pile are predominantly affected by the sliding depth and axial load. 30~60% increase in maximum bending moment and deflection could be anticipated with increase in sliding depth, or axial load. Furthermore, axial load can incur 7~8 times higher pile deflection. The soil resistance mobilized in the moving soil is about 1.5~3 times those estimated using current methods, but it is consistent with those back-figured from laterally loaded piles. Therefore, the existing methods may be conservative.

Acknowledgements

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NON-LINEAR ANALYSIS OF LARGE PILE GROUPS FOR THE NEW WEMBLEY STADIUM

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The paper describes state of the art non-linear analyses of the pile groups that were employed during the raising of the 133m high triumphal arch which forms the centre piece of the new Wembley stadium in North London. The deformation behaviour of these piles groups was critical during the lifting process, due to the slenderness of the arch structure. The issue was made more critical by the high profile nature of the project. The results of Class A (Lambe (1973)) predictions of the pile group movements are compared with observed displacements and rotations which were accurately measured on site. To illustrate the importance of soil non-linearity on pile group behaviour, comparison is also made with pile group analyses which assume a linear soil stiffness.

Introduction

The new Wembley Stadium's main architectural feature is an iconic 133m high triumphal arch. The arch is 7.4m in diameter, and has a total weight of 1,750 tonnes. With a span of 315m, it is one of the largest and most slender structures of its type in the world. Probably the most critical engineering activity during the stadium construction was raising the arch to its final position (112° to the horizontal). Because of the slenderness of the structure, the load-deformation performance of its foundations is a key factor in safely raising the arch. Observations of actual pile group behaviour are extremely rare, especially for complex vertical, horizontal and moment load combinations. Hence, making realistic predictions of pile group deformation was a major challenge. This paper describes the non-linear analyses which were carried out in order make class A predictions (Lambe (1973)) of the movements of these foundations.

During the raising of the Wembley arch monitoring was installed to determine the movement of the pile groups when subjected to these complex load combinations. These movements are compared to the predictions obtained from the class A non-linear analyses.

Further comparisons with more conventional linear pile group analyses are also presented.

Site Geology and Geotechnical Parameters

The site geology comprises London clay underneath made ground of varying thickness, overlying the Lambeth group at a depth of 30-35m, which in turn overlies Chalk. The uppermost 8 to 10m of London clay is weathered. The water table is at a depth of 2.5m below the London clay surface and the pore pressure profile at depth is underdrained. The small strain Young's moduli derived from seismic cone (SCPT) and self-boring pressuremeter (SBP) testing during the ground investigation are shown in Fig. 1.

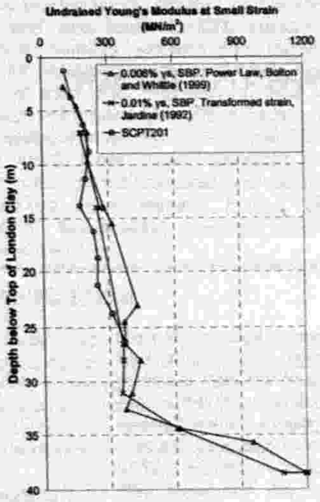

Fig. 1 Undrained Young's Modulus at Small Strain vs. Depth Below Top of London Clay (γ_s = shear strain)

Based on these results and advanced laboratory and other in-situ testing, the best estimate of the undrained vertical and horizontal Young's moduli (E_v and E_h) with depth below ground level (z) for the London clay are given inTable 1.

The best estimate of undrained shear strength (S_u) is given by 40+7.5z for 0<z<10 and 115+4.5z for z>10.

Table 1 Best Estimate of Undrained Young's Moduli

z m	E_v MN/m^2	dE_v/dz MN/m^2/m	E_h MN/m^2	dE_h/dz MN/m^2/m
0	100	6.8	100	15.0
5	134	9.0	175	11.0
10	179	14.0	230	14.6
25	390	3.0	450	0
35	420	0	450	0

Back Analysis of Single Pile Test Data

Many researchers have developed models that attempt to capture the non-linear load-displacement curves observed in tests on individual piles (for example Fleming (1992) and England (1999)). The importance of non-linearity on pile group behaviour has also been previously discussed in the literature (for example by Poulos (1989) and Randolph (2003)), however, linear analyses are still common practice in routine design.

Repute (Geocentrix (2002)) has been developed to analyse pile groups with a non-linear load-displacement response. The current soil stiffness in the analysis ($E_{current}$) is related to the initial elastic stiffness ($E_{initial}$) by a hyperbolic relationship of the type shown in Eq. 1.

$$E_{current} = E_{initial}\left(1 - R_f \frac{S}{T}\right) \qquad (1)$$

Where R_f is a hyperbolic constant controlling the degree of non-linearity, S is the mobilised pile-soil shear stress and T is the limiting value of shear stress determined from the soil-pile interface strength parameters (derived from either total stress or effective stress conditions). A constant soil stiffness can also be modelled, resulting in a linear load-displacement response.

Vertical and lateral pile tests were undertaken on a wide range of pile diameters (0.45m, 0.6m, 0.75m and 1.5m) and lengths (10m-25m) during a preliminary pile testing contract. The load-displacement curve for an 11.5m long test pile with a diameter of 0.75m is shown in Fig. 2. The result of a lateral load test on the same pile is shown in Fig. 3. Also shown in Figs. 2 and 3 are three linear Repute back analyses. The first represents the initial elastic response, the second represents the response of the pile at a factor of safety (FOS) against bearing capacity failure of 1.7 and the third represents the stiffness of the pile at the failure load. The horizontal and vertical Young's moduli used in these analyses are given in Table 2. A constant stiffness with depth was assumed for simplicity.

Table 2 Elastic Stiffness Used in Linear Pile Analyses

Run	Analysis Type	Ev MN/m^2	Eh MN/m^2
1	Initial Elastic Response	232	171
2	Stiffness at FOS = 1.7	118	23
3	Stiffness at Failure	60	9

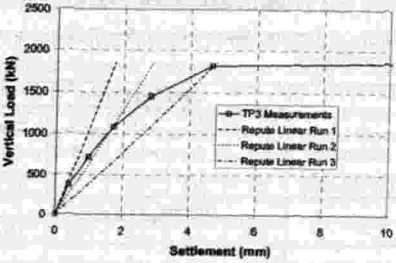

Fig. 2 Load-displacement Behaviour During Vertical Pile Test to Failure

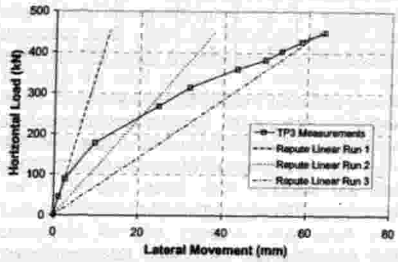

Fig. 3 Load-displacement Behaviour During Lateral Pile Test

Figs. 4 and 5 show the load-displacement curve for the above test pile calculated using a non-linear Repute analysis with the stiffness and strength profiles obtained from the site investigation (Table 1) and the following non-linear parameters; $R_f = 0.50$ for the shaft load, $R_f = 0.99$ for base load, $R_f = 0.90$ for lateral load and $T = \alpha\ S_u$, $\alpha = 0.6$. Below 40m depth the lower layers of the Lambeth group were assumed to be rigid.

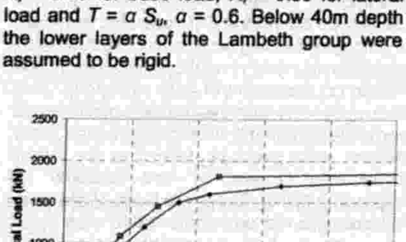

Fig. 4 Pile Test Data Compared to Non-Linear Repute Analysis for Vertical Load

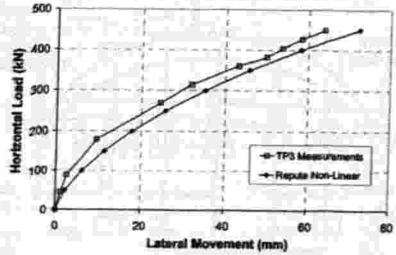

Fig. 5 Test Data Compared to Non-Linear Repute Analysis for Lateral Load

Figs. 4 and 5 illustrate that the non-linear analysis can capture the short-term load-displacement response of the single pile accurately for both vertical and lateral loads. A similar correlation was also found for the other pile tests that were back analysed. The soil parameters derived from the site investigation and adopted in the single pile analysis could therefore be used in a pile group analysis with some confidence.

Lifting Mechanism

The arch was constructed at ground level from individual sections and each end of the arch was connected to an arch base by a hinge joint which allowed free rotation in one direction. Five hydraulic jacks (or jacking points) were attached to the arch via a system of cables to pull it into a vertical position. Five intermediate struts (or turning struts) were used to give the pulling force a vertical component. The lifting mechanism is illustrated schematically in Figs. 6 and 7.

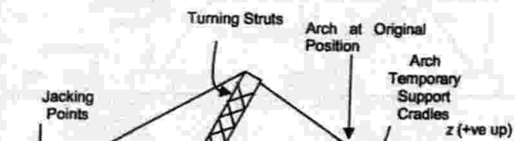

Fig. 6 Schematic Cross Section Showing Arch Lifting Mechanism

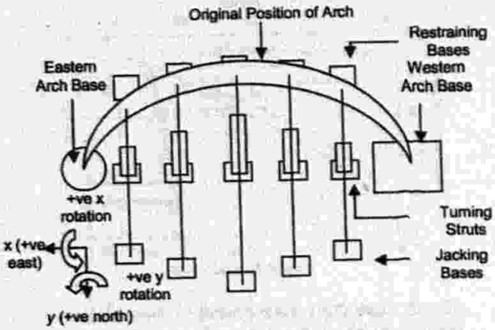

Fig. 7 Schematic Plan View Showing Arch Lifting Mechanism

Once the arch was close to the vertical position, tension was taken in five further bases (restraining bases) which would prevent the arch falling forward in an uncontrolled manner. This is illustrated schematically in Fig. 8.

When the arch went past the vertical position (at around 100°) the jacking points and turning struts became redundant and were dismantled. From this position to the final position (112°), the arch was held by the restraining lines. These lines will be holding the arch in place until the roof is constructed and the whole structure becomes self supporting.

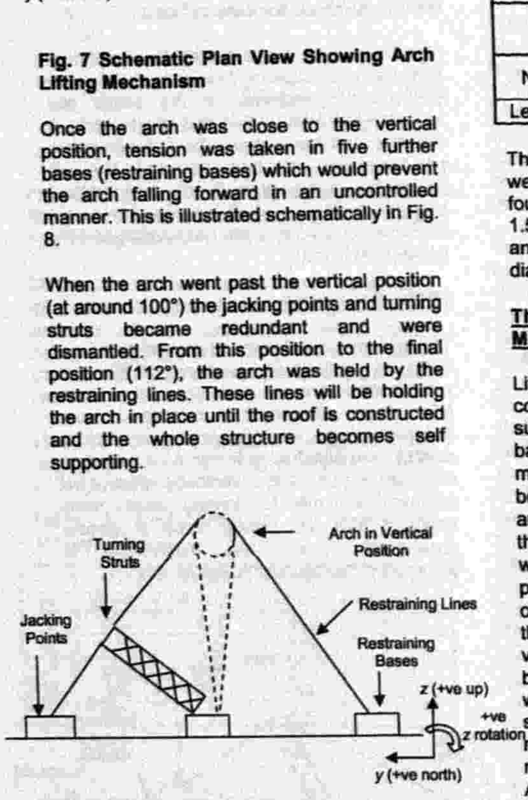

Fig. 8 Schematic Cross Section Through Arch Showing Activation of Restraining Lines

The fifteen temporary arch bases (5 jacking bases, 5 turning strut bases and 5 restraining bases) are constructed on piled foundations. The number and length of piles for each temporary arch base are shown in Table 3 (note that all pile diameters are 1.5m).

Table 3 Group Geometries for Temporary Arch Bases

	Jacking Bases				
	J1	J2	J3	J4	J5
Number	7	9	12	9	7
Length (m)	32	20	15	17	42
	Turning Strut Bases				
	T1	T2	T3	T4	T5
Number	9	10	10	10	6
Length (m)	24	25	26.5	25	37
	Restraint Bases				
	R1	R2	R3	R4	R5
Number	6	8	6	8	6
Length (m)	21.5	10	23	11.5	22

The permanent arch bases (eastern and western) are also constructed on piled foundations. The eastern arch base has 19 1.5m diameter piles with lengths of up to 33m and the western arch base has 60 1.5m diameter piles with lengths of up to 29m.

The Application of the Observational Method to Pile Group Behaviour

Lifting the arch into its final position applies complex load combinations to the pile groups supporting the temporary and permanent arch bases. If one of these pile groups were to move excessively, the consequences would be catastrophic. To safely manage the arch lift and in particular the pile group performance, the observational method (Powderham (1998)) was applied. To assess the performance of the pile groups during the arch erection, a system of precision levelling was devised to determine the in plane (*x* and *y*) rotations (see Fig. 7) and vertical (*z*) movement of each pile group. As a back-up to the surveying data, electrolevels were installed on the most critical bases. Total stations were used to determine the lateral movements (*x* and *y*) and torsional (*z*) rotations (see Fig. 6). On the basis of the class A predictions red and amber limits were set. If any component of movement was found to exceed the amber limit, an increased surveying frequency was specified. If a red limit was exceeded, remedial action would be carried out.

Predicted Pile Group Response

For each pile group 121 load combinations were considered, covering each angle of the arch (at 5° intervals) and any possible combination of structural loading. By utilising the symmetry in the pile group layout (see Table 3) and identifying the most critical pile groups, the number of temporary arch bases that were analysed in detail was reduced to eight, along with both arch bases. The critical load cases were selected from the 121 provided for each pile group and non-linear Repute analyses were run for each to obtain an envelope of displacements and rotations. The amber limits chosen for each movement component took into account the results of the non-linear Repute analyses and the surveying accuracy that could be achieved during the arch lift (±0.1mm for precision levelling and ±1.5mm for total station). To define the red limits each load component in the cases analysed was multiplied by a constant factor until one of the piles in the group reached its structural or geotechnical capacity.

Two examples of the movement envelopes calculated for the eastern arch base are shown in Figs. 9 and 10 together with the associated amber and red limits. For comparison the predicted movements when adopting the linear elastic stiffness profiles derived previously (and shown in Table 2) are also shown.

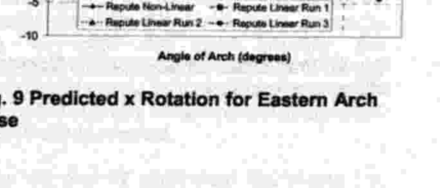

Fig. 9 Predicted x Rotation for Eastern Arch Base

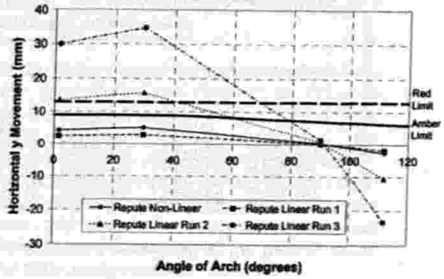

Fig. 10 Predicted Horizontal y Movement for Eastern Arch Base

The maximum structural forces predicted from the three linear and the one non-linear Repute analyses are compared with the bending moment and shear force capacities of the piles supporting the eastern arch base (assuming zero axial load) in Figs. 11 and 12 respectively.

Fig. 11 illustrates that the maximum predicted bending moment from the pile group analyses is heavily dependent on the stiffness model used in the analysis. The maximum shear forces however, do not vary significantly.

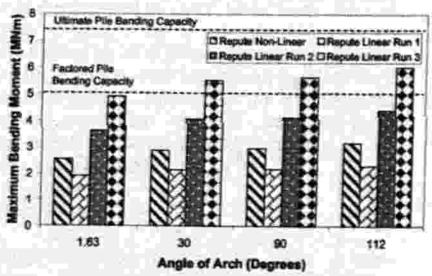

Fig. 11 Maximum Bending Moment Predicted from Linear and Non-linear Analyses

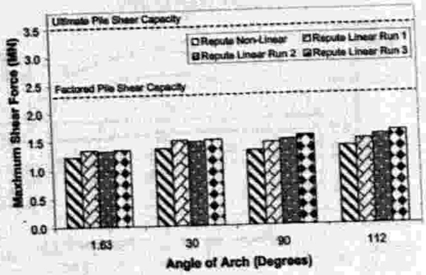

Fig. 12 Maximum Shear Force Predicted from Linear and Non-linear Analyses

Comparison of Observed and Predicted Displacements

During the arch lift the six components of movement (three displacements and three rotations) were calculated from the surveying data at each position of the arch and compared to the amber and red limits to determine if the lift could continue safely. As an example, the six movement components of the eastern arch base are compared to the results of the non-linear Repute analysis in the following figures.

Figs. 13 to 15 compare the vertical and horizontal movements, while Figs. 16 to 18 compare the *x*, *y* and *z* rotations.

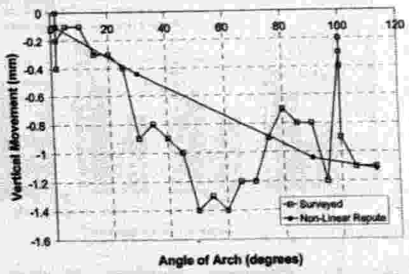

Fig. 13 Predicted and Observed Vertical Settlement of Eastern Arch Base

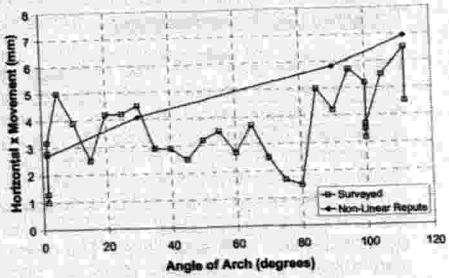

Fig. 14 Predicted and Observed Horizontal Movement in the *x* direction of Eastern Arch Base

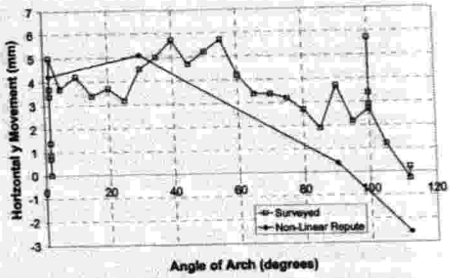

Fig. 15 Predicted and Horizontal Movement in the *y* direction of Eastern Arch Base

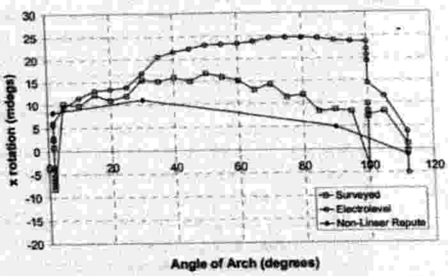

Fig. 16 Predicted and Observed *x* Rotation of Eastern Arch Base

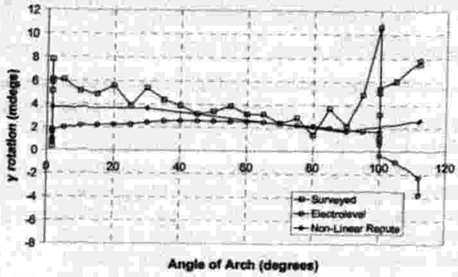

Fig. 17 Predicted and Observed *y* Rotation of Eastern Arch Base

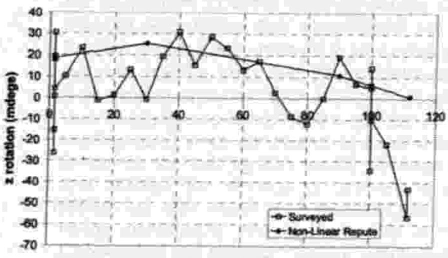

Fig. 18 Predicted and Observed *z* Rotation of Eastern Arch Base

The above figures show that there is a remarkably good correlation between the predicted and surveyed movements. It should be noted that at an angle of 100° sudden jumps can be observed in the measured data. This is due to the turning struts being dismantled and the restraining lines becoming operation, as mentioned previously.

As the arch goes past an angle of 30°, the horizontal force in the *y* direction reduces and at an angle of around 95° it is predicted to change direction. The movements shown in Fig. 15 follow this trend however, the measured displacements do not show a complete reversal. This could be due to the high initial stiffness of the soil being reinvoked upon the load reversal.

Figs. 16 and 17 also show the *x* and *y* rotations obtained from the electrolevels (which were used as a back up to the surveying data). Fig. 16 shows that the *x* rotations obtained from the electrolevels are generally higher than those obtained from surveying. In Fig. 17 the opposite is true, however, the measured movements are of a similar magnitude.

Having demonstrated the accuracy of the non-linear predictions (see Figs. 13 to 18), it is clear from Figs. 9 to 12 that if any of the linear elastic soil models had been used, the movement of the pile groups (and therefore the structural forces) would have been inaccurately predicted. Consequently the amber and red limits would have been set incorrectly and the safety of the arch lift could have been compromised.

Conclusions

The lifting of the arch at the new Wembley stadium has presented an opportunity to undertake class A predictions for the load-displacement behaviour of several large pile groups subjected to complex load combinations. Due to the sensitivity of this high profile project, high quality surveying was undertaken and instrumentation was installed to ensure the timely and safe completion of the arch lift.

The predicted movements correlated very well with the observed movements and illustrate the importance of modelling non-linear load-displacement behaviour of pile groups. If linear analyses had been undertaken (as is routine for pile group analysis) the structural forces and displacements could have been under or over estimated by a considerable amount, depending on what linear stiffness was chosen for the analysis. Red limit breaches could have been triggered incorrectly or not triggered when necessary due to the non-linear behaviour not being correctly replicated by the pile group analysis.

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DRIVEN PILES IN GLACIAL DEPOSITS EXHIBITING TIME DEPENDENT CAPACITY

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The pile capacity gain with time evaluated and implemented for the construction of the Route 21 Viaduct in Newark, NJ, U.S.A is discussed in this paper. The project consisted of the replacement of more than 3.2 km of viaduct and ramps. The project site consists of thick residual materials deposited by the Wisconsin Glacier over sedimentary rocks. The upper portion is glacial-lacustrine deposit underlain by marginal morainic till and stratified drift. Subsurface investigations identified significant variations in layer thickness and grain size consistency. The site was characterized into 4 Soil types, based on subsurface soil behavior.

Closed end 600 mm and 450 mm diameter driven pipe piles were used for the foundation. Loading tests conducted during an advanced construction contract, established that a significant soil set-up could be expected. For the first construction contract, a pilot loading test program consisting of 112 dynamic loading tests and 77 re-strikes (at 2 to 4 weeks) using the PDA (Pile Driving Analyzer) with CAPWAP (Case Pile Wave Analyses Program) was conducted to establish the soil set-up behavior. Nine (9) quick static loading tests were conducted to substantiate the long-term pile capacity. Based on the loading test results, average set-up factors were established and utilized for developing production pile driving criteria. Bearing resistance with depth was analyzed and reviewed. The set-up factors varied significantly, ranging from 1.05 to 1.89, and averaged about 1.30. Conclusions are derived. Set-up factors behavior for each soil type plotted by linear regression are presented and discussed. An appropriate set-up with time relationship for a glacial soil is suggested.

INTRODUCTION

In order to study the time dependent pile capacity characteristics in glacial deposits, the pile driving and testing data from 63 foundation units (First Construction Contract) were utilized. Closed end pipe piles (600 mm and 450 mm diameter) were utilized for the foundations. The dynamic and static loading tests, conducted during an advanced contract, revealed variable soil conditions with soil set-up behavior. For the first Contract a pilot load testing program was established for evaluating the set-up behavior. Dynamic loading tests monitored by Pile Driving Analyzer (PDA) and Case Pile Wave Analysis Program (CAPWAP), were conducted on 112 piles. Nine (9) static loading tests were also conducted.

A set-up factor, as defined by Poulos and Davis (17), is the ratio of soil strength a considerable time after driving to that immediately after driving. Set-up occurred within the site. Significant variations occurred in set-up over a short distance or even within a footing. Mobilized shaft resistance with depth was not consistent. Mobilized capacity varied significantly even within the same soil type. Correlating above observations, production pile driving criteria was established. Depending upon the subsurface soil conditions, the entire site was divided into four (1 to 4) representative soil types to characterize the set-up behavior. An understanding of the Set-up behavior mechanism was developed by studying relevant papers.

Rausche et al (21) has presented a method to determine static pile capacity by measurement of force and acceleration. Piles driven into soft to medium clays or loose saturated silts and silty sands usually exhibit a time dependent increase in capacity due to the effect of 'soil set-up'. In a normally consolidated clay, the capacity increase is due to thixotrophy (regain of undrained strength) and local consolidation produced by dissipation of excess pore water pressure. Fellenius, et al (8) reported set-up behavior in highly variable glacial deposits. Capacity increased considerably in a week and the behavior was not consistent over the site. Dynamic loading tests matched well with static loading tests. Set-up occurred rapidly during the first day after initial driving and then