COUPLED MODELLING OF OBSERVED PORE PRESSURE DISSIPATION AFTER HELICAL PILE INSTALLATION

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ABSTRACT
The ability of conventional design approaches to predict pile capacity is unreliable at best. Understanding the generation and evolution of pore pressures and associated effective stress during, and following, pile installation is a first step to improved design approaches. In this study, a fully coupled large strain cylindrical cavity expansion numerical model is used with the NorSand critical state soil model to show that conventional site investigations typically do not provide enough information to constrain the predicted pore pressures. It is also shown that the different parameter assumptions result in a very different evolution of lateral effective stress, even for initially quite similar pore pressures at the pile wall. Given the importance of lateral effective stress on the predicted pile capacity, this may explain why pile capacity is so difficult to predict accurately.

RÉSUMÉ
L'habilité des approches conventionnelles de calcul pour prévoir la capacité de pile est incertaine au mieux. La compréhension de la génération et de l'évolution des pressions de pore et des contraintes effectives associés pendant, et après l'installation de pile, est une première étape aux approches de calculs améliorés. Dans cette étude, un modèle numérique d'expansion de cavité cylindrique complètement couplé à grande déformations est employé avec le modèle de sol d'état critique NorSand pour prouver que les investigations conventionnelles de sites, typiquement ne fournissent pas assez d'informations pour contraindre les pressions de pore prévues. On montre également que les différentes suppositions de paramètre ont comme conséquence une évolution très différente des contraintes effectives latérales, même pour les pressions de pore au début tout à fait semblables, au mur de pile. Etant donné l'importance des contraintes effectives latérales sur la capacité prévue de pile, ceci peut expliquer pourquoi la capacité de pile est si difficile de prévoir exactement.

1. INTRODUCTION

Helical piles are gaining popularity in North America as an alternative foundation solution to traditional driven and jacked piles. According to the pile manufacturers' these piles have several distinctive advantages over traditional driven and jacketed piles: they mobilize soil resistance both in compression and uplift; they are quick and easy to install without vibration, and no heavy equipment is required for installation of small diameter piles. It is also possible to install them inside buildings and helical piles are reusable.

To date, research efforts in the field of helical piles have concentrated on their lateral and uplift capacity. This is understandable as current design methods used to predict pile capacity are unreliable and require the use of large factors of safety. This lack of design accuracy is explicitly recognized by Eurocode 7 (1997). Eurocode 7 requires that all pile installations be checked by a representative pile test. At a recent symposium on pile design (Ground Engineering, 1999) had the participants provide a prediction of the capacity of a single driven steel pile. The general success rate was very poor with only 2 or 16 teams getting within 25% of the correct capacity. And the best prediction of the pile's capacity was obtained from compensating errors; a too low lateral capacity was balanced by a too high end bearing. This lack of predictive capacity in pile design was also recognized by Randolph in his Rankine lecture (2003).

One important factor controlling a pile's capacity is the long term effective stress at the pile-soil interface. This stress is controlled by the evolution of pore pressures and effective stress during pile installation and subsequent pore pressure dissipation. For all piles, and particularly helical piles where less case history data exists, an ability to accurately understand and predict the evolution of effective stress and pore pressure at the pile wall would provide a basis for accurate pile design.

A recent field study of helical pile performance carried out by Weech (2002) at the Colebrook site in Surrey, British Columbia, provides quality data on the pore pressure regime during and after helical pile installation. Weech installed six instrumented, full-scale helical piles in soft, sensitive, marine silt and clay soil. He monitored the excess pore pressures within the soil surrounding the piles during and after pile installation by means of piezometers located at various depths and radial distances from the pile shaft, and using piezo-ports, which were mounted on the pile shaft. The measured pore pressure distribution with normalized distance r/R (=radius/ pile shaft radius) is shown in Figure 1, overlain on data from other sites compiled by Levadoux and Baligh (1980). It should be noted that data from the Colebrook site is atypical, showing lower normalized excess pore pressure. This may be attributed to the effect of the helices or unusual site properties.

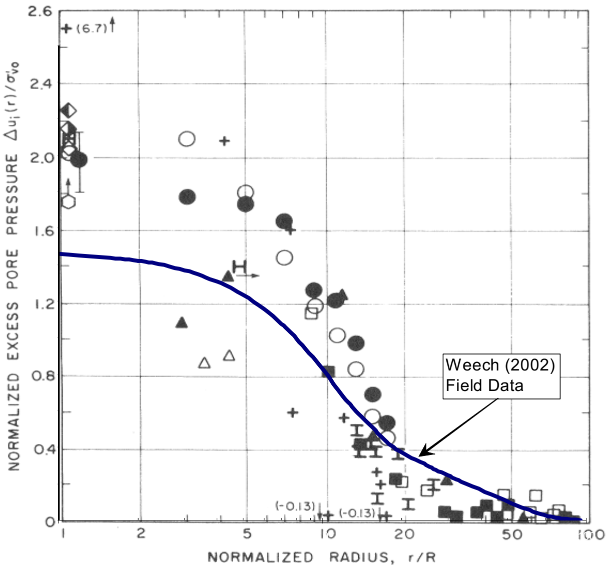

Figure 1 : Distribution of $\Delta u/\sigma'_{v0}$ with distance measured by Weech (2002) and reported by Levadoux and Baligh (1980)

After allowing a recovery period following installation, which varied between 19 hours, 7 days and 6 weeks, piles with two different helix plate spacing were loaded to failure under axial compressive loads. Strain gauges mounted on the pile shaft were monitored during load testing to determine the distribution of loading throughout the pile at the various load levels up to and including failure. Load-settlement curves were generated for different pile sections at different times after installation. The distribution of excess pore pressures was also monitored on the piezometers and piezo-ports during load testing.

It is difficult to explain the complex soil response observed at the Colebrook site based solely on interpretation of the field measurement. This paper uses a numerical approach to investigate the pore pressures and stresses induced during, and following, helical pile installation. Of particular interest is whether the predicted pore pressure is well constrained based on the properties typically measured during a well performed site investigation. Additionally, we examine whether the time evolution of the lateral effective stress at the pile wall is well constrained for reasonable soil parameters, with reasonable dissipation response.

2. MODELLING APPROACH

2.1 Overview

The volume changes in the silty-clay during and following pile installation influence the magnitude and distribution of time-dependent pore pressure and effective stress. Therefore, it is important that the chosen soil model generate realistic volume changes during shearing. A generalized critical state based soil model, NorSand (Jefferies, 1993; Jefferies and Shuttle, 2002), is adopted here. Although its name suggests sand, NorSand is a generalization of the well-known Cam Clay model with a specific capability of realistically representing soil dilatancy. NorSand achieves realistic dilatancy by separating the yield surface from the critical state by a distance in void ratio space defined by the state parameter, Ψ. Overconsolidation exists within NorSand in the usual sense, expressed as a ratio $R = p'_{max}/p'$, where p' is the mean normal effective stress. In effect, a zone of elastic behaviour can be defined using R with Ψ controlling subsequent plastic yielding. Most of the NorSand parameters are familiar and will be discussed below.

In order to predict the changes in stresses and pore pressure under partially drained conditions, an analysis which accounts for the coupling between the rate of loading and the generation of fluid pressures is required. Undrained conditions are not assumed and the pore fluid is free to migrate. The equations of Biot (Biot, 1941) incorporate the effect of coupling the pore pressure behaviour to the soil response.

2.2 Geometric Idealization

During pile installation soil in a region around the pile tip undergoes extensive disturbance and remoulding. Model studies of the displacement pattern in this region have shown that the displacements are between the deformation patterns caused by the expansion of a spherical cavity and a cylindrical cavity (Clark and Meyerhof, 1972; Roy et. al., 1975). These studies have also shown that little further vertical movement of soil occurs at any level once the tip of the pile has passed that level (Randolph et. al., 1979a). Randolph et al. (1979b) report measurements of the radial movement of soil near the mid-depth of the pile, taken from model tests and field data, showing measured radial displacements that agree very well with theoretical predictions, based on the assumption of plane strain and cylindrical deformation at constant volume.

This indicates that the stress changes in the soil over much of the length of the pile shaft (ignoring the regions close to the ground surface and to the pile tip) can be adequately approximated by cylindrical cavity expansion.

If for soil penetration by the helical pile shaft, cylindrical cavity expansion is an obvious analogy, modelling of the helical plates installation is a much more complex process and requires 3-D simulation. However, in this paper we are interested in variability of the pore pressures and lateral stresses, and therefore this effect has been ignored.

2.3 NorSand Input Parameters

As described in Section 2.1, NorSand is a generalized Cambridge-type constitutive model developed from the fundamental axioms of critical state theory and experimental data on sands.

NorSand has seven material properties (see Table 1): Γ, λ describe the familiar semi-log approximation to the critical state locus in void ratio, e - p′ space; elasticity is described by an elastic shear modulus, G and constant Poisson's ratio, υ; M_{tc} (the value of ratio q/p′ at the critical state in triaxial compression, $q_{tc}= \sigma_1'-\sigma_3'$), H_{mod} and χ are the plasticity properties. Of these properties, only H_{mod} and χ are unfamiliar. H_{mod} is a dimensionless plastic modulus, akin to I_r (= G/p') but for the plastic strains. It arises because NorSand decouples the yield surface from the CSL and consequently $\lambda - \kappa$ can no longer serve as the plastic compliance as it does in Cam Clay. Dilation in NorSand is proportional to ψ, and χ is the proportionality coefficient.

The initial density of the soil is represented by the state parameter, ψ, defined as the difference between the current void ratio and the critical void ratio ($e - e_c$) at the same mean effective stress.

Additionally, for a partially drained analysis, the radial hydraulic conductivity of the soil, k_r, is also required.

Table 1. NorSand parameters.

Parameter	Description
M_{crit}	Critical stress ratio
Γ	intercept of the CSL at 1 KPa mean stress
λ	Slope of CSL in e-ln(p) space
χ	Volumetric coupling parameter, function of fabric
H_{mod}	hardening parameter
G	Elastic shear modulus
υ	Poisson coefficient

2.3. Finite Element Formulation

In a saturated soil, when free drainage conditions prevail, the steady state pore-fluid pressures depend only on the hydraulic conditions and are independent of the soil skeleton response to the external loads. In these circumstances a single phase continuum description of soil behaviour is adequate. Similarly, a single phase description is also adequate when no drainage occurs. However, under intermediate boundary conditions in which some flow can take place, there is an interaction between the skeletal strains and the pore fluid flow through the voids. To analyse this situation accurately requires that soil behaviour be analyzed by incorporating the effect of transient flow of the pore-fluid through the voids. Such a theory was developed by Biot (1941).

Biot's theory accounts for solid-to-fluid and fluid-to-solid coupling, where: solid-to-fluid coupling occurs when a change in applied stress produces a change in fluid pressure or fluid mass; fluid-to-solid coupling occurs when a change in fluid pressure or fluid mass is responsible for a change in the volume of the soil.

For the radial symmetry used in these analyses, the Biot governing equation is given by;

$$\frac{K'}{\gamma_w}\left[k_r \frac{\partial^2 u_w}{\partial r^2} + k_r \frac{1}{r}\frac{\partial u_w}{\partial r}\right] = \frac{\partial u_w}{\partial t} - \frac{\partial p}{\partial t}$$

where;

K' = bulk modulus of the soil [kN/m^2]
γ_w = unit weight of water [kN/m^3]
u_w = pore pressure [kN/m^2]
k_r = radial hydraulic conductivity [m/s]
p = mean total stress [kN/m^2]

In these analyses both water and the soil particles are assumed to be incompressible, thus all volume change is the result of a change in the void ratio. Early objections to the assumption of a constant coefficient of consolidation, c_v, in the Biot formulation are irrelevant with modern numerical approaches in which soil properties can vary every increment (Smith and Hobbs, 1976).

The pile installation was modelled as purely radial, using a large strain elasto-plastic finite element code. By representing the helical pile in only the radial dimension the code could be streamlined to minimize simulation time. An incremental viscoplastic formulation was used to represent plasticity (Zienkiewicz and Cormeau, 1974). Although not typically used with more complex soil models, the viscoplastic approach has the advantages of being both simple and fast to converge (Shuttle, 2004). Biot coupling was implemented using the structured approach described in Smith and Griffiths (1997).

3. ESTIMATION OF NORSAND INPUT PARAMETERS

3.1 Overview of Investigations

Three subsurface investigations were performed at, or close to, the helical piles performance research site.

The first investigation was undertaken by the British Columbia Ministry of Transportation and Highways (MoTH) in 1969 prior to construction of the Colebrook Road overpass. The MoTH investigation included dynamic cone penetration tests and drilling with diamond drill to establish the depth and profile of the competent stratum underlying the soft sediments. Field vane shear tests were performed at selected depths. "Undisturbed" samples of the soft soils were recovered with a Shelby tube sampler. Additionally, a number of laboratory tests were carried out on the MoTH samples, including index tests, consolidated and unconsolidated triaxial tests and laboratory vane shear tests.

The second study by Crawford and Campanella (1991) reports the results of a study of the deformation characteristics of the subsoil, using a range of in-situ methods and laboratory tests to predict soil settlements underneath the embankment, and compare them with the actual settlements. In situ tests included field vane shear tests, piezocone penetration tests (CPTU) and a flat dilatometer test (DMT). Laboratory tests were limited to constant rate of strain odometer consolidation tests on specimens obtained with a piston sampler.

The most recent investigations were undertaken by Dolan (2001) and Weech (2002) as a part of study of helical pile performance in soft soils at the Colebrook site. Dolan (2001) obtained continuous piston tube samples from ground level to 8.6 m depth and performed index testing to determine natural moisture content, Atterberg limits, grain-size distribution, organic and salt content. Weech (2002) carried out a detailed in-situ site characterization program, which included field vane shear tests; cone penetration tests with pore pressure (CPTU) and shear wave velocity measurements (SCPT).

These site investigations provided many, but not all, of the input parameters required for the NorSand critical state soil model. However, the major difficulty in deriving input parameters resulted from differences between laboratory and in situ derived values of soil properties. This is not unusual in a silty site where soil disturbance during sampling is a major issue. Local spatial property variation, as seen in the in situ measurements, added to parameter uncertainty.

Due to space limitations it is not possible to provide a full explanation of the parameter derivation. A summary is given in the following sections, and full details are provided in Vyazmensky (2004).

3.2 Estimates of Initial State

A profile of vertical effective stress was established based on an average unit weight of the silty clay layer of 17.8 kN/m^3, estimated from index tests performed by Dolan (2001). For the range of elevations where the pore pressure changes due to pile installation were measured, from -4.57 to –9.92 meters, σ'_{v0} increases from 32.9 to 75.7 kPa. The average σ'_{v0} of 54.3 kPa is used in these analyses.

The profile of equilibrium pore water pressure was established based on piezometer measurements taken prior to helical pile installation. The pore pressure measurements indicate artesian conditions, and can be described by the following equation: u_0 (kPa)= -10.2(Elevation) - 7.1. For the range of elevation from -4.57 to -9.92 meters, u_0 increases from 39.5 to 94.1 kPa.

The coefficient of lateral earth pressure, K_0, was estimated based on empirical correlations developed for interpretation of CPT test data. K_0 for the silty clay layer varies primarily within a range of 0.56 to 0.76. The midrange K0 of 0.66 was assumed for these analyses.

OCR profiles were interpreted from CPT soundings by Weech (2002), and estimated from the consolidation tests performed during MoTH (1969) and Crawford and Campanella (1991) investigations. A mid-range value of OCR derived from CPT and laboratory data is 1.7.

The initial state, ψ, of the silty clay layer is unknown, and the absence of triaxial test data complicates its assessment. Hence, the state has been estimated based on the stress history of the soil. The Colebrook site is lightly overconsolidated over most of the elevations of interest, consistent with a relatively loose soil state. Additionally, the shear vane testing indicates a low undrained shear strength and highly sensitive response. Using the NorSand constitutive model, a sensitive response is only possible with positive (i.e. loose of critical state) values of state. Therefore a very loose range of ψ = -0.02 to +0.16 is assumed, with +0.08 as a best estimate.

3.3 NorSand Parameters

In the absence of triaxial testing, the NorSand specific soil properties were the most difficult to estimate.

Two elastic properties, G and υ, are needed. Values of G_{max} are available from seismic cone measurements, with the input value of G inferred by Weech (2002) being used for the best guess. No data are available on a Poisson's ratio for the silty clay layer For most soils Poisson's ratio, υ, is within a range 0.1 to 0.3. The current analysis uses υ=0.2 as the best estimate.

Crawford and deBoer (1987) quote a friction angle, ϕ', for the silty clay layer in the range 33° to 35°. Although not stated, it is assumed that these values are peak values. Assuming a very loose soil gives ϕ'_{cv} of the order of 31° to 33°, and M_{crit} in the range 1.24 to 1.33. A value of M_{crit} = 1.24, corresponding to ϕ'_{cv} = 31°, was used for the Reference case.

The model property, χ, is a function of fabric, and typically does not vary significantly for different soils (Jefferies and Been, 2005). In the absence of more detailed information it is often taken as 3.5. In the current modelling, a range of χ = 3.0 to 4.0 is assumed.

In the absence of triaxial data, the slope of the critical state line, λ, in e-ln(p') space may be estimate from an empirical relationship involving the plasticity index, PI. Schofield and Wroth (1968) found the relationship between PI, specific gravity, Gs, and slope of the critical state line, λ to be λ = PI Gs / 160. Assuming Gs equal to 2.75 and given that plasticity index for the silty clay layer varies from 7.6% to 21.1%, λ is in the range $0.13 \le \lambda \le 0.362$. This range is large, but is in a good agreement with the range, $0.08 \le \lambda \le 0.363$, reported by Allman and Atkinson (1992) for Bothkennar silty clay. For modelling purposes the lower bound of λ was extended to enable representation of the sensitive behaviour observed from shear vane testing. Γ is back-calculated from the measured void ratio and credible state range.

The final NorSand parameter, hardening modulus, H_{mod}, is typically the most difficult property to estimate in the absence of element test data. A fairly soft response, H_{mod} = 100 was used as the best estimate.

The modelled pore pressures generated during pile installation are insensitive to the value of k_r assumed as the installation is quick. For the dissipation analyses, however, a value of k_r is also required. This paper assumes a value of 10^{-9} m/s, uncorrected for direction, in the middle of the range suggested by Crawford and Campanella (1991).

Table 2. Reference and Best Fit NorSand Parameters

Param.	Ref. Case	Range	Best Fit	Units
G	7.8	5.8 - 9.8	7.8	MPa
ν	0.2	0.1 - 0.3	0.3	-
OCR	1.7	1.45 – 2.2	1.45	-
K_0	0.66	not varied	0.66	-
M_{crit}	1.24	1.24 - 1.33	1.33/1.5	-
χ	3.5	3.0 - 4.0	3.5	-
ψ	0.08	-0.02 - 0.16	0.16	-
λ	0.212	0.07 - 0.362	0.08	e-ln(p)
Γ	2.04	related to e	1.55	@1kPa
H_{mod}	100	50 – 450	200	-

A reference case (see Table 2), based on mid-range values of the parameters, is used as the starting point for this modelling. A second parameter set, derived by fitting the model to the pore pressure distribution at the end of installation, is also given in Table 2. Although referred to as the “Best Fit”, this parameter combination was still constrained to be reasonably consistent with the site properties. The fit is non-unique; there is slight flexibility on parameters which can produce a similar pore pressure response, and the choice of the “best” fit is subjective.

The fit can also be improved by allowing wider flexibility in the parameter ranges.

The predicted consolidated undrained triaxial response for the Reference and Best parameter sets are shown in Figures 2 and 3 respectively. The base case properties do not produce a sensitive soil response. The Best Fit is sensitive (S_t ~ 4.6), although of much lower sensitivity than indicated from shear vane testing. Two values of M_{crit} are shown in Table 2. The lower gives a closer fit to the measured pore pressure. The higher value used for Figure 3 is outside of the reasonable range, but gives an undrained shear strength and pore pressure at the pile wall following installation closer to the Reference case.

Figure 4 compares the pore pressure distribution measured at the end of pile installation with those predicted by the Reference and Best simulations. For both of the numerical fits, the field measured pore pressures are lower at the pile and extend further into the surrounding soil.

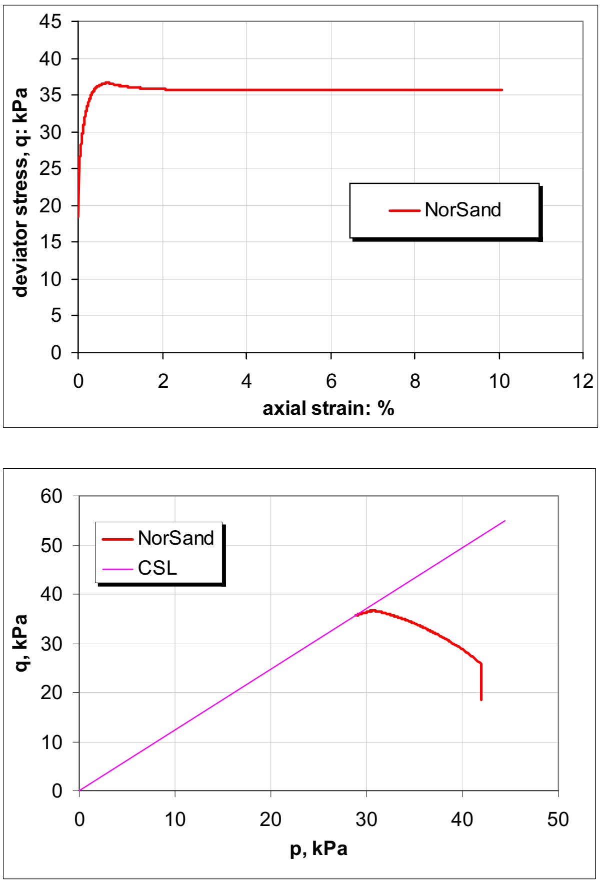

Figure 2 : CU Triaxial Response for Reference Parameters

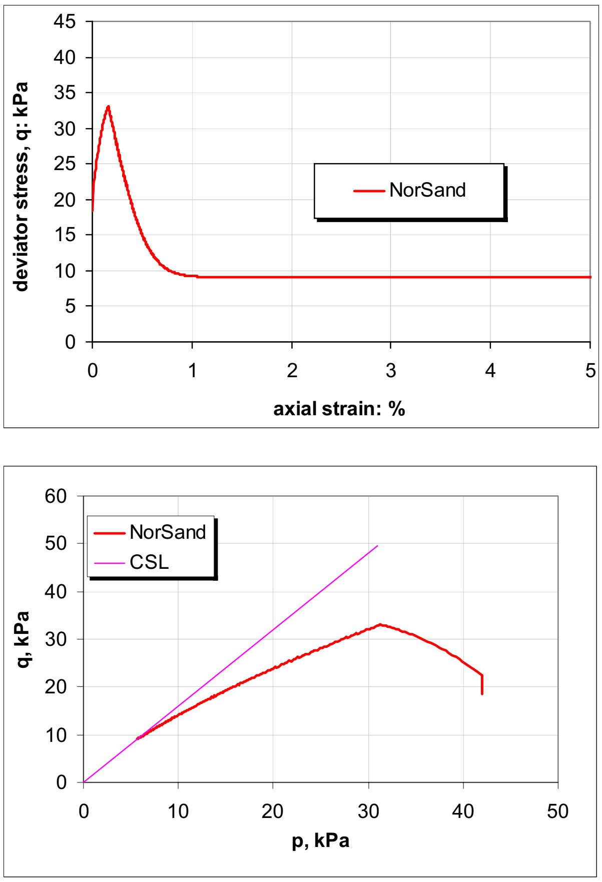

Figure 3 : CU Triaxial Response for Best Parameters

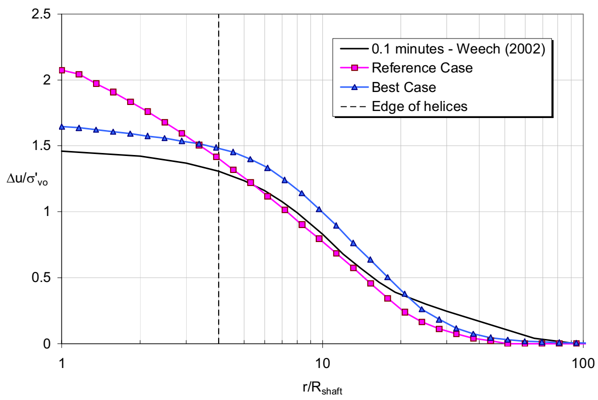

Figure 4 : Comparison of Weech (2002) measured $\Delta u/\sigma'_{v0}$ vs. distance with Reference and Best Fit Results

The Reference case predicts too high pore pressures between the pile wall and the edge of the helices. Beyond the helices the measured and predicted trends are approximately parallel until r/R_{shaft} exceeds 20, when the modelled pore pressures reduce more quickly. If a M_{crit} value in the middle of the parameter range had been assumed, rather than the minimum, the pore pressures between r/R_{shaft} of 4 and 20 would match for the Reference case.

The Best Fit was biased towards achieving a flatter pore pressure distribution between the pile wall and the helices' edge. This was achieved primarily by reducing λ to obtain a sensitive soil response and increasing ψ to make the soil more contractive.

Although the two numerical simulations have very similar peak undrained shear strengths the lateral effective stresses at the end of installation are quite different; 54.5 kPa and 13.5 kPa for the Reference case and Best case respectively.

4. COMPARISON OF MODEL WITH MEASURED PORE PRESSURE DISSIPATION

4.1 Pore pressures

The pore pressure dissipation measured at the pile wall is compared with those predicted using the Reference and Best case parameter sets in Figures 5 and 6. Figure 5 plots the excess pore pressure normalized by the vertical effective stress. Figure 6 normalizes by the excess pore pressure at the end of expansion (i.e. elapsed time = 0), giving a measure of the inferred percentage dissipation at the pile. Figure 5 shows both simulations provide reasonable estimates of the pore pressure (e.g. $\Delta u/\sigma'_{v0}$ <0.2) at the wall for times greater than 10 minutes. The Best case is closer to the measured values at early time, and gives a better estimate of the percentage dissipation. This is to be expected as the lower gradient at the pile wall under field and Best case conditions affects dissipation rates.

4.2 Lateral Effective Stresses

The magnitude of the pore pressure measured at the pile wall during installation could be expected to affect the long term stresses at the pile. Figure 4 shows a large difference between the end of installation pore pressures for the Reference and Best cases. Therefore, in order to make the comparison between the simulations clearer, the Best Case parameters were adjusted, to make the pore pressure at the pile wall closer to that predicted by the Reference case. This was achieved by increasing M_{crit} to 1.5. This change does not change the shape of the pore pressure response with distance, and increases the peak undrained shear strength of the soil closer to that of the Reference case. Figure 7 compares the evolution of pore pressure and lateral effective stress with time for both of the Best simulations. The figure shows the difference in effective stress against the pile is insensitive at small elapsed times. If the pore pressures from revised Best case are plotted on Figure 6, the response is indistinguishable from the original Best case.

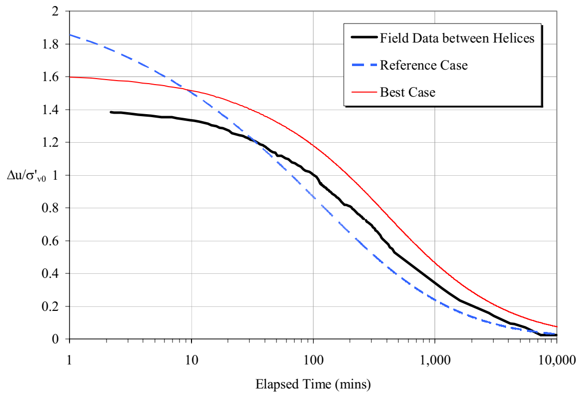

Figure 5 : Comparison of Weech (2002) measured $\Delta u/\sigma'_{v0}$ vs. time with Reference and Best Fit Results

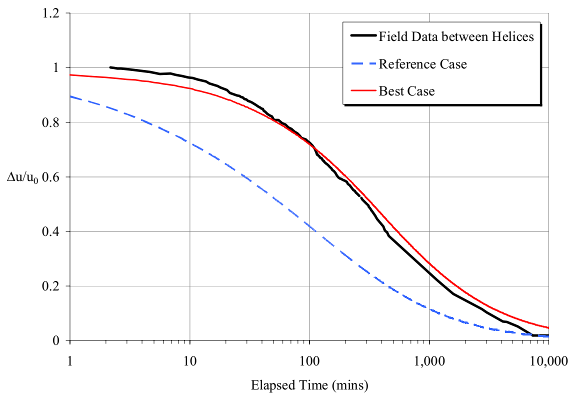

Figure 6 : Comparison of Weech (2002) measured $\Delta u/u_0$ vs. time with Reference and Best Fit Results

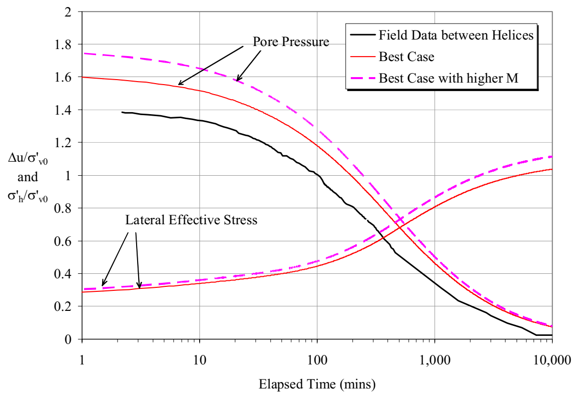

Figure 7 : Comparison of $\Delta u/\sigma'_{v0}$ and σ'_v/σ'_{v0} vs. time for Best Fit (M_{crit} = 1.33) and Revised Best Fit (M_{crit} = 1.50) Simulations

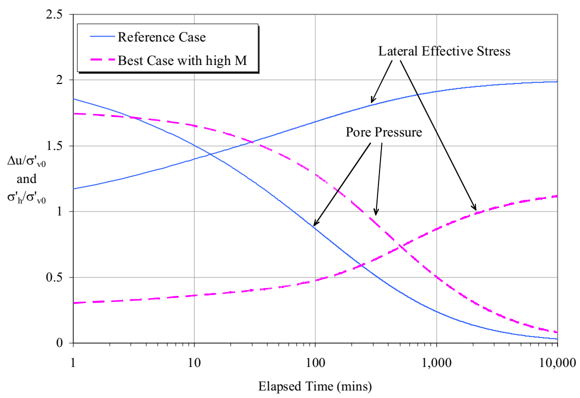

Figure 8 : Comparison of $\Delta u/\sigma'_{v0}$ and σ'_v/σ'_{v0} vs. time for Reference and Revised Best Fit (M_{crit} = 1.50) Simulations

Pile design methods in fine grained soils are typically based on undrained shear strengths. The peak undrained shear strength of the two soils shown in Figure 8 is quite similar. However, it is clear that although the pore pressures generated are similar, the evolution of lateral effective stress is very different. At the end of pile installation, the Reference case has a normalized lateral effective stress at the pile wall of 0.978. The corresponding normalized lateral effective stress at the pile wall for the Best case is only 0.265; a factor of 3.7 difference.

Over the next week (10,000 mins) the pore pressures dissipated and the effective stresses increased. During this time the Reference case normalized pore pressures fell by 2.04 and the normalized lateral stresses increased by 1.01 to 1.99, corresponding to 50% of the pore pressure reduction being translated into effective stresses. At the end of the simulation the vertical effective stress had increased from 54.3 to 75.1 kPa.

Over the same time period the Best case normalized pore pressures fell from 1.79 to 0.08, a fall of 1.71, and the lateral stresses increased from 0.27 to 1.12, an increase of 0.85. Like the Reference case, this corresponds to 50% of the pore pressure reduction being translated into lateral effective stress. The Best case simulation predicts a vertical effective stress at the end of the simulation of 44.5 kPa.

For very similar peak undrained shear strengths prior to installation, the final lateral effective stresses on the pile were very different. The Base case stress was 108 kPa and the Best case stress was 60 kPa, a factor of 1.8 difference, a much lower reduction than would be attributed to sensitivity.

It is worth noting that these simulations cause stress changes and yield in the vertical direction, even though no vertical strain is allowed. Most analytical solutions do not account for out of plane yielding, and hence may give slightly different results even for simple soil models such as Tresca and Mohr-Coulomb.

5. DISCUSSION AND CONCLUSIONS

The results presented in this paper form a small part of a numerical study (Vyazmensky, 2004) to improve our understanding of the time dependent behaviour of helical piles in silty soils; specifically focused on changes in pore pressure and how this influences pile capacity. The results presented in this paper highlight the difficulties in designing piles based on standard site investigations. All three site investigations close to the site were of good to excellent quality. Despite the quality of the investigations, wide variations exist between the laboratory and in situ derived values. This is not unusual at silty soil sites where undisturbed sampling is difficult. Additionally, some critical model parameters were not explicitly measured. These include two parameters commonly used by numerical models, ψ and H_{mod}. State parameter (similar to relative density in purpose) was inferred from the geology. H_{mod} is a NorSand specific parameter, but all plasticity models assume some relationship between the expansion/ contraction of the yield surface with plastic strain. Unfortunately, this parameter is sensitive to sampling disturbance and difficult to obtain in situ.

The effect of this parameter uncertainty had a large effect on the predicted pore pressures (e.g. Figure 5). Even if the pore pressure at the cavity wall following pile installation was correctly estimated, the time-dependent magnitude of the lateral stresses and pore pressures over time can be very different (Figures 6 and 8). This is the case, even for smaller differences in the input parameters. It is therefore not surprising that the ability of the geotechnical community to predict pile capacity based on standard site investigations is poor, and that so much of pile design is empirically, or local experience, based.

Despite the difficulties, numerical modelling remains an important tool for testing hypotheses of pile behaviour and has the potential to move pile design from the empirical to a true application of engineering mechanics. The next stage is to see whether pile capacities inferred from numerically calculated lateral stresses provide an improved estimate of the pile capacity measured at the Colebrook site.

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