe resistance. CAPWAP analyses also predict residual stresses.

Because it acquires data during driving, dynamic monitoring is uniquely suited to determining capacity instantaneously at the end of driving. Restrike testing can provide for multiple tests at various times after driving. When CAPWAP analyses are performed on end-of-drive and restrike data, the distribution of set-up along the shaft can be determined.

To mobilize all available capacity during end-of-initial-drive or restrike testing, the pile has to move (i.e., have a suitably low penetration resistance). At penetration resistances greater than approximately 10 blows per inch, capacity is likely not fully mobilized, and dynamic monitoring likely underpredicts full capacity. Design and implementation of dynamic testing programs, particularly restrike testing, should address mobilizing full capacity. Interpretation and application of dynamic testing results (both EOID and BOR) should include determination of whether or not full capacity was mobilized.

Estimation/Prediction of Set-Up

Empirical Relationships. Empirical relationships have been offered for quantifying set-up. By far the most-popular relationship was presented by Skov and Denver [1988], which models set-up as linear with respect to the log of time. They proposed a semi-logarithmic empirical relationship to describe set-up as

$$Q_t/Q_o = 1 + A[\log(t/t_o)] \qquad \{\text{Eq. 1}\}$$

where Q_t = axial capacity at time t after driving,
Q_o = axial capacity at time t_o,
A = a constant, depending on soil type, and
t_o = an empirical initial time value.

In this relationship, t_o (initial time) is the time at which the rate of excess porewater pressure dissipation becomes uniform (linear with respect to the log of time), and is illustrated on Figure 1. In practice, closely timed multiple capacity determinations are required to define t_o. Such determinations are seldom practical, so t_o must be assumed, back-calculated from field data, or gleaned from empirical relationships in the literature. It should be noted that t_o is a function of soil type, and pile size. The larger the pile diameter, the larger is the value of t_o [Camp and Parmar, 1999]. Using pre-stressed concrete piles and H-piles, Camp and Parmar [1999] empirically determined t_o equal to 2 days, but stated that t_o equal to 1 day seems reasonable. Using pre-stressed concrete piles installed in non-cohesive soils, Axelsson [1998] set t_o equal to 1 day. Long et al. [1999] recommends using t_o equal to 0.01 day. Svinkin et al. [1994] used t_o equal to 1 to 2 days. Bullock [1999] and McVay et al. [1999], recommend standardizing t_o equal to 1 day.

The A parameter is a function of soil type, and pile material, type, size, and capacity [Camp and Parmar, 1999; Svinkin et al., 1994; Svinkin and Skov, 2000], but is independent of depth, and porewater pressure dissipation [Bullock, 1999; McVay et al., 1999]. The A parameter also must be assumed, back-calculated from field data, or gleaned from empirical relationships in the literature. Chow [1998] reported that data from 14 researchers indicated values of A ranged from 0.25 to 0.75. Studies by Axelsson [1998] yielded A values ranging from 0.2 to 0.8. Data from studies by Bullock [1999] yielded an average A value of 0.21, and suggests that in the absence of any set-up testing it would be conservative to use an A value of 0.2 for all depths in all soils. It should be noted that determination of A, whether from field data or data in literature, is a function of the value used for t_o, and visa-versa; these 2 variables are not independent of each other [Bullock, 1999].

It should also be noted that Equation 1 was developed using combined resistance data (lumping shaft and toe resistance). The majority of studies which empirically determined recommended values of t_o and A were also based on combined resistance data. As demonstrated earlier, the relative contributions from shaft and toe resistance affect the back-calculated values of t_o and A. And since both variables are a function of soil type, values of t_o and A back-calculated using combined resistance data in non-uniform soil profiles are aggregate values averaged over the entire shaft length. To correlate values of t_o and A to soil type in a non-uniform soil profile would require determination not just of aggregate set-up, but of the set-up distribution along the pile shaft.

Prediction of set-up for pile groups using Equation 1 should be modeled using an increased pile size, and a higher value for t_o, as compared to individual piles [Camp and Parmar, 1999]. Svinkin and Skov [2000] state that Equation 1 is pertinent for clay and cohesive soils. Others also developed empirical relationships for predicting set-up. The empirical formulas for predicting set-up are presented in Table 1.

Adhesion Factor. Lukas and Bushell [1989] suggest that in clayey soils, set-up can be estimated by first determining the average undrained shear strength along the length of the pile. The adhesion at the time of driving is then estimated by dividing the undrained shear strength by the sensitivity of the soil. The long-term adhesion can be taken as the undrained shear strength times an adhesion factor, and set-up is the difference between the adhesion at the time of driving and the calculated long-term adhesion. Their study, conducted in the Chicago area, concluded that a reasonable upper bound value of shaft resistance could be the undrained shear strength of the soil. This would indicate that the adhesion factor could be taken as equal to the sensitivity of the soil. In stiff clays, the adhesion factor was 0.83 times the sensitivity of the soil; in soft to medium clays, the adhesion factor was 0.64 times the sensitivity of the soil.

Table 1. Empirical Formulas for Predicting Pile Capacities with Time

Author(s)	Equation	Comments
Huang (1988)	$Q_t=Q_{EOID}+0.236(1+\log(t)(Q_{max}-Q_{EOID}))$	Q_t= pile capacity at time t *(days)* Q_{EOD}= pile capacity at EOD Q_{max}= maximum pile capacity
Svinkin (1996)	$Q_t=1.4Q_{EOID}t^{0.1}$	Upper bound
	$Q_t=1.025Q_{EOID}t^{0.1}$	Lower bound
Guang-Yu (1988)	$Q_{14}=(0.375S_t+1)Q_{EOID}$	Q_{14}= pile capacity at 14 days S_t = sensitivity of soil
Skov and Denver (1988)	$Q_t=Q_o[A\log(t/t_o)+1]$	t_o A sand 0.5 0.2 clay 1.0 0.6
Svinkin and Skov (2000)	$R_u(t)/R_{EOID} - 1=B[\log_{10}(t)+1]$	B similar to A

Exploration-Phase Field Testing. For an exploration-phase field test to be valuable for evaluating set-up, the test must have a significant side shear component, and the ability to separate side shear from end bearing [Bullock, 1999]. A *standard penetration test torque test* (SPT-Torque test, or SPT-T test) is performed on a split-spoon sampler after driving, and measures the shear strength of the soil (in torsional side shear) by turning the drill rods and split-spoon sampler from the surface [Decourt, 1989]. Friction acting on the drill rods is minimized by the use of casing and drilling mud. Early tests were performed using a torque wrench; later tests have been performed using an instrumented (full Wheatstone bridge foil strain gages) torque rod [Raushe et al., 1995]. During the test, both torque and the angle of rotation are recorded. The tests can measure both peak and residual torque, and can be performed at various times after driving. It may be possible to correlate the ratio of initial peak torque to some later peak torque (sampler set-up) to pile set-up. SPT-T tests have yielded consistent and repeatable results [Lutenegger, 1998; Rausche et al., 1995, 1996].

The soonest an initial SPT-T test can be performed is approximately 4 minutes after driving the split-spoon sampler. Bullock [1999] found that with staged testing (testing performed at multiple times after sampler driving), SPT-T side shear decreased in sands, and increased in cohesive soils. He also found that SPT-T set-up data in cohesive soils fits well with pile set-up data after excess porewater pressure dissipation, and that SPT-T test set-up data in sands does not, but that the poor fit in sands may improve if set-up were measured over months, rather than hours. Axelsson [2002] found that with staged testing, the increase in peak torque is considerably higher than the increase in residual torque. The SPT-T test may have direct application in estimating pile shaft resistance in sand [Lutenegger, 1998].

In addition to SPT-Torque tests, Rausche et al. [1995, 1996] also performed SPT-Uplift tests on split-spoon samplers in cohesive soils (between USCS classifications of MH and OH) of medium to high plasticity. The uplift tests were performed 10, 25, and 70 minutes after driving, and measured the soil's shear strength. The measured initial shear strengths related to a normalized value of the Standard Penetration Resistance Test blow count ("N" value). The ratio of the uplift resistance of the 70-minute test to the 10-minute tests is termed the "set-up factor." The soil strengths determined from uplift tests showed good agreement with soil strengths determined from SPT-T tests (uplift strength = 0.685 peak torque strength), and uplift values of soil strength fell between peak and residual SPT-T test values.

An *electric cone penetrometer* with porewater pressure measurements is called a piezocone ("CPTu"). The penetrometer is hydraulically advanced into the ground, and provides a nearly continuous record with depth of tip bearing, side friction, pore pressure, inclination, and temperature. CPTu testing provides more a more-elevation-specific side shear estimate than SPT-T testing, and is not subject to borehole disturbance [Bullock, 1999]. CPTu data aids in the estimation of undrained shear strength [Strniša and Ajdi, 1991].

Because the results of CPTu testing are a function of penetration rate, a standardized penetration rate for the test has been established. The standardization of a penetration rate notwithstanding, since porewater pressures are measured during the test, staged testing can be performed after dissipation of excess porewater pressures. It may be possible to correlate the ratio of initial sleeve friction to some later sleeve friction to pile set-up.

Bullock [1999] found that with staged testing, sleeve friction increased significantly during an initial wait period, then decreased in sands, and increased in cohesive soils. He also found that CPTu set-up data in cohesive soils fits well with pile set-up data after excess porewater pressure dissipation, and that SPT-T test set-up data in sands does not. His findings also indicate that CPTu results in mixed soils do not match pile data as well as results from SPT-T testing.

A *Marchetti dilatometer* ("DMT") is a blade-shaped device which is hydraulically advanced into the ground, and measures membrane stress during expansion in the horizontal direction. If penetration thrust is measured, end bearing and side shear of the blade can be estimated. Staged DMT testing can be performed.

Bullock [1999] found that for piles, the end-of-drive horizontal effective stress was nearly the same as predicted by DMT testing. In turn, end-of-drive horizontal effective stress can be used to predict end-of-drive side shear capacity. He also found that staged DMT testing did not produce useful results for set-up analyses.

It is emphasized that none of the exploration-phase field tests discussed herein (SPT-T, CPTu, nor DMT) produced good results related to prediction of set-up in sands [Bullock, 1999].

Potential Application of Promising Methods to Current Practice

Of the references reviewed, two proposed that applying a set-up factor of 1.2 to end-of-drive capacity to estimate long-term capacity would likely be conservative in all soil types at all depths [Turner and Attwooll, 2002; Koutsoftas, 2002]. Application of such an approach to incorporating set-up in design would simply require that piles be installed to 83 percent of their required capacity. A correlation study for such a relatively simple empirical relationship would require that less-sophisticated testing data be available than for correlation studies for more-complex relationships.

Of the currently available empirical relationships, the one presented by Skov and Denver appears to have the most universal application. The multiple parameters used being functions of each other, soil type, and pile type, permeability, size, and capacity notwithstanding, this formula could be applied using published values for these parameters. Alternatively, the existing state database of pile-testing information could be surveyed for pertinent data, which could be used to back-calculate the parameters in the Skov and Denver relationship. Although the amount of existing appropriate data is limited, such a correlation study may provide more-region-specific values than is possible by using published values.

With the application of any empirical relationship, a factor of safety may need to be applied to the calculated set-up, which adequately reflects the uncertainties involved in the relationship. Davisson [1973] presents a project case history which incorporated set-up into design, but maintained a minimum factor of safety of 1.5 disregarding set-up.

As discussed previously, staged dilatometer test results did not produce useful results for set-up analyses. For this reason, further investigation would be required before dilatometer testing would be considered a viable field-testing methodology for evaluating set-up. As also discussed previously in that same section, SPT-Uplift test results showed good correlation with SPT-T test results, so for evaluation of set-up only one of the tests needs to be performed. Of the two tests, because of ease of performance, cost, applicability, repeatability, and support in the literature, the SPT-T test is considered the more-viable field-testing methodology for evaluating set-up. SPT-T testing appears to show promise as an exploration-phase methodology, which could be employed to evaluate set-up [Bullock, 1999].

Piezocone sleeve friction staged testing also appears to show promise as an exploration-phase methodology which could be employed to evaluate set-up [Bullock, 1999]. Advancement of the piezocone is analogous to pile installation. However, piezocone testing does not work in all soil types, and it has found limited use in predominantly glacial deposits such as found in the Midwestern states.

A Case History

To illustrate methodology by which set-up distribution and magnitude may be determined, as well as to provide an idea of set-up magnitude, results from an indicator test pile from a Milwaukee project are presented. The test pile consisted of a 10.75-inch-O.D. closed-end steel pipe pile, and was installed and tested as part of the indicator pile test program for the Midwest Airlines Center in Milwaukee, Wisconsin. At the test pile's location, soil conditions included loose fine-grained granular, to medium to stiff cohesive, fill deposits to a depth of 13.8 feet. Underlying estuarine deposits consisted of loose clayey organic silt, with fine sand seams and layers, to a depth of 35.0 feet. Underlying inorganic soils consisted of interbedded stiff to hard silty clay to clayey silt, and medium dense silt deposits to the toe elevation of the pile.

The test pile installation was dynamically monitored using a Pile Driving Analyzer® ("PDA"). A subsequent CAPWAP analysis was performed on end-of-initial-drive ("EOID") data. The CAPWAP analysis calculated an EOID shaft resistance of 32.5 tons. Thirty days after its installation, the test pile was statically load tested. Although the static load test was not carried to geotechnical (plunging) failure, extrapolation of the plotted head deflection data resulted in an assigned capacity of on the order of 270 tons, with an internal-strain-gage-determined shaft resistance of 202 tons. Twelve days after being statically load tested, dynamic monitoring was performed during restrike testing of the test pile. A subsequent CAPWAP analysis was performed on beginning-of-restrike ("BOR") data. The CAPWAP analysis calculated a capacity (incorporating both shaft and toe resistance) of 270 tons, showing good correlation with the static load test result. The CAPWAP analysis calculated a BOR shaft resistance of 198 tons, again showing good correlation with the static load test result. The resulting shaft set-up (obtained by subtracting the EOID shaft resistance from BOR shaft resistance) was 165.5 tons, corresponding to a shaft resistance set-up factor of 6.1.

By knowing the unit set-up distribution (unit set-up as a function of depth), depth-specific penetration resistance criteria were developed. For a given pile embedment, cumulative set-up was calculated and subtracted from the required pile capacity to yield the dynamic (EOID) capacity required at that embedment. The penetration resistance required to provide that dynamic capacity was determined by wave equation analysis. Since cumulative set-up increased with depth, the required dynamic capacity decreased with depth. Such incorporation of set-up resulted in the piles having the shortest possible embedded length. On a project like the Midwest Airlines Center which involved hundreds of piles, saving even a few feet per pile can result in savings that pay for the testing required to so characterize set-up many times over.

Recommendations for Incorporating Set-Up into Current Practice

Currently, the vast majority of existing data is of the type collected during conventional subsurface exploration and testing programs. Without additional field testing, correlation studies currently possible would be limited to conventional information (Standard Penetration Test results, unconfined compressive strengths, water contents, and groundwater position). Correlations based on such data are not expected to be meaningful or useful.

For projects on which good pile-testing data is available, it may be possible to supplement existing subsurface information with data from promising exploration-phase field testing methodologies. After a review of the existing database has identified the most-valuable pile-testing data, depending on access and proximity to test-pile locations, it may be possible to perform additional exploration and testing. In this way, the scope of specialized exploration and testing could be tailored to the quality of existing data.

For example, a recent private-sector-designed transportation-related project with pile foundations is the 6th Street Viaduct Replacement project in Milwaukee, Wisconsin. A full indicator pile test program (including end-of-initial-drive and restrike testing, with dynamic monitoring and CAPWAP analyses, and an internally instrumented static load test) was performed at each of 4 major structure locations, representing a variety of subsurface conditions. Set-up distributions have been determined at each of the 4 test sites. The 4 structure locations are post-construction drill-rig accessible. Correlation data could be obtained, and the full benefit of the comprehensive test programs realized, for an investment in subsequent specialized testing.

The largest potential for developing set-up correlations is on future projects for which specialized exploration-phase testing is performed, and for which set-up is directly measured. This could be done on both private- and public-sector projects. Private-sector owners would likely be willing to pay for project-specific pile testing required to characterize set-up if it is anticipated that the testing will ultimately save cost on their project. For this to happen, there has to be an adequate number of piles on the project, so that the savings in pile footage resulting from incorporation of set-up will more than offset the cost of the pile testing program. However, the private-sector owner would likely not be willing to pay for the specialized exploration-phase testing required to advance set-up correlations, because such correlations would offer no benefit to the immediate project. A public-sector sponsor (e.g., a DOT) could pay for specialized exploration-phase testing necessary to advance set-up correlations on the private-sector project, recognizing that correlations will ultimately pay dividends on future public-sector projects.

State DOTs are in the unique position of continually purchasing new pile foundations. On a typical small- to medium-sized bridge project, there often is not an adequate number of piles to warrant a pile testing program necessary to adequately characterize set-up. However, in recognition that there is potential for future cost savings, DOTs could invest in the pile testing and exploration-phase field testing required to advance set-up correlations. Once an appropriate amount of exploration-phase testing data has demonstrated reasonable correlation to pile testing set-up results, use of this type of correlation could be used on subsequent projects where project-specific pile testing to characterize set-up is economically unjustifiable. Using such correlations, set-up could be incorporated into design for the cost of supplemental exploration-phase testing (e.g., SPT-T testing), without the cost of project-specific pile testing.

Conclusions

Obviously, characterization of set-up by direct, project-specific testing is preferable to characterization based on empirical relationships. Where project-specific pile testing to characterize set-up is economically unjustifiable, a number of exploration-phase field tests offering potential value in predicting set-up have been identified. Of these, the SPT-Torque test appears to offer the most-favorable combination of applicability of results, ease and simplicity of performing the test, and cost.

Incorporating estimated set-up (based on empirical relationships developed from historical and future data) into current design practices might be possible. The data for such correlations could be obtained by performing field testing for completed projects where site access and quality of past set-up characterization warrant, and/or by performing exploration-phase field testing for future projects (both public- and private-sector) where set-up will be appropriately characterized. It will be necessary to recognize the limitations of such relationships, and apply appropriate factors of safety to set-up so determined.

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