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Estimating Soil/Pile Set-Up

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ESTIMATING SOIL/PILE SET-UP

FINAL REPORT

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16. Abstract

Soil/pile set-up is time-dependent increase in pile capacity, and can contribute significantly to long-term pile capacity. If it were possible to incorporate the effects of set-up during design, it may be possible to reduce pile lengths, reduce pile sections (use smaller-diameter or thinner-wall 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. Accordingly, this research investigated, through a thorough review of the literature and the state of the practice, if it is possible and practical to estimate set-up during design, using information obtained during a relative routine subsurface exploration program. A literature search was conducted; a references list is provided.

Set-up is predominately associated with an increase in shaft resistance. The complete mechanisms contributing to set-up are not well understood, but the majority of set-up is likely related primarily to dissipation of excess porewater pressures within, and subsequent remolding and reconsolidation of soil, which is displaced and disturbed during pile driving. After excess porewater pressures have dissipated, aging may account for additional set-up. A number of empirical relationships have been proposed to estimate or predict set-up, and have demonstrated reasonable success in a number of studies. Empirical relationships are limited in widespread application by the relationships having been based on combined (shaft and toe) resistance determinations, inter-dependence of back-calculated or assumed variables, and the complexity of the mechanisms contributing to set-up.

A number of exploration-phase field tests offering potential value in predicting set-up have been identified: SPT-Torque test, SPT-Uplift test, piezocone testing, dilatometer testing, and vane shear testing. 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. It is recommended that SPT-Torque testing be performed on a number of sites where satisfactory set-up data is available to determine if a meaningful relationship exists between the "set-up" which develops on a split-barrel sampler, and the set-up which develops on a driven pile, in various soil strata. If a meaningful relationship is determined, SPT-Torque tests could be added to routine subsurface exploration programs to measure sampler/soil set-up in various soil strata, and predict production pile set-up.

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EXECUTIVE SUMMARY

Project Summary

Soil/pile set-up is time-dependent increase in pile capacity. Set-up has long been recognized, and can contribute significantly to long-term pile capacity. Its incorporation into pile design can offer substantial benefits. In recent years, the Wisconsin Department of Transportation (WisDOT) has typically spent between $1.8 and $2.9 million annually on foundation piles. If it were possible to incorporate the effects of (to predict) set-up during design, it may be possible to reduce pile lengths, reduce pile sections (use smaller-diameter or thinner-wall 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 to the WisDOT.

Background

A limited number of practitioners are utilizing dynamic monitoring using a Pile Driving Analyzer™ (PDA) during initial driving, and additional dynamic monitoring during restrike testing several days to a few weeks after initial driving (often in concert with instrumented static load tests), to characterize set-up. Subsequent CAse Pile Wave Analysis Program™ (CAPWAP) analysis of the PDA data allows assessing set-up distribution along the pile shaft. Once set-up has been characterized, it can be applied to pile design. On projects involving a large number of piles, the savings in pile costs can greatly exceed the cost of testing required to characterize set-up. However, on projects involving a relatively small number of piles (e.g., the vast majority of WisDOT bridge projects), the cost of testing to characterize set-up will likely exceed pile installation cost savings. Therefore, on small projects, testing required to characterize set-up directly can rarely be economically justified.

Accordingly, the goal of this research is to investigate, through a thorough review of the literature and the state of the practice, if it is possible and practical to estimate set-up during design, using information obtained during a relative routine subsurface exploration program. The desire is to be able to estimate set-up reasonably accurately without substantially increasing subsurface exploration costs.

Process

A number of researchers, academicians, and practitioners have investigated set-up, and prepared papers presenting the results of their findings and conclusions. A literature search was conducted to find all currently available papers on the subject. The literature search was conducted jointly by Dr. Tuncer Edil with the Department of Civil and Environmental Engineering at the University of Wisconsin–Madison, and Wagner Komurka Geotechnical Group, Inc. (WKG^2) of Cedarburg, Wisconsin. In excess of 100 papers dealing with set-up were found. The papers were thoroughly reviewed by WKG^2 to glean technical information considered relevant to this set-up study. A complete references list is provided.

Findings and Conclusions

Set-up is predominately associated with an increase in soil resistance acting on the sides (shaft) of a pile. Unit set-up has units of force divided by pile side area. The complete mechanisms contributing to set-up are not well understood, but the majority of set-up is likely related primarily to dissipation of excess porewater pressures within, and subsequent remolding and reconsolidation of, soil which is displaced and disturbed as the pile is driven. Depending on soil permeability and amount of disturbance, dissipation of excess porewater pressures is non-uniform (non-linear) with respect to the log of time for some time after driving. Subsequently, excess porewater pressure dissipation becomes uniform (linear) with respect to the log of time. After excess porewater pressures have dissipated, and independent of effective stress, additional set-up may occur due to aging.

Set-up is recognized as occurring in most parts of the world, for virtually all driven pile types, in organic and inorganic saturated clay, and loose to medium dense silt, sandy silt, silty sand, and fine sand, and is related to both soil and pile properties. In cohesive soils, the shear strength of the disturbed and reconsolidated soil has been found to be higher than the soil's undisturbed shear strength. In fine-grained granular soils, the majority of set-up is related to creep-induced breakdown of driving-induced arching mechanisms, and to aging. The more permeable the soil, the faster set-up develops. Set-up rate decreases as pile size increases.

Measurement of set-up requires that a pile's capacity be determined at a minimum of 2 different times. 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. Capacity determinations should always separate shaft and toe resistance, and are most-valuable if shaft resistance distributions are determined. Such capacity determinations can be achieved with top- or bottom-loaded static load tests, or with dynamic testing with subsequent CAPWAP analyses.

A number of empirical relationships have been proposed to estimate or predict set-up, and have demonstrated reasonable success (accuracy) in a number of studies. Empirical relationships are limited in widespread application by the relationships having been based on combined (shaft and toe) resistance determinations, inter-dependence of back-calculated or assumed variables, and the complexity of the mechanisms contributing to set-up.

A number of exploration-phase field tests offering potential value in predicting set-up have been identified: SPT-Torque test, SPT-Uplift test, piezocone testing, dilatometer testing, and vane shear testing. 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. Unfortunately, there are no existing Wisconsin project data which combine adequate characterization of set-up with any of these field tests.

Recommendations for Further Action

Incorporating estimation of set-up based on exploration-phase field testing into WisDOT practices may be possible, provided that the limitations of such estimations are recognized, and that appropriate factors of safety are applied. Incorporating set-up into pile design could be done by developing correlations between exploration-phase field testing results and measured set-up.

The data for such correlations could be obtained for relatively small cost by performing exploration-phase field testing on sites where satisfactory set-up characterization has been (or will be) completed by others, and where access for exploration equipment (drill rigs) is possible. Since detailed set-up characterization is relatively expensive, but exploration-phase field testing is relatively inexpensive, performing exploration-phase field testing on sites where set-up has been (or will be) characterized by others will potentially offer WisDOT a relatively low-cost means of developing set-up correlations.

Specifically, it is recommended that SPT-Torque testing be performed on a number of sites where satisfactory set-up data is available. In an SPT-Torque test, a standard split-barrel sampler is driven into the soil, and torque is applied to the drill rod in a controlled manner (immediately after driving) to rotate the sampler. The measured torque is an indication of the unit shearing resistance between the sampler and the soil. The sampler is then left in place for a period of time, after which torque is again applied to rotate the sampler. By staging the application of torque applied to the sampler, time-dependent characteristics of sampler/soil resistance can be determined. Time-dependent torque increase is a measure of “set-up” between the sampler and the soil.

The focus of the recommended research will be to determine if a meaningful relationship exists between the “set-up” which develops between a split-barrel sampler and the soil, and the set-up which develops on a driven pile in various soil strata.

If a meaningful relationship is determined, SPT-Torque tests could be added to routine subsurface exploration programs to measure sampler/soil set-up in various soil strata. By using the correlations developed by the research program, it would be possible to estimate the magnitude of set-up which will develop on driven piles in the same strata. In this way, set-up could be incorporated into the design of pile foundations for projects having too few piles to make project-specific direct measurement of set-up cost-effective.

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ESTIMATING SOIL/PILE SET-UP

BACKGROUND AND PROBLEM STATEMENT

The majority of bridge structures designed by 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. In calendar year 2001, the WisDOT bid 170,000 linear feet of driven piling, with a bid value of $2.89 million.

It is known that after installation, pile capacity often increases with time. This capacity increase is known as soil/pile 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 value [Titi and Wathugala, 1999]. The rate and magnitude of set-up 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 (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 to the WisDOT.

The primary objective of this research is to investigate, through a thorough review of the literature and the state of the practice, if it is possible to estimate set-up during design, using information obtained during the subsurface exploration program. The desire is to be able to estimate set-up reasonably accurately without substantially increasing subsurface exploration costs.

DESCRIPTION OF SET-UP PHENOMENON

General

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 to an approximate distance of 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 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; Tomlinson; Yang, 1956]. Wardle and Freeman concluded that no additional increase in capacity (i.e., in addition to set-up) could be attributed to maintained loads.

Mechanisms

Related to Effective Stress

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 pressures 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 is also not uniform (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 pore pressures, the affected soil experiences an increase in effective and 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 is 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 [Karlsud and Haugen, 1986; Soderberg, 1961].

<u>Logarithmically Linear Rate of Excess Porewater Pressure Dissipation (Phase 2)</u> – 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 in effective vertical and 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 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.

Independent of Effective Stress – Aging (Phase 3)

For a consolidating soil layer, conventional consolidation theory holds that infinite time is required for dissipation of excess porewater pressure to be complete. Practically speaking, there is a time after which the rate of dissipation is so slow as to be of no further consequence, at which time it is accepted that primary consolidation is complete. However, secondary compression continues after primary consolidation is complete, and is independent of effective stress.

Similarly, since the rate of set-up corresponds to the rate of excess porewater pressure dissipation, it follows that in some cases infinite time would be required for set-up to be complete. Again 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[1], 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 effects 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

[1] Schmertmann (1991) states "(thixotropy) does not appear to have an accepted, precise meaning in soil mechanics, although Mitchell (1960, 1976) has suggested one. The dictionary defines thixotropy as the property of certain gels that liquefy when subjected to vibratory forces, then solidify again when left standing. To most geotechnical engineers it has a general, somewhat vague meaning associated with strength regain with time after some type of remolding. Mitchell has focused his definition to include isothermal conditions and reversible behavior at constant composition and volume. ASTM D653 has long defined thixotropy as 'the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible.'"

[Schmertmann, 1991]. Thixotropic aging effects occur primarily at very low effective stresses under drained conditions in cohesive soils [Schmertmann, 1991]. One way to think of this third phase of set-up is that aging is to set-up as secondary compression is to settlement. This imagery notwithstanding, aging appears to have a more-significant contribution to set-up in granular soils than in cohesive soils. Schmertmann (1991) admits 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 1 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., a clayey silt, or a 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.

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 Hauger, 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 (silts or fine sands), 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, 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 straight-line 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]. Chow et al. (1998) showed capacity increases of 50 to 150 percent in 100 days in sand, with large scatter in the data. Long et al. (1999) suggests 20 to 100 percent capacity increase per log cycle of time in sand, but only for the first 100 days. Tomlinson showed a 2.7-fold increase in shaft capacity in sand at 9 months due to long-term aging, and shaft capacity increases of approximately 50 percent per log cycle of time. Koutsoftas (2002) demonstrated total capacity increases of 1.25 to 1.5 times initial capacity in dense sand. Axelsson (1998) showed an average capacity increase of 40 percent per log cycle of time in non-cohesive soil, with a large scatter in the data. In sand, Long et al. (1999) determined that although the largest set-up occurred in the first 10 days after driving, set-up appeared to continue for up to 500 days, and measured shaft resistances increased 2 times initial. A study by Chow et al. (1997) on open-end pipe piles in dense sand demonstrated an 85 percent increase in shaft resistance in 5 years.

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]. There is meager understanding as to what influence gradation has on set-up [York, et al., 1994]. Set-up is greater for dense sands, and well-graded sands, than for loose and 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

General

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 it.

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.

Recognizing that various mechanisms control settlement at different times after driving, and that the various mechanisms may result in different set-up rates, care must be exercised when back calculating empirical relationship parameters, and when extrapolating set-up rates.

Determination of Resistance Allocation

Combined Resistance – 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 drive ("EOD"), 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 EOD and at the time of retesting). This first pile had an EOD 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 EOD and at the time of retesting). This second pile had an EOD 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 which exhibited 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.

Aggregate Shaft Resistance and Toe Resistance – Some determinations of pile capacity differentiate between aggregate shaft resistance and toe resistance. These types of capacity determinations are more-valuable than those which 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 – This most-valuable determination of pile capacity 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.

Static Load Tests

Top-Loaded

A top-loaded static load test is a full-scale proof test, and if carried to geotechnical failure, defines a pile's ultimate 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

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., end bearing 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.

Residual Stresses

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 and their results, in particular restrike testing, should address mobilizing full capacity.

ESTIMATION/PREDICTION OF SET-UP

Empirical Relationships

Skov and Denver

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

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

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 value measured in days.

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 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 to be 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 (1999), recommend standardizing t_o equal to 1 day.

The *A* parameter is a function of soil type, 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 [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

Svinkin (1996) developed a formula for set-up in sands based on load test data. Guang-Yu (1988) presented an equation for capacity of piles driven into soft soils. The estimates are for capacity on the 14th day after driving, and are based on the sensitivity of the fine-grained soil. Guang-Yu suggested that sands and gravels experience no set-up. Huang (1988) presented a formula for predicting set-up rate in the soft-ground soil of Shanghai. Svinkin and Skov (2000) present a variation of Equation 1, using t_o = 0.1 day. Like the Skov and Denver relationships, all these formulas were developed using combined resistance data (lumping shaft and toe resistance). Unlike the Skov and Denver relationship, these formulas all include the instantaneous capacity at end-of-drive, Q_{EOD}, which can be determined by dynamic monitoring.

The empirical formulas for predicting set-up are presented in Table 1.

TABLE 1

Empirical Formulas for Predicting Pile Capacities with Time

Author(s)	Equation		Comments
Huang (1988)	$Q_t = Q_{EOD} + 0.236(1 + \log(t)(Q_{max}\text{-}Q_{EOD}))$	{Eq. 2}	Q_t = pile capacity at time t, (days) Q_{EOD} = pile capacity at EOD Q_{max} = maximum pile capacity
Svinkin (1996)	$Q_t = 1.4Q_{EOD}t^{0.1}$ $Q_t = 1.025Q_{EOD}t^{0.1}$	{Eq. 3} {Eq. 4}	upper bound lower bound
Guang-Yu (1988)	$Q_{14} = (0.375S_t + 1)Q_{EOD}$ S_t = sensitivity of soil	{Eq. 5}	Q_{14} = pile capacity at 14 days
Skov and Denver (1988)	$Q_t = Q_O[A\log(t/t_o) + 1]$	{Eq. 6}	where t_o / A sand 0.5 / 0.2 clay 1.0 / 0.6
Svinkin and Skov (2000)	$R_u(t)/R_{EOD} - 1 = B[\log_{10}(t) + 1]$	{ Eq. 7}	Derivation of Eq. 1, t_o = 0.1 day B similar to A in Eqs. 1 and 6

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 Chicagoland 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.

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].

SPT-Torque Test

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 side shear torsionally) 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 piles’ shaft resistance in sand [Lutenegger, 1998].

SPT-Uplift Test

In addition to SPT-torque tests, Rausche, et. al. (1995, 1996) also performed uplift tests on split-spoon samplers in cohesive (between USCS classifications of MH and OH) soils 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.

Piezocone Testing

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 of a point 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.

Dilatometer 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].

SURVEY OF EXISTING WISCONSIN DATABASE

In order to provide information valuable for evaluating set-up, or to validate new or existing empirical relationships, or to derive new empirical relationships correlated to exploration-phase field testing, existing pile-testing data must provide: 1) accurate and reasonably complete characterization of subsurface conditions, 2) accurate determination of end-of-drive capacity, preferably with separation of shaft and toe resistance, and further preferably with determination of shaft resistance distribution, and 3) accurate determination of subsequent capacity, preferably with separation of shaft and toe resistance, and further preferably with determination of shaft resistance distribution. Determination of time-dependent capacity at multiple time intervals after driving would be a bonus.

Utilization of existing data for this research effort requires a higher standard of evaluation for the type and integrity of the data than may be required for a given project. For example, to evaluate set-up for a given project, a single pile may be driven to a pre-determined depth or penetration resistance, allowed to set for a period of time, and tested. If the capacity meets project requirements, the test program fulfills its purpose. But without better characterization of the subsurface conditions and results, application to other sites or to deriving empirical relationships is not possible.

Conventional Subsurface Information

To be valuable, pile-testing data must be accompanied by accurate and reasonably complete characterization of subsurface conditions. A specific test pile should have a boring sufficiently proximate to it. The boring should extend a reasonable distance below the pile toe. To minimize bottom blow-in and maintain accurate Standard Penetration Test results if advanced through granular deposits below the groundwater table, borings should be advanced using mud-rotary techniques. The location of the groundwater table should be determined. The ground-surface elevation at the boring location at the time it was performed, referenced to a reproducible datum, should be determined. Pile testing data should be correlated to the same elevation datum. At a minimum, the information obtained from the borings should include Standard Penetration Test blow counts in all soil types, and unconfined compressive strength and water content determinations in cohesive soils. The type of hammer used to perform the Standard Penetration Tests (automatic, safety, cable winch, etc.) should be recorded.

End-Of Drive Capacities

Because it is the only method of pile testing which estimates capacity instantaneously at the end of driving, and also provides not only for separation of shaft and toe resistance but also determines shaft resistance distribution, end-of-drive dynamic monitoring with subsequent CAPWAP analysis of the field data is the most-useful end-of-drive capacity determination method for evaluating set-up. Dynamic testing methods are described in Goble et al. (1980), Rausche et al. (1985), Hannigan (1990), and Holeyman (1992). In order to be of value, the field data and the CAPWAP should satisfy a few criteria.

As discussed previously, to mobilize full capacity, penetration resistance should be sufficiently low. The data should be “clean,” and hence “CAPWAPable,” which requires good alignment between the hammer and pile, that the cushion material be in good condition, that the hammer be operating properly, and precludes significant bending of the pile, or damage to the pile, during driving, etc. The relative quality of a CAPWAP analysis is determined by comparing the CAPWAP-predicted pile behavior to the field-measured pile behavior. This comparison is reported as a match quality number, which qualitatively represents the difference between predicted and measured pile response; the lower the number, the better match quality, and the more confidence can be placed in the CAPWAP results.

As discussed previously, during driving, residual stresses are locked in the pile. To account for this, CAPWAP analyses should be performed using the residual-stress option. Although the total CAPWAP-predicted capacity is the same with both the residual-stress and non-residual-stress option, the 2 analysis options result in different allocation between shaft and toe resistance, especially near the toe of the pile. Appropriate allocation of shaft and toe resistance is important to evaluate set-up.

Subsequent Capacities

At a minimum, pile testing at some time after initial driving should establish allocation of resistance between shaft and toe, and would be most-beneficial if the shaft resistance distribution were determined. As discussed earlier, the testing methods capable of these determinations are dynamic monitoring during restrike testing with subsequent CAPWAP analysis, internally instrumented top-load static load tests, and internally instrumented bottom-loaded static load tests, which fail in shaft resistance.

For dynamic monitoring during restrike testing, the same considerations for obtaining and analyzing useful data at end-of-initial-drive remain valid. Of particular concern is properly accounting for changes in the required drive system after set-up occurs. To increase the likelihood of mobilizing full capacity, a bigger hammer may be required for restrike testing than was used to install the pile, a pipe pile may benefit from the increased impedance which results from being filled with concrete, and a pile cushion may be required. Compared to top-loaded static load testing, dynamic monitoring during restrike testing is economically attractive because it can be performed on numerous piles, and at various times after driving, for a fraction of the cost of a comparable top-loaded static load test program.

For internally instrumented static load tests, proper interpretation of the results requires accurate determination of the section modulus, which in turn usually requires accurate determination of the concrete modulus. A further refinement of modulus determination is to evaluate the concrete modulus as strain-dependent [Fellenius, 2000]. As with dynamic testing, the static load test pile should be loaded to geotechnical failure, as opposed to some multiple of the desired allowable load (i.e., a proof test). Performing multiple bottom-loaded static load tests (which fail in shaft resistance) at various times after driving may be economically attractive compared to performing multiple top-loaded static load tests.

Exploration-Phase Field Testing

The "non-standard" exploration-phase field testing methodologies identified by this research which might be considered for further study related to prediction of set-up are SPT-torque, piezocone, and dilatometer testing. The performance of these types of testing during routine subsurface exploration programs is extremely limited in Wisconsin. To our knowledge, there has been piezocone testing performed on a very small number of projects, which also included a pile-testing program, and none of the piezocone testing was staged. We are not aware of any SPT-torque or dilatometer testing performed on any projects, which also included a pile-testing program.

The amount of existing data which encompasses all the desired subsurface exploration and capacity determination parameters as described in this section is quite limited, and reduced even further when coupled with the non-standard exploration-phase field testing methodologies of interest.

POTENTIAL APPLICATION OF CURRENT METHODS TO WISDOT PRACTICES

Empirical Relationships

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 allowable load. The existing Wisconsin database of pile-testing information could be surveyed to see if a set-up factor of 1.2 is appropriate. 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 limitations of this formula were discussed previously in the “Estimation/Prediction of Set-Up – Empirical Relationships – Skov and Denver” section of this report. 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 Wisconsin 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.

For clayey soils, estimating set-up by applying an adhesion factor to the adhesion at the time of driving as suggested by Lukas and Bushell would require determining or estimating the soil’s undrained shear strength, its sensitivity, and an adhesion factor. The existing Wisconsin database of pile-testing information could be surveyed for pertinent data, which could be used to back-calculate these parameters.

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 in the design, but which maintained a minimum factor of safety of 1.5 disregarding set-up.

Field Testing

As discussed previously in the “Estimation/Prediction of Set-Up – Exploration-Phase Field Testing” section, 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-torque test results, so for evaluation of set-up only one 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-torque test is considered the more-viable field-testing methodology for evaluating set-up.

SPT-Torque Test

Applicability – SPT-torque testing appears to show promise as an exploration-phase methodology which could be employed to evaluate set-up [Bullock, 1999]. The driving of the split-spoon sampler is analogous to pile installation, it works in all soil types, and it uses a conventional drill rig. The SPT-T testing itself can be staged to determine time-dependent characteristics, has relatively minor equipment requirements beyond those used for conventional drilling and sampling, with proper training may require no additional personnel, and by obtaining both peak and residual torque values it may provide insight into soil sensitivity. A standardized testing procedure may have to be developed, and correlations would have to be developed between time-dependent increases in SPT-T test results and set-up. Among the exploration-phase field testing studied (SPT-T, CPTu, and DMT), Bullock (1999) recommended continuing research with SPT-T testing because of its simplicity and its success in mixed soils.

Cost – According to Pile Dynamics, Inc. ("PDI") which provided the SPT-T test instrumentation used by Rausche, et al. (1995, 1996), the instrumented torque rod produced for that work has not been formally sustained in the interim (i.e., has not been formally maintained for use, nor developed for commercial application). This notwithstanding, PDI indicated that the requisite instrumentation to perform SPT-T tests could be made available for a cost of approximately $5000.

Data acquisition and interpretation would require the use of a field-portable computer capable of recording and analyzing the instrumentation (strain gages and displacement transducer) output, potentially using test-specific software. Compared to conventional drilling and sampling procedures, additional time (and therefore cost) would be associated with performing SPT-T tests. The additional time required would depend on the number of staged tests performed, and the time intervals (waiting time) between tests.

Piezocone Testing

Applicability – Piezocone sleeve friction staged testing 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. The piezocone testing itself may provide insight into soil sensitivity, can be staged to determine time-dependent characteristics, and since porewater pressure measurements are obtained during the test, staged tests can be correlated to percent excess porewater pressure dissipation.

Piezocone testing does not work in all soil types; refusal can be experienced on obstructions in fill (potentially requiring pre-drilling with a conventional drill rig), cobbles and boulders, or very dense or hard soil deposits. In some higher-strength clays, point resistance and sleeve friction alone can be overcome, but rod friction can increase to the point of precluding further penetration. The piezocone tends to push gravel aside in a clay matrix, but has more difficulty with gravel in a dense all-granular matrix. It requires a special truck which has sufficient ballast (reaction) to resist the force required to hydraulically advance the piezocone. This makes the piezocone rig heavier than conventional drill rigs, and access to some sites can be problematic. In some cases, these issues of weight and access can be addressed by the use of soil anchors to provide suitable reaction. Although correlations between tip and sleeve resistance are used to estimate reported soil types, no soil samples are obtained for visual examination or testing. A standardized testing procedure may have to be developed, and correlations would have to be developed between staged piezocone testing results and set-up.

Cost – Four 80-foot-deep piezocone test holes, including two staged (excess porewater pressure dissipation) tests in each hole, would likely require 1 full day. It is estimated that for staged testing, 30 and 50 percent dissipation of excess porewater pressure would take approximately 15 and 45 minutes, respectively. The cost for this testing, hole abandonment, and reduction and presentation of data is estimated to be approximately $5000.

In-Situ Vane Shear Testing

Applicability – By determining peak and residual shear stresses, in-situ vane shear testing would provide insight into soil sensitivity, which is in turn applicable to estimating set-up using an adhesion factor as proposed by Lukas and Bushell. Vane shear testing uses a conventional drill rig, but is only applicable in soft clays (free of gravel) having a maximum unconfined compressive strength of 1 ton per square foot (tsf). At a maximum, it can measure undrained shear strengths on the order of 1,100 to 1,200 pounds per square foot (psf). It requires special equipment (generally operated by personnel other than the drill crew) which is not conducive to performing the test in below-freezing temperatures. Correlations would have to be developed between adhesion factors and set-up.

Cost – Vane shear testing performed at 5-foot intervals (interspaced between 5-foot sampling intervals) in a 60-foot boring would likely require one full day. The cost for this testing, and reduction and presentation of the data, is estimated to be on the order of $1000 (in addition to conventional drilling and sampling costs for the boring, and drill crew stand-by time during performance of the vane shear tests).

POTENTIAL FOR FURTHER STUDY / DEVELOPMENT OF CORRELATIONS

Correlation with Existing Data

Empirical Relationships

Several specific potentials for correlation of empirical relationships with existing data were discussed previously in the "Potential Applications of Current Methods to WisDOT Practices – Empirical Relationships" section. Currently, the vast majority of existing data were 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).

Any correlation studies should carefully address the applicability of existing data with respect to the parameters pertinent to set-up, and to the relationships of interest.

Field Testing

Previous – There is a very limited database of results from promising exploration-phase field testing methodologies (SPT-T, piezocone, or in-situ vane shear) which were previously performed for projects with pile-testing programs. Meaningful correlation studies with these testing methodologies are not possible with existing data.

Future – 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 determined the most-valuable pile-testing data, depending on access and proximity to test-pile locations, it may be possible to initiate additional exploration and testing. In this way, the scope of specialized exploration and testing could be tailored to the quality of existing data.

Correlation with Future Data

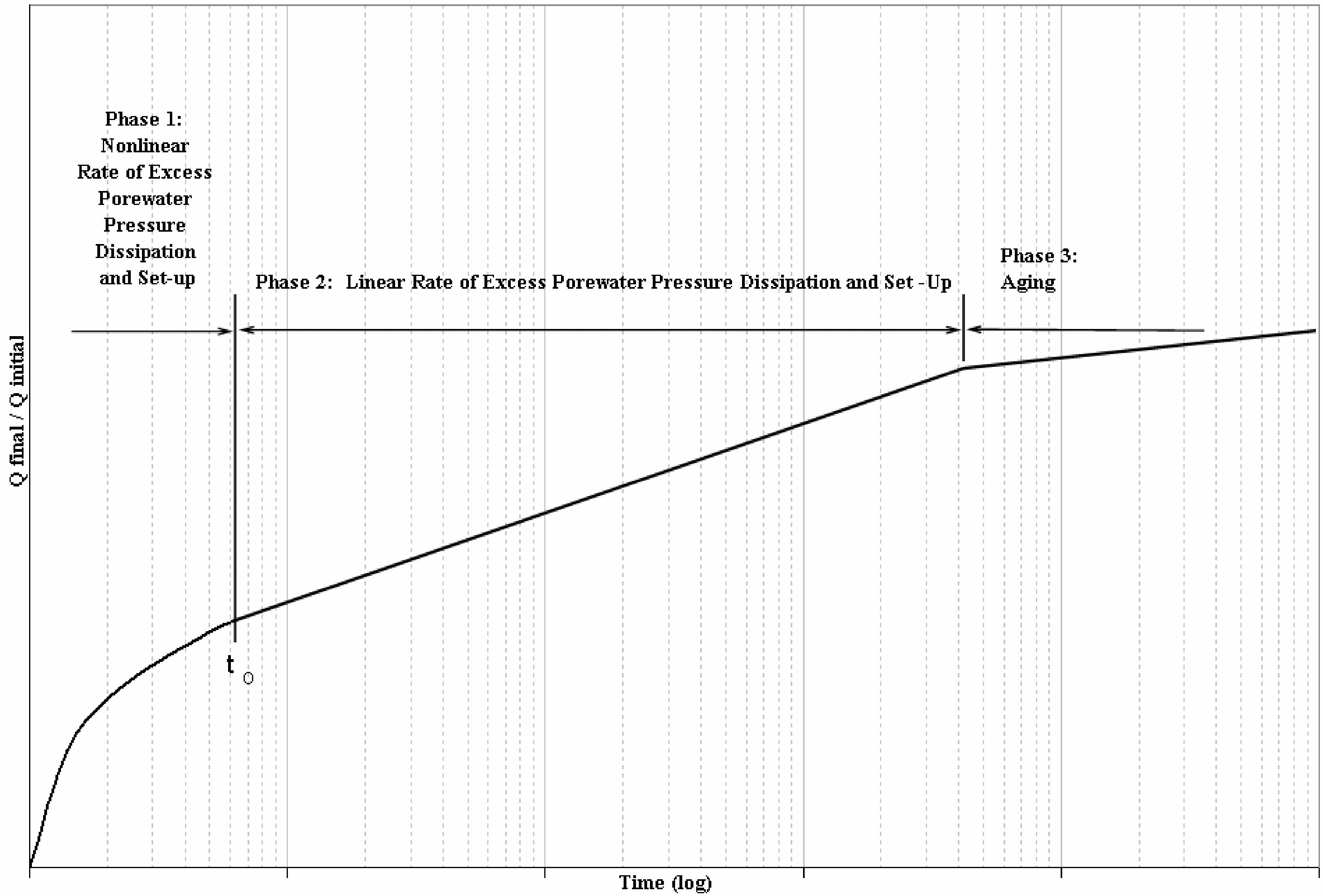
The largest potential for correlation studies lies with data from future projects. The WisDOT is in the unique position of continually purchasing new pile foundations. Collection and assimilation of set-up data could be on-going, and could pay continuing benefits. This broad, long-term, multi-project application of information may warrant scopes of pile-testing programs which are economically unjustifiable in the private sector.

If the scope of a WisDOT project warrants, the WisDOT could enter into pile-testing programs on a project-by-project basis. In this way, the type of data obtained during exploration-phase subsurface exploration, and pile testing, could be tailored to the WisDOT's purposes (both for a particular project, and for particular correlation studies). The recouping of large-project testing costs would likely come from design refinements (e.g., accounting for set-up) for the subject project, with potential additional benefit to future projects coming from correlation studies.

Concurrently, the WisDOT could periodically canvas the private sector to learn of completed, or upcoming, pile-testing programs. The WisDOT could provide potentially minimal additional exploration and/or testing of interest to correlation studies, which was not originally proposed to be performed. In this way, the WisDOT could realize full benefit from a pile-testing program at a cost of only the additional exploration and/or testing

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-drive and restrike testing dynamic monitoring with 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.

Figure 1: Idealized Schematic of Set-Up Phases



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LITERATURE SURVEY / ANNOTATED BIBLIOGRAPHY

Literature Survey

In order to reach the published information on pile set-up, an electronic search of the literature was conducted. The search started broadly focusing on "pile" and yielded 113 entries. It was further narrowed to "pile set-up," yielding a much smaller number of entries. The entries on the short list were obtained and reviewed. A short annotated description of each of the entries is provided below.

Annotated Bibliography

One of the most recent and focused source of information is the 1999 issue of Transportation Research Record No. 1663 entitled "Pile Setup, Pile Load Tests, and Sheet Piles." The papers in this issue are summarized below.

Attwooll et al. (1999) "Measured Pile Setup Load Testing and Production Piling"

This paper reports the results of static pile load tests ("SPLT") and restrike events on companion piles in connection with the I-15 Corridor Construction Project. Dynamic restrike results compared well with SPLT results. Large capacity gains with time, (i.e., set-up) were observed regardless of clay or dense alluvial sand subsurface conditions. Dynamic capacity at the end of installation ("EOI") and SPLT after 11 to 45 days were compared to arrive at a set-up ratio and also set-up unit friction. Set-up ratio varied from 1.2 to 5.9, and set-up unit friction from 47 to 73 kPa. Use of set-up unit friction (which attributes all setup to changes in shaft friction) provided a better means of predicting pile capacity than set-up ratio. The best fit of SPLT results yielded a setup unit friction of 57.5 kPa added to EOI capacity. This approach, because it does not take test time into account, is conservative, and yields a spread of $\pm$17 percent in predicted pile capacity, which was found to be acceptable.

Long et al. (1999) "Value of Methods for Predicting Axial Capacity of Piles"

This paper compares the value of predicting and precision of capacity using dynamic formulas, wave equation analysis, and dynamic monitoring without load tests. The comparison is made using the FHWA database of some 100 pile load tests. The predictive methods were based either on penetration resistance measurement (i.e., Engineering News ("EN") formula, Gates formula, and wave equation analysis program ("WEAP")), or pile acceleration and force measurement with time (i.e., Pile Driving Analysis ("PDA"), measured energy ("ME") approach, and Case Pile Wave Analysis Program ("CAPWAP")). Each method can use pile driving resistance at the end of driving ("EOD") or beginning of restrike ("BOR").

BOR incorporates some pile set-up depending on when it is performed after driving. Authors define a "wasted capacity index" ("WCI") as a measure of how inefficiently a method predicts capacity. A statistical analysis of the results show that CAPWAP with BOR results in the greatest precision of all methods investigated. WEAP, PDA, and CAPWAP benefit from the use of BOR data, but EN, Gates, and ME benefits little from using BOR data. The cost of using a method is expressed as WCI, which is a function of the reliability required for a foundation and the precision with which capacity can be determined.

Long et al. (1999) "Measured Time Effects for Axial Capacity of Driven Piles"

This paper uses a database of pile load tests (both static and dynamic) collected from the literature by the authors to quantify time effects on pile capacity. The database is divided into three groups according to the primary subsurface profile: clay, sand, and mixed soil. Pile types included a variety (i.e., pipe, concrete, timber, etc.). Time-dependent variation of pile capacity was determined by dynamic testing in most cases. Time effects are mostly positive (i.e., increase of capacity), but also relaxation (i.e., decrease of capacity) is reported (for instance in some sands). While pore pressure dissipation contributes to set-up, it is not the only mechanism responsible for time effects. The setup was up to 6 times in clays and 2 times in sands. Authors recommend SPLT be performed at least 10 days after driving in sands to take advantage of set-up.

Camp and Parmar (1999) "Characterization of Pile Capacity with Time in the Cooper Marl: A Study of the Applicability of a Past Approach to Predict Long-Term Pile Capacity"

This paper focuses on the Cooper Marl, a stiff (overconsolidated) cohesive calcareous soil in which piles bearing in the Cooper Marl experience significant set-up, (i.e., BOR/EOD ratios of up to 8). The data available from 14 sites involving 114 tests (44 CAPWAP, 12 EOD, 2 SPLT, and 100 restrike) were analyzed to see if the Skov and Denver (a linear relationship between set-up and logarithm of time) would apply. They proposed use of 2 days as the time for the onset of the linear set-up on the semi-logarithmic plot. With this assumption, they found a fairly consistent of the linear relationship. They found that the smaller piles gained capacity at a faster rate than the larger ones. Such dependency on size was reported by other investigators.

Titi and Wathugala (1999) "Numerical Procedure for Predicting Pile Capacity-Setup/Freeze"

This paper focuses on set-up in saturated clays and silts, and develops a numerical procedure that takes into account the changes in effective stresses due to pile installation and the resultant consolidation. Set-ups of up to 12 are reported as well as some cases of decrease in pile capacity with time (i.e., relaxation). The procedure focuses on consolidation as the main mechanism of capacity gain. The developed numerical model was verified using two instrumented pile segment models in the field with measurement of radial total stresses and pore pressures during installation. Additionally, pile-soil shear transfer and displacement were measured and predicted by the numerical model.

Whittle and Sutabutr (1999) "Prediction of Pile Setup in Clay"

This is a theoretical paper that develops a numerical model of predicting set-up as a function of time using nonlinear finite element method. It validates the proposed set-up analysis with data from an instrumented model piles at two sites. The proposed analysis is based on the strain path method coupled with consolidation. It predicts radial effective stress as a function of consolidation (i.e., time). It deals with the effect of overconsolidation, and takes nonlinear behavior into account. The radial effective stress at full set-up (i.e., the maximum set-up at the end of consolidation) is shown to increase with increasing overconsolidation ratio (i.e., soil stiffness), and also being higher for closed-end than open-end pipe piles.

Walton and Borg (1998) "Dynamic Pile Testing to Evaluate Quality and Verify Capacity of Driven Piles"

The experience of NYSDOT in using PDA in verifying initial capacity and set-up is described. NYSDOT has replaced SPLT largely by PDA. WEAP analysis is used to check drivability. The paper indicates that CAPWAP is useful in determining post-driving capacity. There is an awareness of the significance of set-up and it is used during pile installation by allowing a longer waiting period and additional restrike for piles in cohesive soils if the pile is rejected based on its driving record.

Tevera and Wathugala (1999) "Pile Capacity Setup/Freeze at Bayou Boeuf Bridge Extension, Louisiana"

This is a detailed field investigation of pile set-up in a soft to medium clay deposit using an instrumented square precast concrete pile. The pile was fitted with an Osterberg cell at the tip which allowed conducting a static load test. The observations and testing continued for 2 years. A PDA was used during pile installation to obtain driving resistance, stresses, and the maximum transferred energy. Both EOD and BOR (1-day) data were monitored by PDA. CAPWAP analyses were performed on the EOD and BOR data collected, and the shaft resistance was determined. Starting 7 days since EOD, Osterberg cell load tests were performed for 2 years providing the changes in shaft resistance (i.e., set-up) with time. The shaft resistance increased 196 percent in 14 days after EOD. It continued to increase another 28 percent until 247 days. Potential cost savings by using long-term capacity is demonstrated in terms of pile length reduction (about 13 percent).

Yang (1970) "Relaxation of Piles in Sand and Inorganic Clays"

Based on extensive pile re-driving data from New York, it is demonstrated that piles driven in dense fine sand, inorganic silt, or stiff fissured clay may have relaxation (i.e., loss of capacity or driving resistance with time). It is also shown that piles in soft clay or organic silt tend to show freeze or set-up upon re-driving after an interruption. Use of re-driving resistance is suggested for static capacity of piles under actual loads.

Svinkin and Teffera (19XX) "Some Aspects of Determination of Pile Capacity by the Wave Equation"

WEAP was run repeatedly to match force, energy, and velocity measured during restrikes. Then soil parameters were selected to match the pile capacity thus computed with the one measured from static test. This effort led to some observations such as blow count being the basic indicator of pile capacity. Soil damping is determined to be the basic parameter for adjustment of WEAP solutions. So adjustments of Smith damping was made to adjust WEAP capacity and blow counts to match with static capacity. Side damping is found to vary linearly with time. Initial side damping coefficient was taken from the first restrike for pile in clay and from EOD for piles in sand.

Lucas and Bushell (1989) "Contribution of Soil Freeze to Pile Capacity"

A practical field investigation involving SPLTs on piles installed in clayey soils at five different Chicagoland sites at different times up to 80 days. The results were analyzed by estimating adhesion initially and increasing times. The adhesion was not always determined rigorously such as from tension tests but calculated based on certain assumptions. The calculated adhesion was found to be lowest shortly after driving and corresponding roughly to the remolded strength of the clay deposit. Adhesion increased rapidly the first 10 days following driving and continued to increase after 10 days but at a reduced rate, climbing up to undisturbed undrained strength of the clay. There are practical lessons such as performing SPLT after 10 days, or reducing the pile driving criteria to account for pile set-up.

Randolph, Carter and Wroth (1979) "Driven Piles in Clay-The Effects of Installation and Subsequent Consolidation"

This is an analytical paper based on the modified Cam-clay model. The changes in total stresses and pore pressures at different radial distances from the side surface of the pile are calculated using a finite element method at the end of pile installation, which is described by cavity expansion. The subsequent consolidation is described as radial consolidation resulting in a differential equation in the same form as Terzaghi's one-dimensional consolidation equation. Using the analytical solutions, effective stresses at the end of consolidation and time for the essential completion of consolidation are determined. The results are used to calculate the shear strength of clay surrounding the pile after installation and after consolidation as a function of radial distance. The authors compare their findings with field tests on piles, and successfully show the increase in pile capacity with time. They indicate that the undrained shear strength close to the pile increases by a factor of 1.3 to 2. Most interestingly, they show that stress changes normalized by the initial value of the undrained shear strength are effectively independent of the overconsolidation ratio of the clay.

Tomlinson "Some Effects of Pile Driving on Skin Friction"

This well-known paper provides adhesion factors back-calculated from numerous driven pile load tests. Most significantly, the effect of granular soil skin on the pile driven from a granular soil layer into a clay layer is described. The paper has little to offer regarding pile set-up. It is stated that piles in London clay showed no significant increase in capacity in the tests one month and one year after installation. This is consistent with other findings that most of set-up in glacial clays take place in less than one month.

Paikowski, LaBelle and Hourani (1996) "Dynamic Analyses and Time-Dependent Pile Capacity"

Using an extensive database including EOD, BOR, and SPLT information on variety of types and sizes of piles driven into clayey soils, the authors once again demonstrate set-up. They first use the CAPWAP analysis of EOD and compare it with the SPLT capacity for 28 piles driven into clay. They show that SPLT capacity on the average is 1.6 times the CAPWAP capacity based on EOD records. Considering the 23 BOR cases only, this ratio drops to 1.25. Authors attribute this to the time effects. They then look at rate of capacity gain determined using dynamic analysis of restrike data over time with that determined from SPLT data over time and show that these rates match very well. They conclude that restrikes and CAPWAP can be used to monitor set-up. They also recommend that restriking should be scheduled for a time corresponding to 75 percent capacity gain; however, this value shows significant difference based on CAPWAP or SPLT.

Wardle, Price and Freeman (1992) "Effect of Time and Maintained Load on the Ultimate Capacity of Piles in Stiff Clay"

Constant rate of penetration loading tests were carried out on four instrumented piles in London clay intermittently over a period of three years. Piles were placed by different methods including driving. What distinguishes this research is the manner in which loading was carried out. End-bearing and shaft resistance were not rigorously measured but were calculated. They report a 14 percent increase in the capacity of driven piles after 108 days, and a 37 percent increase at the end of the testing period. They also found little difference in the performance of jacked and driven piles suggesting that jacked piles (more convenient to instrument) can be used to study set-up. They could not attribute an increase in pile capacity to maintained loads applied to the piles for long periods.

Attwooll et al. (2001) "Measured Pile Setup During Load Testing and Production Piling I-15 Corridor Reconstruction Project, Salt Lake City, Utah"

This paper presents the results of dynamic and static load testing on concrete-filled closed-end pipe piles at nine locations consisting of clay but also sand deposits. Dynamic test results from restrike events compared well with SPLT results. Large capacity gains were observed regardless of subsurface conditions. This paper is essentially the same paper by Attwooll et al. (1999) described above.

Rausche et al. (1996) "Determination of Pile Driveability and Capacity from Penetration Tests"

They recommend static load testing or restrike testing to be performed after the excess pore pressures are dissipated and at least 2 weeks after driving. They calculated a set-up factor defined as the static test load divided by the EOD wave equation capacity based on 99 test piles from 46 sites. Based on the calculated set-up factors, they developed a table presenting recommended set-up factors depending on predominant soil type along pile shaft. The highest recommended value is 2 for clay and the lowest is 1 for sand and gravel. They recommend confirmation of the set-up factor with local experience and with site-specific restrike dynamic tests and SPLTs.

McVay et al. (1996) "Determination of Pile Driveability and Capacity from Penetration Tests"

This report describes setup measurements in the laboratory and field in Florida. Five fully instrumented piles were tested in a variety of soils. The instrumented piles were square precast concrete piles fitted with Osterberg load cells. In the field, SPT-Torque, piezo CPT, and DMT stage testing was performed to estimate the pile freeze with time. In the laboratory, simulations were conducted in a centrifuge which showed set-up. Field load tests were stage tested at different time intervals until 77 to 1727 days. These tests showed a general linear increase in pile shaft friction with the logarithm of time. All soils, sand to clay, showed the same rate of set-up. They observed setup beyond excess pore pressure dissipation (i.e., independent of effective stress). The SPT-Torque test showed the most promise as a practical tool for assessing set-up in clays, but no test was useful in sands.

APPENDIX A: SET-UP DETERMINATION EXAMPLE

To illustrate methodology by which set-up distribution and magnitude may be determined, as well as to provide an idea of the magnitudes of set-up which have been documented in Wisconsin, results from a single indicator test pile from a Milwaukee project are presented.

Indicator Test Pile X-10.5 (the test pile's designation reflecting its row and column location) 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 Wisconsin Center project in Milwaukee, Wisconsin[2]. 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 (Elevation -2.3[3]). Underlying estuarine deposits consisted of loose clayey organic silt, with fine sand seams and layers, to a depth of 35.0 feet (Elevation -23.5). 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.

Indicator test pile installation was dynamically monitored using a Pile Driving Analyzer™ ("PDA"). A subsequent CAse Pile Wave Analysis Program (CAPWAP) analysis was performed on end-of-initial-drive (EOID) data. The EOID CAPWAP-determined unit shaft resistance distribution is presented in Figure A-1. As reported on Figure A-1, the CAPWAP analysis calculated an EOID shaft resistance of 32.5 tons.

Thirty days after its installation, Indicator Test Pile X-10.5 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 ultimate 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 indicator test pile. A subsequent CAPWAP analysis was performed on beginning-of-restrike (BOR) data. The CAPWAP analysis calculated an ultimate pile capacity (incorporating both shaft and toe resistance) of 270 tons, showing good correlation with the static load test result. The BOR CAPWAP-determined unit shaft resistance distribution is presented in Figure A-2. As reported on Figure A-2, the CAPWAP analysis calculated a BOR shaft resistance

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[2] Wagner Komurka Geotechnical Group, Inc., "Indicator Pile Test Program Report for The Wisconsin Center – Milwaukee, Wisconsin," WKG[2] Project No. 96016, prepared for Mr. Scott Smith of Engberg Anderson (Milwaukee), August 29, 1996.

[3] Unless indicated otherwise, elevations are positive, have units of feet, and are relative to Milwaukee City Datum ("MCD").

of 198 tons, again showing good correlation with the static load test result. The resulting set-up distribution (obtained by subtracting the EOID set-up distribution presented in Figure A-1 from the BOR set-up distribution presented in Figure A-2) is presented in Figure A-3. As reported on Figure A-3, this corresponds to a set-up of 165.5 tons in shaft resistance, and a shaft resistance set-up factor of 6.1.

Figure A-1: Wisconsin Center - Indicator Pile X-10.5 (10.75-Inch O.D.) - Unit Shaft Resistance vs. Elevation - End-Of-Drive CAPWAP

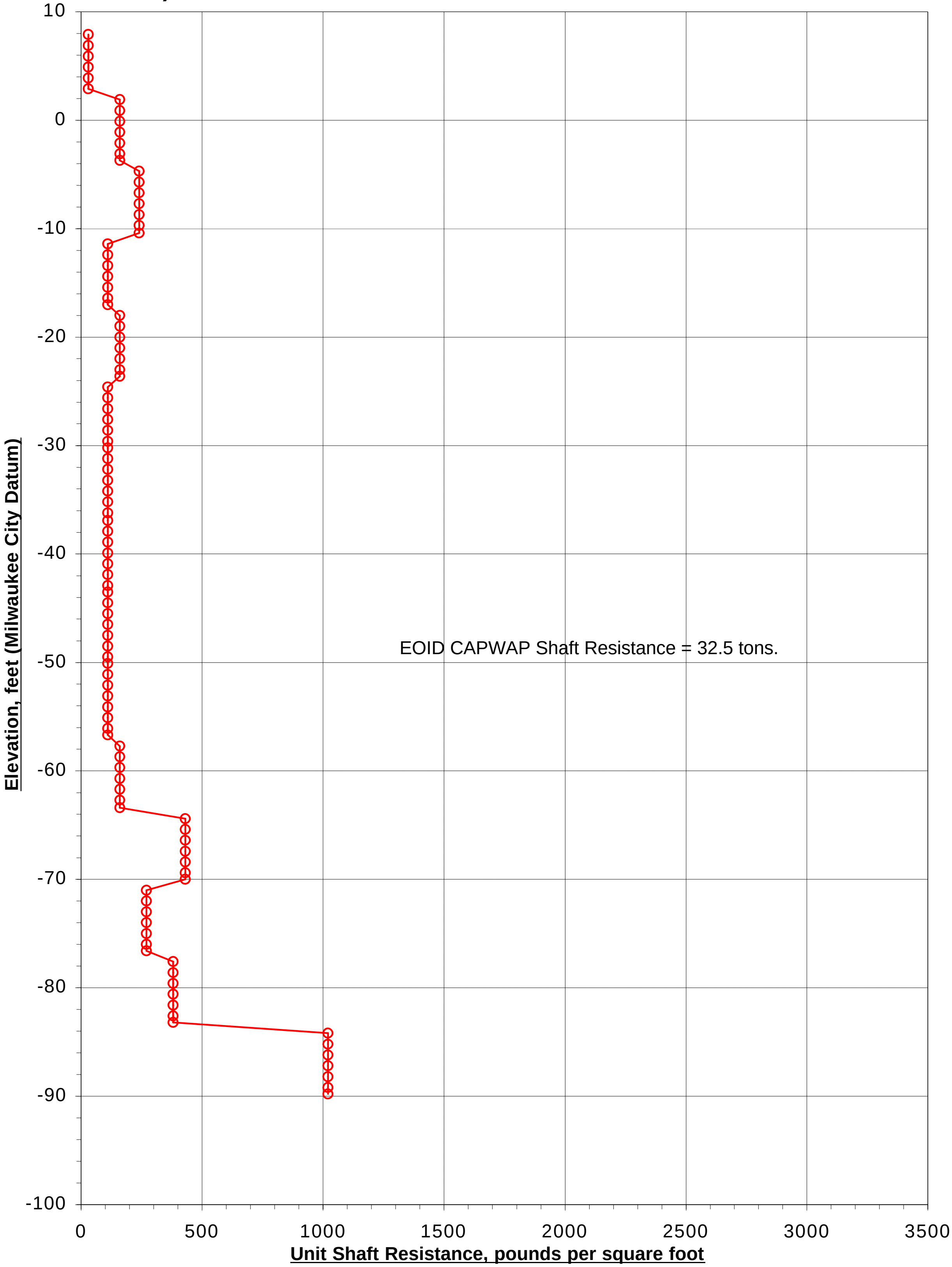

Figure A-2: Wisconsin Center - Indicator Pile X-10.5 (10.75-Inch O.D.) - Unit Shaft Resistance vs. Elevation - Beginning-Of-Restrike CAPWAP

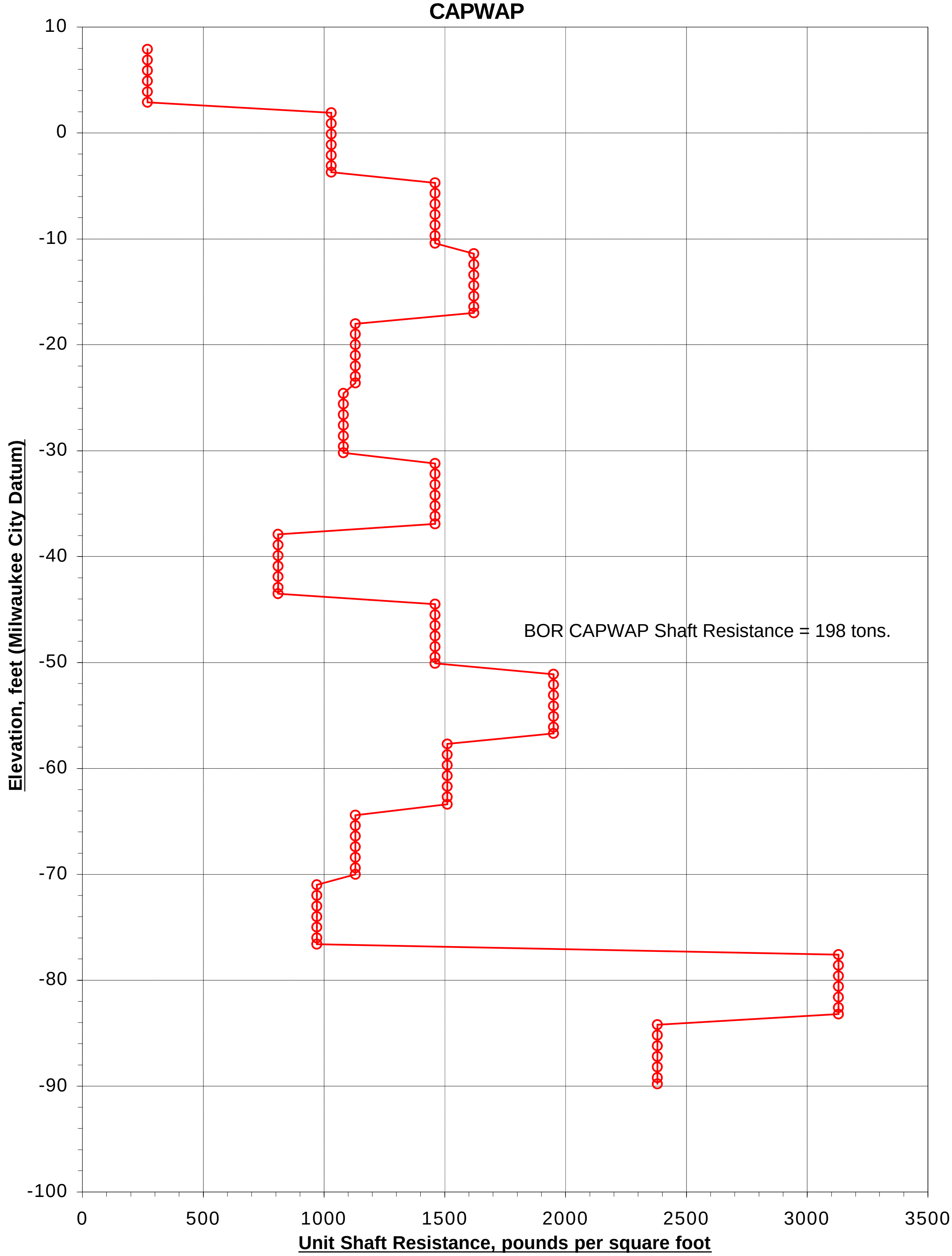

Figure A-3: Wisconsin Center - Indicator Pile X-10.5 (10.75-Inch O.D.) - Unit Set-Up vs. Elevation