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Technical session 2h: Pile foundations (II): Installation, quality control, performance, and case histories

Séances techniques 2h: Fondations sur pieux (II): Installation, contrôle qualité, performance et etude de cas

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1 INTRODUCTION

While there have been considerable advances in scientific approaches to pile design in recent decades (Randolph 2003a), there has been equally strong growth in the development of new piling techniques and in the monitoring and quality control of these techniques. The papers presented to this session reflect such trends indicating that:

(i) New or modified piling techniques have been stimulated by the need to produce cost-effective piles of high capacity, while at the same time ensuring that they may be used in urban areas where noise and vibration are restricted to minimal levels. Most case histories described involve 'quiet' pile construction techniques (e.g. bored, screw and jacked piles), and the few that deal with driven or vibrated piles consider the effects of ground accelerations on adjacent properties.

(ii) Pile testing, quality control and monitoring remain essential components of a piled solution. The papers to this session have revealed the growing popularity of the Osterberg cell to conduct static load tests and have identified the need for more reliable and less subjective interpretation techniques for both low and high strain dynamic testing.

Papers assigned to this session also consider:

(iii) predictive approaches that reflect current needs to establish more reliable and cost-effective design techniques for pile groups and laterally loaded piles. Design-chart type methods for the assessment of the behavior of pile groups and approaches for pile design in seismic areas are presented.

(iv) the re-use of piled foundations and soil structure interaction effects.

These four subject areas provide the framework for the following review and discussion of the papers presented to Technical Session 2h of 16th ICSMGE. The references to these papers are shown in italics to distinguish them from other papers referred to.

2 PILE TYPES

A significant number of papers have highlighted interesting design and construction issues for a variety of pile types; these are summarized and assessed in the following sections.

2.1 *Cast in-situ auger and screw piles*

The continuous flight auger (CFA) pile is currently one of the most popular types of cast-in-situ concrete piles. However, its dominant position in the marketplace is being challenged by a new generation of screw piles, which are designed to induce displacement to the soil, thereby yielding a capacity far in excess of that which can be developed by a replacement pile. *Albuquerque et al.(2005)* present a well documented case history from Brazil involving nine 12m long, instrumented piles (3 No. CFA, 3 No. bored-without bentonite and 3 No. Omega) with nominal diameters (D) between 0.36m and 0.4m. The upper 6m of soil at the test site was a collapsible (high porosity) sandy clay; this was underlain by residual clayey-sandy silt. All piles were equipped with instrumentation to measure the shaft friction distributions and the contributions of end bearing to the capacity during static load tests. On extraction of one of each pile type from the ground after load testing, the characteristic 'ribs' of the omega pile were observed and the CFA pile showed signs of over-break with an effective diameter of up to 490mm between 1m and 3m depth.

It was found that the end bearing capacity of the Omega screw pile at a pile displacement of 10% of the diameter, $q_{b0.1}$, was ≈0.6 times the CPT q_c value at pile tip level; this ratio is the same as that deduced by Xu & Lehane (2005) for full displacement driven piles in sand. In contrast, $q_{b0.1}/q_c$ ratios observed for the CFA and bored piles were only 0.2 and 0.05 respectively. The shaft friction developed by the CFA pile in the upper collapsible sandy clay was 2.5 times that of a bored pile (which is partly attributed the over-break observed) but similar to the bored pile in the underlying sandy silt. The Omega pile was, however, also the best performing for shaft friction, particularly closer to the pile tip where the shaft friction generated was three times that of the CFA pile.

Katzenbach & Schmitt (2005), although primarily focusing on modelling the load displacement behaviour of bored piles,

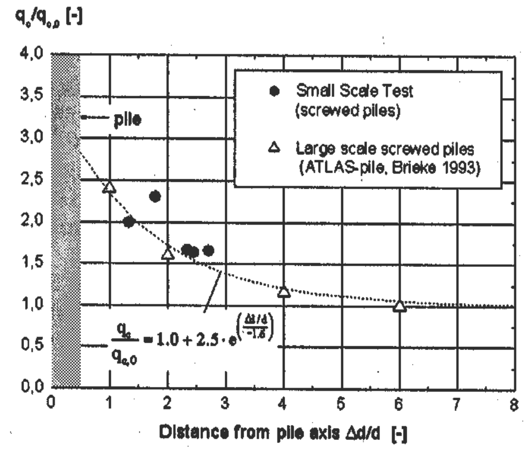

Fig. 1. Increase in CPT q_c resistance adjacent to a screw pile (reproduced from *Katzenbach & Schmitt 2005)*

provide data for small scale CFA and screw piles (D=280mm, L=3m) conducted in a (large) 8m ×6m × 6m testing chamber filled with loose and medium dense sand. These tests indicated that the shaft friction developed on the screw pile was at least double that on a CFA pile. This factor of 2 is, as shown on Fig. 1, the same as the ratio to the original CPT q_c value ($q_{c,0}$) of the q_c value measured at a distance of 1D from the shaft after installation of the screw pile.

This evidence and the summary of European piling practice provided by De Cock et al. (2003a) indicate that both the shaft and base resistance of cast-in-situ screw piles (such as Omega, Fundex, & Atlas) are between 1.5 and 3 times those of equivalent CFA piles in sands, silts and clays. It is therefore likely to be only a matter of time before screw piles dominate the market for medium scale (300 to 800mm diameter) bored piles.

2.2 *Jacked piles*

Jacked piles are becoming a more popular displacement pile option for urban developments in view of the minimal noise and vibration associated with their installation. *Medzvieckas and Sližyte (2005)* describe observations made using three 220mm diameter (D) instrumented piles that were jacked to a depth of about 3m in a large 'sand box' and then load tested statically. Their results, which are reported to show similar trends to those of other full scale jacked pile installation in Lithuania, indicate that the piles' ultimate static capacities (i.e. measured at plunging failure) were only 80% of the jacking force required for their installation. This trend, which is referred to as 'relaxation' by Mitchell & Mander-Jones (2004), was not observed by *Deeks et al. (2005)* who present data for jacked pipe piles at a sand site in Japan and report that installation loads were generally equivalent to the static capacity at a displacement of 0.1D.

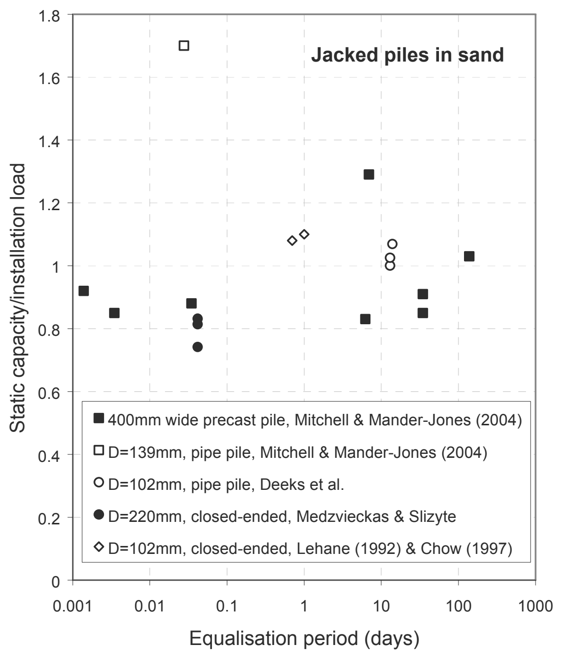

Fig. 2. Ratio of static compressive capacity to installation capacity of jacked piles in sand

The tendency for 'relaxation', if it is a general trend for jacked piles in sand, would detract from the often cited advantage for jacked piles i.e. the installation load is measured and can be considered in sand to be equivalent to (or less than) a pile's static ultimate capacity. The test data of *Medzvieckas and Sližyte (2005)* indicate higher shaft capacities in static load testing than during installation, which is in keeping with the now well established tendency for shaft friction in sand to increase with time (e.g. Axelsson 1998 and Chow et al. 1998). The corresponding ultimate base capacities were, however, only 0.6 ±0.06 times the base capacity measured at the final jacking stage of installation, indicating that 'relaxation' is a phenomenon affecting the base capacity.

Ratios of installation loads to static capacities are plotted on Fig. 2 against the time since installation (or equalization period) for a selection of jacked piles in sand. Allowing for the effects on ageing on the shaft resistance component of the static capacities, the data on Fig. 2 appear to suggest that, on average, the static ultimate base capacity will be less than that mobilized during installation. This effect, which appears more dominant for short piles with a low shaft resistance, requires further investigation. If 'relaxation' does not occur, *Deeks et al. (2005)*, and others, show that the base stiffness of a jacked pile in sand may be much larger than that of a driven pile.

Huy et al. (2005) examine the influence of loading rate in dry sand and show a surprisingly strong rate effect of the peak deviator stress measured in triaxial tests. In contrast, no rate effect on penetrometer resistance in dry sand was observed between velocities of 1mm/s and 250 mm/s. This absence of a rate effect for penetrometer resistance suggests that differences between installation loads and static capacities of jacked piles are not related to rate effects.

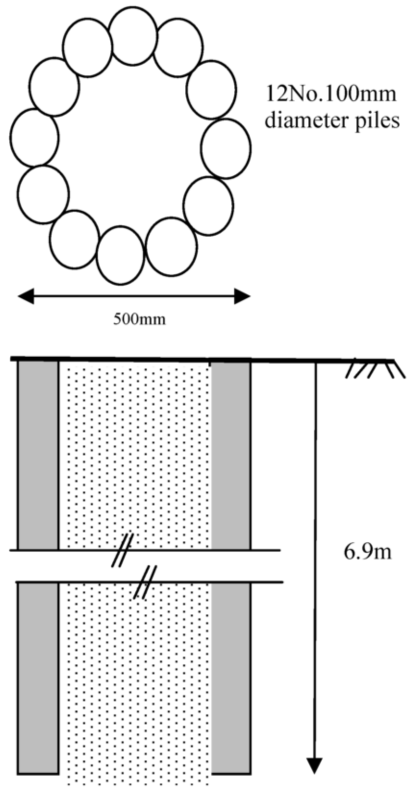

Fig.3. Creation of a large diameter pipe pile using small diameter jacked piles (*Deeks et al. 2005*)

Deeks et al. (2005) also present results from an interesting test involving 12 No. 100mm diameter, 6.9m long jacked piles at the same sand site in Japan. As shown on Fig. 3, the piles were arranged in a circular configuration to form what was essentially a 500mm diameter pipe pile with a wall thickness of 100mm. The authors note that the enclosed sand was observed to move downwards during failure (i.e. the sand plug did not move). This behaviour, which is the same as that shown by the plugs of pipe piles in sand under static loading, is in marked contrast to the response shown by groups of 12 piles arranged in a more typical grid configuration (where this type of 'block' failure is uncommon). The stiffness of the pile group shown on Fig. 3 must therefore be largely controlled by the stiffness of the relatively undisturbed sand beneath the plug – and hence be similar to the (lower) operational stiffness of a typical bored pile. This inference is in keeping with the comment of *Deeks et al. (2005)* who state that the stiffness of the group was over-predicted using the super-position approach that they used effectively to predict the stiffness of other pile groups with more conventional spacings.

2.3 *Conventional bored piles*

Because of the limitations on the diameter and length of screw piles and CFA piles, conventional bored piles employing temporary casing or bentonite/polymer slurries continue to dominate the market for large capacity cast-in-situ piles.

Both *Seah et al. (2005) and Corbet et al (2005).* describe the pile design, construction and testing for a 345 km long, high-speed rail guide-way in Taiwan. The southern portion of this guide-way was primarily constructed as a viaduct and involved about 20,000 No. 1.5m to 2m bored piles with working loads often in excess of 10 MN. The piles were bored to a typical depth of 50m in inter-bedded layers of sand, clay and silt and used a polymer mud coupled with a reverse circulation drilling rig. Loose sediment was removed from the pile bases by airlifting prior to concreting and, in some instances, pile bases were grouted after concreting.

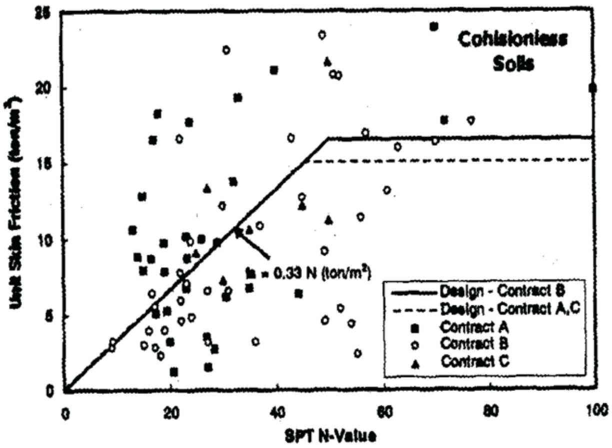

Fig. 4. Backanalysed correlation between the SPT N value and shaft friction on bored piles in sand (reproduced from *Seah et al. 2005.*).

The pile designs were based on correlations, such as shown on Fig. 4, derived from a comparison of SPT N data with results from preliminary static tests performed using conventional and Osterberg-cell testing techniques. These correlations were very poor with, for example, a sand with N=20 indicating ultimate average shaft friction (q_s) varying between 10 kPa and 200 kPa. It is difficult to see how the data on Fig. 4 provided a good basis for design. The suggested trend lines clearly do not provide an adequate correlation and are also higher than normally assumed in practice. For example, Poulos (1989) indicates that q_s (kPa) $\approx$1.0N is a typical (although conservative) correlation for shaft friction on a bored pile in sand while the design trend line adopted leads to predictions that are three times this value i.e. q_s (kPa) $\approx$3.0N. The correlations adopted for base capacity are however more conservative than usually adopted and compensate for the apparent optimism.

Pardini (2005) describes a case history which included Osterberg-cell load tests on 1.8m and 2m diameter pile bored under bentonite; these were used for a large infrastructure project in Argentina, which included a 550m long cable-stayed bridge. The test piles were about 30m in length with the final 9m length embedded in high plasticity Miocenic clay. The measured ultimate end bearing resistance was about 10 times the in-situ undrained strength assessed from triaxial test data (when corrected for the reported non-zero total stress friction angle) and therefore in keeping with conventional bearing capacity theory.

Bustamante and Boato (2005) discuss three case histories which involved the use of various polymer slurries to construct large diameter (D=1.2m to 2.2m) bored piles to depths of up to 75m. The selected polymer was, however, only satisfactory at one of the sites (for which sand was the dominant soil type). Nevertheless, the authors are generally supportive of the use of polymer slurries but conclude that site specific trials need to be conducted in advance of the main piling works to verify the compatibility between the polymer and soil type.

2.4 *Cast-in-situ piles using temporary casing*

Two papers discuss the application of cast-in-situ concrete piles, involving the use of temporary steel casing which is vibrated into the ground. Both of the piles types gain additional capacity due to the displacement induced by the casing insertion.

Liu et al. (2005) describe a novel, recently patented, cast in-situ concrete pipe pile, referred to as a PCC pile. A casing comprising an inner and outer pipe, which are connected at the pile tip using the detail shown on Fig. 5, is vibrated to the required depth (with a limit of 25m). Concrete is then placed in the annular void between the pipes (which is typically about 125mm wide) and forces the 'flap' at the pile base to open. Concreting is continued while withdrawing the casing by vibration. The end result is an un-reinforced concrete pipe pile.

This type of pile has been used successfully to provide support to embankments approaching bridge abutments in China. *Liu et al. (2005)* provide an example from an instrumented embankment (which employed a geogrid as a pile cap for the PCC piles) showing the piles' effectiveness in reducing settlements. The 'PCC' pile provides an efficient ratio of pile capacity to concrete volume. However, the absence of reinforcement and hence bending and tensile capacity limits the application of the 'PCC' pile (at least for the reported configuration).

Fig. 5. End detail of temporary casing for cast-in-situ (PCC) pipe pile (*Liu et al. 2005*).

David & Mail (2005) describe a case history from Israel involving the use of 'Vibrex' piles with lengths of 14 to 20m in sand (with an upper 3-6m layer of stiff clay); these piles involve vibration of a temporary casing with a sarcrificial shoe prior to concreting. The vibratory action is likely to have benefited the in-situ sand at the test site and, as with 'PCC' pile described by *Liu et al. (2005)*, the pile capacities were also enhanced by the displacement induced by installation. Strain gauges fixed to the reinforcement indicated that the ultimate friction generated in the sand was about 180 kPa. This level of friction, which equates to about 4.5 N (kPa), falls within the range of correlations summarised by Poulos (1989) for full displacement piles. Further static load tests at the same site indicated that the capacity of a 'Vibrex' pile was not affected by installation, one hour later, of another 'Vibrex' pile at a distance of 4D away.

Geophones were employed to monitor vibrations during the 'Vibrex' casing installations and showed that peak particle velocities (ppvs) were within acceptable levels at a distance of 10 m from the installation location. These measurements also showed that the 'ppvs' recorded in the basement of a nearby structure were typically less than 30% of those of the adjacent ground surface. Such a soil-structure interaction effect may well form part of the study presented by *Yeung et al. (2005)* who describe the techniques that will be adopted in Hong Kong to assist formulation of practical noise and vibration guidelines for driven piling contractors.

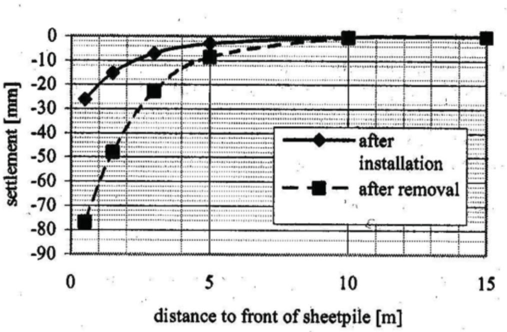

Fig. 6 Settlements induced by sheet pile installation and extraction *(Meijers and van Tol 2005)*

2.5 *Sheet piling*

Meijers and van Tol (2005) highlight the importance of settlement induced by sheet piling in sand and provide a summary from Dutch case histories indicating that there is a one in ten probability that the settlement induced 1m from a sheet pile will be in excess of 100mm. The same authors focus on settlements induced by vibrated sheet piles and, based on their own model tests, argue that the model proposed by Sawicki et al. (1998) is most suitable for settlement predictions. This model relates the sand compaction to the square of the shear strain amplitude (imposed by the vibratory hammer) and employs empirical coefficients that are a function of the initial sand relative density. A field trial, however, showed that this method over-predicted the observed surface settlements after installation by a factor of 4. This over-prediction may have been due to a 1.5m thick near surface layer of clay, as the predictions were more in line with settlements observed at depth.

On removal of the test sheet piles, as shown on Fig. 6, the maximum surface settlement increased from 28mm to 77mm. The increase in the settlement tough, which was shown to be consistent with the volume of the sheet pile, is an important consideration that is often over-looked in design e.g. for this particular case history, the sheet pile removal caused the ground surface distortion at a distance of 4m from the sheet pile wall to increase from 1/400 to 1/130.

3 PILE GROUP ANALYSIS

Comodromos & Bareka (2005) and *Kempfert & Rudolf (2005)* present results from 3-D numerical investigations into the behaviour of relatively small pile groups. Both papers acknowledge the effects of non-linearity on predicted group response and use the numerical analyses to develop design charts.

Comodromos & Bareka (2005) employed FLAC-3D, with about 38000 elements and a linear elastic perfectly plastic model for the soil to produce plots for stiffness efficiencies at a range of normalised settlements (w/D) for 2×3, 3×3 and 5×5 pile groups with various spacing ratios (s/D) and slenderness ratios (L/D). Their results for a 3×3 group with s/D=3 and L/D=25 in a clay with an undrained shear strength of 50 kPa and a limiting pile shaft shear stress of 50 kPa are compared in Fig. 7 with the same predictions obtained using the RATZ computer program (Randolph 2003b), which employs non-linear, discrete load transfer curves (i.e. springs) to represent soil at various levels along the shaft and at the pile base. The RATZ predictions for the group response are obtained by factoring the elastic component of the load transfer curves for the single pile using a settlement ratio term given by Randolph (1979). It is clear that the RATZ and FLAC predictions are in close agreement, indicating that, for the type of problem analysed, the much simpler and less time consuming load transfer approach is adequate.

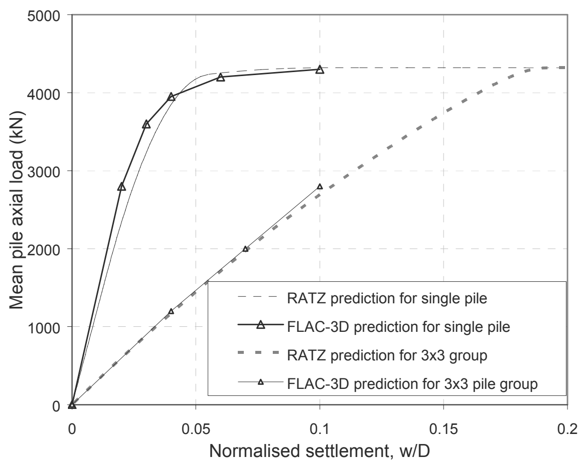

Fig. 7 Comparison of FLAC-3D analyses of *Comodromos & Bareka (2005)* with predictions using RATZ for a 3 × 3 pile group (s/D=3, L/D=25)

Kempfert & Rudolf (2005) employed ABAQUS to develop design charts for estimating the average settlement of a pile group (w_{group}) in a drained elastic soil with a Mohr Coulomb failure criterion. The authors' parametric study indicated that w_{group} does not depend on the pile diameter and that the ratio of the pile group to single pile settlement (w_{group}/w_{single}) depends primarily on the pile spacing to length ratio (s/L) and the number of piles in the group, N; groups were assumed to have a square geometry in plan. Their proposed trend for 3×3 and 5×5 groups is compared on Fig. 8 with corresponding predictions obtained using the well known curves published by Fleming et al. (1992), which relate w_{group}/w_{single} to L/D, s/D and the ratio of pile to soil stiffness. It is evident that the Fleming et al. (1992) predictions (for purely elastic conditions) lead to the inference of a slower degradation of w_{group}/w_{single} with s/L than that given by *Kempfert & Rudolf (2005)*; the scatter shown by the Fleming et al. (1992) points arises because this approach does not lead to a unique dependence of w_{group}/w_{single} on s/L.

The predictions of w_{group}/w_{single} for 3×3 and 5×5 pile groups using the approach of *Comodromos & Bareka (2005)* are also included on Fig. 8 and are seen to be, on average, a little larger than those of *Kempfert & Rudolf (2005)* and Fleming et al. (1992). Measured settlement ratios for two 3×3 pile groups in clay (reported by O'Neill et al. 1982 and Koizumi & Ito 1967) are plotted on Fig. 8a and evidently fall below the three sets of predictions i.e. the three approaches (all of which assume linear elastic conditions before failure) over-estimate interaction effects, indicating that soil stiffness non-linearity needs to be included in prediction methods.

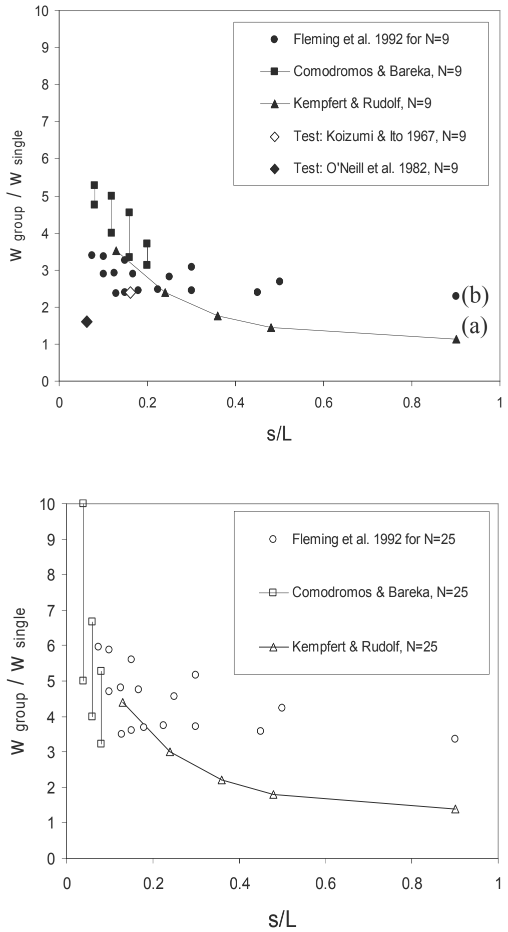

Fig. 8. Pile group settlement ratios for (a) 3 ×3 pile group and (b) 5 × 5 pile group

4 LATERALLY LOADED PILES

Hafez & Budkowska (2005) and *Rahman & Budkowska (2005)* present sensitivity analyses for laterally loaded piles conducted within the framework of variational calculus. These analyses demonstrate the relative importance of factors such as the pile's bending stiffness, soil stiffness and strength and show the expected dominance of the properties of the soil in the vicinity of the pile head. The findings are in keeping with less formal approaches to sensitivity analyses involving input of a range of parameters to computer programs for laterally load pile analysis.

Given the dominant influence of near surface layers on a pile's lateral performance, ground improvement of these layers may be expected to lead to a significant enhancement of this performance. *Tomisawa & Nishikawa (2005)* present results from a field trial and centrifuge tests to demonstrate this effect and also propose a simplified approach to assess the level of improvement. It is proposed that ground improvement, which for the field case reported involved soil-cement mixing, should extend to a depth of $[4EI/(kD)]^{0.25}$, where EI is the pile's flexural rigidity, D is the pile diameter and k is the coefficient of subgrade of the improved soil; this k value is estimated from the proportion of ground occupied by the improved soil-cement columns and the unconfined strength of these columns. The field trial indicated that the use of ground improvement reduced the number of piles required for a bridge abutment by a factor of 6 and reduced overall construction costs by 45%.

Sesov et al.(2005) present results from experiments which examined the effects of lateral spreading of liquefied soil on single piles and pile groups installed on gently sloping ground. The experiments were conducted using small diameter piles at 1*g* in a testing chamber with a maximum sand height of only ≈400mm. The results are therefore likely to have been affected significantly by scale effects. The experiments did show, however, that lateral movements induced by shaking (at 10 Hz) vary with the relative permeability of the sand layers employed. Peak bending moments for single piles occurred at peak soil lateral velocity and not at the maximum imposed lateral displacement while the pattern of peak bending moments induced in a 3×3 pile group was complicated and required more research.

Uzuoka (2005) examines the response of piles to liquefaction induced lateral spreading of sand by performing finite element (FE) analyses for a liquefied sand at a given depth (i.e. plane stress analysis with constant vertical stress) and using the coupled constitutive model of Oka et al. (1994). These analyses indicate that, in keeping with the observations of *Sesov et al. (2005)*, the stress cycles induced on the piles are linked to the soil velocity rather than to the displacement, but only when the loading frequency is high (10 Hz) or the degree of liquefaction is moderate to high (i.e. effective stress is 10^{-5} times the initial effective stress). At low excitation frequencies (e.g. 0.1Hz), lateral pressures relate directly to the soil displacement and the inertia forces are negligible.

5 PILE TESTING

The papers submitted to this session discussed the use of high strain dynamic load testing, low strain integrity testing and both conventional and Osterberg-cell static load testing.

5.1 *Dynamic load testing*

Green & Kightley (2005) describe one of the first applications of the CAPWAP method of analysis for dynamic testing in South Africa. Predictions of base capacity at a silty sand site were in good agreement with static load test data whereas predicted shaft capacities (and shaft stiffness) were less than those measured in static load tests. These authors attributed this shaft friction discrepancy to 'set-up' effects and also state that dynamic test interpretation must be carried out by experienced personnel.

Such a need for expert interpretation could be reduced if automated procedures for signal matching of dynamic pile test data, such as described by *Charue & Holyman (2005)*, became more robust and reliable. These procedures, which are also being examined using genetic algorithms (Tyler 2004), are becoming more feasible as computer processing speeds increase and will, in future, hopefully reduce the subjective nature of pile dynamic analysis.

Maertens (2005) comments that there is a need for specification of clearer acceptance criteria for contract piles from dynamic test data and also that a contingency plan or remedial programme should be in place in the event that these criteria are not met.

5.2 *Integrity testing*

Passalacqua et al. (2005) describe observations from an extensive series of sonic integrity tests (i.e. employing a hammer and accelerometer at the pile head) conducted on bored and CFA piles. This paper highlights some of the deficiencies of this testing method showing, for example, the sensitivity of the recorded signal to the relative position of the hammer and accelerometer at the pile head and the influence of reflections from highly stratified soil deposits. Even matching of records from cores taken from test piles did not indicate clear compatibility with the corresponding integrity test. It is also noteworthy that the pile that was cored indicated that is was 35% shorter than it was supposed to be!

5.3 *Osterberg cell static load testing*

The ever increasing popularity of the Osterberg cell is evident from the high proportion of papers in this session which report static load test data derived from the Osterberg testing technique. For example*, Erol et al. (2005)* report data from Osterberg-cell tests applied to measuring rock socket friction in an extremely weathered amphibolite. The measured ultimate socket friction was, however, only 200 kPa and the base stiffness only 200 MPa. These results fall well short of empirical correlations relating friction and stiffness with the reported point load strengths, indicating that the 'rock' had little or no effective bond strength and behaved as a gravel.

Osterberg cells are employed on bored piles and some displacement piles and recently have been used to set a world record for static load testing – in which a static load of 27,800 tonnes was applied on a 3m diameter offshore pile for the Incheon 2nd Link project in Korea. With this increased use of the Osterberg-cell testing, it would appear necessary to have more comparisons than are presently available, which compare the shaft responses measured in conventional static compression and tension tests with those recorded in the standard Osterberg-cell test (i.e. where the cell is placed close to the tip of the test pile).

One such comparison is presented on Fig. 9, which is reproduced from test data reported by Kumar et al. (2004) for a series of tests conducted at the Southern Illinois University Carbondale (SIUC) in the U.S. The specific tests plotted were those conducted on 300mm square precast concrete piles driven through firm-stiff silty clay (with an average SPT N value of 12) to found on a hard sandy clay/shale at depth of about 6.5m.

A clear dependence of both the inferred stiffness and shaft capacity on the load testing method is apparent on Fig. 9. This trend is not unexpected given differences in the location of the load application and the mode of deformation of the pile itself. The effects of the residual load distribution, progressive failure and the pile length (or the influence of the stress free surface at ground level) are also likely to vary with between the two testing techniques.

Tan (2005) reports a similar tendency for Osterberg cell tests to lead to higher shaft capacities in clay. Such discrepancies need to be addressed if shaft friction test data derived from Osterberg cell tests are to be applied in an appropriate and effective way.

5.4 *Interpretation of static load tests*

Davisson (1972) defines the ultimate pile capacity in compression (Q_{cult}) as the pile head load at a settlement (δ_{ult}) equal to the pile's axial compression plus [D(mm)/120 + 4mm]. This definition of failure is popular in the U.S, although as indicated by *Senapathy et al (2005)*, it is not usually the only definition employed. *Baligh & Abdelrahman (2005)* propose a 'modification of Davisson's method' based on experience in Egypt.

The Davisson definition of δ_{ult} often leads to lower inferred Q_{cult} values than those obtained using the more common definition of Q_{cult} at δ_{ult}=D/10. This latter definition of failure is used extensively, but not exclusively, in Europe (e.g. see De Cock et al. 2003a), and there remains a clear need to achieve consensus on the assessment of the ultimate load from a static load test. However, achieving such a consensus would also require standardisation of static load testing procedures (e.g. see De Cock et al. 2003b) and ultimate limit state design procedures/factors.

Senapathy et al (2005) also describe some of the foundation cost savings that may be achieved, with an appropriate contract form, through the use of pre-contract static and dynamic testing. 'Design & construct' type contracts are ideally suited to this type of optimisation.

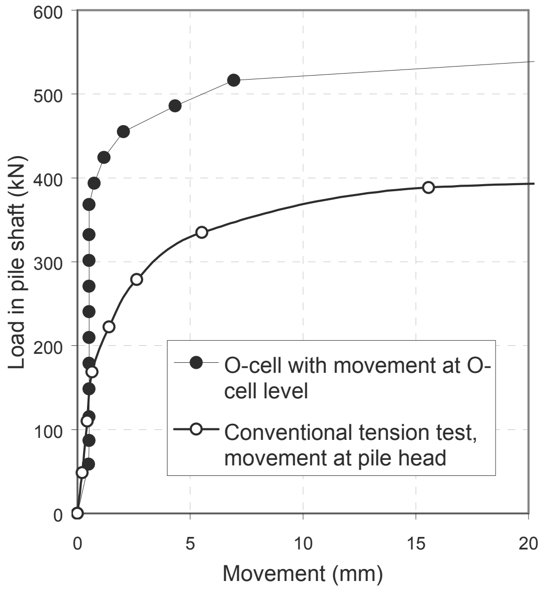

Fig. 9. Comparison of Osterberg-cell test with an equivalent conventional static tension test (Kumar et al. 2004).

6 RE-USE OF PILED FOUNDATIONS

The average life span of office buildings in some of the major cities of the World is only about 25 to 30 years (Chapman 2003) and it is becoming more difficult to avoid consideration of the option to re-use old foundations for new developments. *Vaziri (2005)* describes the numerous issues that needed to be addressed when piled foundations for a 30 year old, seven storey building were re-used for a new six storey development in the centre of London. The new design, which included transfer structures to minimise bending of the existing piles, was influenced by the quality of design information for the piles of the original building and the assessed integrity of these piles. The site investigation work comprised sonic integrity testing of all piles, hand excavated observation pits (to expose the upper sections of about 5% of the piles), and coring with subsequent chemical and strength tests of the concrete (which, incidentally, indicated cube strengths that were often more than three times the nominal strength). A critical issue raised by *Vaziri (2005)* was that of design responsibility, as the original piling contractor was not in a position to warrant the piles for the change in use and increased design life.

The basis of the design described by *Vaziri (2005)* was that pile settlements would be low if the new design loads on the existing bored piles in London clay did not exceed their previous maximum loads. This design assumption may be viewed as conservative on the basis of Fig. 10, reproduced from Powell et al. (2003), which shows that the re-loading capacity of jacked and driven piles in London clay after a period of about 20 years is 70% higher than that measured during initial loading (which took place after pore pressure dissipation); a similar increase in initial stiffness is also reported. Karlsrud & Haugen (1985), and others, provide evidence in support of the view that the gain in stiffness and capacity with time after pore pressure dissipation is a general characteristic of displacement piles in clay.

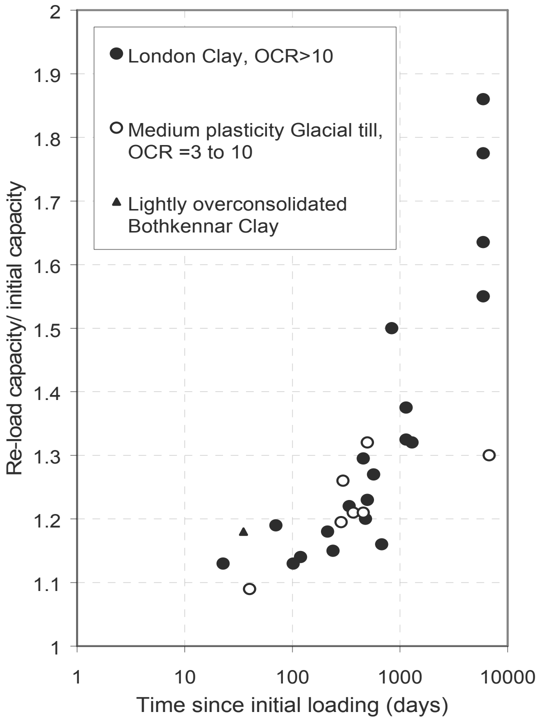

Fig. 9. Gain in capacity on re-loading of displacement piles in clay (reproduced from Powell et al. 2003)

For bored piles, however, the position is less clear. For example, Osterberg-cell load tests reported by Unwin & Jessep (2004) indicate that the re-loading shaft capacity of 1.05m diameter bored piles in London clay reduces with time to about 0.8-0.85 times the 2-week capacity after a period of a year; the same dataset indicated that the pile stiffness at typical working load levels did not show any effect of re-loading or ageing.

It would therefore appear that, for the re-used bored piles in London Clay described by *Vaziri (2005)*, the option to limit the axial load to that applied on first time loading was justified. While there is much evidence to support an increase in driven pile capacity with time, an assumption of enhanced capacity due to ageing for bored piles in clay is presently not justified in the absence of site specific static load test data.

7 SOIL-STRUCTURE INTERACTION

Maia et al. (2005) present an interesting case history, supplemented by 3D numerical analyses, which clearly illustrates the effects of soil-structure interaction for a 12 storey building north of Rio de Janeiro. They also remind us of the comment of Skempton & MacDonald (1956) who state that: "it does not matter how accurate the settlement analysis is if the actual value which can be supported by the structure is not known".

Each column of the building examined by *Maia et al. (2005)* was supported on a single 22m long, 500 to 600mm diameter CFA pile and the settlement of each column was monitored during the entire construction period. The settlement data indicated that there was no load redistribution between the columns over the early stages of construction and column settlements varied from 0.5 to 1.5 times the overall average settlement. Increments of differential settlements reduced as each subsequent floor level was constructed and virtually disappeared with the addition of the masonry infill. This infill substantially increased the overall rigidity of the structure and all increments of column settlements were virtually identical thereafter.

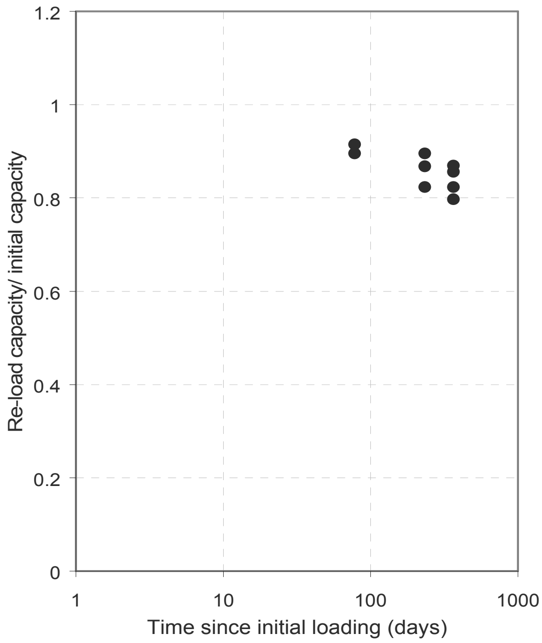

Fig. 10. Reload shaft capacity of bored piles in London Clay (reproduced from Unwin & Jessep 2004)

8 CONCLUSIONS

A review of the papers presented to this session has revealed a number of interesting trends and allowed identification of areas in need of further research. These are summarised as follows:

(i) As evidence grows for the improved capacity offered by cast in-situ screw piles over CFA and bored piles, screw piles are likely to begin to dominate the market for medium scale (300 to 800mm diameter) cast-in-situ bored piles.
(ii) The importance of soil displacement on the capacity developed by cast-in-situ piles is also recognised in new pile construction techniques such as the PCC pile.
(iii) Jacked piles are becoming more popular, but there is a need to investigate negative set-up effects on their base capacity in sands.
(iv) Although 3-D numerical approaches for pile groups are becoming more common, most reported analyses do not incorporate soil stiffness non-linearity and consequently do not match the observed response of pile groups in the field.
(v) The degree of subjectivity associated with the interpretation of dynamic pile test and sonic integrity test data may, in the future, be reduced as 'expert systems', currently under development, mature.
(vi) There is a shortage of comparative studies between Osterberg-cell and conventional static load tests; such studies are required urgently given observed differences between the respective shaft friction measurements and the growing popularity of Osterberg cell tests for both non-displacement and displacement piles
(vii) The re-loading capacity of bored piles in London Clay reduces with time while that of displacement piles increases with time. Further research is required to understand the (non-pore pressure related) mechanisms contributing to these opposing trends.
(viii) Realistic differential settlement predictions for a building must take account of the increase in the structure's rigidity as construction progresses.

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