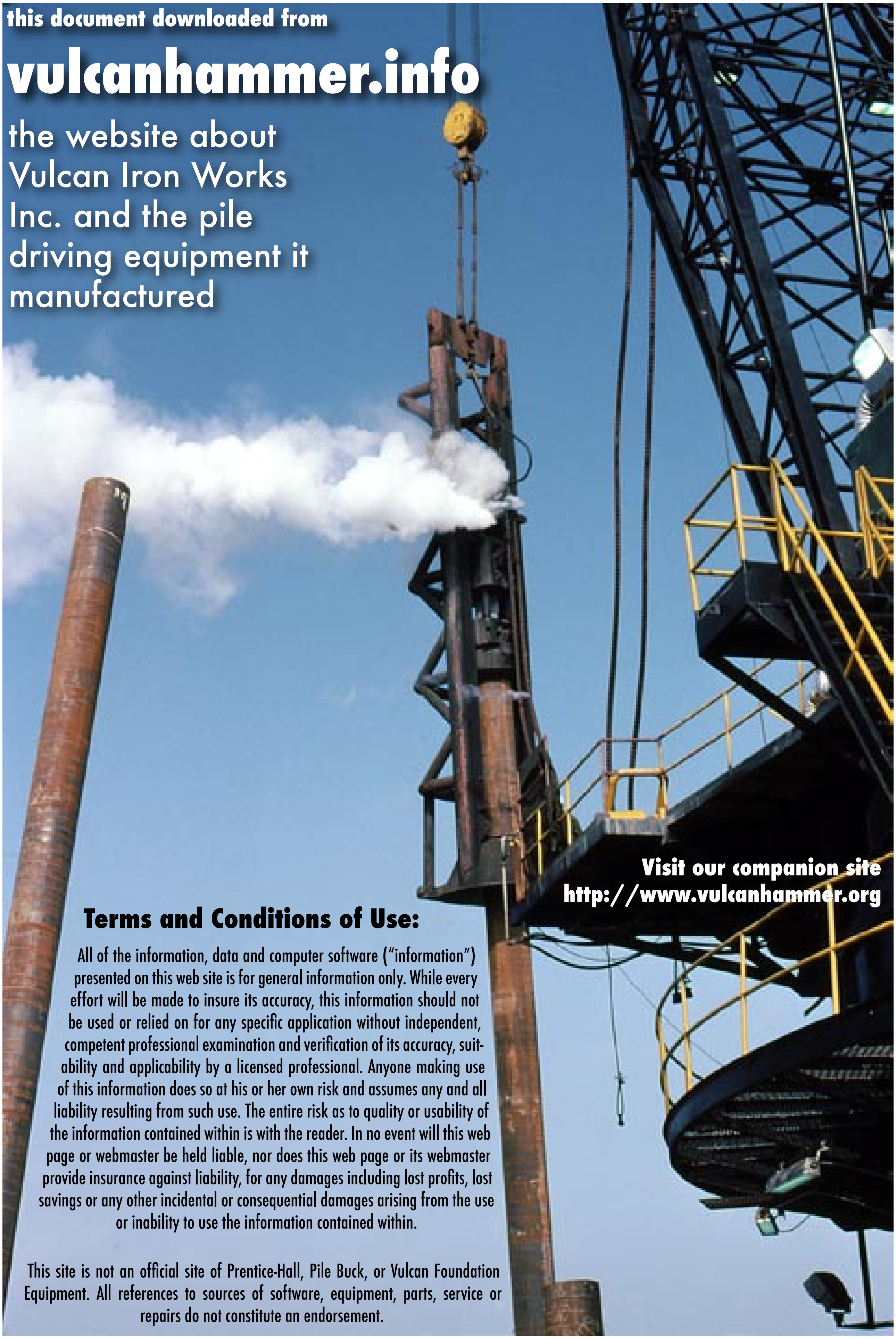

THE ACTION OF SOFT CLAY ALONG FRICTION PILES; BAY MUD REVISITED

Lymon C. Reese*

INTRODUCTION

Research at Berkeley more than three decades ago gave new insight into the behavior of axially-loaded piles in clay. Numerous experiments and analytical studies performed since, many of which have been supported by the oil industry with respect to offshore platforms, have added significantly to a better understanding of the problem. Yet, the prediction of the "real" behavior of such piles with effective-stress methods remains far beyond the present capabilities of geotechnical engineers.

A brief discussion is presented to elucidate the factors involved in the interaction between a pile and the clay with a view of establishing fundamental concepts. Models are described that serve as guidance to further research. Some results of studies at Berkeley and elsewhere are presented that are relevant to an improved understanding.

The thrust of the paper is to lay out the kinds of experiments that must be performed if the problem of an axially-loaded pile in clay is to be solved rationally. Further development of methods to predict the load versus settlement of a pile in soft clay must await the collection of a body of reliable data from field measurements.

The discussion that is presented is built around the behavior of a single pile for simplicity but the concepts presented apply to a group of closely-spaced piles. Also, for simplicity only the load transfer in side resistance is considered. End bearing is important, of course, but for a pile in soft clay the load carried in end bearing is

*Nasser I Al-Rashid Chair Emeritus at The University of Texas at Austin, Civil Engineering Department, ECJ 9.228, Austin, Texas 78712.

frequently a small fraction of the load carried in side resistance.

EFFECTS ON SOIL OF INSTALLING A PILE

Bored piles are not well adapted for use in soft clay so it can be reasonably assumed that the pile will be driven by an impact hammer. The interaction of an elastic pile and the inelastic soil under the last blow of the hammer will likely result in the distribution of a residual load nonuniformly along the length of the pile.

Immediate Effects of Pile Driving

The first effect is the displacement of a volume of soil nearly equal to the volume of the material that is inserted. Soil displacement around a driven pile has been discussed by a number of authors, for example by Zeevaert, 1950 [1] and Hagerty and Peck, 1971 [2]. The displacement of the soil during pile driving can cause uplift and lateral movement of piles previously driven [3]. Baligh, 1985 [4] has proposed a method for obtaining the deformations around an inserted pile by assuming that the insertion was at a constant velocity, that the soil was incompressible, and that the results were independent of the constitutive relations of the soil. The ability to predict the deformations of the soil even for such a special case is useful; however, the solution to the general problem remains unsolved. For example, if a tubular pile is driven, it is not now possible to predict if, or at what depth, the pile will plug.

There will inevitably be some lateral vibration of the pile during driving because of the eccentricity of the application of the impact loading. The lateral deflection could be enough to cause the soil near the groundline to be pushed away from the pile and the axial capacity would be adversely affected.

The driving of a pile into soft clay causes the total pressure at the pile wall to increase above the at-rest pressure and the pore pressure to increase above the equilibrium pressure. Increases in the pressures around piles in clay have been observed at the pile wall and at several diameters from the pile wall [5] [6] [7] [8] [9] [10]. The increase in the pore pressure around a driven pile in clay is analogous to the increase in temperature in a slab due to the imposition of a line source of heat.

As a pile is driven past a particular point in the soil, shearing deformation of large magnitude will occur. Information is unavailable as to the conditions under which the sliding will occur at the pile wall or at some distance into the soil. However, Tomlinson reported that the excavation of driven piles revealed that soil from upper strata were moved downward into completely different strata [11] [12].

Effects of Time with No Axial Loading

A considerable amount of time must elapse before excess pore pressure is substantially dissipated around a pile driven into a saturated, homogeneous clay. Along with the decrease in excess pore pressure is an increase in axial capacity. Figure 1 [13] [14] presents curves for eight experiments that show the rate of increase in axial capacity as a function of time. The curve for test B-1 in Fig. 1 was developed from data on the decay of excess pore pressure and by making the assumption that the axial capacity of the small rod went up as the pore pressure went down [14]. As may be seen in the figure, some of the piles had not reached the full capacity for more than a month after installation.

The data from Fig. 1 were used to construct Fig. 2 that shows a log-log plot of the time for 50% of final capacity as a function of pile diameter. Only the data from full-displacement piles were used in the plot. The results shown in Fig. 2 are consistent with the concepts that the magnitude of the "bulb" of increased pore pressure is a function of the amount of soil that is displaced and that the capacity of a pile under axial loading is inversely related to the decay of excess pore pressure.

The state of stress in a homogeneous, saturated clay immediately after pile driving is complex. The excess pore pressure at a particular point on the pile wall probably reaches its maximum value just after the pile is driven. Then, there is an exponential decrease in pore pressure and a corresponding decrease in water content. The change in water content is significant [15] [6] and the shear strength of the clay at the pile becomes markedly higher than the strength just after the pile was driven. The physical process of the outward flow of water and the inward packing of soil grains is not well understood, and no theory has been proposed for predicting the state of stress and the properties of clay as a function of distance from the pile wall.

If a driven pile in saturated clay remains unloaded, there is some evidence to show that compressive stress develops in the pile. The remolding of the clay due to pile driving causes its consolidation characteristics to change so that it will settle under self weight. Thus, some downdrag will result with a consequent change in the residual stresses that existed immediately after pile driving [1] [16].

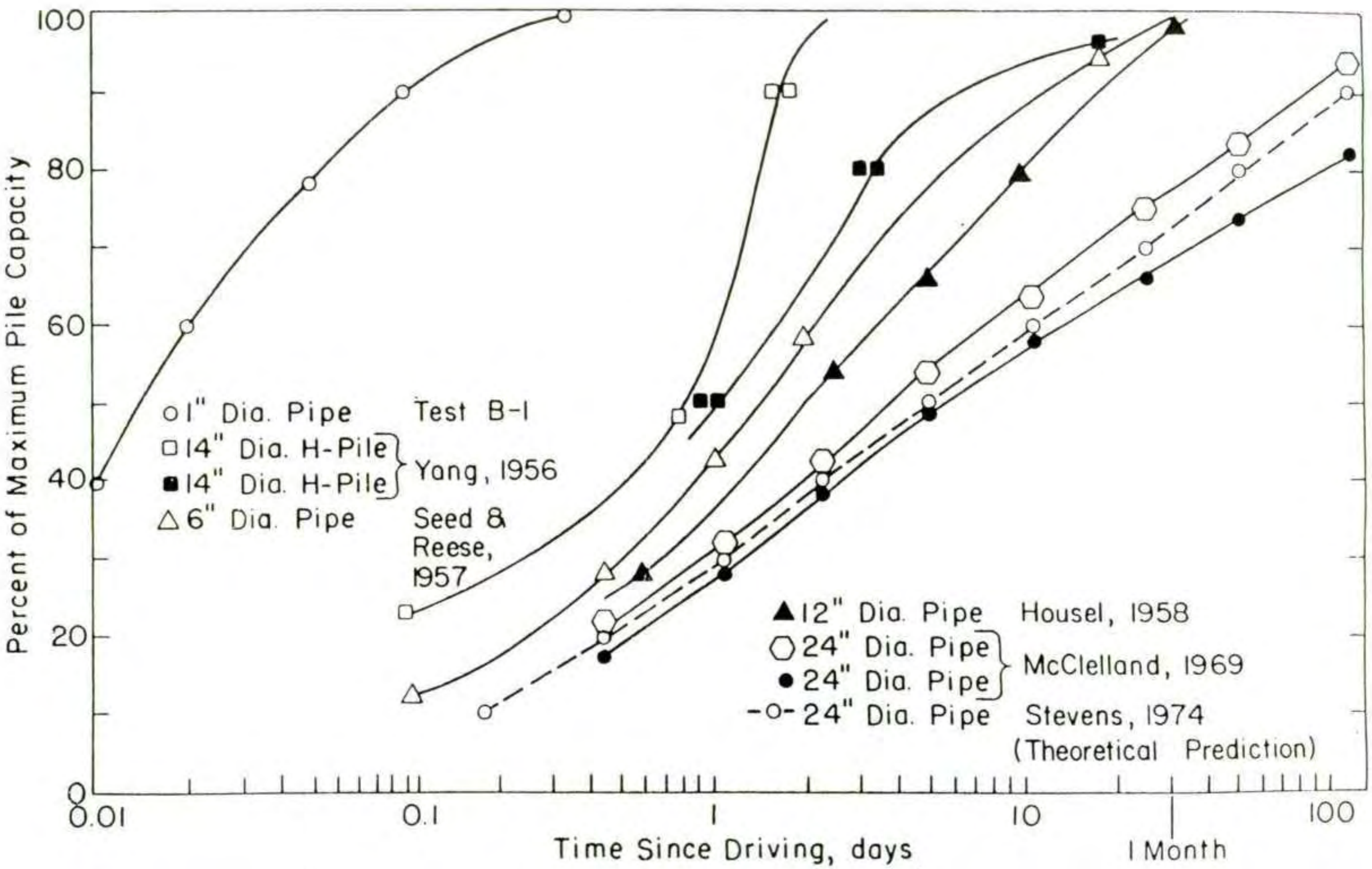

Figure 1 Pile capacity versus time (after Vesic) 1 inch = 25.4 mm.

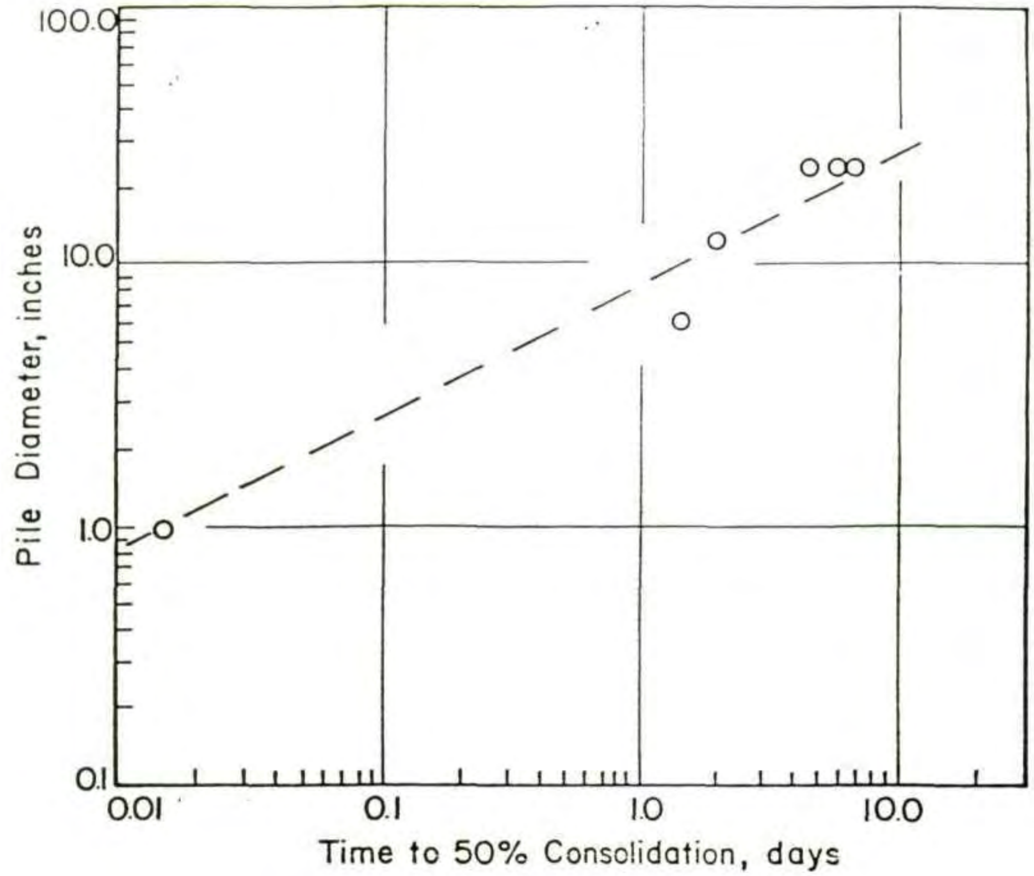

Figure 2 Pile Diameter versus time to 50 percent consolidation (log-log plot) (after Grosch and Reese, 1980) 1 inch = 25.4 mm.

Effects of Loading

The axial load on a pile will cause an increase in the stress in the pore water that leads to consolidation and settlement. For most offshore structures, the settlement due to consolidation is of little or no consequence. The increase in shear strength of the clay as a result of consolidation may or may not be significant in terms of the additional capacity of a pile.

The cyclic, axial loading of a pile could result in an increase in the pore pressures so that the axial capacity would be affected during a storm [17] [14]

Summary

The soft clay around a driven pile experiences drastic time-related changes that are dependent on the geometry of the pile, the roughness of the pile wall, the method of installation, the distance from the pile wall, the stratigraphy at the site, and the properties of the clay. The result is that the clay that controls the axial capacity of the pile in side resistance will have properties that are quite different from the in situ properties of the soil.

MODELS FOR COMPUTING BEHAVIOR IN SIDE RESISTANCE

Pile

A wide variety of types of piles can be driven into soft clay where the variables are the geometry of the pile and the material of which it is made. In all cases the pile will deform under axial load and the deformation will usually be elastic or nearly so.

For the purposes of this discussion, the assumption will be made that load transfer from pile to soil will occur at or near a pile-soil interface that is flat or curved, and it is convenient in the presentation to consider a cylindrical pile. Tapered piles and H-piles are eliminated from consideration, although many of the concepts that are developed will apply generally.

Soft Clay

A section from a driven pile is shown in Fig. 3. Elements are shown at the pile wall at Points (a) and (b) and elements are shown radially away from the pile wall. If it can be assumed that the properties of the clay were perfectly uniform prior to pile driving, the properties of each of these elements will be different. Furthermore, the state of stress will differ from the in situ state. An element of the clay at some radial distance from the pile wall will retain its in situ properties and state of stress.

An element of the clay at the interface of the pile is shown in Fig. 4; a generalized state of stress is shown on the face next to the pile. The three stresses that are indicated are all related to time. The normal stress is a function of the in situ soil properties, the in situ state of stress, and the effects of driving the pile. The normal stress could be zero or nearly so near the ground surface, especially if the pile had experienced lateral deflection during placement.

The shearing stress in the vertical direction is due to the residual stresses, if any; the possible effects of downdrag; and the loads that are applied. The shearing stress in the horizontal direction is probably very small and may be due to variation in soil properties and the bending of the pile during driving.

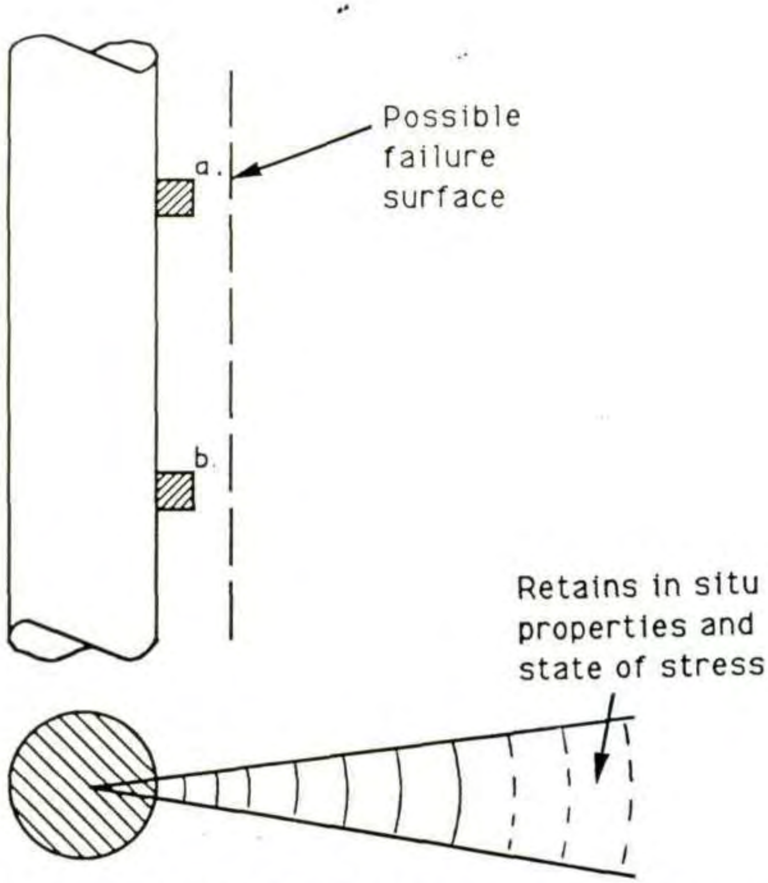

Figure 3 Variation of state of stress and soil properties around a driven pile.

A failure surface, not at the interface of the pile and the soil but in the soil at some distance from the wall of the pile, is shown in Figs. 3 and 4. A conceptual approach to the location of the failure surface at some point along the length of a pile is shown in Fig. 5. The shearing resistance of the clay, determined either by the undrained-strength approach or by the effective-stress approach, is shown as a curved line extending from the wall of the pile. Field measurements, some of which are mentioned later, show that the water content of the clay is less at the pile wall than the in situ water content. Furthermore, piles in clay that are recovered have a layer of clay clinging to the wall of the pile. If soil particles move toward the pile while excess pore pressures are dissipated, it follows that water molecules must move outward; thus, at some distance from the interface, the water content could be greater than the in situ value and the strength would be less than the in situ value.

A point at the wall of the pile is shown in Fig. 5 to represent the shearing resistance at the interface. This point is plotted below the strength of the soil at that point but it could possibly be above the soil strength if the wall of the pile is rough. Also, there could be a chemical reaction between the clay and the material of the pile that could cause a strong resistance at the interface.

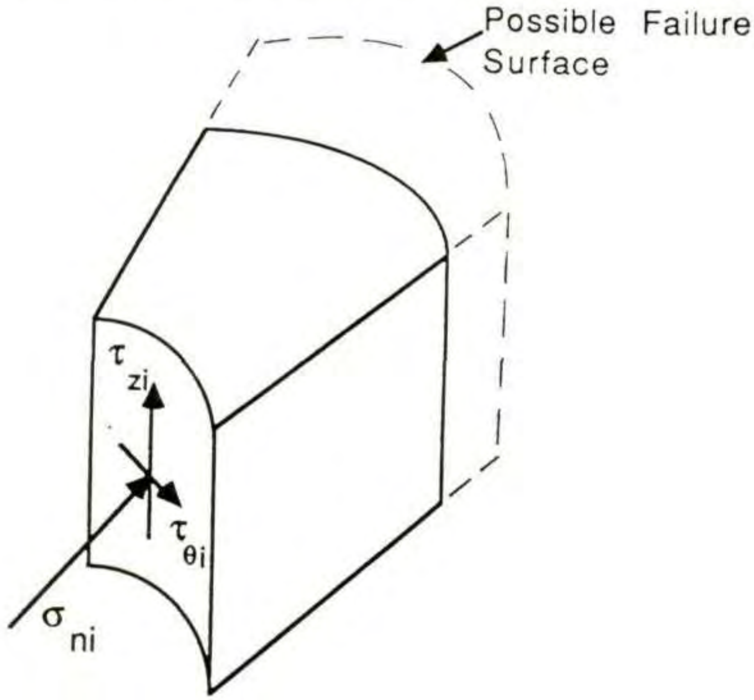

Figure 4 Generalized state of stress at interface of pile and soil.

The dashed line in Fig. 5 shows the applied stress, and that stress will decrease with distance from the wall of the pile. At some distance from the pile the applied stress will equal the shearing resistance of the clay and sliding will tend to occur when the pile is fully loaded. The thickness of the clay layer that will move with the pile is indicated in the figure. That layer of clay in effect enlarges the cross-sectional area of the pile. However, it is unlikely that the distance to the potential sliding surface will be the same at all points along the pile so that the actual thickness of the clinging layer would involve consideration of bearing stresses and kinematics.

The curve given in Fig. 5 conceptually could represent the undrained strength of the clay; however, the use of effective stresses is more satisfactory and obligatory if the true behavior of the pile is found. Thus, the parameters that define the strength of the clay must be ascertained on surfaces, millimeter by millimeter, from the pile wall. These parameters will be related to the in situ parameters as influenced by the driving of the pile. Also, the effective stresses must be found at the pile wall and out into the continuum.

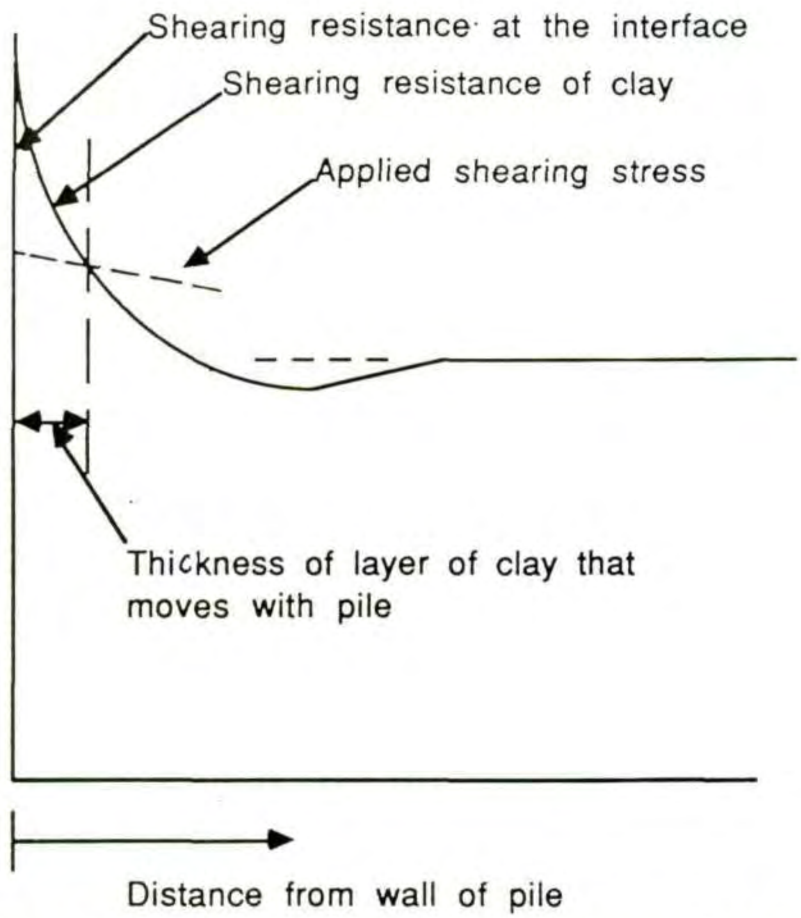

Figure 5 Conceptual curves for locating position of failure surface.

Interaction Between Pile and Soil

Many procedures for piles in clay are directed only at the computation of the ultimate capacity under short-term loading at the time when pore pressures are fully dissipated. However, it is desirable to be able to predict the load versus settlement and the distribution of load along a pile for any time after a pile has been driven. Such predictions can be made by utilizing the model shown in Fig. 6.

The sketch in Fig. 6 shows the pile as an deformable body; the soil has been replaced by mechanisms that merely indicate that the unit load transfer in skin friction is a nonlinear function of the movement of the pile. The unit load transfer is characterized by a family of s-z curves (sometimes called t-z curves), where s is the load transfer in side resistance at a point along the pile and z is the movement of a point on the pile relative to the position of the point prior to loading. The sketch indicates that the movement of the pile head under a given load requires the solution of a nonlinear differential equation because the movement of the pile and the load transfer are mutually dependent. The concept of load-transfer curves as a means of explaining the interaction was presented by Seed and Reese, 1957 [6] and developed further by Reese, 1964 [18] and Coyle and Reese, 1966 [19].

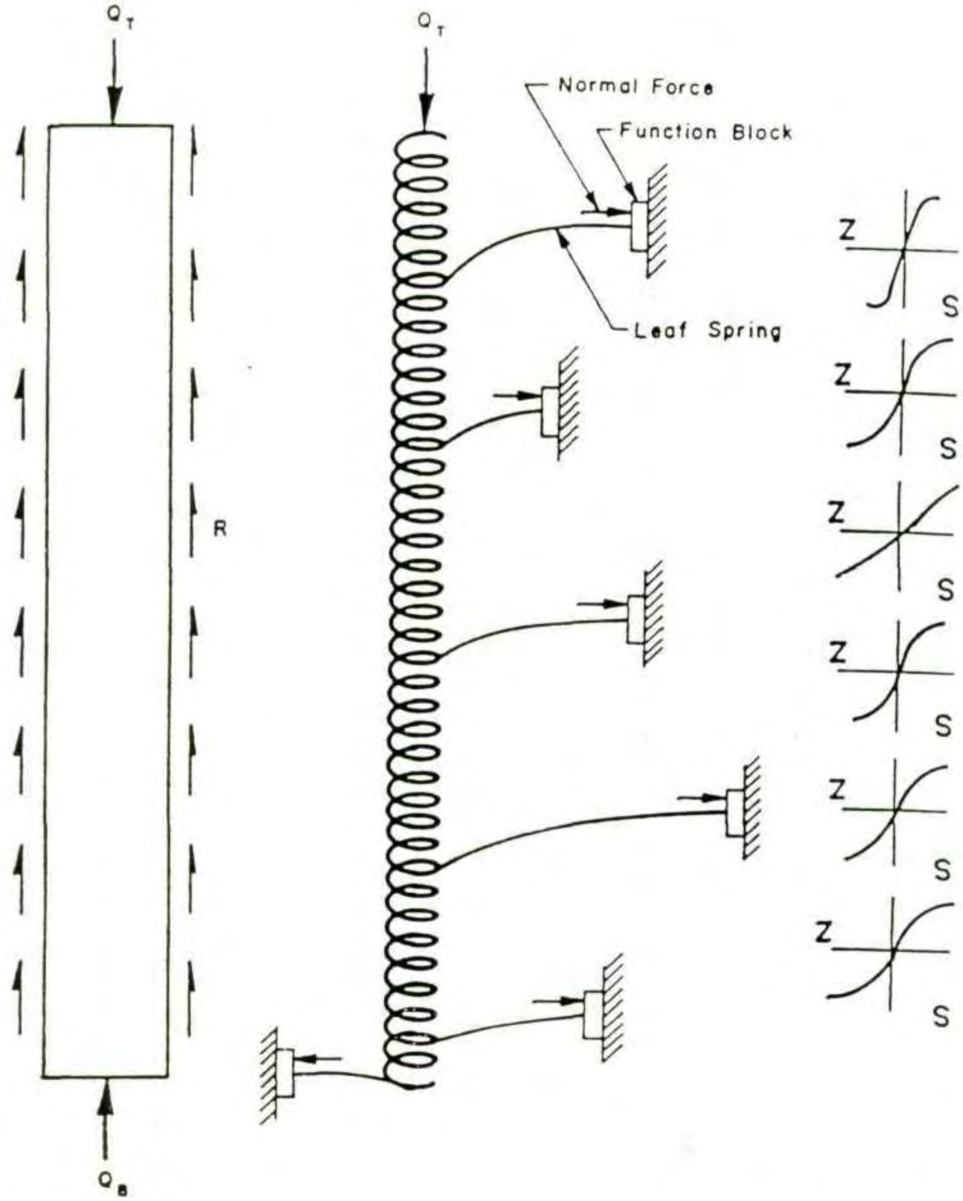

Figure 6 Model of axially loaded pile (after Reese, 1964).

The model can be criticized because single-valued curves, as shown in the sketch, cannot be used to characterize the behavior of a continuum. However, the curves that are reported in technical literature have been obtained from experiments with instrumented piles where the continuum effect was satisfied. Furthermore, the solution for the behavior of a pile could proceed equally well if multi-valued curves, including the influence of the continuum, could be predicted for various points along a pile.

Other investigators have proposed formulations for load-transfer curves for piles in clay since the earlier studies that were cited; for example, Kraft, et al., 1981 [20] and Vijayvergiya, 1977 [21]. The method is being used in practice, but the number of tests with instrumented, full-scale piles is insufficient to allow such predictions to be developed with confidence.

The load transfer at some time after the installation of a pile could be represented by a family of curves that are movement-softening such as indicated by the Points A, B, and C in Fig. 7. Then, the ultimate capacity, and the load-settlement curve, must be computed in consideration of the axial stiffness of the pile. Because of the deformation of the pile under load, the pile movement and load transfer near the bottom could be represented by Point A, while Point B could represent the soil response at the midheight, and Point C could represent the soil response near the top. Thus, the solution of a differential equation would be required if soil response is represented by curves such as in Fig. 7.

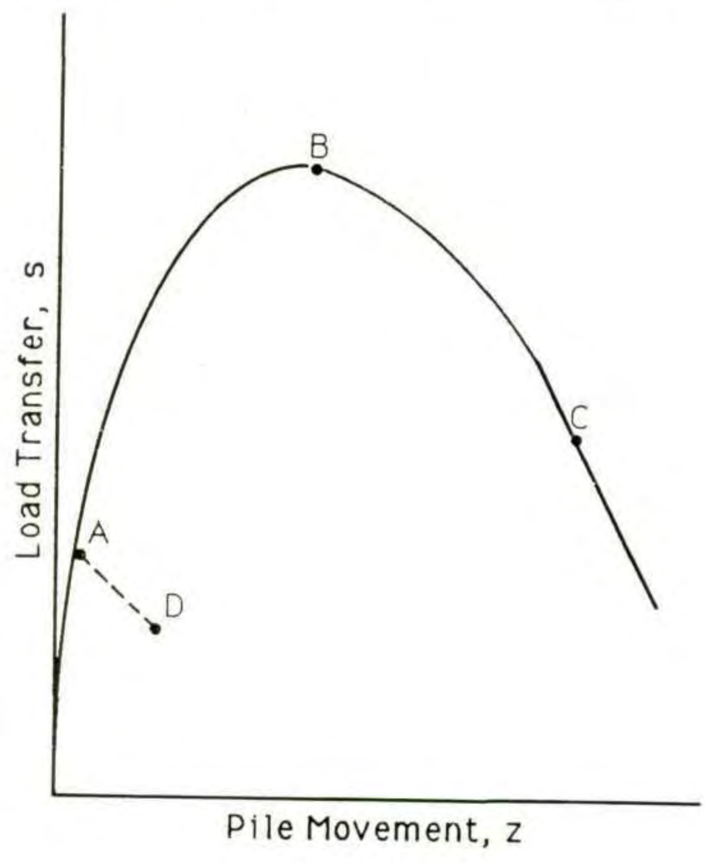

Figure 7 Hypothetical movement-softening load-transfer curve.

If a pile in soft clay is subjected to sustained loading such that Point A in Fig. 7 gives the load transfer and pile movement, consolidation and creep could cause a displacement to Point D'. Research on the behavior of piles in soft clay can lead to the ability to predict load-transfer curves as a function of time and loading on a pile. Thus, the complete response of a pile to axial loading at any time after installation could be found analytically.

Summary

The models describe a system that can explain the action of soft clay along an axially-loaded pile. While the system is complex in terms of many that are in current use, researchers currently have the capability to gain data to implement the models. Such research is necessary if significant advances are to be made in predicting the response of piles in soft clay.

SOME PERTINENT FIELD MEASUREMENTS

The influences of the driving of piles on soft clay are so dominant that laboratory studies are of limited benefit. Furthermore, the amount of field research in the last several decades is so large that it is impossible to review all of it. Therefore, starting with the work of Seed and Reese, 1957 [6] and Reese and Seed, 1955 [5] (called the Berkeley studies for convenience), a few studies are described briefly. Useful studies have been made with probes that are inserted into clay [14] [22] but only those results from driven piles are included below. The nature of the research is indicated that will further the understanding of the subject problem.

The Norwegian Geotechnical Institute carried out a noteworthy set of experiments at Haga with a fully instrumented, cylindrical pile that was 5 m in length and with a diameter of 153 mm [23]. The tests were carried out over a period of 4 years and data were acquired concerning the behavior of both pile and supporting soil.

Total and Pore Pressures

The pipe pile (152 mm in diameter) installed at Berkeley contained gauges that measured pressures at six positions along its length. Significant differences were shown between the results of measurements of pore pressures and total pressures, indicating that there was effective stresses at the pile wall during the driving. There was an exponential decrease in the pressures with time and data, taken at the end of the test period of 800 hours, showing that good agreement was obtained between the measured pore pressures and the computed hydrostatic pressures. The pore pressures were subtracted from the residual total pressures and the result was divided by the computed effective overburden pressure; the values ranged from 0.24 to 0.54. Pressures were measured at nearby piles when other piles were driven in the vicinity with interesting results.

The Haga experiments revealed that the maximum values of total pressure and pore pressure were almost equal and that these values varied almost linearly with depth. The residual values of the total pressure varied almost linearly with depth and was about one-fourth of the values measured immediately after installation. The residual pore pressure was very close to the hydrostatic over the length of the pile. Data on the decay of the excess total and pore pressures were not presented in the 1986 paper.

Water Content at Wall of Piles

Samples were taken close to the walls of piles that were driven in the Berkeley experiments and the orientation was carefully marked so that the portion of the sample next to the wall of a pile could be identified. The results showed that the average values of water content were 48.1% prior to pile driving, 43.6% next to wall of pile one day after pile driving, and 41.1% next to wall of pile 30 days after pile driving.

The researchers at Haga made an excavation near the wall of the 153 mm pile and obtained block samples for study. The water content within a zone of about 16 mm from the pile wall was was about 23.6% while the water content of the undisturbed soil ranged from about 48% to 59%. These remarkable studies showed that the change in water content and soil distortion occurred principally in a zone that was about 200 mm from the pile wall.

Studies with a timber pile with a length of 13.1 m and an average diameter of 250 mm were described by Eide, et al, 1961 [24]. At the conclusion of loading tests, samples were taken to a depth of 9 to 10 m that skimmed the pile wall. Distortion in the layers of soil was noticed to a distance of 60 mm from the pile wall and the average moisture content in the "reconsolidated" zone was found to be 24%. The average moisture content of the natural soil at the same depth was found to be 35%.

Axial Capacity as a Function of Time

The test pile in the Berkeley studies was loaded the first time at 3 hours after pile driving and then at 1, 3, 7, 14, 23, and 33 days after installation. At a particular time after driving, one of the piles at the site was loaded to failure three times in quick succession with almost identical load-settlement curves, indicating that the previous loading had little if any effect on subsequent loadings. The plotted results from the tests showed a rapid increase in capacity with time during the early hours with the full capacity being achieved at about 23 days after installation.

The results from the Haga experiments showed that preloading had a significant effect on subsequent loading; thus, different piles had to be tested in order to obtain information on increase in capacity with time. The researchers found that capacity increased almost linearly with time from about 60 kN at 1 week after installation to about 74 kN at 5 weeks after installation. The authors postulated that the increase occurred due to the increase in the effective angle of internal friction near the pile wall as well as to the increase in effective stress.

The timber pile tested by Eide, et al, 1961 [24] was subjected to both short-term loading and sustained loading. The first test was 3 days after installation and the final test was 799 days after installation. As observed by others, the pile showed a sharp increase in capacity during the early days but some increase was shown after the pile had been in place for three months.

Four steel pipe piles (356 mm in diameter) were tested by Cox, et al, 1979 [25] in under-to-normally consolidated clays. The piles were driven through casings so that strata of soil at different depth below the ground surface could be investigated and tested both in tension and compression. The first tests were a few days after driving and other tests were done after more than 300 days after driving. Some significant increases in pile capacity with time were noted.

A pipe pile (762 mm in diameter) was driven into silty clay to a depth of 80.2 m below the ground surface; the top 57.9 m of the pile was isolated from the soil by a casing [26]. The authors report a four-fold increase in pile capacity with time. Increases of 9% to 32% in capacity within a few hours after a previous test were attributed to the dissipation of shear-induced pore pressures.

Load-Distribution Curves

Data from the Berkeley tests showed that little of the load was carried in end bearing and that the load transfer in the top few diameters was much less than that over the lower portion of the pile.

Cox, et al, 1979 [25] obtained data that showed load transfer increasing slightly with depth. For one of the piles with 15.2 m of length in contact with the soil, only 11% of the load was carried in end bearing.

Load-Transfer Curves

Results from vane-shear tests at the Berkeley site were used to predict curves showing the transfer of load in skin friction as a function of pile movement. The distribution of load with depth, considering the elastic shortening of the pile, was predicted reasonably well.

Cox, et al, 1979 [25] obtained a number of such curves and most showed that the maximum load transfer occurred at a pile movement of 10 mm or less. The curves remained approximately constant with additional pile movement or showed some slight decrease.

Comments on Requirements of Methods of Investigation

Investigation of the behavior of a pile driven into clay presents some formidable challenges. Instrumentation must have appropriate sensitivity and reliability. not only during the installation but with time after installation.

With regard to the measurement of the axial loading with depth, instrumentation installed prior to driving must survive the dynamic loading and stresses. Measurements must be made at more than one point around the circumference in order to eliminate the effects of bending moment from eccentric loading. The instrumentation should not experience a zero-shift during driving so that the residual loading in the pile can be found.

With regard to the measurement of total or pore pressures, the rapid rise of these pressures during the installation of a pile presents challenges. In the case of total pressures, a system is required that allows no deflection; otherwise, arching would occur around the gauge. A similar nulling method is required for pore pressures because the drainage of a few cubic millimeters of water could cause undesirable inaccuracies. A fundamental difficulty in measuring the pressures along a driven pile is that the maximum pressures are undoubtedly generated when the pile first penetrates the soil at a particular depth; then, as the pile is driven farther, the pressures at that depth would change with time. Thus, not only is instrumentation required that will eliminate the inward movement of soil or water, but pressures are needed at several points along a pile during the driving.

The measurement of water content and other properties of the soil around the pile is of considerable importance. The technique of opening an excavation along the pile so that samples can be taken perpendicular to the pile wall is an ideal solution but is not always feasible. Arrangements can be made to sample from the interior of a closed-end pipe pile and in other cases special arrangements can be made to allow vertical sampling next to the wall of the pile.

The angle of internal friction (f) of the clay probably changes millimeter by millimeter from the wall of the pile outward. Assuming that specimens of the clay of acceptable quality can be obtained, a techniques is required that can obtain these values of ϕ. The development of such a technique would appear to be a formidable challenge.

CONCLUDING COMMENTS

The understanding of the response of a pile in soft clay to loading awaits the performance of a series of field tests that provide data on a number of relevant parameters. With such data at hand, analytical tools are available that allow the development of methods of prediction. In addition to the Berkeley researchers who employed the theory of the diffusion of heat [27] [28], a number of investigators have proposed methods of predicting the interaction of a pile and the supporting clay [29] [30] [31] [4] [32] [33] [34] [35] [36]. Each of the methods makes a contribution to the eventual solution to the behavior of piles in soft clay under axial loading but each of them suffers from a lack of high-quality data.

The relatively recent high-quality experiments at Haga and the earlier ones at Berkeley, while somewhat crude by today's standards, are examples of those that are necessary before predictions can be much improved. But because the response of clay to the driving and loading of a pile is plainly related to the kinds of soil profiles and pile geometry, those tests only provide "points of light" in a large matrix.

The scope of the problem of gaining the amount of data that will be required to allow rational predictions is large and may seem overwhelming. However, major construction projects could easily support the cost of the necessary instrumentation and the collection of data that allow progress. The writer closes the paper, as done in the past, with a quote from the late Karl Terzaghi [37] that remains fresh and pertinent: "Our theories will be superseded by better ones, but the results of conscientious observations in the field will remain as a permanent asset of inestimable value to our profession."

ACKNOWLEDGEMENT

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